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Guide to Remedial Measures for Liquefaction Mitigation at Existing Highway Bridge Sites

by

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Technical Report MCEER-99-0015

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NOTICE

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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 University at Buffalo, State University of New York, 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:

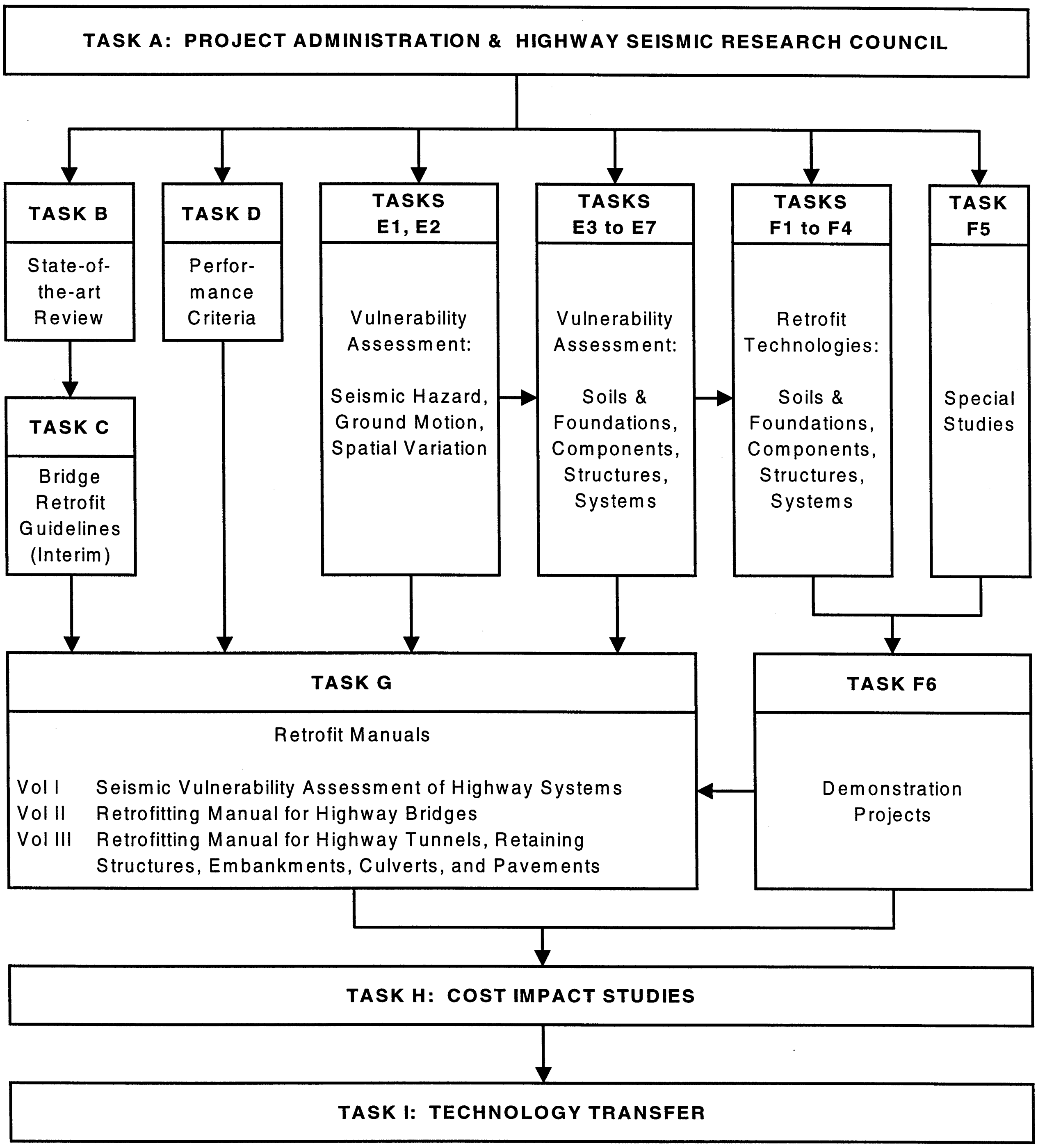
- assess the vulnerability of highway systems, structures and components;
- develop concepts for retrofitting vulnerable highway structures and components;
- develop improved design and analysis methodologies for bridges, tunnels, and retaining structures, which include consideration of soil-structure interaction mechanisms and their influence on structural response;
- review and recommend improved seismic design and performance criteria for new highway systems and 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-F-1/1(b), "Guide to Remedial Measures for Liquefaction Mitigation at Existing Highway Bridge Sites" of that project as shown in the flowchart on the following page.

The overall objective of this task was to develop methodologies to assess liquefaction risk at existing highway bridges, select appropriate ground and/or foundation improvement methods to reduce the risk of damage, design the improvements, and verify that these improvements had been

achieved in the field. The report describes the methodologies developed, which focus on assessing liquefaction risk at existing bridges. Procedures to assess liquefaction risk are presented in flowchart form, progressing from simple to more complex methods. A number of remediation methods are considered, including grouting compaction, vibro systems, surcharge and buttress fills, reinforcement and containment, vertical drains and underpinning. Relevant information is summarized from some numerical and laboratory (shaking table and centrifuge tests) studies conducted by other researchers on the performance of liquefaction remediation.

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



ABSTRACT

The seismic evaluation of an existing highway bridge requires the assessment of the liquefaction potential of supporting soils and resulting damage to the structure and its foundations. Where the risk of damage due to excessive deformations associated with the development of liquefaction is judged to be unacceptable, ground or foundation improvement can potentially be used to reduce the level of damage to acceptable levels. This report presents considerations for (1) evaluating the potential for liquefaction-induced damage to an existing highway bridge and the need for remediation, (2) selecting and designing remedial measures, and (3) implementing the proposed improvement schemes in the field.

Flow charts are included for assessing whether liquefaction remediation is necessary at an existing bridge with shallow or deep foundations based on evaluations of liquefaction potential, stability, and deformations of the ground and foundations. For a bridge where liquefaction remediation is judged necessary, a procedure is outlined for selection and design of remedial measure schemes. The applicability of various ground and foundation improvement methods at different abutment and pier types is presented. Factors that influence the design and performance of improved ground schemes are discussed, including treatment zone size and location. Guidelines are provided for the construction and verification of improvement measures including specifications, field tests, and quality assurance and control.

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SECTION 1
INTRODUCTION

1.1 Overview of Problem

Many existing bridges in the United States were built prior to a significant understanding of seismic liquefaction risks. As a result, some of these bridges are susceptible to damage due to the loss of soil strength and large ground deformations associated with the increase in pore water pressures that occur with complete or partial liquefaction of saturated, cohesionless soil deposits during and after earthquake shaking. In areas where the ground is level, bridges supported on foundations located above or in liquefiable soils can experience excessive movements due to bearing capacity failure or densification of loose deposits. Bridge foundations extending through liquefiable soils, such as piles or drilled shafts, can be damaged from the deformations and stresses that result from the loss of vertical and lateral support, as well as downdrag. If the ground surface is sloping or there is an abrupt change in grade, such as at a river channel, a bridge foundation can be subjected to lateral forces and displacements due to the movement of the liquefied soil stratum and overlying layers parallel to the direction of the slope or grade change (Youd, 1993).

Although existing bridges may be susceptible to seismic-induced liquefaction damage, remediation methods can be undertaken to reduce the risk of this damage. These methods include improving the liquefiable soil properties or foundations supporting the bridges (Mitchell, 1992; Mitchell and Cooke, 1995). Ground and foundation improvement techniques and usage for liquefaction mitigation have grown substantially in the last two decades. The performance of improved ground in the 1989 Loma Prieta, 1994 Northridge, and 1995 Kobe earthquakes has demonstrated that ground improvement can successfully be used to mitigate liquefaction (Mitchell et al., 1995).

Among the special characteristics of bridges and bridge sites that must be considered in the assessment of liquefaction potential and the mitigation of ground failure by improvement methods are:

- foundations located above, in, or through unstable ground;
- abrupt elevation changes at abutments;
- isolated supports;
- need to maintain the continuity of a long, linear structure;
- presence of thick, unconsolidated subsoil layers that are interbedded and non-uniform;
- sloping ground surface;
- difficult working conditions (i.e. underwater work, soft surficial soils, uneven terrain, and limited clearance room); and
- need to maintain bridge open to traffic during construction.

The user of this report should keep these issues in mind and their implications for the particular bridge being evaluated.

1.2 Purpose of Report

Efficient evaluation of the risk of liquefaction-induced damage to bridges and the development of effective remediation measures requires a consistent methodology. Such a methodology is presented in this report. It focuses on:

- assessing liquefaction risk at existing bridges,
- determining if ground or foundation improvements may be used to reduce the risk,
- selecting and designing a method or combination of methods to achieve the needed improvements by following a logical series of steps, and
- implementing the improvements in the field.

This report has been designed to be a general guidance document for liquefaction mitigation at existing highway bridges. The focus therefore is on the procedural steps, evaluations, and decision making involved in this process. Recommendations are presented regarding the information required in the process and methods of obtaining and evaluating it. The user is directed to other references for more specific details associated with these tasks. More detailed design recommendations regarding liquefaction remediation will be provided in a forthcoming MCEER report.

One of the goals of this report is to provide an understanding of the behavior of improved ground zones used for liquefaction mitigation and their impact on the performance of highway bridge foundations. Although there are field case histories demonstrating the positive impact of improved ground on the performance of supported structures, they frequently lack enough detailed information and measured data to allow a complete understanding of treated ground response, and factors affecting it. On the other hand, laboratory centrifuge and shaking table tests, along with numerical modeling, provide detailed data of the response of improved ground models during simulated earthquake shaking and liquefaction. For this reason, Section 5 of this report (dealing with liquefaction mitigation) presents results from centrifuge tests, shaking table tests, and numerical modeling studies performed by various researchers which provide insight into the factors and parameters affecting improved ground behavior, design, and performance.

Information presented in this report includes a discussion of potential liquefaction-induced failure mechanisms in Section 2 and bridge performance evaluation with regard to foundation stability and movement in Section 3. In Section 4 a methodology for assessing liquefaction remediation needs is presented. Potential remediation methods and an overview of the process for selecting and designing remedial measures are presented in Sections 5 and 6, respectively. Considerations in implementation of foundation and ground improvements at a bridge site are reviewed in Section 7 followed by conclusions in Section 8.

SECTION 2
LIQUEFACTION-INDUCED FAILURE MECHANISMS

2.1 Types of Failure Mechanisms

Many bridges have collapsed as a result of earthquake-induced liquefaction. Ross et al. (1969) studied bridges that collapsed during the 1964 Alaskan earthquake. Similarly Youd (1993) reviewed bridge collapses due to liquefaction-induced deformations caused by the following earthquakes (date and location): 1868, Hayward, California; 1886, Charleston, South Carolina; 1906, San Francisco, California; 1964, Prince Williams Sound, Alaska; 1964, Niigata, Japan; and 1991, Limon Province, Costa Rica.

Failure mechanisms which can develop during liquefaction and cause damage to bridges due to resulting ground deformations and foundation movements include lateral spreading, loss of bearing capacity and settlement, ground oscillations, and flow failures (Youd, 1992).

2.1.1 Lateral Spreading

Damage to many bridges due to earthquake-induced liquefaction has resulted from lateral spreading of gently sloping ground towards river channels. Lateral spreading consists of the displacement of ground down slopes typically having inclinations of 3 degrees or less, or towards an incised channel, as a result of liquefaction of underlying soils (Youd, 1992 and 1993). The displacements are usually incremental, occurring at periods during the earthquake when the strength of the liquefied material is less than needed to resist the lateral forces acting on the overlying non-liquefied soil (Kramer, 1996). The overlying soil is usually broken up in blocks which displace downslope or toward the incised channel, on top of the liquefied soil, as shown in Figure 2-1. General characteristics of lateral spreading that are manifested in the ground, as described by Youd (1993), are "extensional deformations at the head of the feature, shear deformations along the margins, and compressed ground at the toe." Displacements can range from a few centimeters to several meters.

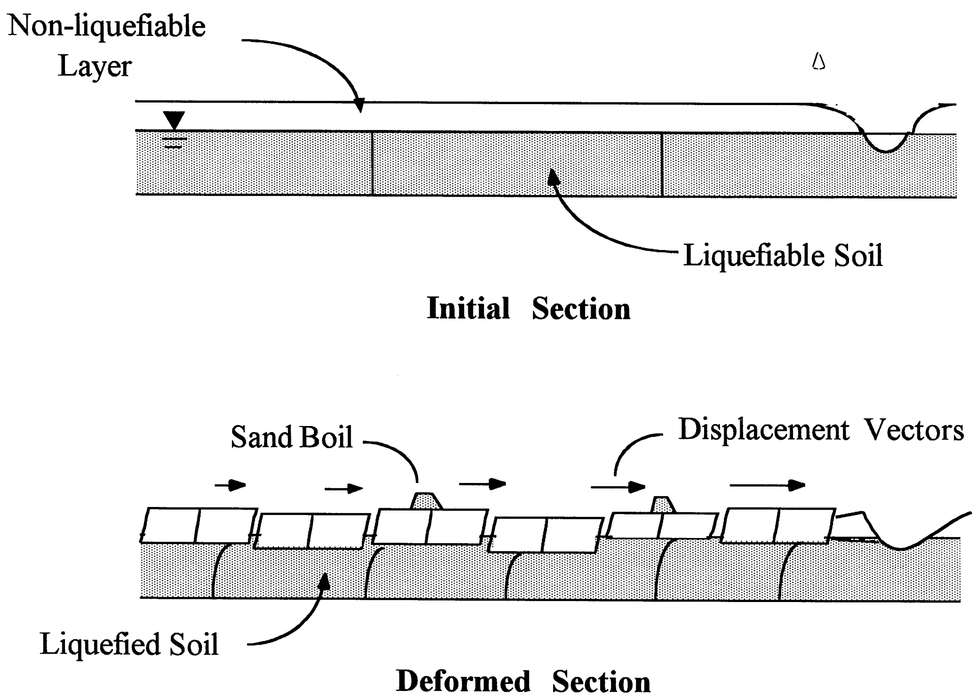


FIGURE 2-1 Lateral Spreading Mechanism (after Youd, 1984)

Since bridges are typically located at the toe of a lateral spread, they are commonly subjected to compression. Damage to the bridge is generally caused by differential lateral ground displacement. The type and magnitude of damage depend on such factors such as the foundation, superstructure, substructure, and connection characteristics.

2.1.2 Loss of Bearing Capacity and Settlement

Loss of bearing capacity results from the loss of strength associated with the increase in pore water pressures and softening of the soil occurring during partial or full liquefaction. The reduction in bearing capacity can result in excessive settlements/movements of a bridge pier or abutment whose foundation bearing pressure exceeds the reduced capacity, as shown schematically in Figure 2-2.

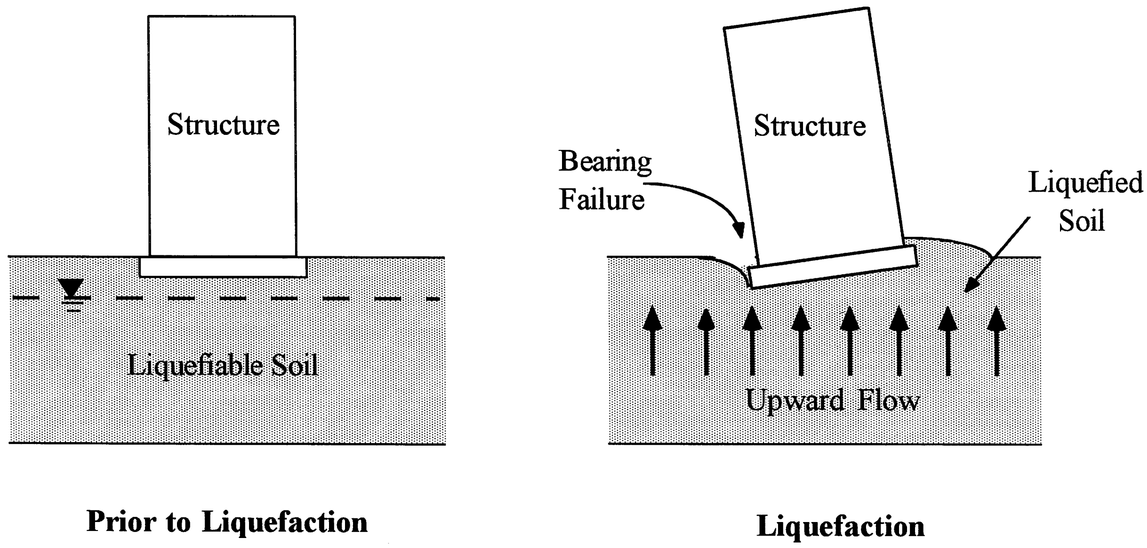


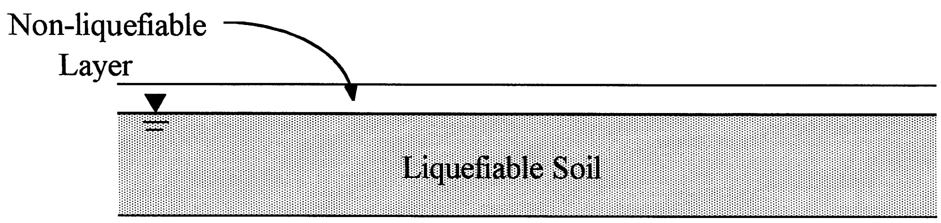
FIGURE 2-2 Bearing Capacity Failure (after Youd, 1984)

Excessive movements can also occur in the absence of a catastrophic or sudden ground failure, as a result of the cyclic loading of the foundation which causes it to gradually penetrate into the weakened soil. In addition, settlements can be induced due to the densification which occurs when excess pore water pressures dissipate in partially or fully liquefied soils.

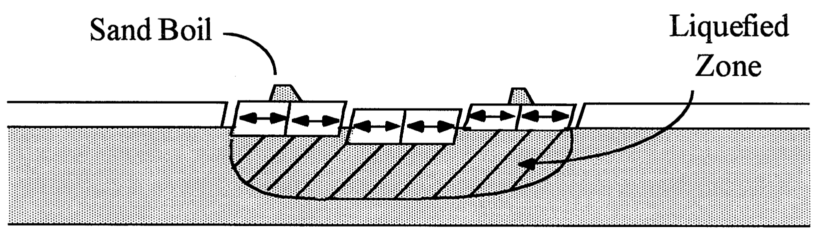
Similar to a loss of bearing capacity is the loss of axial and lateral support for deep foundations extending through liquefiable soil. This loss of support can cause excessive deformations and stresses in piles or drilled shafts resulting in damage.

2.1.3 Ground Oscillation

Ground oscillation is a phenomenon that occurs on relatively level ground where lateral spreading does not occur. In this phenomenon, broken blocks of non-liquefied surficial soil oscillate back and forth and up and down on top of an underlying liquefied layer during an earthquake, as shown in Figure 2-3 (Youd, 1992). A bridge supported by the surficial layer can experience severe



Before Earthquake



During Earthquake

FIGURE 2-3 Ground Oscillation Phenomena (after Youd, 1984)

deformations when substructure columns or walls supported by shallow foundations on different blocks undergo differential movements.

2.1.4 Flow Failure

Flow failure is the rapid movement of liquefied soil and overlying layers down slopes having angles typically greater than 3 degrees, as illustrated in Figure 2-4. According to Youd (1992), "these failures commonly displace large masses of soil tens of meters and in a few instances, large masses of soil have traveled tens of kilometers down long slopes at velocities ranging up to tens of kilometers per hour." The large displacements result from the residual strength of the liquefied soil being less than necessary to resist the static gravitational forces acting on overlying unliquefied soils during and after earthquake shaking (Kramer, 1996). Although such failures have primarily been observed to occur in offshore seabeds or tailings dams, they may be possible at a bridge site given sufficient ground slope and the proper subsurface conditions. This type of failure could cause severe damage to a bridge supported on, or even through, the liquefiable soil.

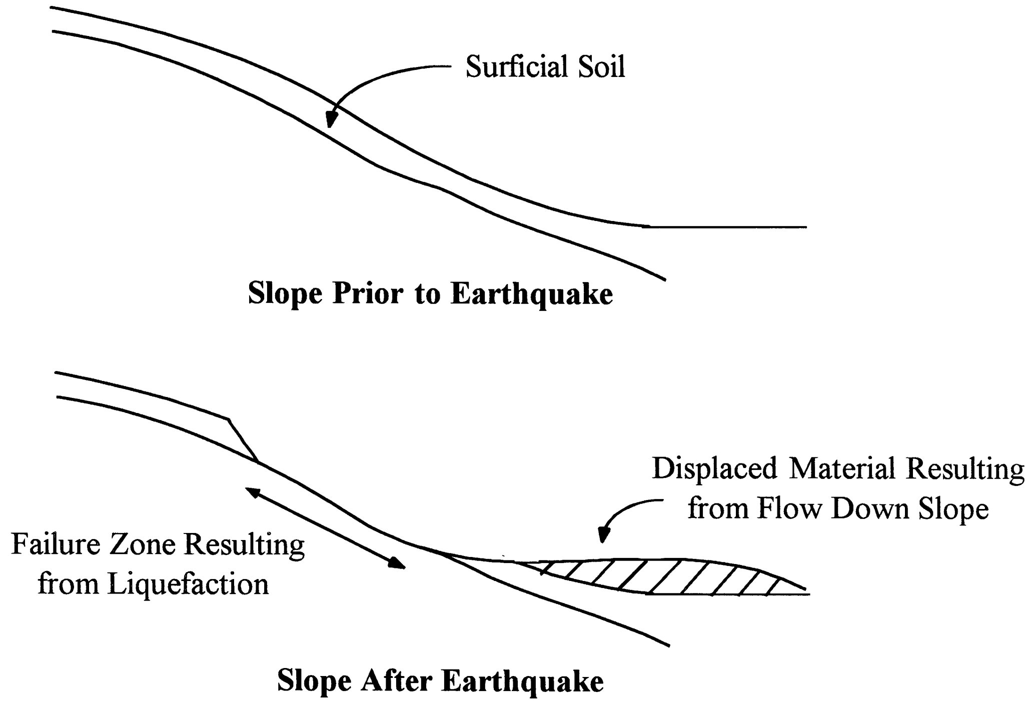


FIGURE 2-4 Flow Failure (after Youd, 1984)

2.2 Implications for Assessment and Remediation

During the assessment of potential liquefaction-induced damage to an existing bridge, it is important to identify the potential liquefaction-induced ground failure mechanisms that the bridge may experience. The failure mechanism will affect the magnitude of the forces and displacements to which the bridge is subjected. Likewise, the ground deformation pattern resulting from the failure mechanism will be a key indicator regarding the potential location for remedial measures. The magnitudes of forces that the remedial measures must withstand are also dependent on the failure mechanism. For these reasons, the evaluation of bridge performance described in the next section considers these different failure mechanisms, with particular emphasis on bearing capacity failure, settlement, and lateral spreading.

SECTION 3
BRIDGE PERFORMANCE EVALUATION

3.1 Introduction

Stability and deformations are both important considerations for evaluating whether the foundations of a bridge will perform satisfactorily during a seismic event that induces liquefaction of the surrounding soils. Both the factor of safety against a stability failure and the magnitude and direction of displacements should be estimated for bridge piers and abutments supporting the superstructure to assess whether the bridge performance is expected to be satisfactory or remedial measures will need to be considered. The performance of bridge foundations during a seismic event that induces liquefaction must be evaluated for conditions which develop both during earthquake loading and afterwards.

Although stability and deformations should both be evaluated, it is the deformations of the bridge foundations which ultimately govern performance. Pseudostatic stability analyses can be performed to evaluate the stability of foundations, where the seismic forces acting on the foundation and soil mass during earthquake loading are represented as equivalent static loads. However, inadequate safety factors from these analyses do not necessarily indicate inadequate performance, because the condition of instability may only exist for short periods of time during shaking. The final deformations that accumulate during shaking due to transient instability may be acceptable. By the same token, a factor of safety greater than one from a pseudostatic analysis does not guarantee adequate performance, because accumulated deformations which occur during shaking could be excessive. Therefore, although the factor of safety for stability during earthquake loading may be used as a general indicator of potential performance, the ultimate indicator should be computed deformations. Factor of safety alone can not be used as a direct indication of potential deformation under seismic loading due to the dependency of the deformation on the frequency of the input motion (Riemer et al., 1996).

Unlike the earthquake-loading case, evaluation of stability for post-earthquake conditions will be a definitive indicator of inadequate performance if the factor of safety against failure based on post-earthquake soil strength is less than one. On the other hand, if the factor of safety is adequate, deformations will usually still need to be evaluated to ensure that they are within acceptable limits.

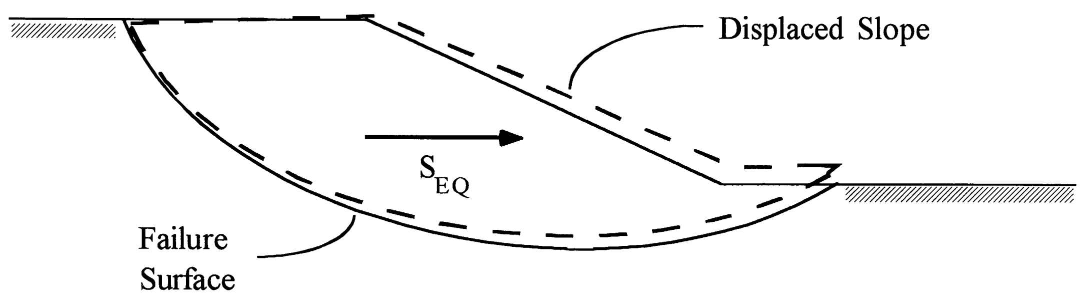
3.2 Stability

3.2.1 Footings and Slopes

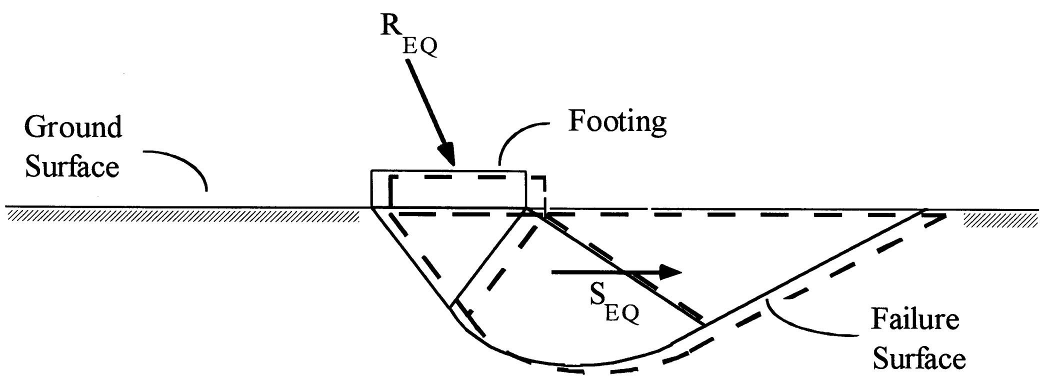
Methods for evaluating the stability of footings and slopes under earthquake loading have traditionally utilized pseudostatic approaches. Evaluating footing stability using pseudostatic approaches has been discussed by Dormieux and Pecker (1995), Shi and Richards (1995), and Paolucci and Pecker (1997). Pseudostatic slope stability methods are fairly well established and described in Kramer (1996). Illustrations showing typical seismic forces and failure surfaces considered in pseudostatic analyses for slopes and footings are presented in Figure 3.1.

As explained by Kramer (1996), one of the difficulties of pseudostatic analyses is selecting appropriate seismic coefficients to use in the analyses to adequately represent the earthquake-induced forces. Table 3-1 summarizes information presented by Kramer on pseudostatic coefficients proposed by others, primarily for seismic evaluations of earth dams. Kramer concludes there are "no hard and fast rules for selection of a pseudostatic coefficient for design."

Another difficulty in performing pseudostatic analyses is assigning appropriate strengths for the soils. The strengths of most soils degrade with continued cyclic loading. Loss of strength is particularly great for soils that develop high excess pore water pressures during shaking, such as loose, saturated sands. Kramer (1996) states that "pseudostatic analyses can be unreliable for soils that build up large pore pressures or show more than 15% degradation of strength during shaking."



(a) Slope



(b) Footing (after Shi and Richards, 1995)

Notes:

R_{EQ} = Seismic load exerted on footing by structure

S_{EQ} = Seismic-induced inertial load acting on soil above failure surface

Magnitude and direction of forces and displacements vary with time during earthquake.

FIGURE 3-1 Seismic Forces and Potential Failure Surfaces for Slopes and Footings

TABLE 3-1 Reported Pseudostatic Seismic Coefficients for Earth Dams (after Kramer, 1996)

Author	Suggested Pseudostatic Coefficient, k_h	Comments
Terzaghi (1950)	$k_h = 0.1$ for "severe" earthquake $k_h = 0.25$ for "violent, destructive" earthquakes $k_h = 0.5$ for "cata-strophic" earthquakes	"Severe" earthquakes correspond to Rossi-Forel IX and "violent, destructive" earthquakes correspond to Rossi-Forel X. Terzaghi states that the most sensitive materials include "submerged or partially submerged loose sand."
Seed (1979)	$k_h = 0.10$ for $M = 6.5$ to $k_h = 0.15$ for $M = 8.25$	Applicable for dams that: (1) are constructed of soils that do not generate high excess pore pressures or lose more than 15% of strength under cyclic loading, and (2) have crest accelerations of 0.75g or less. Deformation of dam should be acceptable if pseudostatic factor of safety at least 1.15.
Marcuson (1981)	$k_h = 1/3\ a_{max}/g$ to $1/2\ a_{max}/g$	Maximum dam acceleration should include amplification and deamplification effects.
Hynes-Griffin and Franklin (1984)	$k_h = 0.5\ a_{max}/g$	Concluded that earth dams with pseudostatic factor of safety greater than 1.0 would not develop "dangerously large" deformations.

Use of the pseudostatic method for evaluating the stability of footings or embankments underlain by liquefiable soil strata requires that the strength of these materials be reduced to account for the effects of high excess pore water pressures that develop during earthquake loading. The reduced strength used for these materials is dependent on the excess pore water pressures expected during shaking. It is also dependent on the density of the liquefiable soil. Loose, granular soils, such as clean sands, will tend to collapse and have low residual strengths at large strains. On the other hand, dense soils will tend to dilate and have higher strengths. Guidance regarding the selection of residual strengths can be found in Seed and Harder (1990).

For cases where a soil stratum is expected to liquefy during shaking, resulting in a significant loss of strength in the liquefiable soil, the standard-of-practice is to evaluate the potential for a flow slide failure. In this case, stability is evaluated for static loading using undrained residual shear strength for the liquefied soil, as commonly done for earth dams.

Post-earthquake analyses of all embankments and foundations should be performed. These analyses should reflect the worst conditions existing after shaking has stopped. An important consideration in this case may be the migration of excess pore water pressures from liquefied zones into denser zones that have not liquefied and are supporting bridge foundations or embankments. This migration needs to be estimated and the effect on strengths taken into account.

3.2.2 Deep Foundations

3.2.2.1 Issues

The stability of deep foundations (i.e. piles and drilled shafts) extending through soils which liquefy is difficult to evaluate due to the many factors involved. The combined axial and lateral loading performance of the deep foundations must be evaluated taking into consideration the seismic loads exerted by the bridge, downdrag forces exerted as liquefied soils and overlying deposits settle, and the transfer of lateral and axial loads along the length of the foundation as soil strata liquefy or soften. In addition, at locations where lateral spreading is expected to occur, additional lateral loads on the foundation must be taken into account.

3.2.2.2 Axial Loading

The capacity of deep foundations to resist axial loading conditions during earthquake loading and liquefaction can be evaluated using traditional pile capacity analysis procedures. In this evaluation, the transfer of vertical load along the length of the deep foundation from soil layers which liquefy or soften during earthquake shaking to more competent layers needs to be taken into account. In the analysis, the strength of liquefiable soils will need to be reduced based on the level of excess pore water pressure expected to develop. Non-liquefiable soil strengths need to be reduced for the level of cyclic degradation expected.

A simplified method for assessing downdrag forces exerted on piles/shafts due to liquefaction and settlement of surrounding soils consists of estimating the relative settlement of the liquefied soils to the piles/shafts. Based on the predicted magnitude of the relative settlements and the post-earthquake strength properties of the soils, an assessment of potential downdrag forces exerted on the sides of the piles/shafts should be possible. Settlements of liquefiable soils due to seismic loading can be estimated using Tokimatsu and Seed's (1987) or Ishihara and Yoshimine's (1991) methods.

3.2.2.3 Lateral Loading

The ability of deep foundations to resist lateral loads is a problem involving soil-structure interaction in which displacements of the foundation are an important factor. Discussion of lateral loading is presented in Section 3.3 dealing with deformations.

3.3 Deformations

3.3.1 Footings and Slopes

3.3.1.1 No Lateral Spreading

Footings and slopes will experience movements when subjected to seismic loading. For cases where conditions are not conducive to lateral spreading, the movements experienced by footings and slopes founded over liquefiable soils can be estimated using methods which range from relatively simple and empirical approaches to more complex numerical methods.

One simple method for estimating slope deformations is through application of Newmark's (1965) sliding block analysis, as illustrated in Figure 3.2. In this method, pseudo-static slope stability

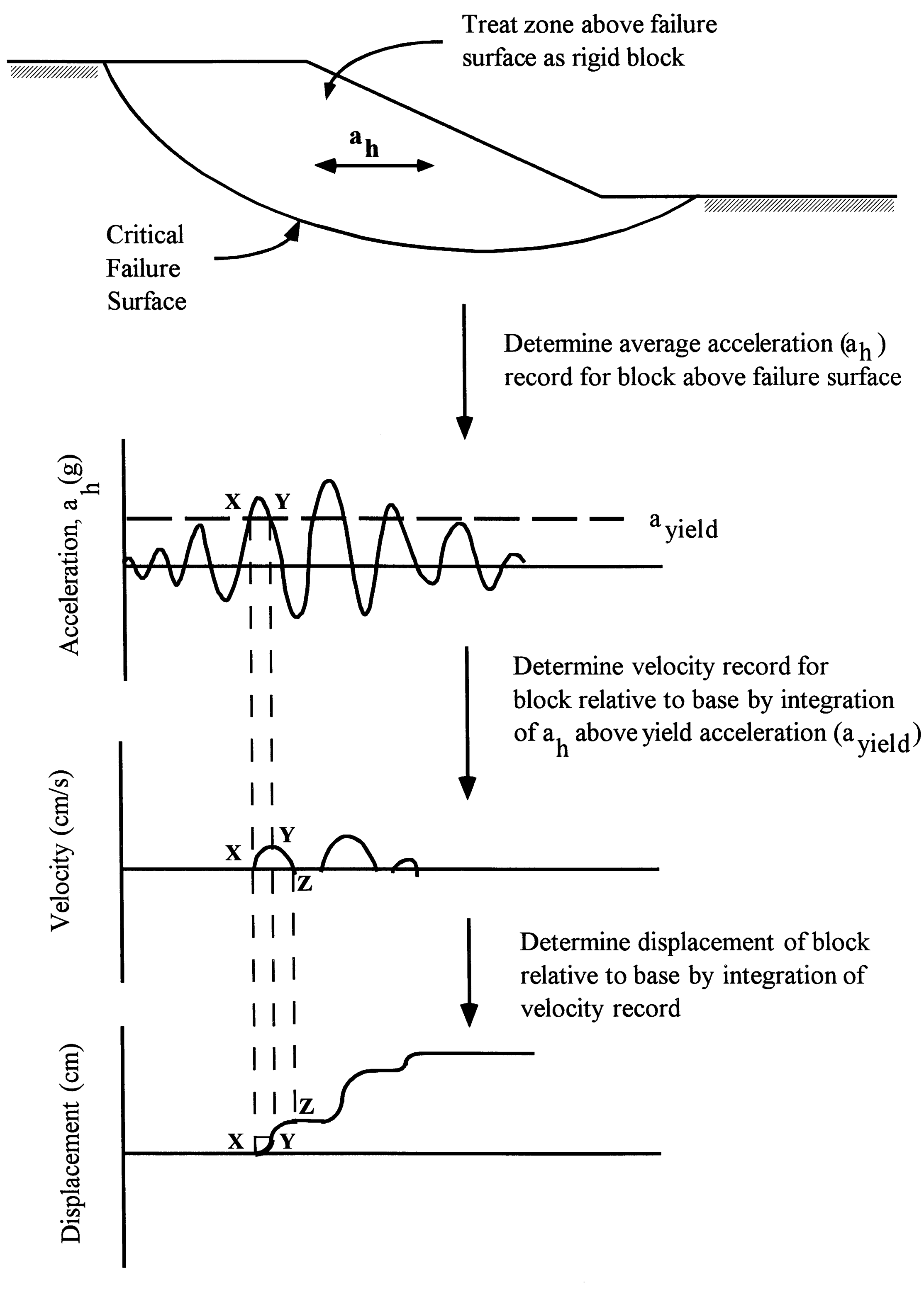


FIGURE 3-2 Estimation of Displacements by Newmark (1965) Sliding Block Method (after Jibson, 1993)

analyses are performed for several potential sliding surfaces to determine the seismic acceleration producing a factor of safety of one along each surface (known as the yield acceleration). The surface with the lowest yield acceleration is generally selected for evaluation of deformation. A seismic response analysis is then done for the profile, using the specified input motion, from which an average acceleration record for the zones above the sliding surface is developed. This record is used to compute the permanent lateral displacements which occur during shaking (Newmark, 1965) by double integration of the portion exceeding the yield acceleration, assuming rigid-plastic soil behavior. More details regarding Newmark analyses can be found in Munfakh et al. (1998). Important factors in this analysis are the strength properties assigned to the soil strata that liquefy during shaking. Recent studies by Dobry and Abdoun (1998) indicate that even liquefied, loose soils may exhibit dilative behavior which could result in Newmark predictions being conservative if residual strengths (i.e. - Seed and Harder, 1990) are used. Finn (1998) indicates Newmark's method is only applicable when liquefiable layers are relatively thin. Vertical deformations from densification of liquefiable soils can be estimated with simplified methods based on the Standard Penetration Test (SPT) (i.e. - Tokimatsu and Seed, 1987, or Ishihara and Yoshimine, 1991).

As mentioned earlier, the factor of safety for stability can not be used as a direct indicator for embankment or slope deformations. However, Youd (1998) indicates that based on past experience and analytical calculations, a slope with an adequate static factor of safety will generally experience small deformations. He states that "adequate factors of safety that generally reduce displacements to acceptable limits (less than 100 millimeters) are approximately 1.5 for magnitude 6.5 earthquakes, 2.0 for magnitude 7.5 earthquakes, and 2.5 for magnitude 8.5 earthquakes." These values are recommended by Youd as screening guides for classifying slopes as minimally deformable and non-hazardous.

A simplified method of determining the settlement of a footing supported on the surface of a level, uniform sand deposit under seismic loading is presented by Shi and Richards (1995). This method is similar to the Newmark analysis described above with the exception that the yield acceleration is determined using a simple seismic bearing capacity analysis that takes into account the shear between the footing and underlying soil, as well as the inertial force acting on the supporting soil.

The analysis includes using a pair of simplified failure surfaces (to account for displacements that occur due to movement in opposite directions during shaking) consisting of active and passive wedges, as shown in Figure 3-3. Settlements, w, are computed from the horizontal displacements of the assumed Coulomb failure wedges beneath the footing using equation 3-1.

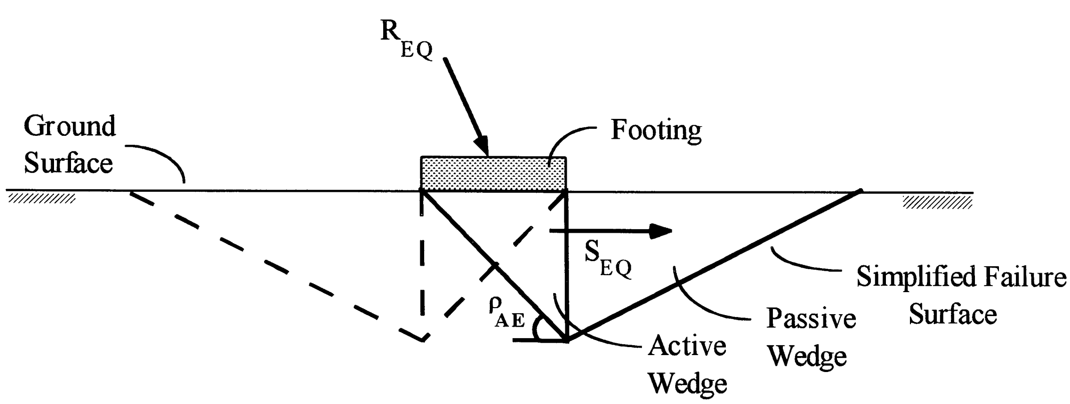
$$w = 2\,\Delta \tan \rho_{AE} \tag{3-1}$$

where:

Δ = Horizontal sliding block displacement of the fictitious wall dividing the active and passive wedges

ρ_{AE} = Active wedge angle

Shi and Richards (1995) developed this method for estimating settlements of footings on stable sand deposits, so using it for footings supported on or above fully or partially liquefiable sands may be tenuous. In addition to the settlement computed from this formula, settlements due to densification of soil deposits beneath the structure which partially or fully liquefy need to be estimated and included.



Notes:

R_{EQ} = Seismic load exerted on footing by structure

S_{EQ} = Seismic-induced inertial load acting on soil above failure surface

ρ_{AE} = Active wedge angle

Dashed lines represent wedges and failure surface when earthquake loading reverses direction

FIGURE 3-3 Loading and Failure Mechanisms for Footing Settlement Estimation (after Shi and Richards, 1995)

Additional insights into the potential damage to structures on foundations located over or in liquefiable soils can be obtained from charts developed by Ishihara (1995), shown in Figure 3-4, as cited by Moriwaki (1997). These charts, which are based on case history data, indicate combinations of liquefiable layer thickness and non-liquefiable surface layer thickness that were observed to cause damage or no damage to structures having foundations with different embedment depths.

3.3.1.2 Lateral Spreading

Approximate estimates of the movements of footings and slopes due to lateral spreading can often be made using empirical procedures. One such procedure for estimating lateral spreading deformations by Bartlett and Youd (1995) consists of using empirical equations for estimating deformations for sites near steep banks, where free face conditions predominate (Eqn. 3-2), and for sites located on gently sloping ground (Eqn. 3-3).

For free-face conditions:

$$\log D_H = -16.366 + 1.178\,M - 0.927\log R - 0.013\,R + 0.657\log W + 0.348\log T_{15} + 4.527\log(100 - F_{15}) - 0.922\,D50_{15} \tag{3-2}$$

For gently sloping ground conditions:

$$\log D_H = -15.787 + 1.178\,M - 0.927\log R - 0.013\,R + 0.429\log S + 0.348\log T_{15} + 4.527\log(100 - F_{15}) - 0.9224\,D50_{15} \tag{3-3}$$

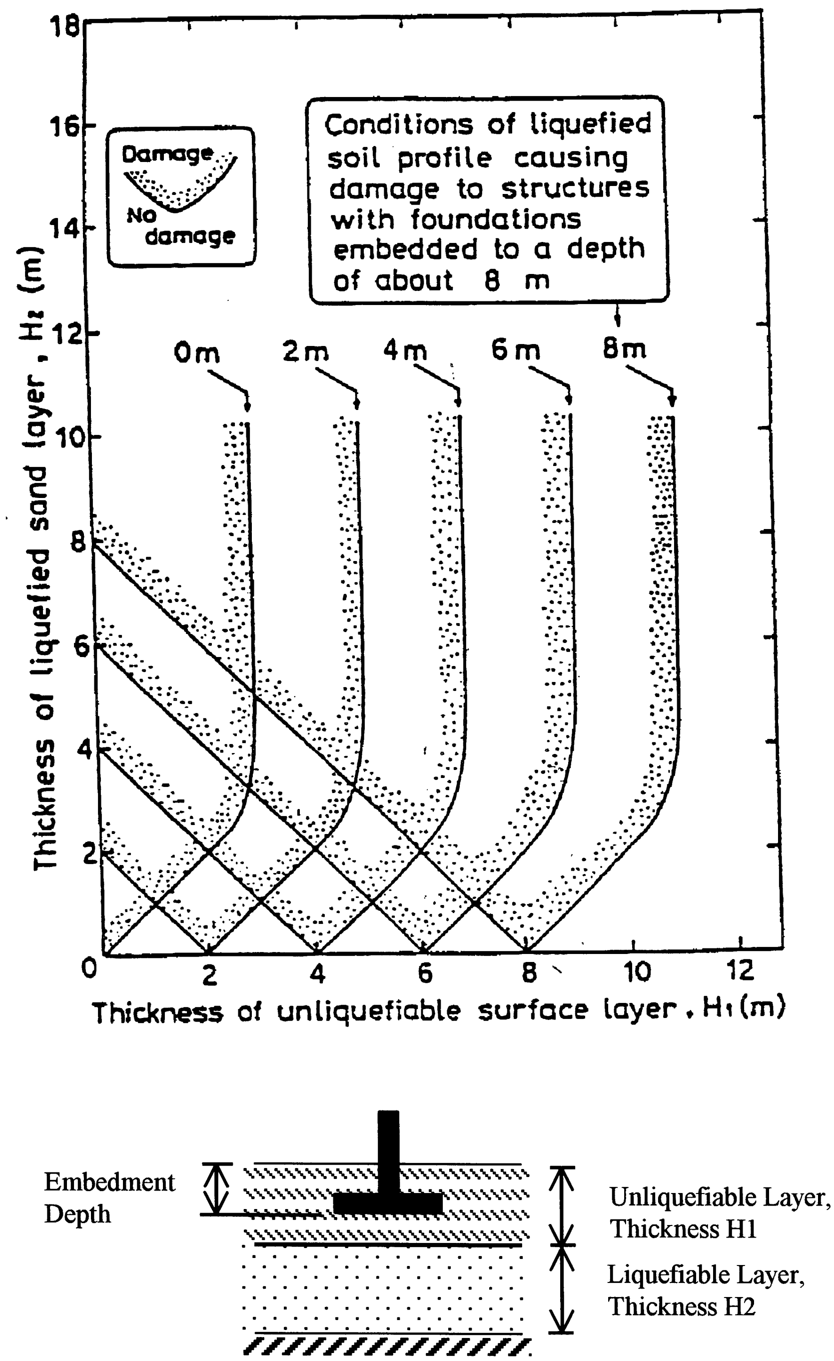
where:

D_H = Estimated horizontal ground displacement, in meters

M = Earthquake magnitude (moment magnitude)

R = Horizontal distance from the seismic energy source, in kilometers

T_{15} = Cumulative thickness of saturated granular soils with corrected blow counts, $(N_1)_{60}$, less than 15, in meters



Notes:

1. Liquefiable soil layer having thickness H_2 is located beneath unliquefiable surface layer of thickness H_1.
2. Chart is based on select case history information. Refer to Ishihara (1995) for more details and potential limitations.

Figure 3-4 Observation of Damage vs. No Damage to Foundations Embedded at Various Depths in Unliquefiable Surface Layer Overlying Liquefiable Sand Layer (annotated from Ishihara, 1995)

F_{15} = Average fines content (fraction of sediment sample passing a No. 200 sieve) for granular layers included in T_{15}, in percent

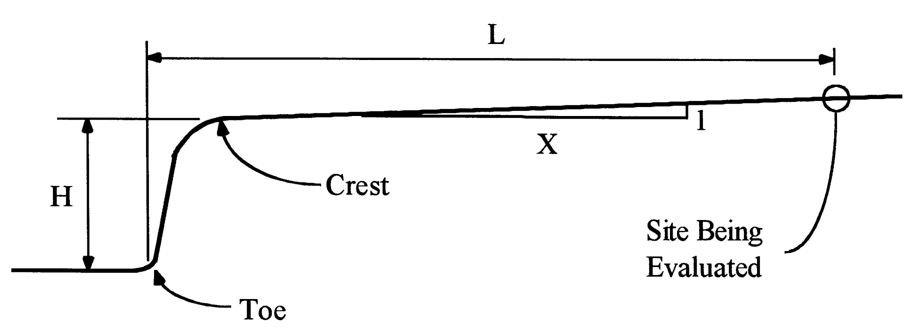
$D50_{15}$ = Average mean grain size in granular layers included in T_{15}, in millimeters.

S = Ground slope, in percent (refer to Figure 3-5)

W = Free-face ratio (refer to Figure 3-5)

In some cases where both free-face or gently sloping ground conditions could be applicable, as illustrated in Figure 3-5, lateral spreading estimates are made independently using Equations 3-2 and 3-3. The more conservative estimate is then used for the site.

A comparison of actual to predicted displacements made by Bartlett and Youd using their original database indicates the predicted displacements from the empirical equations varied from one-half to two times the actual displacements, with 90 percent confidence. Although Bartlett and Youd's procedure does not inherently account for the presence of foundations, the displacement of the foundation can conservatively be assumed to be the same as the predicted ground displacement, thus neglecting the restraining effects of the structure.



L = Distance from Toe of Free Face to Site Being Evaluated
H = Height of Free Face (Crest Elevation - Toe Elevation)
W = Free-Face Ratio = (H / L) (100), in percent
S = Slope of Natural Ground Toward Channel = (1/ X) (100), in percent

FIGURE 3-5 Definitions of Slope, S, and Free-Face Ratio, W, in Bartlett and Youd (1995) Equations for Lateral Spreading Estimates (after Youd, 1993)

3.3.1.3 Complex Methods

More complex methods for estimating seismically-induced permanent deformations generally use 2-D or 3-D dynamic deformation analyses. These analyses are typically based on an effective stress analysis and use a non-linear, elastic-plastic constitutive soil model. The most comprehensive of these analyses use a fully-coupled solution scheme which satisfies both the equilibrium and flow equations at the same time (Smith, 1994; Arulanandan, 1996). Such a solution scheme is incorporated in the computer codes DYNAFLOW (Prevost, 1981), DIANA (Kawai, 1985), DYSAC2 (Muraleetharan et al., 1988, 1991) and SWANDYNE 4 (Zienkiewicz et al., 1990a, 1990b). An alternative scheme consists of using a partially-coupled solution where the equilibrium equations are typically solved first followed by calculation of pore water pressures from volumetric strains estimated using relationships, such as those developed by Martin et al. (1975) or Byrne (1991). This type of solution scheme is utilized in the computer program TARA-3 (Finn et al., 1986) and provided with the program FLAC (Itasca Consulting Group, Inc., 1996).

One advantage of the complex analysis methods is they allow problems to be analyzed which can not be readily evaluated using the procedures described above for footings and slopes. However, care should be exercised when using these more complex analyses for lateral spreading problems that involve large deformations, because the analytical methods may become numerically unstable. Finite difference solution methods, such as the type employed by the computer program FLAC (Itasca Consulting Group, Inc., 1996) seem to be particularly well suited for handling large deformation problems.

The performance of complex methods for predicting the response of liquefiable soils under dynamic loading has been mixed. Results from the Verification of Liquefaction Analysis by Centrifuge Studies (Arulanandan and Scott, 1994), or VELACS, project indicate that the partially-coupled or uncoupled solution schemes used in that study did not predict the performance of liquefiable soils under dynamic loading particularly well. By the same token not all fully-coupled solutions performed well, particularly for cases involving footings or walls supported on liquefiable soils. However, recent information in the literature suggests that reasonable results can be obtained from complex codes. Arulanandan et al. (1997) report the successful use of DYSAC2 to predict the behavior of liquefiable soil deposits in the Marina District of San Francisco during the 1989 Loma Prieta earthquake. Both the codes DYSAC2 and DYNAFLOW are reported by Arulanandan (1996) as having adequately predicted the response of a submerged embankment subjected to dynamic loading in a centrifuge test. According to Madabhushi and Zeng (1998), the code SWANDYNE successfully predicted the seismic response of a gravity quay wall on a liquefiable sand modeled in a centrifuge. Finn (1988, 1991) reports the successful validation of TARA-3 using centrifuge studies. The program SUMDES (Li et al., 1992), which is a code utilizing a fully-coupled solution for evaluating liquefiable soil deposits under 1-D dynamic loading, has been reported by Li et al. (1998) as providing reasonable predictions of a site response for the 1986 Lotung earthquake in Taiwan.

3.3.2 Deep Foundations

Estimates of pile/shaft deformations under axial and lateral loading can be obtained using traditional analytical methods. If refined estimates of pile deformation are needed, more complex analyses are required.

3.3.2.1 Axial Loading

The settlement of piles/shafts under axial loading can be evaluated using traditional analytical methods (e.g. t-z curves) for the axial load/deformation behavior of deep foundations. As part of this evaluation the strength of liquefiable soils will need to be reduced based on the level of excess pore water pressure expected to develop. Non-liquefiable soil strengths need to be reduced for the level of cyclic degradation expected. Downdrag loads should also be included, as appropriate.

3.3.2.2 Lateral Loading without Lateral Spreading

Assessment of pile/shaft performance under lateral loading can be made by superimposing on the pile/shaft the ground deformations estimated from a free-field ground response analysis and pile deformations estimated from a p-y analysis of the piles due to inertial loads from the structure. When performing analyses of piles/shafts under lateral loading using the superposition method, the piles/shafts are typically analyzed for a portion of the peak soil displacements and peak inertial forces from the structure due to the low probability that both peaks will occur at the same time. For example Caltran's (1994) interim procedure for evaluating the performance of piles/shafts under earthquake loading consists of evaluating the deformations and stresses in the piles for the following load and soil strain conditions:

- 140 percent of the vertical dead load (to account for higher loads in perimeter piles due to overturning moments) from the supported structure, 100 percent of the horizontal (shear) inertial load from the supported structure, and no soil strain (soil strain is due to deformation of the soil mass in response to the propagating seismic wave);
- 140 percent of the vertical dead load, 25* percent of the horizontal inertial (shear) load, and the peak uni-directional soil-strain; and
- 140 percent of the vertical dead load, 25* percent of the horizontal inertial (shear) load, and the peak bi-directional soil strain.

* Note: Jackura and Abghari (1995) indicate that using 25 percent of the horizontal inertial (shear) load may be unconservative and 50 percent may be more appropriate.

The analysis of pile behavior due to the lateral loading component from the supported structure during an earthquake requires that the resistance provided by liquefiable soil, as well as non-liquefiable soils subject to cyclic strength degradation, be appropriately reduced. ATC-32 (Applied Technology Council, 1996) states that:

> "Recent unpublished centrifuge tests conducted by Ricardo Dobry at the Rennsalaer Polytechnic Institute indicates that fully liquefied sand (from freefield liquefaction effects) has a residual strength of about 10 percent of the initial p-y curve resistance, as determined by Reese's (Reese et al., 1974) p-y procedure. The 10 percent residual

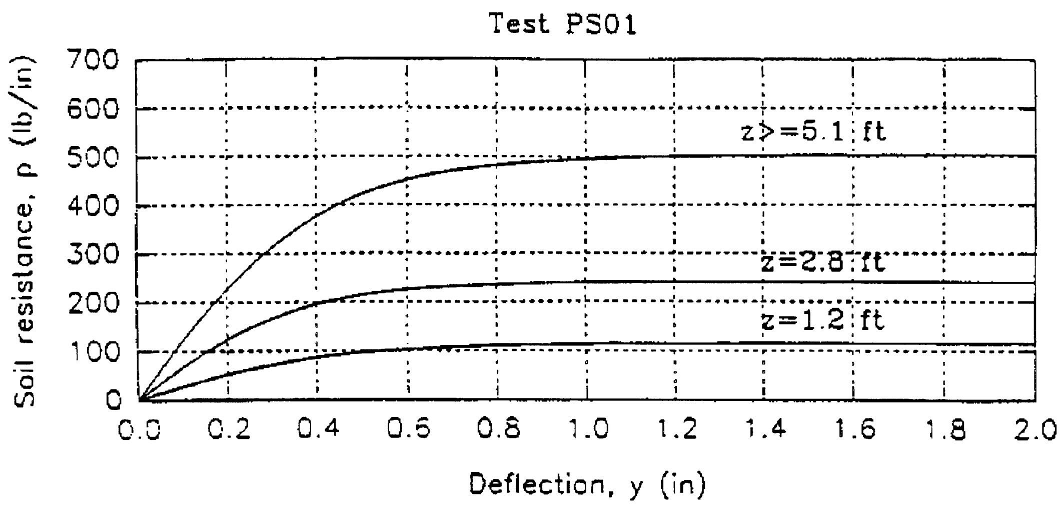
strength is appropriate for fine sands. Other soil types that are less prone to liquefaction are expected to have higher residual strength values."

Based on the preliminary results from the centrifuge tests involving a single pile cited by ATC-32, Liu and Dobry (1994) developed a dimensionless degradation parameter C_u, which is a multiplier to reduce the load p of p-y curves developed for piles in sand under static conditions (similar to those presented in Figure 3.6) to account for excess pore water pressure effects. A plot of the data for the degradation parameter C_u versus excess pore water pressure ratio, r_u (where r_u is the ratio of the excess pore water pressure to initial effective vertical stress), is presented in Figure 3.7. As seen from the plot, C_u can be lower than 0.1 for high r_u values, meaning that the residual p-y resistance can be less than 10 percent of the initial resistance.

The effect of excess pore water pressures on the p-y curves for single piles in fine sand has also been studied by Wilson et al. (1998) using centrifuge model tests. They note the multiplier used to reduce the load p of static p-y curves for a single pile to account for liquefaction varies with relative density (D_r). Their results indicate the multiplier may be about 0.1 to 0.2 for fine sand at a relative density of about 35 percent and 0.25 to 0.35 for a relative density of approximately 55 percent. Wilson et al. (1998) caution that the p-y resistance can be expected to progressively soften during shaking as the excess pore water pressures progressively increase.

The Japanese Code of Highway Design stipulates that the conventional spring constants used in the lateral load analysis of piles involving no softening due to liquefaction be reduced by a factor of 1/6 to 2/3 for analyzing piles when liquefaction is a concern (Ishihara and Cubrinovski, 1998).

The free-field ground response analysis for the portion of the lateral loading evaluation of the piles involving ground deformations can be made using a total stress ground response analysis, such as that performed with the computer program SHAKE (Schnabel et al., 1972), or an effective stress analysis using a program such as SUMDES (Li et al., 1992). It is important to note that a ground response analysis done using a total stress approach will not inherently incorporate the



Note: z indicates depth below ground surface in prototype units

FIGURE 3-6 p-y Curves for Pile in Sand Developed in Centrifuge Test PS01 (from Liu and Dobry, 1994)

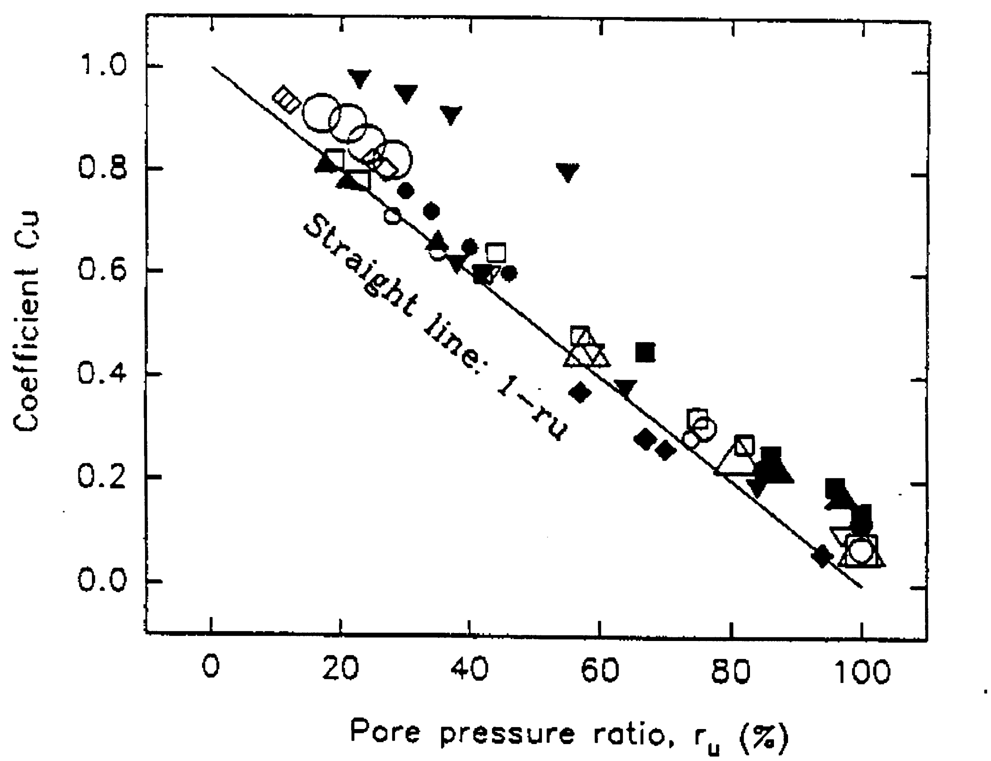


FIGURE 3-7 p-y Curve Degradation Parameter for Pile in Sand having Excess Pore Water Pressures (from Liu and Dobry, 1994)

increase in pore water pressures and decrease in strength during shaking. Therefore, it can be more difficult to select appropriate parameters for the liquefiable soil deposits to use in a total stress analysis than in an effective stress analysis.

For piles and drilled shafts adjacent to embankments, slopes, or retaining walls consideration must also be given to loads imposed on the foundations by deformation of the ground in the down slope direction during and after the earthquake. Where ground deformation at these structures is expected to occur, the piles must be designed to adequately resist the imposed lateral loads.

3.3.2.3 Lateral Loading with Lateral Spreading

Piles/shafts embedded in laterally spreading soils are subjected to lateral loads exerted by both the bridge and the laterally spreading soil. Methods for assessing the performance of piles subjected to lateral spreading have been developed by O'Rourke et al. (1994), as well as Ishihara and Cubrinovski (1998). Both methods use beam and spring elements to model the pile and soil-pile interaction, respectively. Ishihara and Cubrinovski (1998) state that for the case of lateral spreading around a pile where the pile displacement is approximately 50 percent of the soil displacement, the stiffness of the conventional spring constant used to model the soil-pile interaction in their method is on the order of 1/1000 to 1/100 of the value used for evaluating the pile under lateral loading in soil involving no softening due to liquefaction. Comparison of the results from these methods to the actual performance of piles subjected to lateral spreading for a limited number of cases indicates the results obtained are reasonable. However, further confirmation of the methods is still needed.

The loads imposed on piles by laterally spreading soil may be particularly high when there is a non-liquefiable layer overlying the liquefiable soil (Wilson et al., 1998).

3.3.2.4 Complex Methods

In cases where more complete evaluations of the combined axial and lateral loading behavior of deep foundations are needed, finite element or finite difference based computer programs can be used in which the piles/shafts can be modeled as structural elements. Use of such programs requires that an appropriate constitutive model for the soil be input to represent the soil behavior, including progressive liquefaction. Potential programs that can be used include the finite-element based program PAR (PBM Engineering, Inc.) or the finite difference program FLAC. However, there are some potential difficulties with using the program PAR, as explained by Wilson et al. (1998), or the partially-coupled solution provided with FLAC, as described in Section 3.3.1.3.

3.4 Performance Criteria

3.4.1 Stability Safety Factors

The factor of safety against bearing failure is the main criterion for evaluating whether a footing or slope at a bridge is stable based on a pseudostatic analysis for earthquake loading and static analysis for post-earthquake loading. Specific recommendations for appropriate factors of safety for the earthquake and post-earthquake loading cases are not given in codes such as the 1996 Standard Specifications for Highway Bridges by AASHTO (1996) or Improved Seismic Design Criteria for California Bridges: Provisional Recommendations, ATC 32 by the Applied Technology Council (1996). Typically a factor of safety somewhere between 1 to 1.5 would be adopted depending on the conservativeness of the assumptions and parameters used in the analyses. However, the factor of safety for stability during earthquake loading provides only one indicator of performance. Deformations should be estimated to make a final evaluation.

3.4.2 Deformation Limits

Deformations of a bridge foundation and structure ultimately govern its performance for seismic and post-seismic loading conditions. Therefore, evaluation of bridge performance requires that some type of deformation criteria be established. Typically deformation criteria are expressed in terms of either distortion, δ/l, or gross horizontal and vertical movements of the bridge. Distortion, δ/l, is defined as the ratio of the differential settlement, δ, between bridge supports to the distance between the supports, l.

Table 3-2 summarizes some deformation criteria for bridges cited in the literature. The criteria presented, with the exception of that by Youd (1998), are for the bridge itself. For the case of a bridge on shallow foundations, the deformation of the bridge can conservatively be estimated to be the same as that of the supporting ground at foundation level.

As seen from Table 3-2 the deformation limits are primarily for maintaining serviceability of the bridge. It is important to note that when using these limits, with the exception of those given by Youd (1998), all post-construction movements of the bridge must be included in the deformation estimate. Therefore, when evaluating the serviceability for earthquake and post-earthquake conditions using these limits, the deformation estimates must include any post-construction settlements prior to the earthquake, as well as the earthquake-induced deformations. The limits presented by Youd (1998) appear to apply to seismic and post-seismic induced deformations only, and mark the boundary between displacements that can be tolerated without serious damage or loss of function and those that can potentially cause significant damage.

Larger deformations than those given in Table 3-2 can be tolerated if the design objective is either to (1) maintain the bridge in a repairable but not an immediately serviceable state following the earthquake, or (2) prevent collapse of the bridge but not maintain repairability. For these cases, the tolerable amount of ground and foundation deformation becomes increasingly a function of the particular structural details of the bridge.

3.4.3 Recommendations

The deformation limits presented in Table 3-2 are recommended as a starting point for setting deformation criteria for maintaining bridges in a serviceable state. However, criteria should be

TABLE 3-2 Distortion and Movement Criteria for Bridges

Authors	Basis of Assessment	Expected Bridge Performance	Findings		Comments
			Distortion Criteria, δ/l	Gross Movement Criteria	
Duncan and Tan (1991)	Literature review	Serviceability limit state	Simple span: 0.008 Continuous span: 0.004	Total settlement: 50 mm - not harmful 100 mm - somewhat harmful Horizontal movement: 38 mm	Distortion and horizontal movement criteria of Moulton et al. (1985) adopted except simple span value, which was found to be overconservative. State that distortion criteria preferred over gross movement criteria.
Lok and Mitchell (1992)	Literature review	Serviceability limit state	Simple span: 0.005 Continuous span: 0.004	Vertical: 50 to 100 mm Horizontal: 38 mm - without vertical movement 25 mm - with vertical movement	Distortion and movement criteria from Moulton (1985). State that displacements which bridge can tolerate during and after a seismic event is affected by bearing lengths of support spans and any end restraint mechanisms used.
AASHTO (1996)	Moulton et al. (1985)	Serviceability limit state	Simple span: 0.005 Continuous span: 0.004	Horizontal: 38 mm - with small vertical movement 25 mm - with vertical movement	Moulton et al.'s (1985) distortion and gross horizontal movement criteria have been adopted.
Youd (1998)	Review of case histories	Upper limits of serviceability limit state	-	Ground settlement: 25 mm - shallow found 100 mm - deep found. Lateral ground displacement: 100 mm	Settlement of deep foundation itself, as opposed to surrounding ground, should be 25 mm or less. Settlement limits apply when lateral displacement less than 100 mm.

Notes: 1. These criteria do not apply to rigid frame structures which AASHTO (1996) states " shall be designed for anticipated differential settlements based on the results of special analyses."

established by the bridge engineer on a case by case basis, particularly where the interest is only in maintaining repairability of the bridge or preventing collapse. The deformation limits are critical criteria in the method presented in Section 4 for assessing whether liquefaction remediation is necessary at particular bridge sites.

SECTION 4
ASSESSING NEED FOR LIQUEFACTION REMEDIATION

4.1 Introduction

An inherent step in the seismic evaluation of an existing bridge is to assess whether the bridge is susceptible to liquefaction-induced damage. Based on the risk for liquefaction-induced damage the need for remediating the supporting soils and foundations can be established.

A comprehensive stepwise procedure is presented in this section for evaluating the risk of liquefaction-induced damage to bridges and the need for remedial measures. The essence of the procedure is to first use simple evaluation methods to assess the potential for liquefaction. If conditions are conducive to liquefaction, then the procedure progresses to more advanced and detailed analytical methods for evaluating ground stability and deformations, using the procedures discussed in Section 3, until it is established whether ground or foundation improvement are necessary. This procedure has been established in flow chart form to provide a logical progression to the evaluation process, including specific points of information gathering, analyses, and decisions. The charts are presented at the end of this section and explained below.

4.2 Flow Charts

4.2.1 Overview of Charts

Two sets of flow charts have been developed: one set for shallow foundations or deep foundations founded above or partially in liquefiable soils (hereafter referred to as shallow foundation charts) and the second set for deep foundations extending completely through the liquefiable strata (hereafter referred to as deep foundation charts). The general organization of the two sets of charts is similar, as described below, and there are some similarities between the recommended evaluation procedures. However, since there are different factors of concern for shallow and deep foundations, two separate sets of charts were developed. The two sets of charts are independent of each other, with the exception that the deep foundation charts sometimes reference the shallow foundation charts for more details concerning particular analytical procedures or parameter determination methods.

4.2.2 Chart Organization and Types

The general organization of both sets of flow charts for shallow and deep foundations is given below.

- Procedure Summary - This chart outlines the sequence of major evaluation steps to be carried out, starting with the simplest and proceeding progressively to the more complex.

- Specific Evaluations - These charts, which immediately follow the summary chart, provide specific details for each major evaluation step to be carried out. One flow chart is provided for each major evaluation step indicated in the procedure summary. They are arranged sequentially in the same order as they appear in the summary. The user progresses through these charts until a decision is reached that liquefaction remediation is or is not needed.

- Parameter Determination - These charts present methods for determining the parameters needed in the specific evaluations. The parameter determination charts are presented after the charts for specific evaluations.

Flow charts for both shallow and deep foundations are presented at the end of this section as Figures 4-1 to 4-21. Table 4-1, which appears at the front of the flow charts (along with Tables 4-2 and 4-3) indicates the type of chart appearing in each figure, as well as the interrelationship between the figures. More detailed explanations of the information appearing on the different chart types is provided below.

4.2.2.1 Procedure Summary

The procedure summary chart lists the sequence of major evaluation steps to be carried out to determine if liquefaction and liquefaction-induced damage is likely to occur and if ground improvement or foundation retrofitting is necessary. The user starts at the top with the simplest evaluation and progressively works downward to more complex evaluations until it is determined whether remediation is needed. Once the necessity for remediation has been established at a point in the evaluation procedure, there is no need to continue with the remaining (more complex) steps.

4.2.2.2 Specific Evaluations

The main issue addressed in each specific evaluation chart is presented in an ovular enclosure. Where appropriate the issue is divided into relevant sub-issues presented in rectangular boxes immediately beneath the main issue. The factors which must be evaluated for each issue, followed by the parameters needed and analyses to be performed, are all presented sequentially downward in a series of rectangular boxes. Results from the analyses are taken into diamond-shaped decision blocks where they are used to answer relevant "yes" or "no" questions. With the exception of the last evaluation flow chart, "yes" responses to the decisions result in moving to the next "more complex" evaluation. "No" responses result in the conclusion that either ground/foundation improvement is or is not required.

4.2.2.3 Determination of Parameters

At the top of the charts for determination of parameters, the factors from the relevant specific evaluation chart are presented in boxes. The relevant parameters needed in analyses for those factors followed by the recommended methods for determining those parameters are presented sequentially downward in rectangular boxes. Once the parameters have been obtained they are then used in the analyses specified for that particular evaluation.

The soil properties and parameters needed for evaluation of the risk of liquefaction-induced damage to bridges and the need for remedial measures include:

- index properties - plasticity and grain size for soil classification;
- soil state parameters - unit weight, void ratio, relative density;
- stress state parameters - vertical and horizontal confining stresses, pore water pressures, and shear stresses;
- liquefaction resistance - cyclic resistance ratio (CRR);
- strength properties - effective friction angle, undrained steady state (residual) shear strength;
- stiffness properties - shear modulus, damping; and
- recompression properties - volumetric strain.

Table 4-2 (located in front of the flow charts at the end of this section) and the parameter determination flow charts indicate in-situ and laboratory tests that can be used to obtain these properties and parameters, as well as appropriate correlations needed to obtain the properties from in-situ test results. If one of the methods for obtaining a property or parameter is preferred, based on the authors' experience, that method is highlighted in the table.

In-situ tests, particularly the Cone Penetration Test (CPT), are generally preferred for obtaining many of the parameters indicated in Table 4-2. Advantages of using in-situ tests to obtain soil property and state parameters for liquefaction evaluations include: (1) the properties are assessed using measurements made under actual field conditions and (2) disturbance effects are generally less than those associated with obtaining soil samples for laboratory testing. Limitations of in-situ testing include: (1) the need to use empirical correlations to obtain soil properties rather than direct property measurements and (2) the results may not reflect important influences of soil structure, such as cementation, on soil properties and behavior (particularly true for the SPT).

As seen from Table 4-2, no single in-situ test is recommended for obtaining all of the properties/parameters required in the liquefaction assessment procedure presented in the flow charts. Thus, two or sometimes three types of tests should be used in conjunction with each other when evaluating a bridge site. Generally the CPT provides a greater amount of data and has more reliable correlations than other in-situ methods, provided the soil classification has been verified by direct sampling. Therefore the CPT should be used for liquefaction evaluations, provided site conditions permit. The CPT should be supplemented with the SPT to provide soil samples for visual classification and to check soil classifications derived from CPT data. In-situ shear wave velocity tests (V_s) are used to obtain data for determination of the maximum shear modulus. In soils containing sufficient gravel and cobbles to impede SPT or CPT penetration, Becker Hammer penetration test (BPT) resistance should be used.

Laboratory tests can be used to obtain soil index, density, strength, stiffness, and volume change properties for liquefaction assessments. "Undisturbed" samples are necessary to test for these properties, with the exception of index properties, where disturbed samples can be used. Obtaining high quality "undisturbed" samples of potentially liquefiable materials (using methods such as fixed piston tube sampling or ground freezing) is expensive, time consuming, and often of limited success, because it is very difficult to sample saturated, loose granular soils. In addition, replicating field stress conditions in the laboratory is difficult and any differences between the lab and field conditions can affect the properties measured. Therefore, laboratory tests are generally not recommended for evaluating soil properties to be used in liquefaction analyses, except for (1) soil classification, (2) determination of the undrained shear strength of clays for use in complex numerical analyses of ground behavior, and (3) measurement of the liquefaction resistance and volumetric strain properties of soils for which correlations based on in-situ test results have not been established, such as geologically old sand and cemented sands.

4.2.3 Parameter and Analysis Method Issues

4.2.3.1 Detailed Information

The flow charts for evaluations and parameter determinations contain sufficient information for the user to complete each of the evaluation procedures outlined in the summary chart. References are cited in the charts that provide more specific information on the parameters and analytical procedures. A reference list specifically applicable to the flow charts and Table 4-2 is provided at the end of the charts. In addition, the reference information can be found in the general reference list for this report (Section 9).

More detailed discussion of some of the issues and recommendations pertinent to the parameters and analyses presented in the flow charts can be found in the "Screening Guide for Rapid Assessment of Liquefaction Hazard at Highway Bridge Sites" by Youd (1998). The discussion presented in that screening guide is not duplicated herein. However, Table 4-3 (in front of flow charts at end of this section) lists the parameters and analyses in the flow charts which are discussed in that guide, as well as the section of the guide where the discussion can be found.

4.2.3.2 Applicability to Different Conditions

The analytical procedures suggested in the flow charts were selected for evaluating liquefaction potential, stability, and deformations for site and subsurface conditions commonly associated with liquefiable soil deposits at bridges in low-lying areas where fluvial deposits are present. When conditions at the bridge site differ from those for which a particular analytical method was developed, its applicability should be assessed by the engineer. Where the method is not applicable, alternative methods that are more appropriate to those conditions should be used.

4.2.3.3 Reliability of Results

When making an assessment of the potential for liquefaction-induced damage and the need for remediation, the reliability of the analytical results should be considered. In general, the limits set for factors of safety and deformation in the decision blocks of the specific evaluation flow charts have been chosen to reflect the reliability of the analytical procedures themselves. The limits have been chosen to be conservative, in general, such that the outcome in the evaluation of a marginal site will be for remediation more often than not.

The reliability and conservatism of the analytical results are not only influenced by the analytical methods themselves, but also by the reliability and conservatism of the parameters used in the analyses. The factor of safety and deformation limits have been established assuming "average" values will be selected for parameters using available information of "average" to "high" reliability. If a decision is made to consistently use "more conservative" or "less conservative" parameter values, then the impact on the prediction of the need for liquefaction remediation should be understood and acceptable. Likewise, if the reliability of information is lower than average, then the limits used in making decisions should be adjusted accordingly.

4.3 Relationship to Overall Retrofit Evaluation

Evaluating the need for liquefaction remediation at a bridge site is only one aspect of establishing the need for seismic retrofit of an existing highway bridge. The assessment of liquefaction potential and need for remediation should therefore fit into an overall retrofit assessment procedure similar to the one presented in the "Seismic Retrofitting Manual for Highway Bridges" (Buckle and Friedland, 1995).

The structure of the liquefaction assessment procedure presented in the flow charts herein fits well into an overall seismic retrofit assessment procedure. For instance, the first step in the procedure summary (Figures 4-1 and 4-15), consisting of a preliminary evaluation of liquefaction potential, could be used to evaluate the liquefaction vulnerability rating used in the preliminary screening (for future detailed evaluation) of bridges suggested by Buckle and Friedland (1995). The later steps presented in the procedure summary can be part of a more detailed evaluation of the need for bridge seismic retrofit.

4.4 Two-Level Design Approach

One of the recommendations of the ATC-18 Project (Rojahn et al., 1997), which reviewed domestic and foreign seismic design practice, is the adoption of a two-level approach to seismic design, particularly for "important" structures. The essence of the two-level approach is to design a structure to resist both a "less severe" seismic event with minimal damage and a "more severe" seismic event with increased, but acceptable, levels of damage.

The liquefaction assessment procedure presented herein is readily adaptable to such a two-level design approach. The procedure can be performed for two different design earthquakes corresponding to two different design levels. Likewise, two sets of limits can be established for the stability safety factors and deformations used in evaluating the need for liquefaction remediation. One set of limits would be used with the "less severe" seismic design event and the other with the "more severe" design event. If liquefaction remediation is determined to be necessary for either event, then feasible remediation methods can be selected and designed as discussed in Sections 5 and 6, respectively.

4.5 Assessment Example

The flow charts can be used in the manner outlined below to establish if liquefaction remediation measures are necessary at a bridge supported on shallow foundations.

1. A preliminary evaluation of the site and earthquake conditions is performed per Figure 4-2 to see if a liquefaction hazard exists at the bridge site. The parameters necessary in the evaluation are obtained from Figures 4-6, 4-7, and 4-8. If the evaluation suggests a risk of liquefaction at the site based on the subsurface and earthquake conditions, proceed to Figure 4-3 for an evaluation of liquefaction potential.

2. Figure 4-3 is used to determine if the site is susceptible to liquefaction based on computation of the factor of safety against liquefaction. Parameters needed for the evaluation are obtained using Figure 4-9. In addition, densification settlements of the soil deposits are computed. If the factor of safety or densification settlements indicate a potential for damage, then proceed to gross estimates of stability and deformations using Figure 4-4.

3. Gross estimates of stability and deformations are determined using the methods recommended in Figure 4-4. Parameters needed in the analyses are determined using the methods listed in Figure 4-10. If the computed lateral deformations or settlements clearly indicate that liquefaction remediation is or is not needed, then the evaluation stops here. Otherwise, proceed to the refined estimates of deformation given in Figure 4-5.

4. Refined deformation estimates are generally obtained using numerical modeling methods. The parameters typically needed in these analyses are determined using the methods given in Figures 4-11 to 4-14. At the completion of the analyses the lateral deformations and settlements are reviewed to see if liquefaction remediation is required.

For some bridges, the engineer may decide not to use the more complex evaluation steps, such as refined deformation estimates. If the results at the end of the evaluation cycle are not conclusive regarding whether remediation is or is not required, it will be necessary to return to some previous point in the evaluation and repeat the procedure using new or improved information, where possible.

TABLE 4-1 Summary Table for Flow Charts to Assess Need for Liquefaction Remediation Measures

Procedure Summary [1]	Specific Evaluations [2]	Parameter Determinations [3,4]
Fig. 4-1 Procedure for **"Shallow" Foundations** Over or In Liquefiable Soils	Fig. 4-2 Site and Earthquake Conditions	Fig. 4-6 Soil Classification and Experience Fig. 4-7 Boundary Conditions Fig. 4-8 Earthquake Characteristics
	Fig. 4-3 Liquefaction Potential	Fig. 4-9 Earthquake Loading and Liquefaction Resistance
	Fig. 4-4 Gross Stability and Deformation Estimates	Fig. 4-10
	Fig. 4-5 Refined Deformation Estimates	Fig. 4-11 Earthquake Loading and Soil State Fig. 4-12 Stress State Fig. 4-13 Strength Properties Fig. 4-14 Stiffness and Recompression Properties
Fig. 4-15 Procedure for **"Deep" Foundations** Extending Through Liquefiable Soils	Fig. 4-16 Site and Earthquake Conditions	Fig. 4-6 Soil Classification and Experience Fig. 4-7 Boundary Conditions Fig. 4-8 Earthquake Characteristics
	Fig. 4-17 Liquefaction Potential	Fig. 4-9 Earthquake Loading and Liquefaction Resistance
	Fig. 4-18 Gross Loading and Deformation Estimates	Fig. 4-20
	Fig. 4-19 Refined Loading and Deformation Estimates	Fig. 4-21

Notes:
1. Procedure summary outlines major evaluation steps to be conducted in assessment.
2. Specific evaluations proceed in order of figures until it is established if liquefaction remediation is or is not required.
3. Recommended field and laboratory tests, along with correlations, to obtain some of the properties and parameters needed in flow charts are given in Table 4-2.
4. Sections of "Screening Guide for Rapid Assessment of Liquefaction Hazard at Highway Bridge Sites" by Youd (1998) which are relevant to parameters or analyses used in flow charts are given in Table 4-3.
5. References for Tables 4.1 to 4.3 and Figures 4-1 to 4-21 are provided in list following Figure 4-21.

TABLE 4-2 Recommended Tests and Correlations for Soil Properties Needed in Liquefaction and Remediation Evaluations

PROPERTY/ PARAMETER[1]	IN-SITU TESTS			LABORATORY TESTS	COMMENTS
	SPT	CPT	V_s		
Classification (all soils)	* from visual observation of samples	Robertson (1990), Olsen (1988) or NCEER (1997)	-	* particle size analysis, Atterberg limit tests	classification of SPT samples used to confirm CPT classification
Unit Weight, Void Ratio and Relative Density	NAVFAC (1982)	refer to Mayne et al. (1995) for list of correlations	-	*measurement from "undisturbed" samples	-
K_o (for σ_h')	-	*Kulhawy & Mayne (1990) chart	-	-	-
Pore Water Pressure	direct measurement of groundwater level	*CPTU measurement	-	-	-
Liquefaction Resistance	NCEER (1997)	NCEER (1997)	NCEER (1997)	cyclic CU direct simple shear or triaxial tests	lab testing typically used for soil types not in charts
Effective Friction Angle	Terzaghi et al. (1995), NAVFAC (1982)	Robertson & Campanella (1983)	-	static CU triaxial tests	-
Steady State Undrained Shear Strength	*Seed & Harder (1990) chart	*convert q_{c1} to equivalent $(N_1)_{60}$ and use SPT	-	static CU triaxial tests	lab testing typically used for soil types not in charts
Shear Modulus[2] (sands and clays)	-	refer to Mayne et al. (1995) for list of correlations	*use $G_{max}=\rho V_s^2$	resonant column, cyclic simple shear or triaxial tests	use degradation curves[3]: sand - Idriss (1990); clay - Vucetic & Dobry (1991)
Volumetric Strain	*Tokimatsu & Seed (1987) or Ishihara & Yoshimine (1991)	*convert q_{c1} to $(N_1)_{60}$ and use SPT correlation	-	reconsolidate cyclic CU simple shear or triaxial test samples	volumetric strain from lab tests used for soil types not in charts

Notes:
1. Properties/parameters shown are for cohesionless, granular soils unless otherwise noted.
2. Damping can be roughly estimated from resonant column tests.
3. Degradation curves may be needed for use with G_{max} for seismic ground response analyses.
4. V_s = shear wave velocity CPTU = piezocone CU = consolidated undrained

* denotes preferred test for obtaining property/parameter when one exists

TABLE 4-3 Sections of Screening Guide by Youd (1998) Relevant to Flow Chart Procedure

Parameter (P) or Analysis (A) in Flow Chart	Flow Chart Figure No. (in this report)	Relevant Section of Screening Guide (in Youd, 1998)
Plasticity (P), Grain size (P)	4-2, 4-16	4.2
Geology (P)	4-2, 4-16	3.2
Past performance and known properties of soil (P)	4-2, 4-16	3.1
Groundwater levels (P)	4-2, 4-16	3.4
Earthquake magnitude and distance (P), maximum ground acceleration (P)	4-2, 4-16	3.3.1
Comparison of acceleration to limiting values for liquefaction (A)	4-2, 4-16	3.3.2
Maximum ground acceleration (P), Earthquake magnitude (P)	4-3, 4-17	3.3.1, 4.3.2.1
Relative density (P), Grain size(P)	4-3, 4-4, 4-17, 4-18	4.3.2.1
Evaluation of CSR, CRR, and factor of safety against liquefaction, F_L (A)	4-3, 4-17	4.3
Earthquake magnitude (P), Cyclic stress ratio induced by earthquake (P)	4-4, 4-18	4.3.2.1
Moment magnitude (P), Distance from site (P), Soil and slope parameters (P)	4-4	5.3.1
Estimate settlements from Tokimatsu and Seed, 1987 (A)	4-4, 4-18	5.4
Estimate lateral deformation using Bartlett and Youd, 1995 (A)	4-4	5.3

PRELIMINARY EVALUATION OF SITE AND EARTHQUAKE CONDITIONS
(refer to Fig. 4-2)

↓

ASSESSMENT OF LIQUEFACTION POTENTIAL
(refer to Fig. 4-3)

↓

GROSS STABILITY AND DEFORMATION ESTIMATES
(refer to Fig. 4-4)

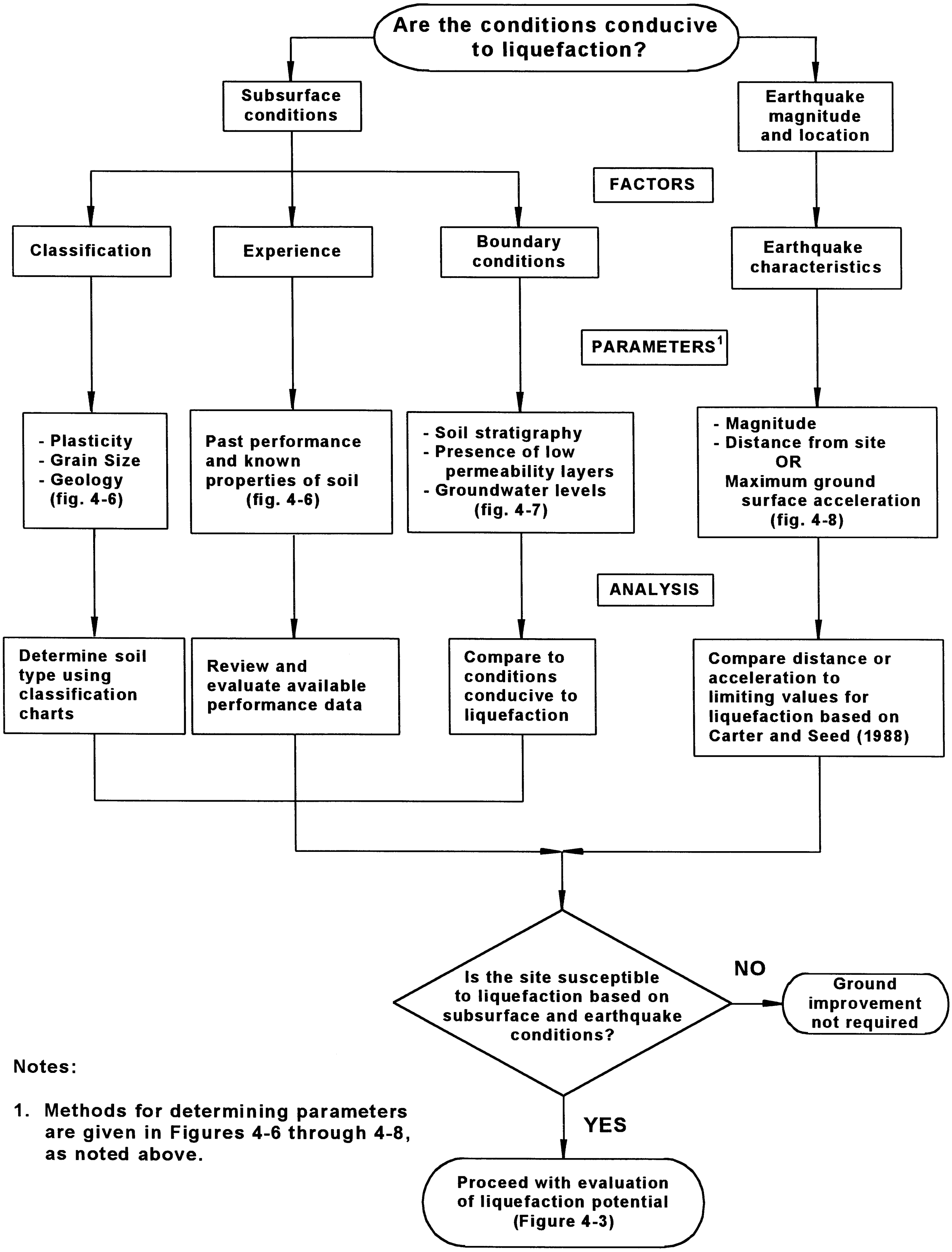
↓

REFINED DEFORMATION ESTIMATES
(refer to Fig. 4-5)

Notes:

1. Details for each evaluation step are presented in Figures 4-2 to 4-5, as noted.
2. This assessment procedure is for evaluation of the need for remediation at sites where soils are fully or marginally liquefiable. Seismic stability and deformations (including settlement) must also be evaluated at sites with non-liquefiable soils, but are not the focus of these flow charts.

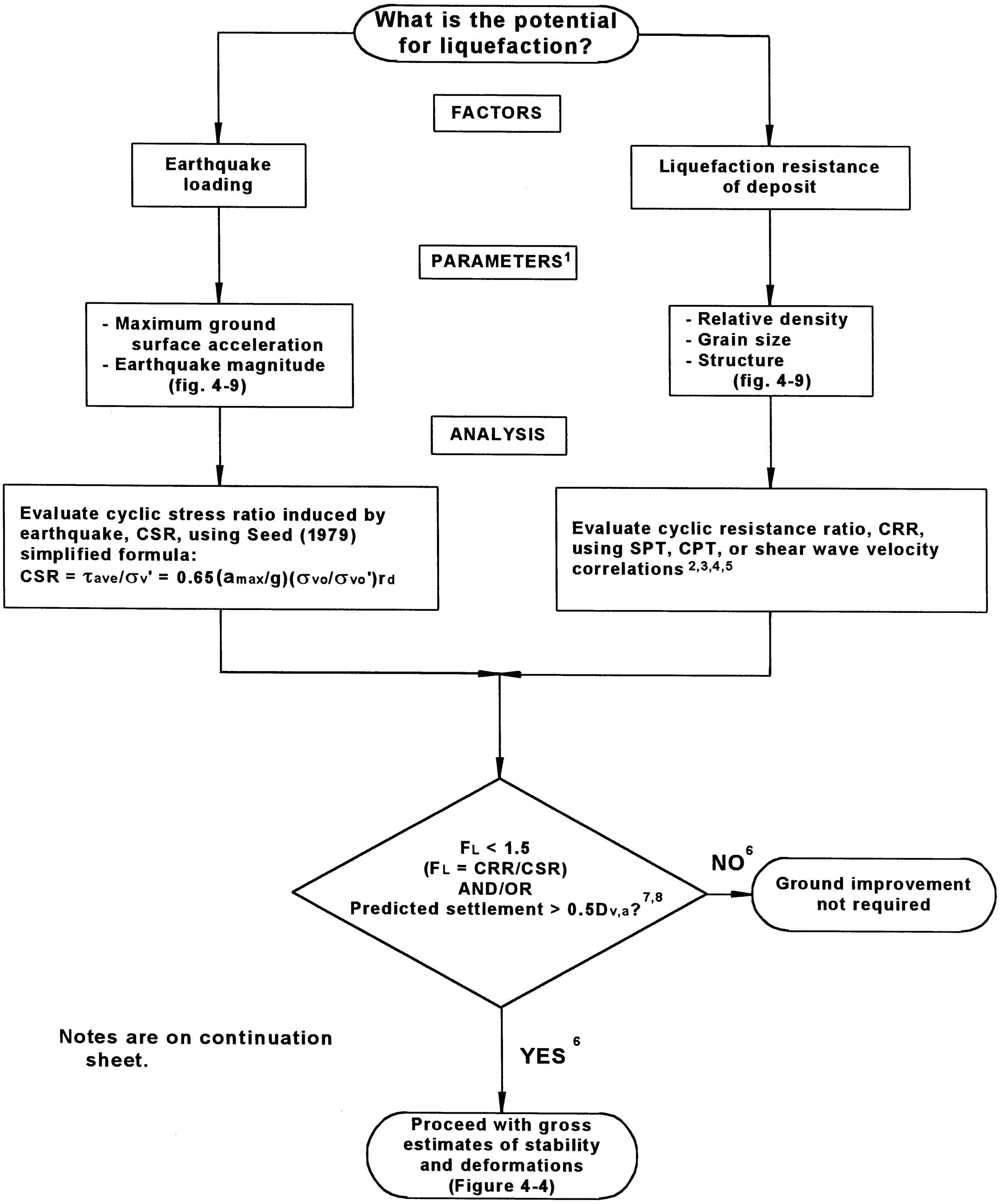
FIGURE 4-1 Procedure to Assess Need for Remedial Measures at Existing Bridges Founded Over or In Liquefiable Soils



Notes:

1. Methods for determining parameters are given in Figures 4-6 through 4-8, as noted above.

FIGURE 4-2 Preliminary Evaluation of Site and Earthquake Conditions



Notes are on continuation sheet.

FIGURE 4-3 Assessment of Liquefaction Potential

Notes:

1. Assessment methods for parameters are given in Figure 4-9, as noted.

2. Evaluation of liquefaction resistance by CPT is generally preferred, provided the results (particularly soil classification) have been confirmed with some SPT data and samples. CPT data is nearly continuous with depth and more reliable. Obtain SPT and CPT correlations with CRR from NCEER (1997) for clean sands.

   Correct SPT and CPT correlations with CRR per NCEER (1997) for: fines content, influence of thin soil layers, earthquake magnitudes different than M = 7.5, vertical effective confining stress using K_σ, and static horizontal shear stress using K_α.

3. Shear wave velocity can be used as a supplemental method to SPT or CPT for evaluating cyclic resistance ratio (CRR) per NCEER (1997).

4. Liquefaction resistance of gravelly soils should be evaluated per NCEER (1997). The Becker Penetration Test (BPT) may be required for soils with high content of gravel and cobbles.

5. If possible, site specific liquefaction potential curves should be developed and used when no liquefaction resistance correlations are available for the soils encountered. These curves can be developed in the laboratory for soils which can be sampled (using specialized methods if necessary and possible) using cyclic CU triaxial or cyclic simple shear tests.

6. "No" response appropriate if it applies to both factor of safety and settlement criteria. "Yes" response appropriate if it applies to either or both criteria.

7. Deposits of cohesionless soils, particularly those which are loose, are susceptible to densification settlement during earthquake shaking. Estimated settlements of these deposits should be calculated using available methods (e.g. Tokimatsu and Seed, 1987, or Ishihara and Yoshimine, 1991) and included in settlement estimates for comparison to acceptable settlement limits.

8. $D_{v,a}$ is the allowable vertical movement (allowable settlement) of the bridge foundation determined by the bridge engineer.

FIGURE 4-3 (continued)

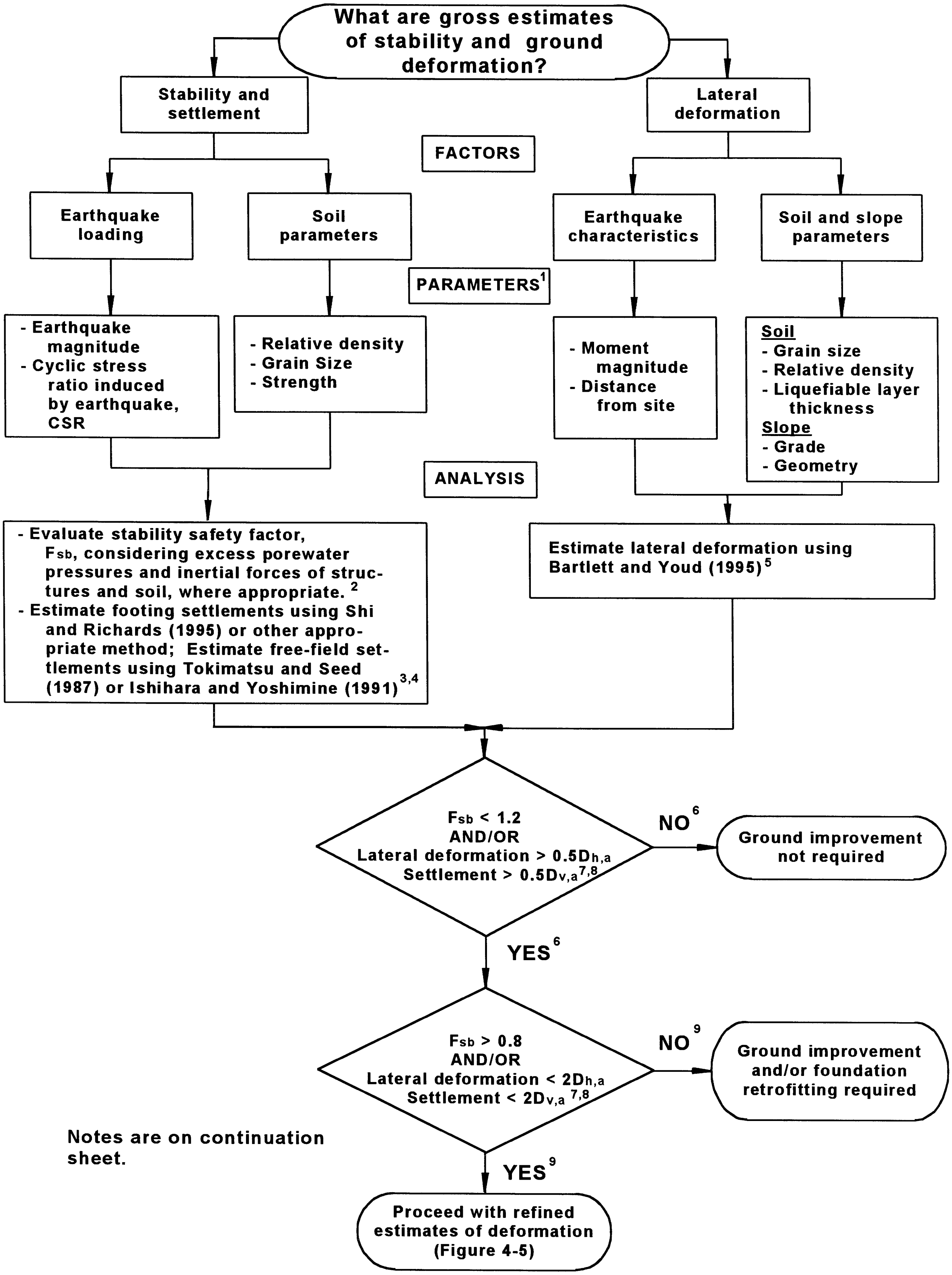


FIGURE 4-4 Gross Stability and Deformation Estimates

Notes:

1. Assessment methods for parameters are given in Figure 4-10.

2. Excess porewater pressures can be estimated per Marcuson and Hynes (1989). For case where soil strata completely liquefies, the standard-of-practice is to analyze stability for flow type failure using static loads and undrained residual strengths for liquefied soils.

3. Shi and Richards (1995) method of estimating seismic footing settlement developed for stable sand deposits. Applicability of method for estimating settlements of footings located over or in fully or partially liquefied soils has not been established and therefore should be done using engineering judgement.

   Ishihara (1995) provides charts summarizing combinations of liquefiable layer thickness and non-liquefiable surface layer thickness observed to cause damage or no damage to structures with different foundation depths.

4. Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1991) procedures were developed for "clean" sands. Munfakh et al. (1998) indicate these procedures will overestimate settlements for sandy silts and silts. For silty sands, Youd (1998) indicates settlement can be estimated for screening purposes using Tokimatsu and Seed's procedure by correcting $(N_1)_{60}$ to an equivalent "clean sand" value using the method described by NCEER (1997).

   For other soil types susceptible to liquefaction, settlements can be estimated using results from cyclic CU triaxial tests on "undisturbed" samples subjected to cyclic stress levels causing liquefaction. Samples are reconsolidated after liquefaction to obtain volumetric strain data. Volumetric strain is then correlated to the factor of safety against liquefaction, F_L, and the relative density/penetration resistance of the soil.

5. For sites not satisfying seismic and site condition limits specified by Bartlett and Youd (1995), lateral deformations can be estimated using Newmark's (1965) method, provided liquefiable layer is relatively thin (Finn, 1998). Appropriately reduced shear strengths should be used along failure surface in liquefied soil.

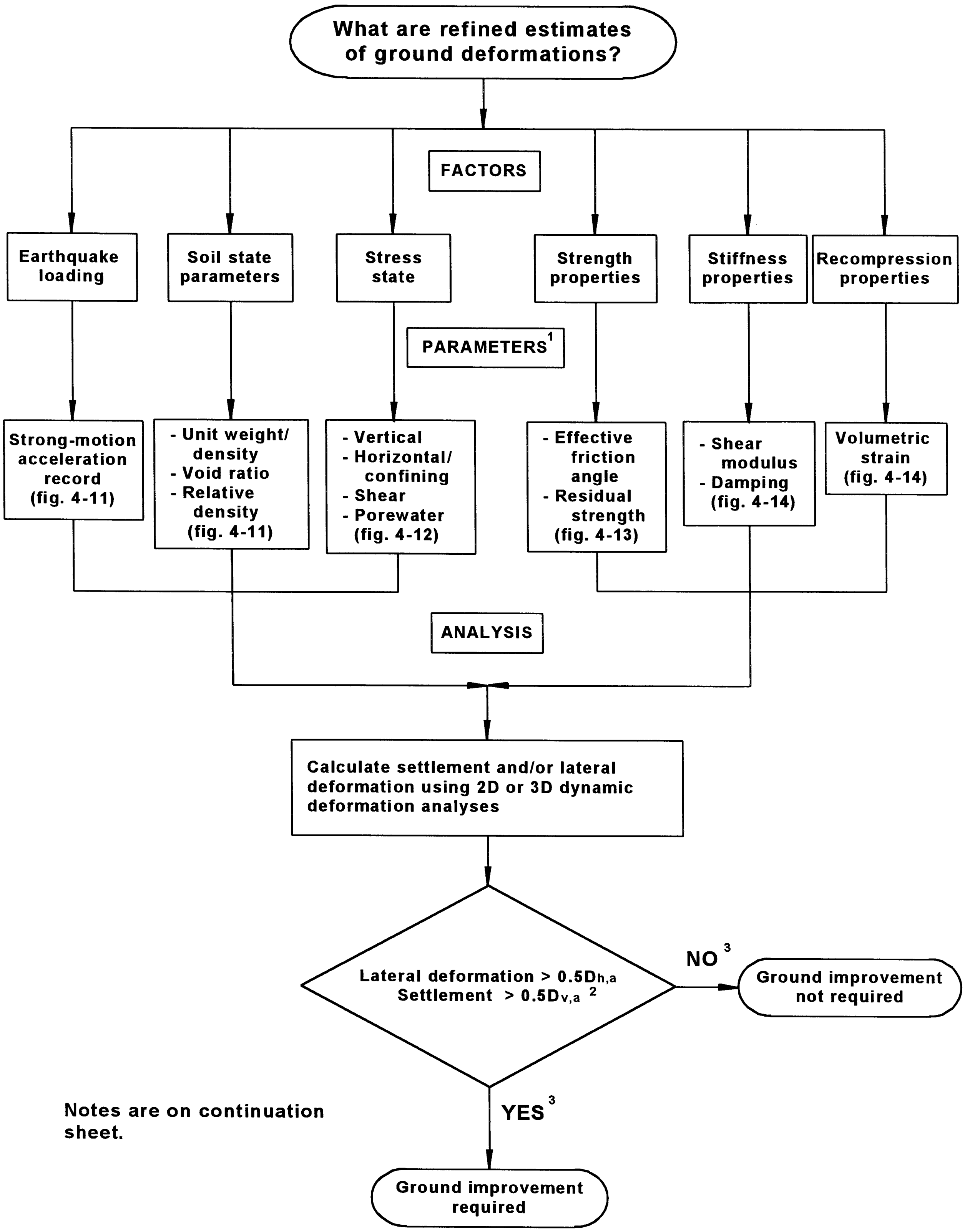
6. "No" response appropriate if it applies to both factor of safety and settlement/lateral deformation criteria. "Yes" response appropriate if it applies to either or both criteria.

7. Estimated settlements should include densification settlements of cohesionless soils above groundwater (per Tokimatsu and Seed, 1987, or Munfakh et al., 1998) and settlements due to deformations from lateral spreading and reduction in bearing capacity, as well as those from dissipation of seismically-induced excess porewater pressures of saturated soils.

8. $D_{v,a}$ and $D_{h,a}$ are the allowable vertical and horizontal movements, respectively, of the bridge foundation as determined by the bridge engineer.

9. "Yes" response appropriate if it applies to both factor of safety and settlement/lateral deformation criteria. "No" response appropriate if it applies to either or both criteria.

FIGURE 4-4 (continued)



Notes are on continuation sheet.

FIGURE 4-5 Refined Deformation Estimates

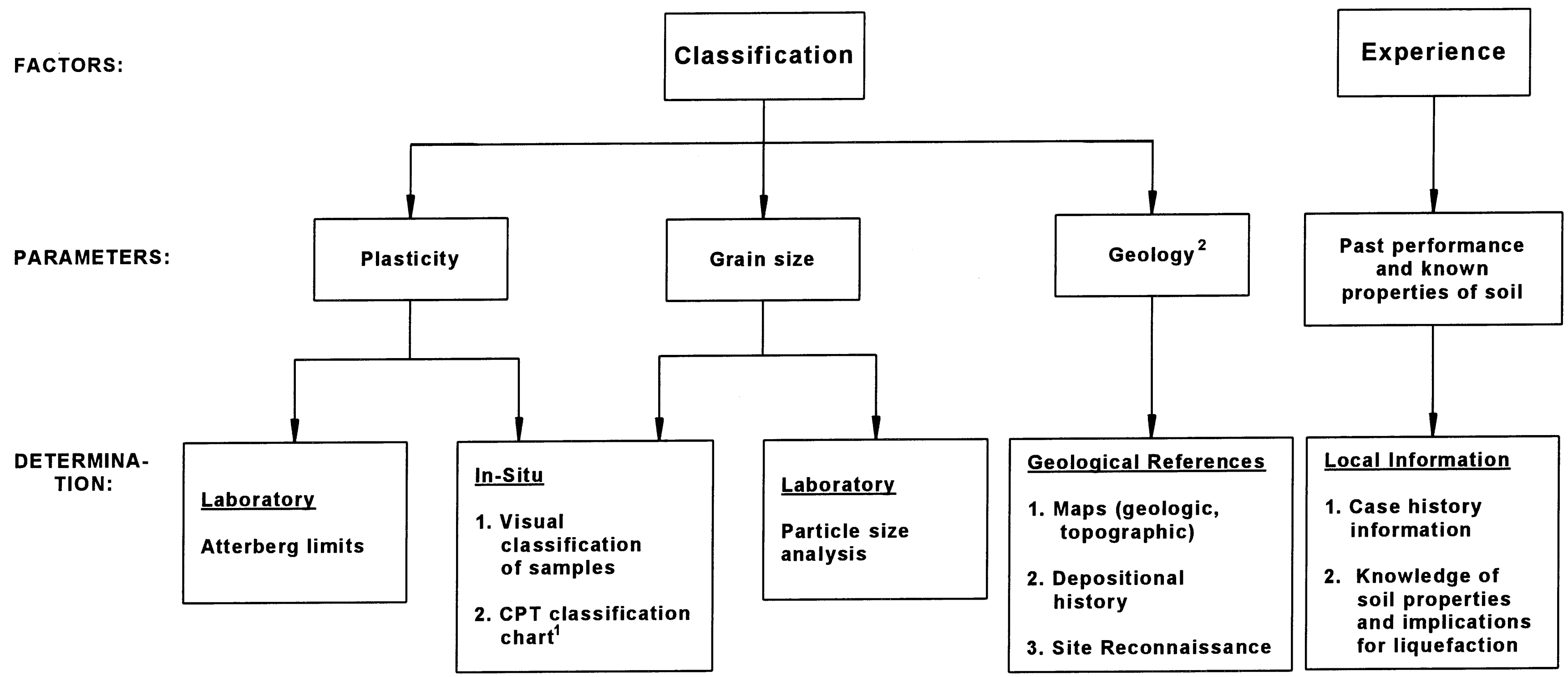
Notes:

1. Assessment methods for parameters are given in Figures 4-11 through 4-14, as noted.

2. Estimated settlements should include densification settlements of cohesionless soils above groundwater (e.g. Tokimatsu and Seed, 1987, or Munfakh et al., 1998) and settlement due to deformations from lateral spreading and reduction in bearing capacity, as well as those from dissipation of seismically-induced excess porewater pressures of saturated soils.

3. "No" response appropriate if it applies to both lateral deformation and settlement. "Yes" response appropriate if it applies to either lateral deformation or settlement, or both.

FIGURE 4-5 (continued)



Notes:

1. Use chart in accordance with NCEER (1997). CPT should only be used for classification after verification of its suitability by soil sampling adjacent to some CPT soundings.

2. Geologic information provides some general insights regarding soil composition, fabric, and structure.

FIGURE 4-6 Determination of Soil Classification and Experience Parameters for Preliminary Evaluation of Site Conditions

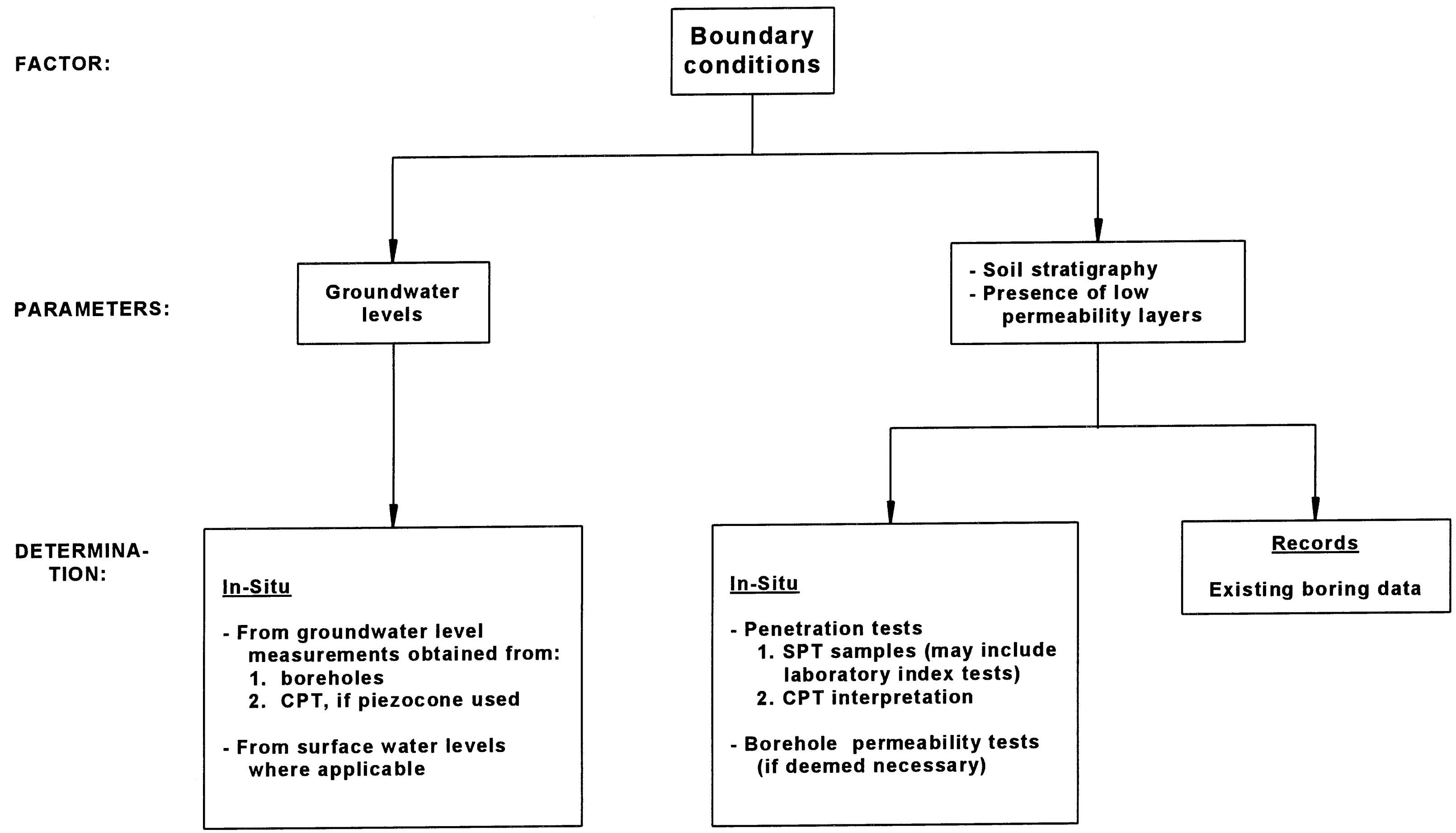
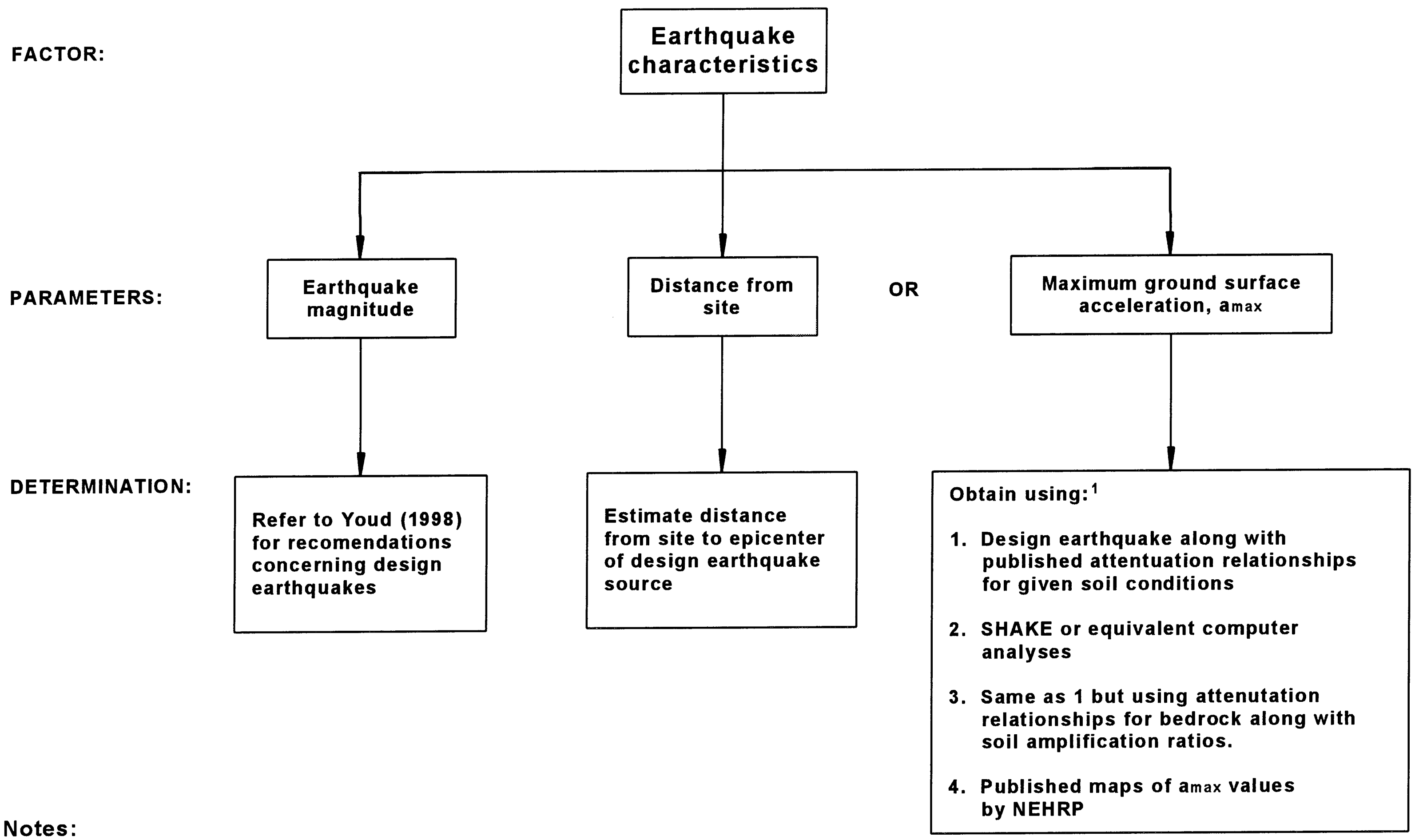


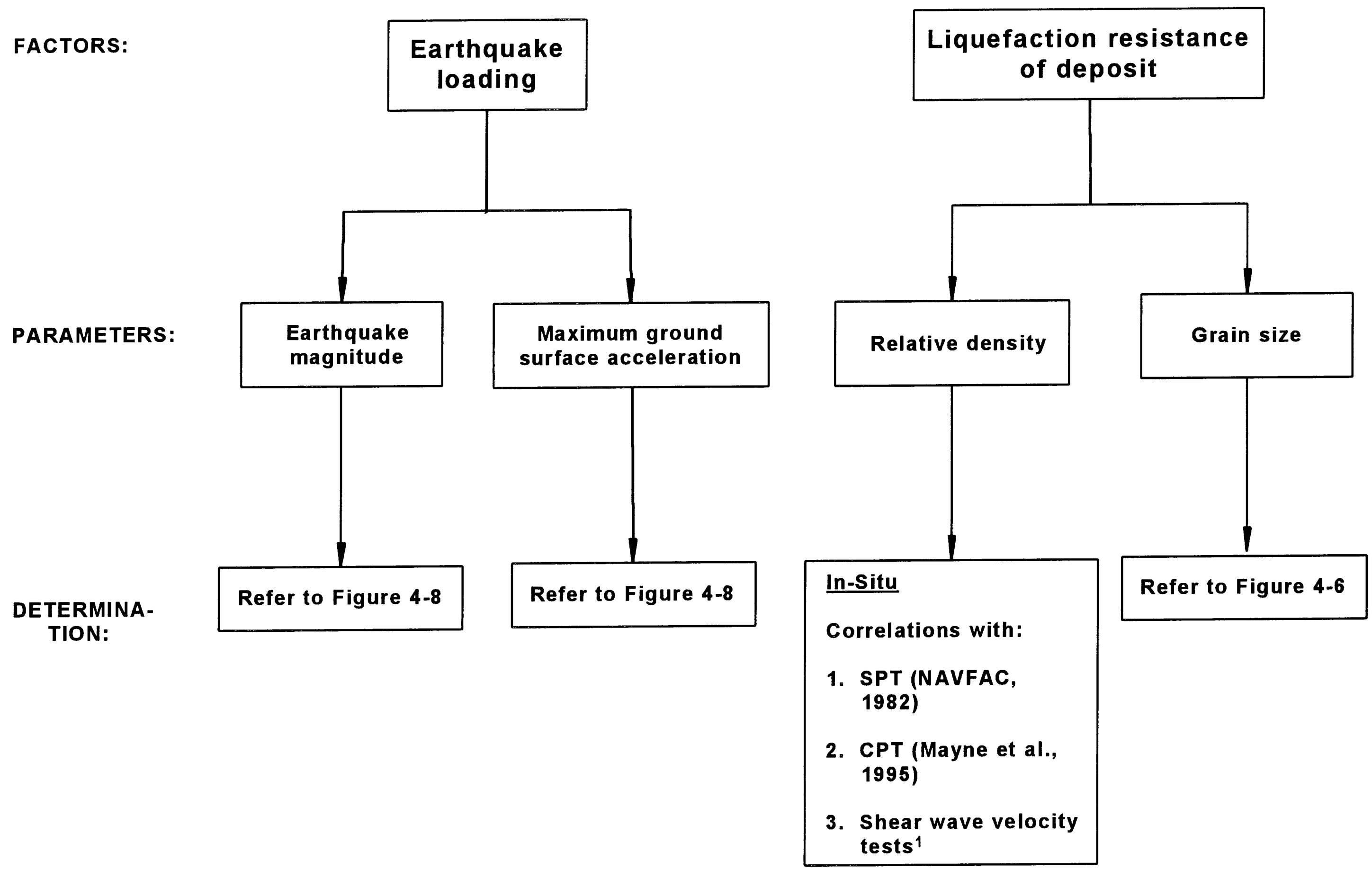
FIGURE 4-7 Determination of Boundary Condition Parameters for Preliminary Evaluation of Site Conditions



Notes:

1. From NCEER (1997): Option 1 is the preferred method when such relationships are available for the given soil conditions. Option 2 should be used if attenuation relationships for given soil conditions (Option 1) are not available. Option 3 is least desirable because of magnitude and potential frequency dependancy of amplification. Option 4 is not discussed.

FIGURE 4-8 Determination of Earthquake Characteristic Parameters for Preliminary Evaluation of Earthquake Conditions



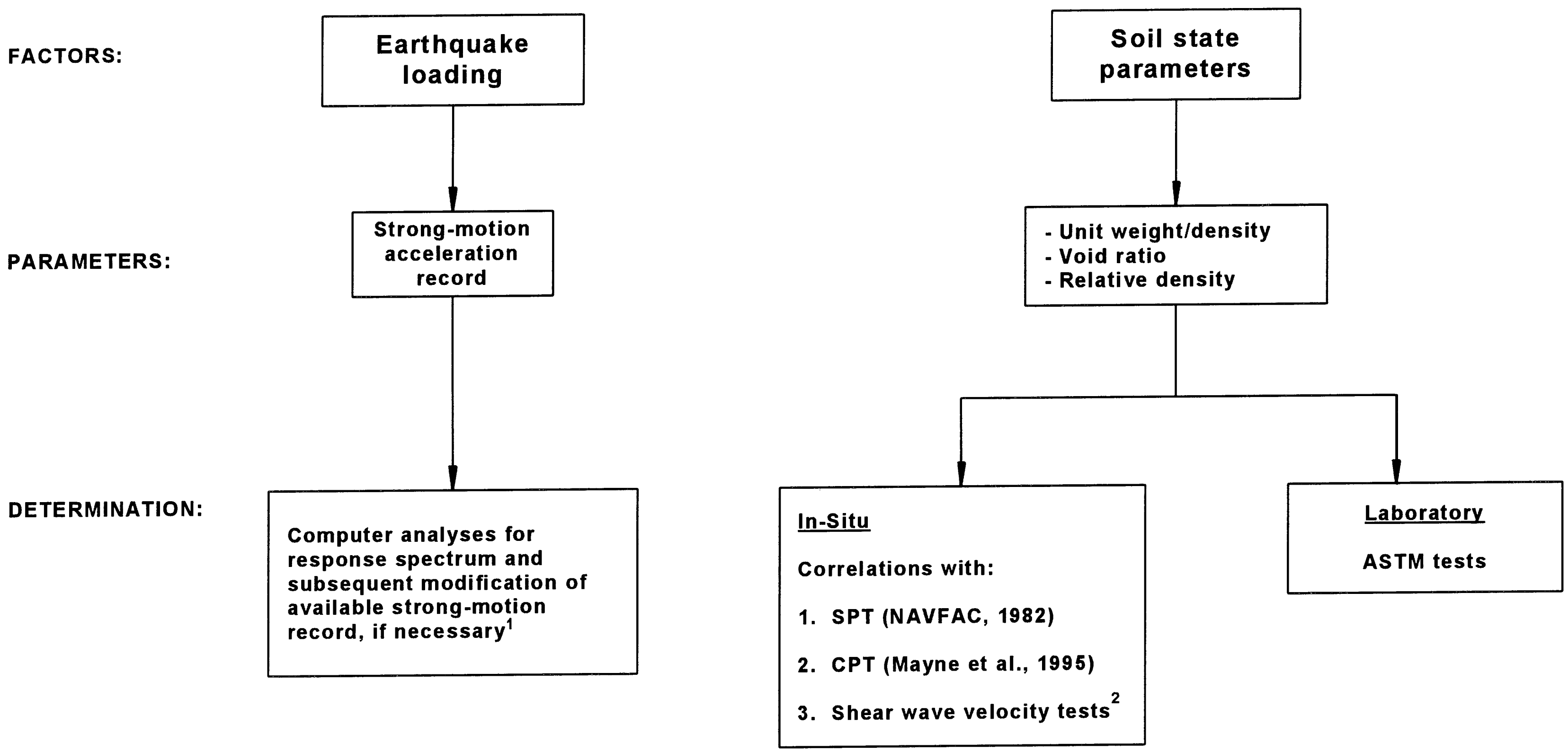
Notes:

1. Correlation of shear wave velocity with relative density is not well established. Therefore, CPT or SPT tests should be used in conjunction with shear wave velocity tests to help evaluate this parameter.

FIGURE 4-9 Determination of Earthquake Loading and Liquefaction Resistance Parameters for Assessment of Liquefaction Potential

Factors	Parameters	Determination
Earthquake loading	Earthquake magnitude	Refer to Figure 4-8
	Cyclic Stress Ratio (CSR)	Refer to Figure 4-8 for a_{max}; CSR = $0.65(a_{max}/g)(\sigma_{vo}/\sigma_{vo}')r_d$
Soil parameters	Relative density	Refer to Figure 4-9
	Grain Size	Refer to Figure 4-6
	Strength	Refer to Figure 4-13
Earthquake characteristics	Moment magnitude	Refer to Figure 4-8
	Distance from site	Refer to Figure 4-8
Soil and slope parameters	Soil	
	Grain size	Refer to Figure 4-6
	Relative density	Refer to Figure 4-9
	Liquefiable layer thickness	Soil borings or CPT soundings
	Slope	
	Grade and geometry	1. Construction plans or project specific survey maps 2. Field reconnaissance 3. Topographic maps

FIGURE 4-10 Determination of Parameters for Gross Stability and Deformation Estimates

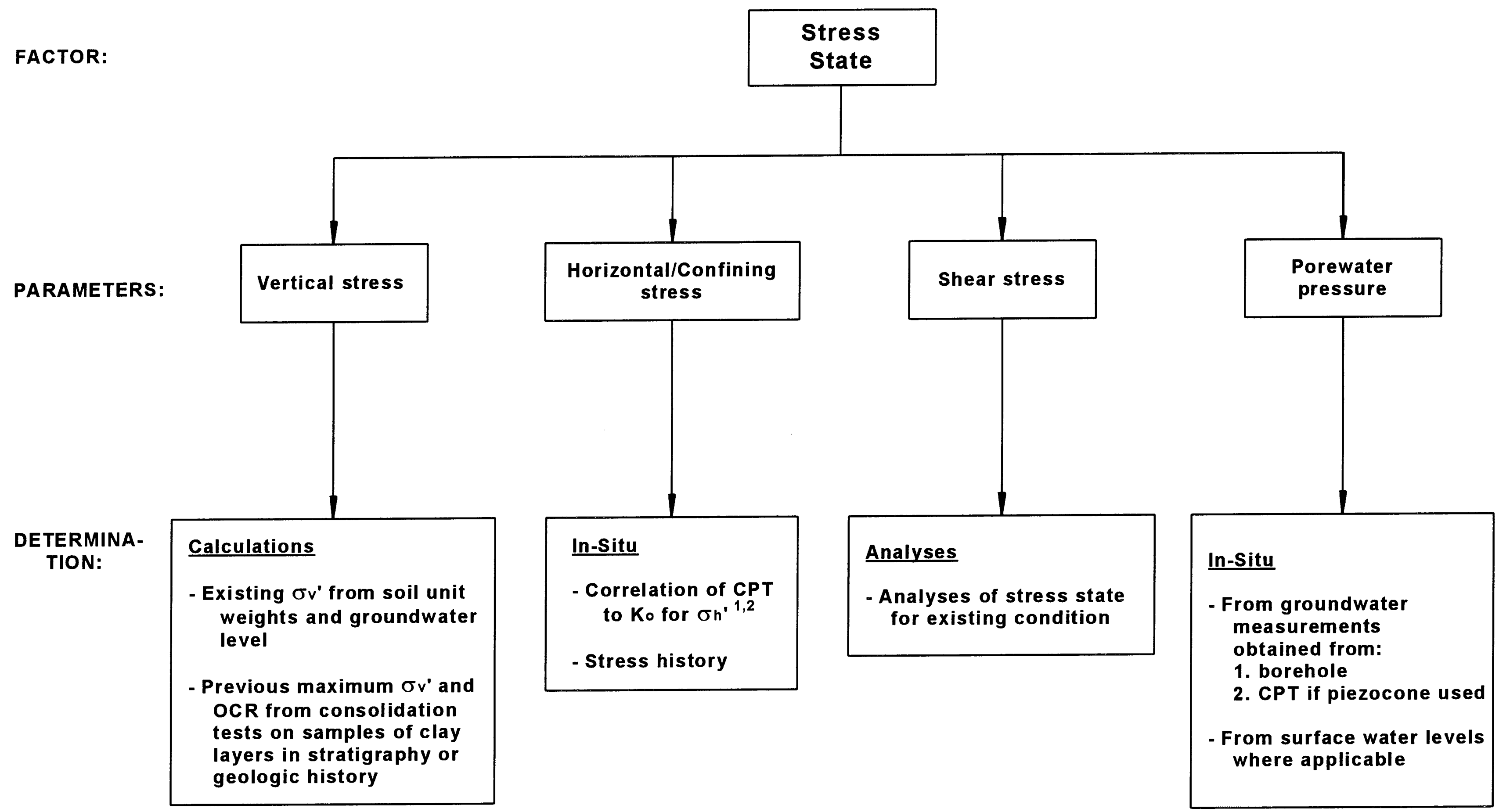


Notes:

1. Computer analyses to determine response spectrum from strong-motion bedrock record can be performed using SHAKE, QUAD4M, or equivalent analysis. Modify strong-motion record as necessary to obtain desired design response spectrum.

2. Correlation of shear wave velocity with void ratio, relative density or unit weight is not well established. Therefore, CPT or SPT tests should be used in conjunction with shear wave velocity tests to help evaluate these parameters.

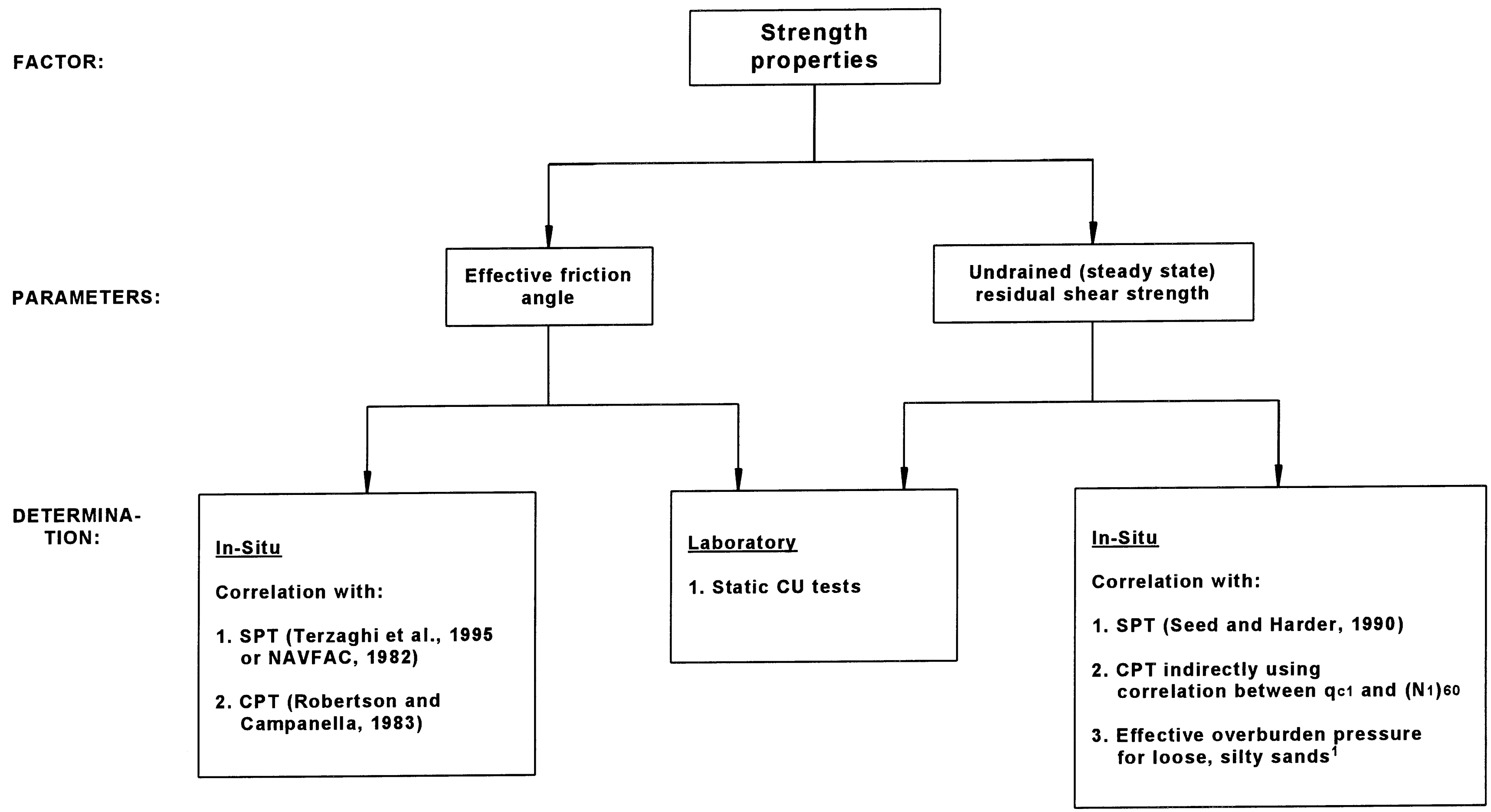
FIGURE 4-11 Determination of Earthquake Loading and Soil State Parameters for Refined Deformation Estimates



Notes:

1. Reference Kulhawy and Mayne (1990)
2. For normally consolidated sand can take $K_o = 1 - \sin \phi'$.

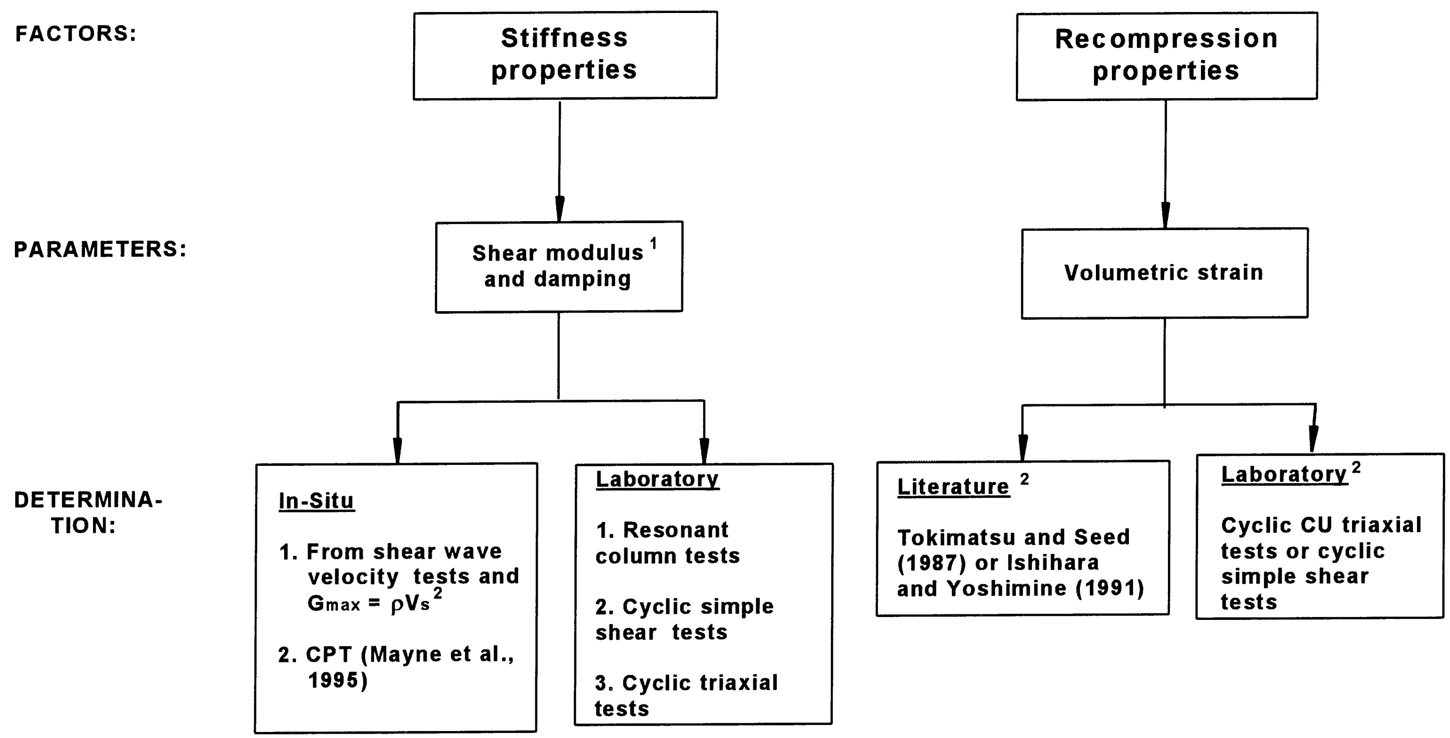
FIGURE 4-12 Determination of Stress State Parameters for Refined Deformation Estimates



Notes:

1. Recommended Su/p ratio per Baziar et al. (1995): $S_u/p \approx 0.145$ or $S_u/p \approx 0.11 + 0.0037(PI)$
   where Su = undrained shear strength, p = effective overburden pressure, and PI = plasticity index

FIGURE 4-13 Determination of Strength Properties for Refined Deformation Estimates

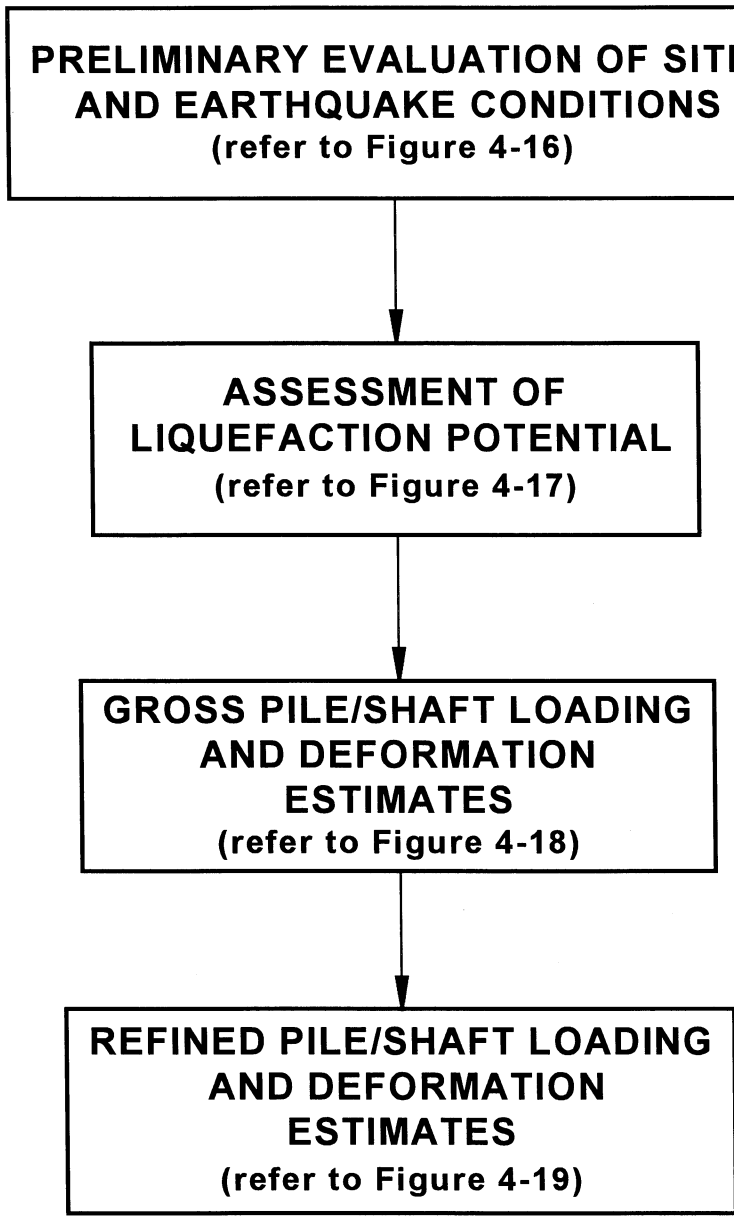


Notes:

1. Variation of shear modulus and damping with shear strain for "clean" sands can be determined using curves by Idriss (1990). Curves by Vucetic and Dobry (1991) can be used for plastic soils.

2. Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1991) procedures were developed for "clean" sands. Munfakh et al. (1998) indicate these procedures will overestimate settlements for sandy silts and silts. For silty sands, Youd (1998) indicates settlement can be estimated for sreening purposes using Tokimatsu and Seed's procedure by correcting $(N_1)_{60}$ to an equivalent "clean sand" value using the method described by NCEER (1997). For other soil types susceptible to liquefaction, volumetric strains can be estimated in the laboratory using results from cyclic CU triaxial tests on "undisturbed" samples subjected to cyclic stress levels causing liquefaction. Samples are reconsolidated after liquefaction to obtain volumetric strain data.

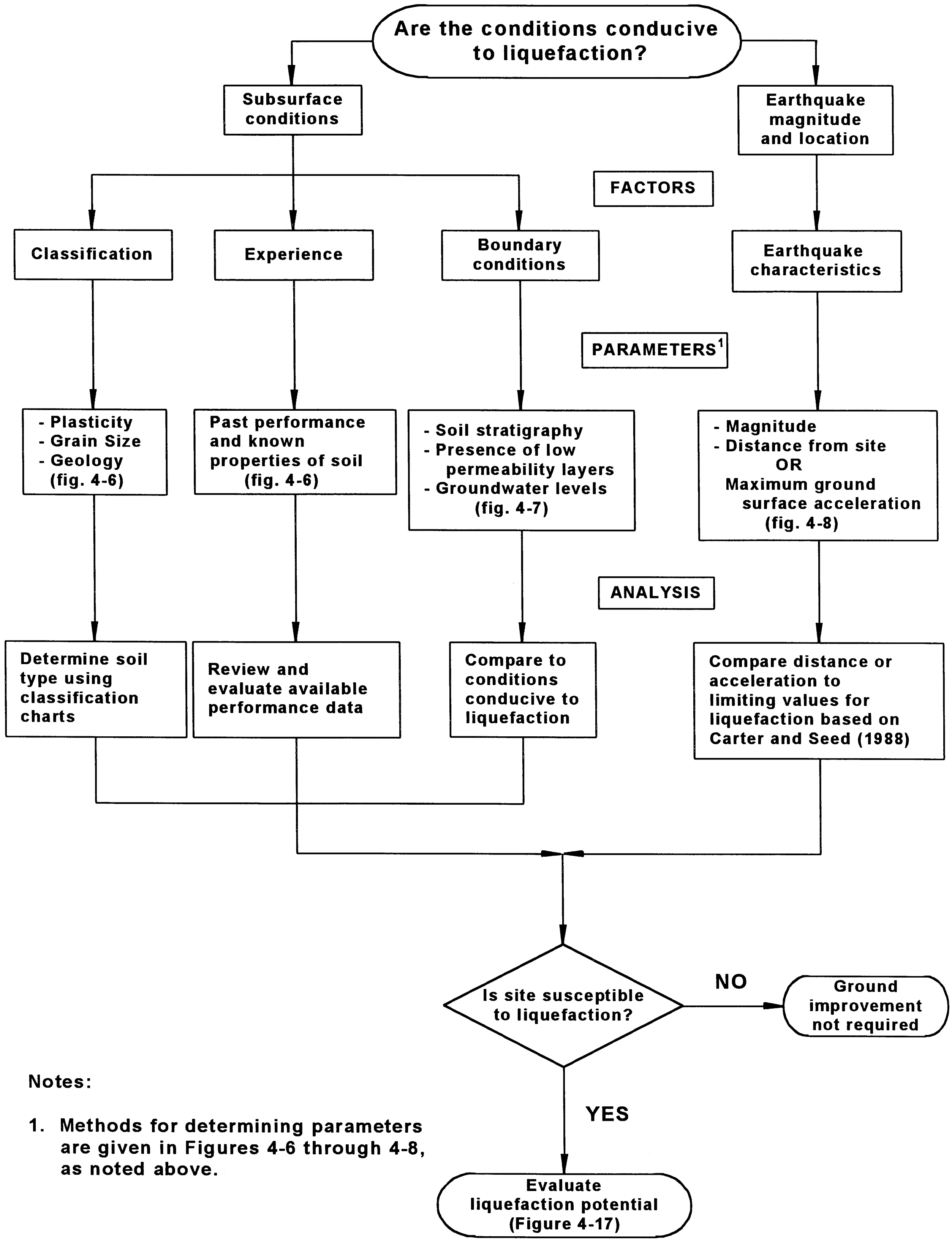
FIGURE 4-14 Determination of Stiffness and Recompression Properties for Refined Deformation Estimates



Notes:

1. Details for each evaluation step are presented in Figures 4-16 to 4-18, as noted.
2. This assessment procedure is for evaluation of the need for remediation at sites where soils are fully or marginally liquefiable. Seismic loading and deformations of piles/shafts must also be evaluated at sites with non-liquefiable soils, but are not the focus of these flow charts.

FIGURE 4-15 Procedure to Assess Need for Remedial Measures at Existing Bridges Founded on Fully-Penetrating Piles/Shafts



Notes:

1. Methods for determining parameters are given in Figures 4-6 through 4-8, as noted above.

FIGURE 4-16 Preliminary Evaluation of Site and Earthquake Conditions for Piles/Shafts

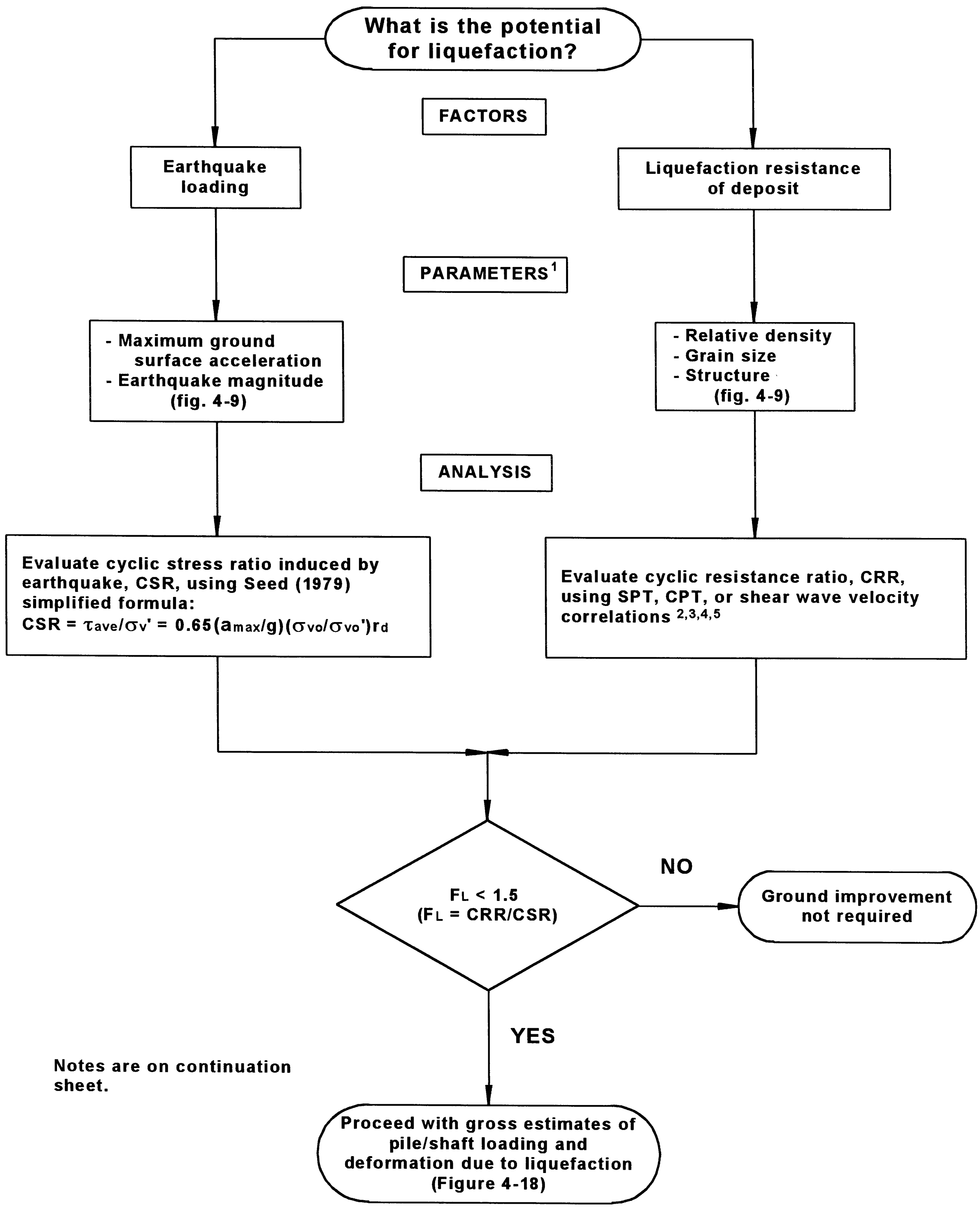


FIGURE 4-17 Assessment of Liquefaction Potential for Piles/Shafts

Notes:

1. Methods for determining parameters are given in Figure 4-9.

2. Evaluation of liquefaction resistance by CPT is generally preferred, provided the results (particularly soil classification) have been confirmed with some SPT data and samples. CPT data is nearly continuous with depth and more reliable. Obtain SPT and CPT correlations with CRR from NCEER (1997) for clean sands.

   Correct SPT and CPT correlations with CRR per NCEER (1997) for: fines content, influence of thin soil layers, earthquake magnitude different than M = 7.5, vertical effective confining stress using K_σ, and static horizontal shear stress using K_α.

3. Shear wave velocity can be used as a supplemental method to SPT or CPT for evaluating cyclic resistance ratio (CRR) per NCEER (1997).

4. Liquefaction resistance of gravelly soils should be evaluated per NCEER (1997). The Becker Penetration Test (BPT) may be required for soils with with high content of gravel and cobbles.

5. If possible, site specific liquefaction potential curves should be developed and used when no liquefaction resistance correlations are available for the soils encountered. These curves can be developed in the laboratory for soils which can be sampled (using specialized methods if necessary and possible) using cyclic CU triaxial or cyclic simple shear tests.

FIGURE 4-17 (continued)

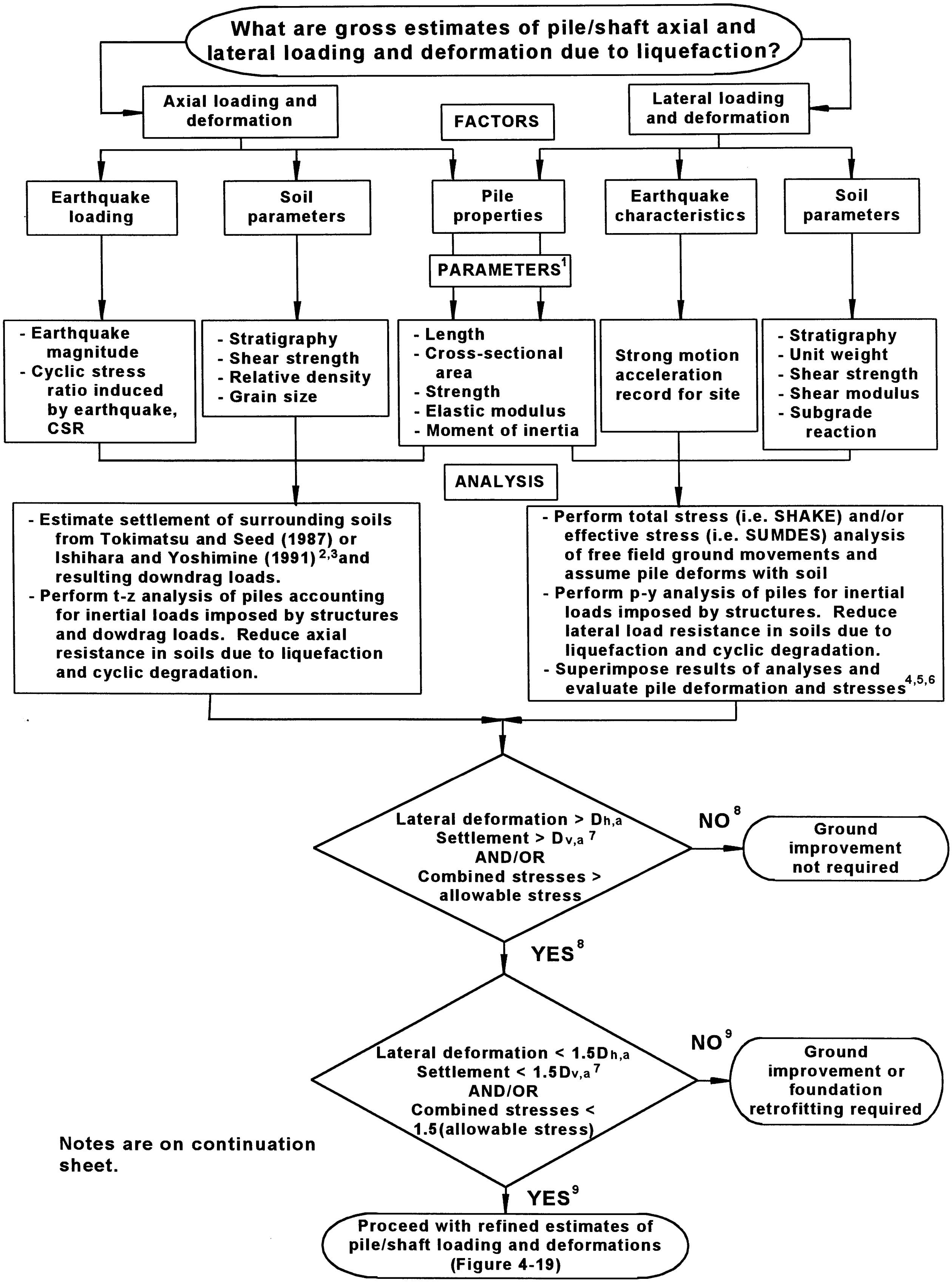


FIGURE 4-18 Gross Pile/Shaft Loading and Deformation Estimates

Notes:

1. Assessment methods for parameters are given in Figure 4-20.

2. Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1991) procedures were developed for "clean" sands. Munfakh et al. (1998) indicate these procedures will overestimate settlements for sandy silts and silts. For silty sands, Youd (1998) indicates settlement can be estimated for screening purposes using Tokimatsu and Seed's procedure by correcting $(N_1)_{60}$ to an equivalent "clean sand" value using the method described by NCEER (1997). For other soil types susceptible to liquefaction, settlements can be estimated using results from cyclic CU triaxial tests on "undisturbed" samples (see note 4 of Figure 4-4).

3. Estimated settlements should include densification settlements of susceptible cohesionless soils above groundwater (per Tokimatsu and Seed, 1987, or Munfakh et al., 1998) and settlements due to deformations from lateral spreading, as well as those from dissipation of seismically-induced excess porewater pressures of saturated soils.

4. Since it is unlikely that the maximum ground/pile deformation and the maximum inertial load due to structure will act on piles in the same direction at the same time, a reduced inertial load can be used for p-y analyses of piles for some cases. Caltrans (1994) recommends piles be analyzed for stresses and moments for the following cases:
   - 100% horizontal inertial (shear) load with 140% vertical dead load and 0% soil strain
   - 25% horizontal inertial (shear) load with 140% vertical dead load and 100% (peak) uni-directional soil strain
   - 25% horizontal inertial (shear) load with 140% vertical dead load and 100% (peak) bi-directional soil strain

   Note: Jackura and Abghari (1995) indicate that using 25% of the horizontal inertial (shear) load may be unconservative and 50% more appropriate.

5. If lateral spreading is anticipated, where the surrounding soil mass moves relative to piles, analyze piles using available methods (e.g. O'Rourke et al., 1994; Ishihara and Cubrinovski, 1998)

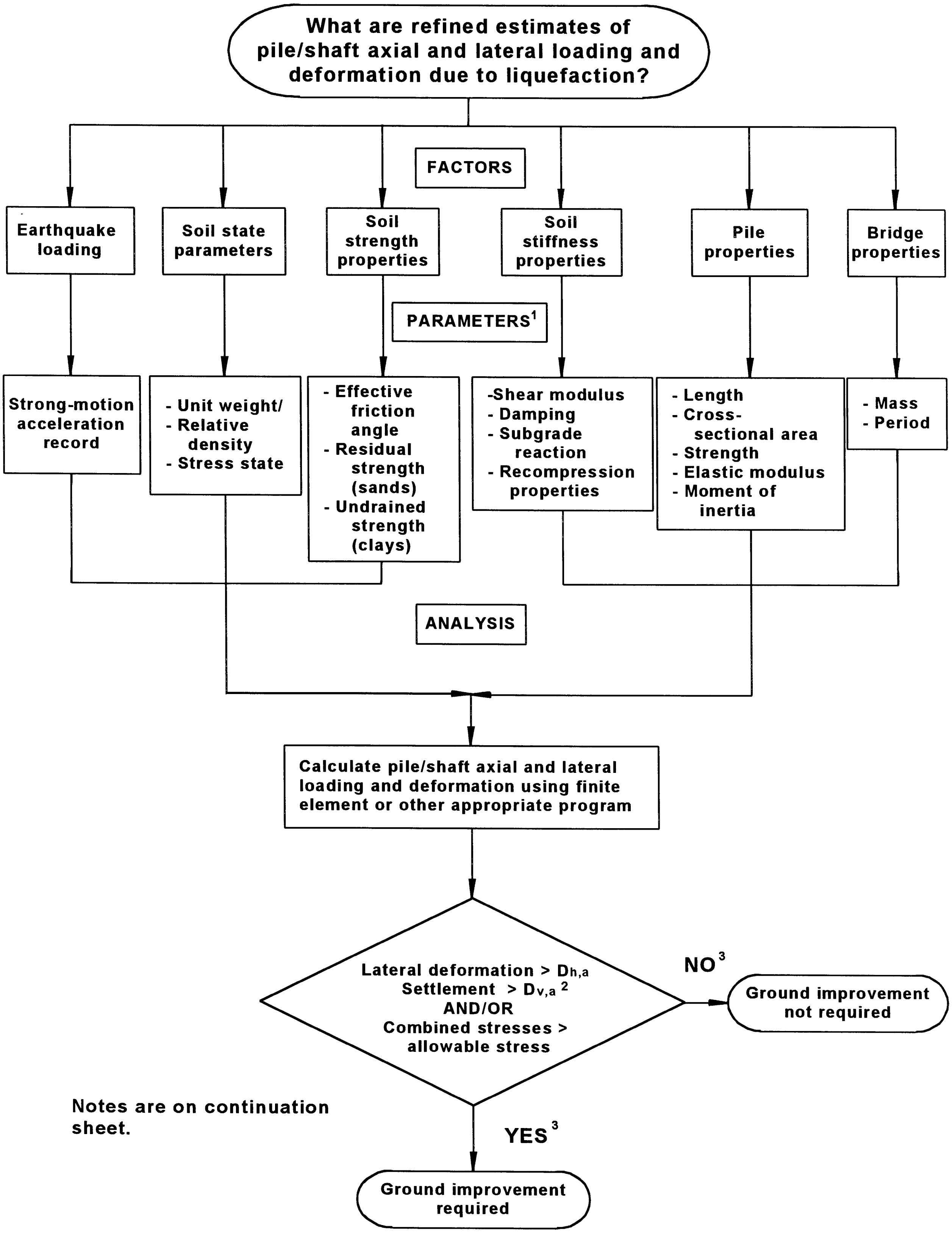
6. For piles located at retaining walls, embankments, or slopes, additional loads may need to be included to account for movement of the retaining wall, embankment, or slope materials.

7. $D_{v,a}$ and $D_{h,a}$ are the allowable vertical and horizontal movements, respectively, of the bridge foundation as determined by the bridge engineer.

8. "No" response appropriate if it applies to both lateral deformation/settlement criteria and stress criteria. "Yes" response appropriate if it applies to either or both criteria.

9. "Yes" response appropriate if it applies to both lateral deformation/settlement criteria and stress criteria. "No" response appropriate if it applies to either or both criteria.

FIGURE 4-18 (continued)



Notes are on continuation sheet.

FIGURE 4-19 Refined Pile/Shaft Loading and Deformation Estimates

Notes:

1. Methods for determining parameters are given in Figure 4-21.

2. $D_{v,a}$ and $D_{h,a}$ are the allowable vertical and horizontal movements, respectively, of the bridge foundation as determined by the bridge engineer.

3. "No" response appropriate if it applies to both lateral deformation/settlement criteria and stress criteria. "Yes" response appropriate if it applies to either or both criteria.

FIGURE 4-19 (continued)

Factors	Parameters	Determination
Earthquake loading	Earthquake magnitude	Refer to Figure 4-8
	Cycle Stress Ratio (CSR)	Refer to Figure 4-8 for a_{max} and Figure 4-10 for CSR formula
Soil parameters (axial loading and deformation)	Stratigraphy	Refer to Figure 4-7
	Shear strength	Refer to Figure 4-13
	Relative density	Refer to Figure 4-9
	Grain Size	Refer to Figure 4-6
Pile properties	Length	Manufacturer's data
	Cross-sectional area	
	Strength	
	Elastic modulus	
	Moment of inertia	
Earthquake characteristics	Strong-motion acceleration record	Refer to Figure 4-11
Soil parameters (lateral loading and deformation)	Stratigraphy	Refer to Figure 4-7
	Shear strength	Refer to Figure 4-13
	Shear modulus	Correlations with CPT; shear wave velocity tests; standard modulus curves (refer to Fig. 4-14)
	Damping	Standard damping curves (refer to Figure 4-14)
	Subgrade reaction	Correlations with soil type and strength

FIGURE 4-20 Determination of Parameters for Gross Pile/Shaft Loading and Deformation Estimates

Factors	Parameters	Determination
Earthquake loading	Strong-motion acceleration record	Refer to Figure 4-11
Soil state parameters	Relative density Stress state	Refer to Figure 4-11 Refer to Figure 4-12
Soil strength properties	Effective friction angle and residual undrained shear strength (sands)	Refer to Figure 4-13
	Undrained shear strength (clays)	Correlations with CPT; field vane shear test; laboratory tests
Soil stiffness properties	Shear modulus and damping	Refer to Figure 4-14
	Subgrade reaction (and degradation)	Correlations with soil type and strength
	Recompression properties	Refer to Figure 4-14
Pile properties	Length Cross-sectional area Strength Elastic modulus Moment of inertia	Manufacturer's data
Bridge properties	Mass Period	Structural evaluation

FIGURE 4-21 Determination of Parameters for Refined Pile/Shaft Loading and Deformation Estimates

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SECTION 5
LIQUEFACTION REMEDIATION METHODS

5.1 Introduction

At bridge sites where the factor of safety against liquefaction is unacceptably low or anticipated liquefaction-induced ground deformations or loadings on the structure are excessive (based on the evaluations outlined in Section 4), ground or foundation improvement measures will be necessary. The objective of ground and foundation improvement for liquefaction mitigation is to provide improved support for key bridge components during and immediately after an earthquake, to prevent unacceptable damage due to liquefaction of surrounding soils. Ground improvement achieves this objective primarily by providing mechanisms which increase the shear strength and lower the excess pore water pressures induced in potentially liquefiable soils around bridge foundations. As a result, the loss of soil strength and densification of soils around the foundations are reduced, thereby keeping vertical and lateral deformations and loads on the structure within acceptable limits. Foundation improvement, on the other hand, involves strengthening the existing foundation elements or adding additional elements so that the deformations and loads on the foundation and bridge structure are within acceptable limits, even if the surrounding ground liquefies.

The applicability and feasibility of using different ground and foundation improvement methods for remediating liquefaction effects at existing highway bridges are dependent on:

- space and geometry limitations of the bridge site affecting accessibility and working space for construction equipment,
- subsurface conditions affecting the effectiveness of a particular method in producing the required improvement,
- potential for construction-induced movements and vibrations of the bridge caused by the remediation method along with the likelihood of resulting damage,
- desired post-treatment performance of the bridge,

- potential environmental effects of improvement implementation, and
- cost of the improvement method relative to other methods.

A review is provided below of potentially feasible ground and foundation improvement options and their applicability to some typical bridge configurations based on the above issues. The importance of the treatment type, size, and location on bridge performance is also discussed.

5.2 Bridge Configurations Considered

5.2.1 Overview of Configurations

The configuration of a bridge will impact the feasibility of using particular ground and foundation improvement methods for liquefaction mitigation. Inherent in the bridge configuration are the abutments and piers with their supporting foundations, approach embankments, span lengths and widths, and limited clearance. Site factors that influence the bridge configuration and also impact the feasibility of remediation methods include the ground surface topography and the presence of a water body, structures or other roadways.

There are many different existing bridge configurations that consist of varying combinations of the elements mentioned above. Since it is not feasible to address ground and foundation improvement relative to all of these configurations, the improvement methods are discussed herein relative to some commonly used abutment and pier types. Based on this information, the engineer should be able to evaluate the use of remediation methods for other bridge configurations.

5.2.2 Abutments

Abutment types considered in this report include pile-supported stub, full-height wingwall, and floating stub abutments. Plan and section views for each of these abutment types are shown in Figure 5-1.

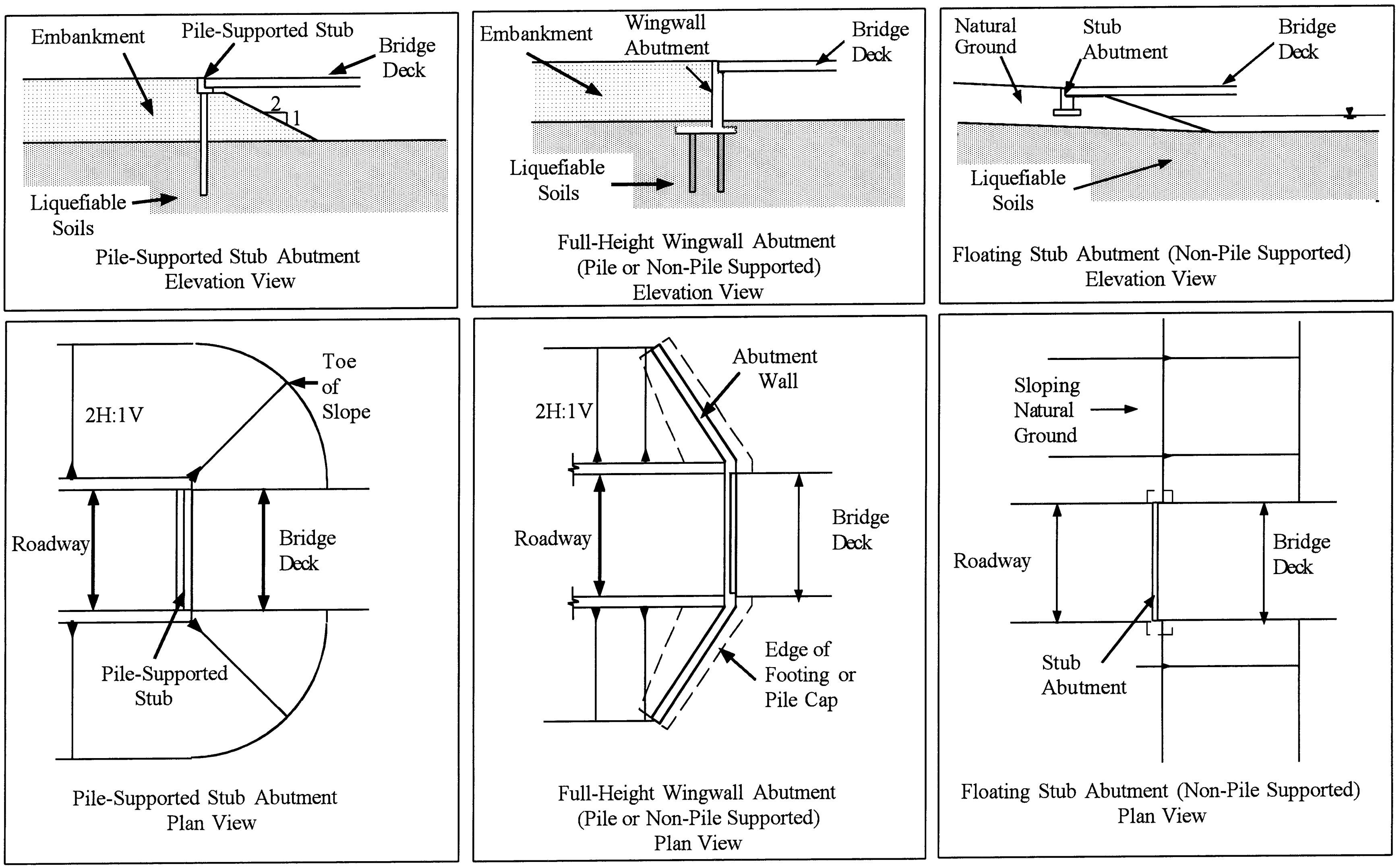


FIGURE 5-1 Some Typical Bridge Abutments (from Mitchell and Cooke, 1995)

Pile supported stub abutments are commonly used at the end of an approach embankment having a sloping face. The piles are typically driven through the embankment fill and several feet into the underlying natural soils.

Full-height wingwall abutments are essentially retaining walls supported by either shallow or deep foundations. They are commonly used where space restrictions forbid a sloping face at the ends of approach embankments.

Floating stub abutments are typically used where the site topography and soil conditions permit the abutments to be founded on shallow spread footings in natural soils having adequate bearing capacity.

5.2.3 Piers

The term pier, as used in this report, refers to the substructure elements constructed between abutments for the purpose of supporting the superstructure of the bridge. Pier is not used herein to refer to deep foundation elements such as drilled shafts, piles, or caissons.

Pier types considered include solid wall or multi-column, hammerhead, and single circular column piers. Bridge piers are commonly supported by deep foundations due to the magnitude of loads supported and, when the pier is located in rivers or streams, concern for scour. Figure 5-2 shows each of these pier types in plan and elevation.

5.2.4 Span Length and Width

The abutment and pier types shown in Figures 5-1 and 5-2 are typical for small (6 to 15 meter spans) to medium (15 to 60 m spans) bridges. Since the majority of bridges in the United States fall in these categories, the discussion of remediation method applicability presented herein is focused on these two bridge sizes. Based on this information, the user should also be able to

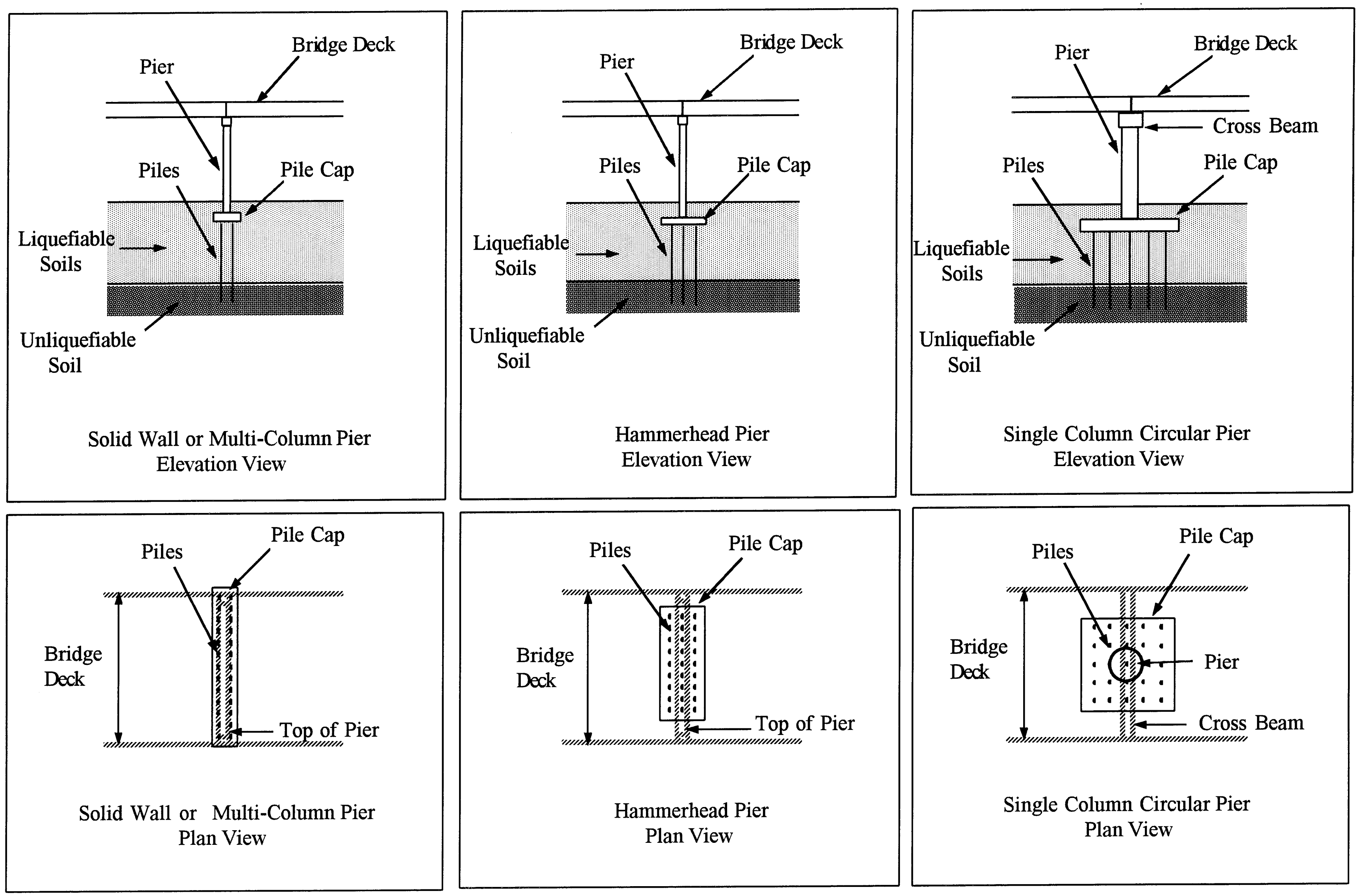


FIGURE 5-2 Some Typical Bridge Piers (from Mitchell and Cooke, 1995)

draw some conclusions about the use of the remedial measures for large (60 to 150 m spans) bridges.

5.3 Ground Improvement Methods

Ground improvement methods considered potentially feasible for remediation of liquefiable soils at existing highway bridges include:

- Grouting - compaction, permeation, jet;
- Vibro systems - vibratory probe, vibro-compaction, vibro-replacement (stone columns);
- Surcharge and buttress fills;
- Reinforcement and containment - root piles, mixed-in-place walls and columns; and
- Drains.

Other methods, such as deep dynamic compaction and deep blasting, are generally not considered feasible due to the constraints imposed by the presence of the bridge, particularly the potential for construction-induced damage. However, recent improvements in the control of vibrations induced by deep blasting may make this method feasible for some bridges (Mitchell et al., 1998).

The characteristics and applicability of the potential ground improvement methods for liquefaction mitigation at existing bridges are summarized in Table 5-1. Included in the table are brief descriptions of the principles behind each method, soil types for which it is suitable, properties of treated materials, relative costs, applicability to the different abutment and pier types shown in Figures 5-1 and 5-2, and general comments. More detailed descriptions of the improvement methods, including illustrative diagrams of the improvement processes, are given in Appendix A. Some references on the improvement methods are also given there.

The stated applicabilities of an improvement method to different abutment and pier types given in Table 5-1 are subjective, based only on the limitations imposed by space and geometry factors and

TABLE 5-1
SUMMARY OF GROUND IMPROVEMENT METHODS FOR LIQUEFACTION REMEDIATION AT EXISTING BRIDGES

Method	Principle	Suitable Soil Types	Treated Soil Properties	Relative Costs	Abutment Applicability*		Pier Applicability*	Comments
					Stub	Full-Height Wingwall		
Compaction Grout	Highly viscous grout acts as spherical hydraulic jack when pumped under high pressure resulting in densification.	Compressible soils with some fines	Increased D_r SPT: $(N_1)_{60}$ = 25 to 30 CPT: q_{c1} = 80 to 150 tsf (Kg/cm^2)	Low material cost; high injection cost.	1. High. Treat anywhere between abutment and embankment toe; treat under and around abutment if excessive settlement expected. 2. High. Treat around pile groups.	1. Generally high. Treat under and around foundation. 2. High. Treat around pile groups.	High for solid wall, multi-column, and hammerhead piers. High to moderate for circular column piers. 1. Treat under and around foundation. 2. Treat around pile groups.	Must control heave and/or hydraulic fracture of soil. Particulate and chemical grouting: verify size and strength of grouted soil mass. Jet grouting: stage work to limit settlements. Evaluate potential damage to piles from jetting pressure.
Particulate Grouting	Penetration grouting: fill soil pores with cement, soil and/or clay.	Clean, medium to coarse sand and gravel	Cement grouted soil: high strength	Lowest of grouting systems				
Chemical Grouting	Solutions of two or more chemicals react in soil pores to form a gel or solid precipitate.	Silts and sands	Low to high strength	High to very high				
Jet Grouting	High speed jets at depth excavate, inject and mix stabilizer with soil to form column or panels.	Sands, silts and clays	Solidified columns and walls	High				

Notes: * Item No. 1 under applicability indicates applicability of improvement method for foundations over or in liquefiable soils. Item No. 2 indicates applicability for pile (or drilled shaft) foundations extending through liquefiable soils.

TABLE 5-1 (cont.)
SUMMARY OF GROUND IMPROVEMENT METHODS FOR LIQUEFACTION REMEDIATION AT EXISTING BRIDGES

Method	Principle	Suitable Soil Types	Properties of Treated Soil	Relative Costs	Abutment Applicability*		Pier Applicability*	Comments
					Stub	Full-Height Wingwall		
Vibratory Probe	Densification by vibration, liquefaction-induced settlement underwater.	Sand (< 15% passing No. 200 sieve)	$\underline{D_r}$: up to 80+% Ineffective in some sands.	Moderate	1. Moderate for lateral spreading; low for settlement. Treat at embankment toe to reduce risk of construction settlement.	1. Low. Potential for excessive settlement and vibrations of bridge. Overhead clearance limitations. 2. Low. Treating around piles difficult due to access problems.	1. Low. Potential for excessive settlement and vibrations of bridge. Overhead clearance limitations. 2. Moderate to high. Treat around pile groups.	Overhead clearance limitations will restrict use. Monitor bridge for excessive vibrations. Construction in water requires special procedures.
Vibro-Compaction	Densification by vibration and compaction of backfill at depth.	Sand (<20% passing No. 200 sieve)	$\underline{D_r}$: up to 85+% <u>SPT</u>: $(N_1)_{60}$ = 25 to 30 <u>CPT</u>: q_{c1} = 80 to 150 tsf (Kg/cm^2)	Moderate				
Vibro-Replacement Stone Columns	Densely compacted gravel columns provide densification, reinforcement, and drainage.	Sands and silts	Increased D_r <u>SPT</u>: $(N_1)_{60}$ = 25 to 30 <u>CPT</u>: q_{c1}= 80 to 150 tsf (Kg/cm^2)	Moderate to high	2. Low. Treating around piles difficult due to access problems.			
Surcharge/ Buttress Fill	Weight of surcharge increases liquefaction resistance by increasing effective stresses. Buttress fill increases stability by reducing overturning moment and increasing resisting moment.	Any soil surface provided it will be stable	Increase in strength	Low	1. High for slope stability and low for settlement. Place at embankment toe. 2. Low. Ineffective in increasing soil stresses at piles.	1 & 2. Moderate. Place buttress fill in front of wall.	1 & 2. Moderate to low. Place surcharge around pier.	Need large area. Evaluate loads and settlement imposed on bridge.

Notes: * Item No.1 for foundations over or in liquefiable soils. Item No. 2 for pile (or drilled shaft) foundations extending through liquefiable soils.

TABLE 5-1 (cont.)
SUMMARY OF GROUND IMPROVEMENT METHODS FOR LIQUEFACTION REMEDIATION AT EXISTING BRIDGES

Method	Principle	Suitable Soil Types	Properties of Treated Soil	Relative Costs	Abutment Applicability*		Pier Applicability*	Comments
					Stub	Full-Height Wingwall		
Mix-In-Place Walls & Columns	Lime, cement or asphalt introduced through auger or special in-place mixer.	Liquefiable and weak soils	Solidified soil walls or columns of relatively high strength confine and/or reinforce potentially liquefiable soils	High	1. Moderate for lateral spreading; low for settlement. Install along toe of embankment. 2. Low. Hard to install around abutment pile group.	1. Moderate for lateral spreading, low for settlement. Install at toe of wall. 2. Moderate to low. Install around abut-ment pile group.	1. Moderate to low. Install completely around pier. 2. Moderate to low. Install completely around pier pile groups.	Extend to firm strata. Stage work to control construction settlements. Space limi-tations may restrict use. Construction in water requires spe-cial proce-dures.
Root Piles	Small-diameter inclusions (typically grouted piles) used to carry tension, shear and compression.	All soils	Reinforced zone behaves as a coherent mass	Moderate to high	1. Moderate to low. Zone for installing piles same as described for for grouting. 2. Moderate to low. Install piles around pile groups.	1. Moderate to low. Install piles beneath and around foundation. 2. Moderate to low. Install piles around pile groups.	1. Moderate to low. Install piles beneath and around pier foundation. 2. Moderate to low. Install piles around pile groups.	Extend piles to firm strata. Large number of piles may be required to provide adequate reinforcement. Avoid damage to existing piles.

Notes: * Item No.1 for foundations over or in liquefiable soils. Item No. 2 for pile (or drilled shaft) foundations extending through liquefiable soils.

TABLE 5-1 (cont.)
SUMMARY OF GROUND IMPROVEMENT METHODS FOR LIQUEFACTION REMEDIATION AT EXISTING BRIDGES

Method	Principle	Suitable Soil Types	Properties of Treated Soil	Relative Costs	Abutment Applicability*		Pier Applicability*	Comments
					Stub	Full-Height Wingwall		
Drains: **Gravel** **Sand** **Wick**	Relief of excess pore water pressure to prevent liquefaction. Intercept and dissipate excess pore water pressure plumes from adjacent liquefied soil.	Sand, silt	Improved drainage	Low to moderate	1&2. Mod-erate. Install drains around zone improved by other method(s).	1&2. Mod-erate. Install drains around zone improved by other method(s).	1&2. Moderate. Install drains around zone improved by other method(s).	Topography and space limitations may restrict use.
Mini-Piles**	Provide piles to carry loads through liquefiable soils to firm stratum.	All soils	-	High	1&2. High to moderate.	1&2. High to moderate.	1&2. High to moderate.	Must provide means of tying mini-piles into existing foundation.

Notes: * Item No.1 for foundations over or in liquefiable soils. Item No. 2 for pile (or drilled shaft) foundations extending through liquefiable soils.
** Foundation improvement method.

the potential for damage to the structure due to movements or vibrations induced by the improvement procedures. The adequacy of any method to achieve the required level of ground improvement and the improvement cost are dependent on site specific conditions; therefore these factors are not included in the assessment of the methods general applicability given in Table 5-1.

5.3.1 Grouting

Grouting methods generally improve cohesionless soils by increasing their strength. In compaction grouting this is achieved by physically densifying the soil matrix, as well as increasing the in-situ effective stresses in the ground. Permeation grouting is typically used to increase strength by (1) replacing the water in soil voids with another material so that excess pore water pressures are not generated during earthquake shaking and (2) binding the soil particles together with cementing material. Jet grouting completely alters the soil matrix by removing columns of soil and replacing them with cement grout or a mixture of soil and cement. Jet and compaction grout columns also act as reinforcing elements in the soil mass.

Review of Table 5-1 indicates grouting methods have high applicability to many abutment and pier types. A major advantage of grouting techniques, particularly compaction and permeation grouting, is the ability to limit the loss of ground support and structure movements while the work is being performed. Grouting operations can generally be performed in small work areas and on slopes. Inclined holes can be drilled and grouted. Significant increases in the relative density of loose, granular soils can be achieved with compaction grouting. Permeation and jet grouting can usually produce grouted soils of high strength.

Figure 5-3 shows a conceptual treated zone for compaction grouting beneath and around a footing. A diagram of a potential treated zone for permeation (chemical or particulate) grouting beneath a footing is shown in Figure 5-4. Jet grout columns could also be used to achieve the treatment zone design requirements shown in Figure 5-4. Four case histories are summarized in Andrus and Chung (1995) where permeation grouting was used to remediate liquefiable soils or reduce seismically-induced settlements for existing or new structures. Likewise, they provide a

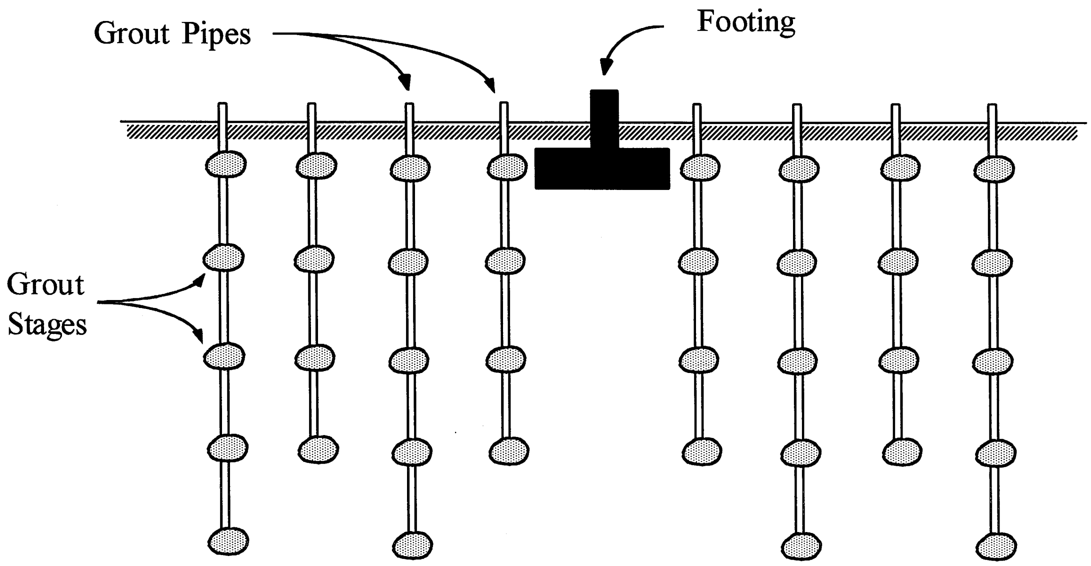


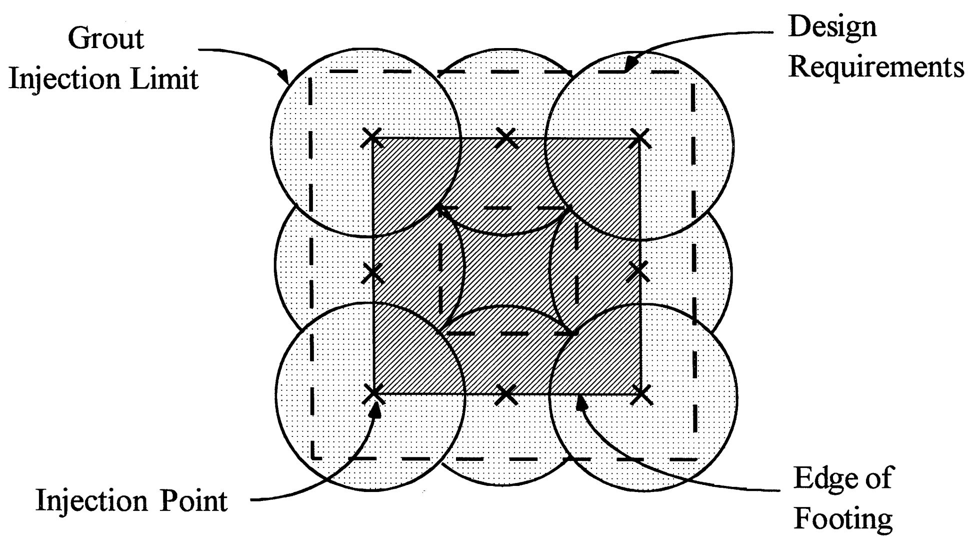
FIGURE 5-3 Section Showing Compaction Grouting Around Footing (after Graf, 1992)

summary of eleven and three case histories, respectively, where compaction and jet grouting were used for liquefaction remediation, including several at existing structures.

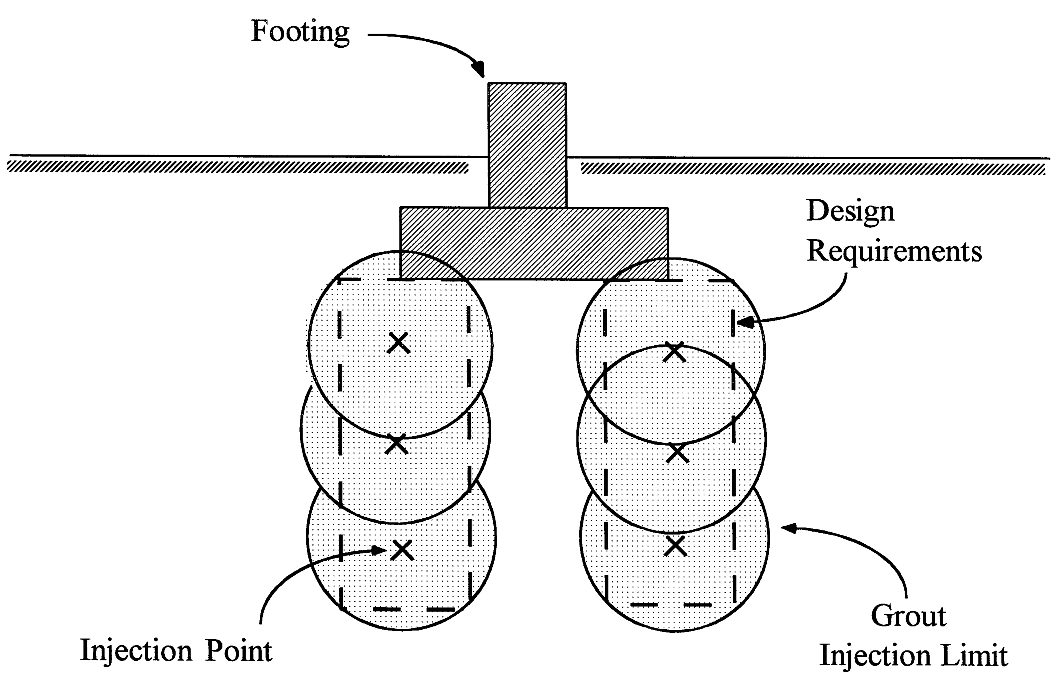
5.3.2 Vibro Systems

Vibro systems improve loose, saturated cohesionless soil by causing temporary local liquefaction of the soil and subsequent densification through the use of a vibrating probe. The densification results in greater shear strength to resist earthquake-induced stresses. When the void which develops in the soil during insertion of the vibrating probe is backfilled with densified gravel or crushed stone, additional soil mass strength is achieved as a result of the reinforcement and improved drainage provided by these zones. Compaction of the backfill with the probe also induces lateral displacements in the surrounding soil resulting in additional densification.

Significant improvements in the relative densities of loose, granular soils can be achieved with vibro systems. However, vibro system methods can cause significant settlement of the ground being treated unless extreme care is taken during the installation process. Therefore, these



Plan View



Profile

FIGURE 5-4 Permeation Grouted Zone Beneath a Footing (after Andrus and Chung, 1995)

methods are generally restricted to creating improved zones around existing deep foundations which extend completely through the liquefiable soils, as shown in Figure 5-5, provided (1) the foundation and superstructure will not be damaged by construction vibrations and (2) the foundation can safely withstand the temporary loss of support in liquefiable zones and subsequent downdrag forces which may occur during densification. Alternatively, these methods can potentially be used for ground improvement at the toe of an approach embankment supporting a stub abutment, as illustrated in Figure 5-6, if there is adequate distance between the foundation and improvement locations. For these types of applications, vibro systems are generally judged to have moderate applicability.

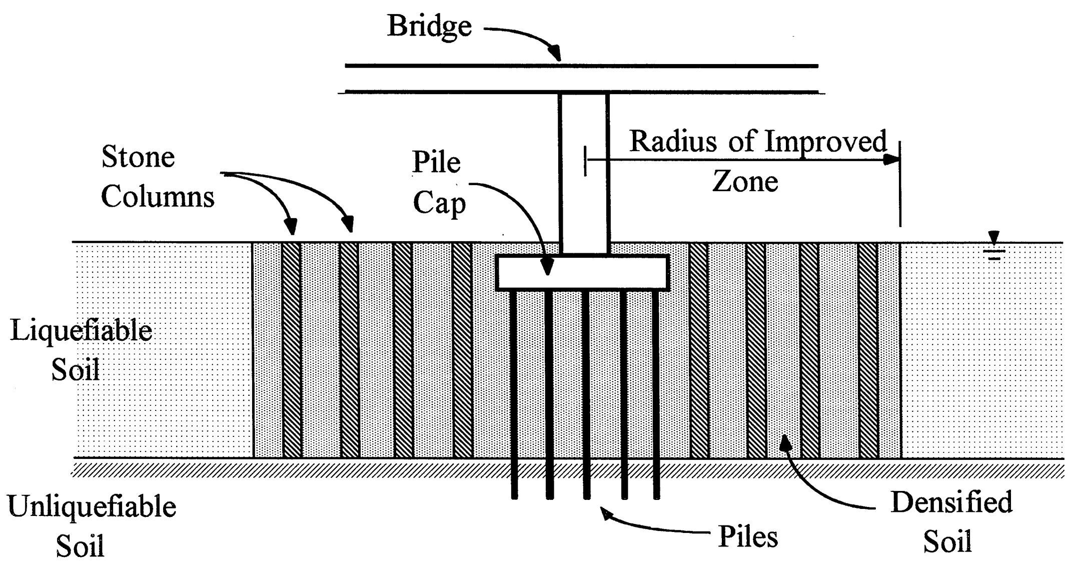


FIGURE 5-5 Vibro System (Stone Column) Treatment Around a Pile-Supported Bridge Pier (after Jackura and Abghari, 1994)

5.3.3 Buttresses and Surcharge Fills

Buttresses consist of select, compacted fills constructed adjacent to embankments to extend potential slope stability failure surfaces, provide additional weight beyond the toe, and increase

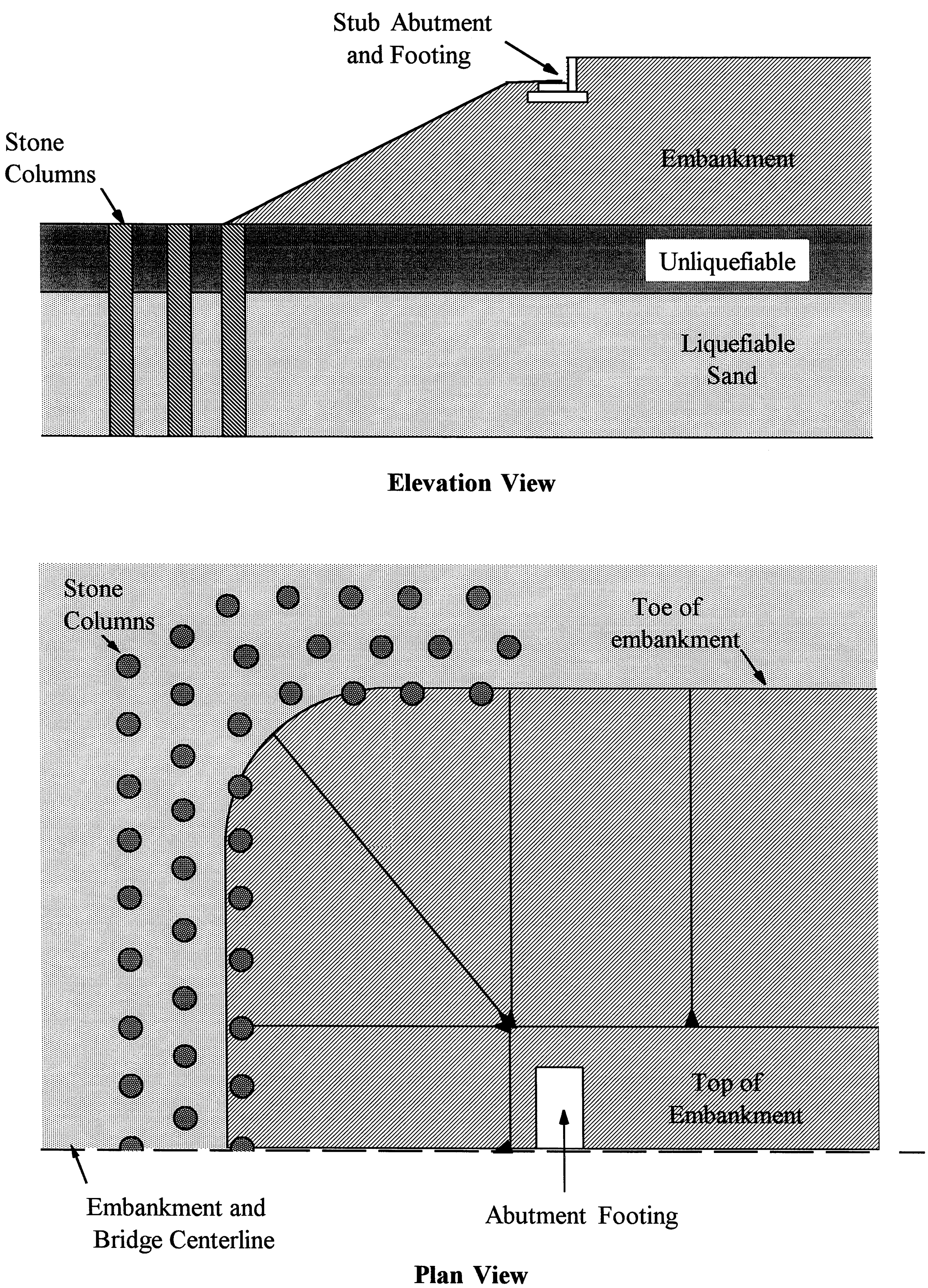


FIGURE 5-6 Vibro System (Stone Column) Treatment at Toe of Approach Embankment

effective stresses in underlying liquefiable strata, all of which improve stability. A buttress fill can be constructed in front of an approach embankment supporting a stub abutment to improve stability, as shown in Figure 5-7. Similarly, such a fill could be placed in front of a full-height wingwall abutment to mitigate liquefaction effects. Use of buttress fills at existing bridges for liquefaction remediation is judged to have high to moderate applicability.

Permanent surcharge fills are placed on the ground surface above liquefiable deposits to increase the effective stresses in the soils, thereby increasing the shear resistance and reducing the liquefaction potential. This type of application is judged to have moderate to low applicability for remediation of liquefiable soils at existing bridges.

Potential limitations of using buttress or surcharge fills at existing structures are restrictions on the size of the fill due to space limitations and the potential for damaging settlement and loading of foundations caused by the fill weight.

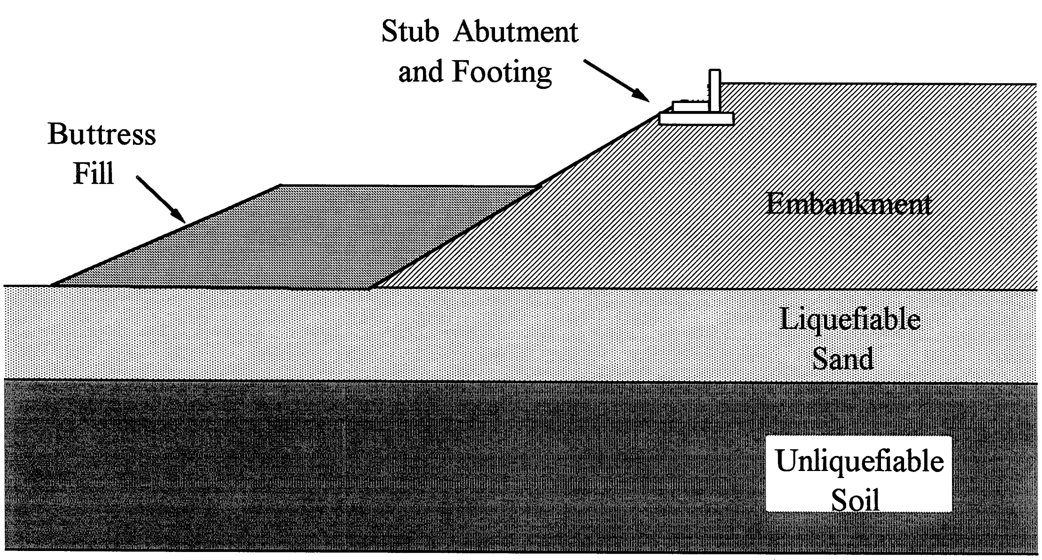


FIGURE 5 -7 Section Showing Buttress Fill at Toe of Approach Embankment

5.3.4 Containment and Reinforcement

Stiff elements in the ground around existing structures can be used to provide containment and reinforcement of liquefiable soils. Reinforcing methods generally consist of reinforcing elements, such as mixed-in-place columns or root piles, installed in a closely-spaced pattern for the purpose of resisting earthquake forces and reducing shear stresses in the liquefiable soils. Some potential treatments at abutments could consist of using mixed-in-place or stone columns in a configuration similar to that shown in Figure 5-6, or mixed-in-place walls as shown in Figure 5-8.

Containment methods provide an enclosure that prevents movement of any liquefied soil from within the boundaries of the enclosure, in addition to providing reinforcement of the ground. Figure 5-9 illustrates the use of a mixed-in-place wall around a pile foundation to provide reinforcement and containment.

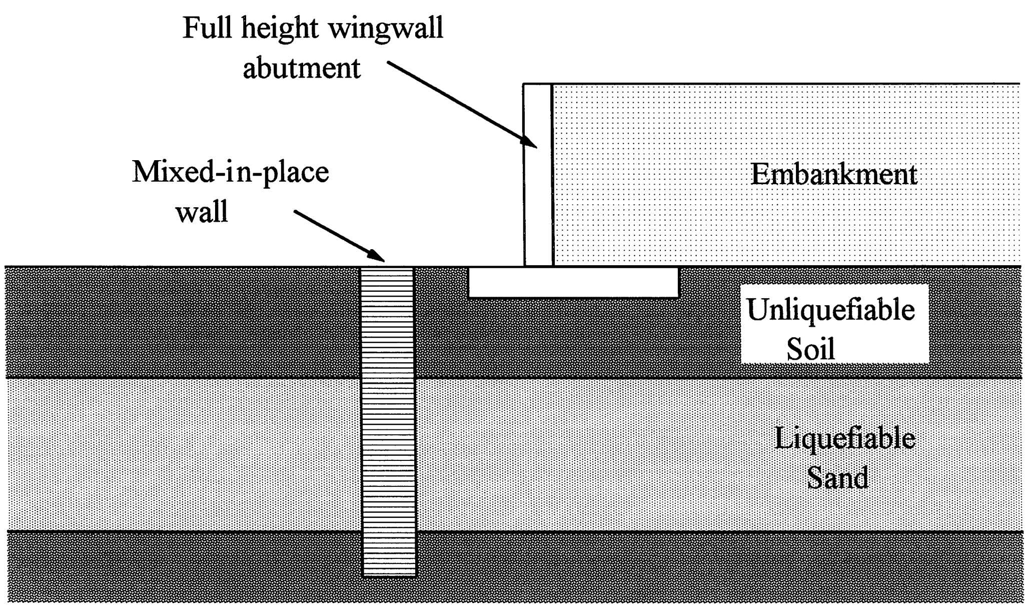


FIGURE 5-8 Section Showing Mixed-in-Place Wall in Front of Full-Height Wingwall Abutment

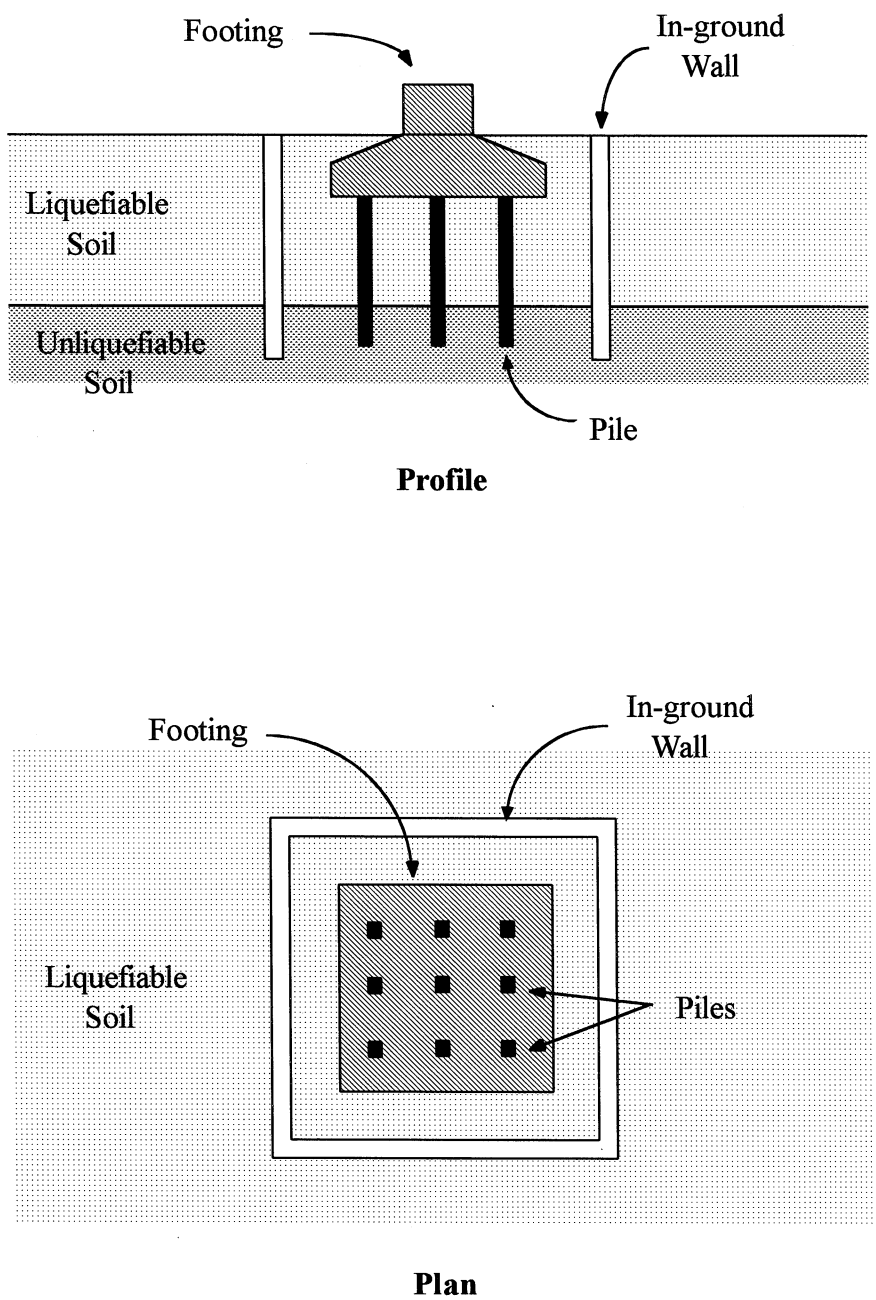


FIGURE 5-9 Mixed-in-Place Wall Around Pile-Supported Pier (after Kawakami, 1996)

Containment or reinforcement alternatives for existing bridges might also include installing sheetpiles at or around the existing foundations. However, there are potential problems with adequate clearance for construction equipment and damaging construction vibrations. For this reason, the use of sheetpiles is not discussed further in this report, although in some cases their use may be possible.

Reinforcement methods are judged to have moderate to low applicability for liquefaction remediation at existing bridges. Containment methods have high to moderate applicability.

5.3.5 Drains

Drainage methods for mitigating liquefaction consist of installing vertical columns of "free-draining" gravel, coarse sand, or prefabricated wicks in the ground. The drains are used to limit the development of excess pore water pressures in the liquefiable soil within the treatment zone, thereby improving the strength and shearing resistance during and after earthquake shaking.

Theoretical analyses, model tests, and field tests have shown that vertical gravel drains can potentially limit the development of excess pore water pressures and thus prevent liquefaction. However, the drains must be closely spaced and the soil being protected must be fairly coarse, having a high hydraulic conductivity. Thus the effective use of drains as a sole mitigation measure for liquefaction beneath an existing bridge foundation is only likely under special conditions. On the other hand, drainage zones can be constructed around the perimeter of improved zones (such as densified zones) supporting bridge foundations to intercept post-earthquake pore water pressure plumes which could otherwise cause pore water pressure build up and strength loss in the improved soils.

An important factor in drain performance is the permeability of the installed drain material in the field. Field case studies have shown that drains installed using stone or gravel can become contaminated with varying amounts of finer material, such as sand and silt, depending on the installation process. For instance, Baez and Martin (1995) report that gravel samples obtained from in-situ stone columns constructed by bottom feed or top feed methods contained approximately 20 percent (by weight) of the surrounding native sand. The presence of this sand decreased the permeability of the in-situ stone column material relative to the permeability of the uncontaminated gravel borrow (prior to installation) and would have significantly affected the performance of the columns if they were used for drainage purposes.

Successful design of vertical drains is dependent on adequately representing the drain permeability and resistance in the design analyses. Boulanger et al. (1998) investigated the design and performance of stone columns and gravel drains for liquefaction mitigation using drainage. They note that a critical factor in predicting the performance is adequately accounting for the effects of drain resistance in the design analyses. They indicate that design diagrams developed by Iai and Koizumi (1986) and Onoue (1988) account for the drain resistance. A discussion of the detrimental effects of contamination and low permeability zones in drains on performance (including a quantitative analysis), drain design and construction issues, and earthquake performance of drainage columns is also given in Boulanger et al. (1998).

5.4 Underpinning with Mini-piles

Underpinning elements can be used to support part of the foundation load during liquefaction. A widely-used underpinning method is to install mini-piles (also called pin-piles) which are tied into the existing bridge foundation and extend through the liquefiable soil into a non-liquefiable bearing stratum, as shown in Figure 5-10. These piles, along with any other existing piles/shafts extending through the liquefiable stratum, must be capable of withstanding the loads imposed on them by the structure, as well as those from deformation of the ground. The capacity and deformations of the piles should account for the reduced support that occurs due to liquefaction of the loose, granular soils and strength and stiffness degradation of non-liquefiable soils under cyclic loading. The piles should also be designed for downdrag loads associated with soil densification and resulting settlement. In areas susceptible to lateral ground deformation due to lateral spreading or movements of embankments, slopes, or retaining walls, mini-piles should be designed to resist the lateral forces imposed by the ground movement.

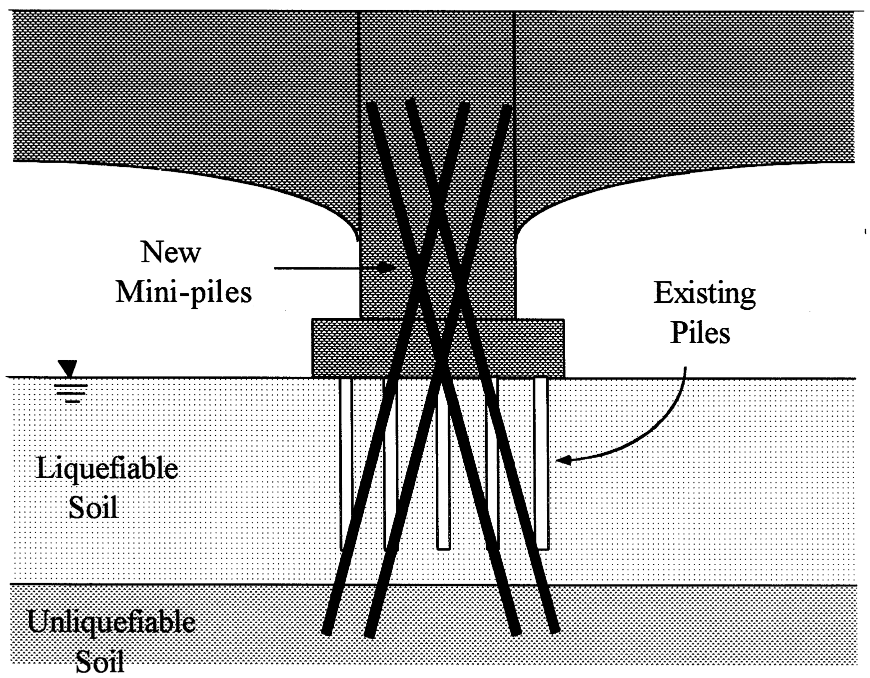


FIGURE 5-10 Conceptual Illustration of Foundation Improvement Using Mini-piles at Bridge Pier (after Hayward Baker Inc., 1995)

The applicability of mini-piles for liquefaction remediation is judged to be high to moderate.

5.5 Effect of Treatment Type, Size, and Location

Three factors of particular importance to the performance of ground improvement measures are the type, size, and location (relative to the bridge abutments and piers) of the treatment. All three factors must be considered in the design process to develop an effective remediation scheme that balances cost and desired performance.

Effects of treatment type, size, and location on improved ground and supported structure performance have been evaluated by various researchers from the results of numerical analyses, centrifuge tests, and shaking table tests for specific cases. The results from these studies are helpful for establishing a qualitative sense for the effects of these parameters on improved ground performance. However, they are not sufficient to draw conclusions that are applicable to all cases and conditions, because the models were limited to a few sets of specific conditions. The potential influences of treatment type, size, and location on the performance of bridge abutments and piers are discussed below, based on information from some of these studies. Summaries of other centrifuge and shaking table tests performed involving improved ground and supported structures can be found in references such as Stewart et al. (1997).

5.5.1 Abutment/Embankment Performance

5.5.1.1 Treatment Size and Location

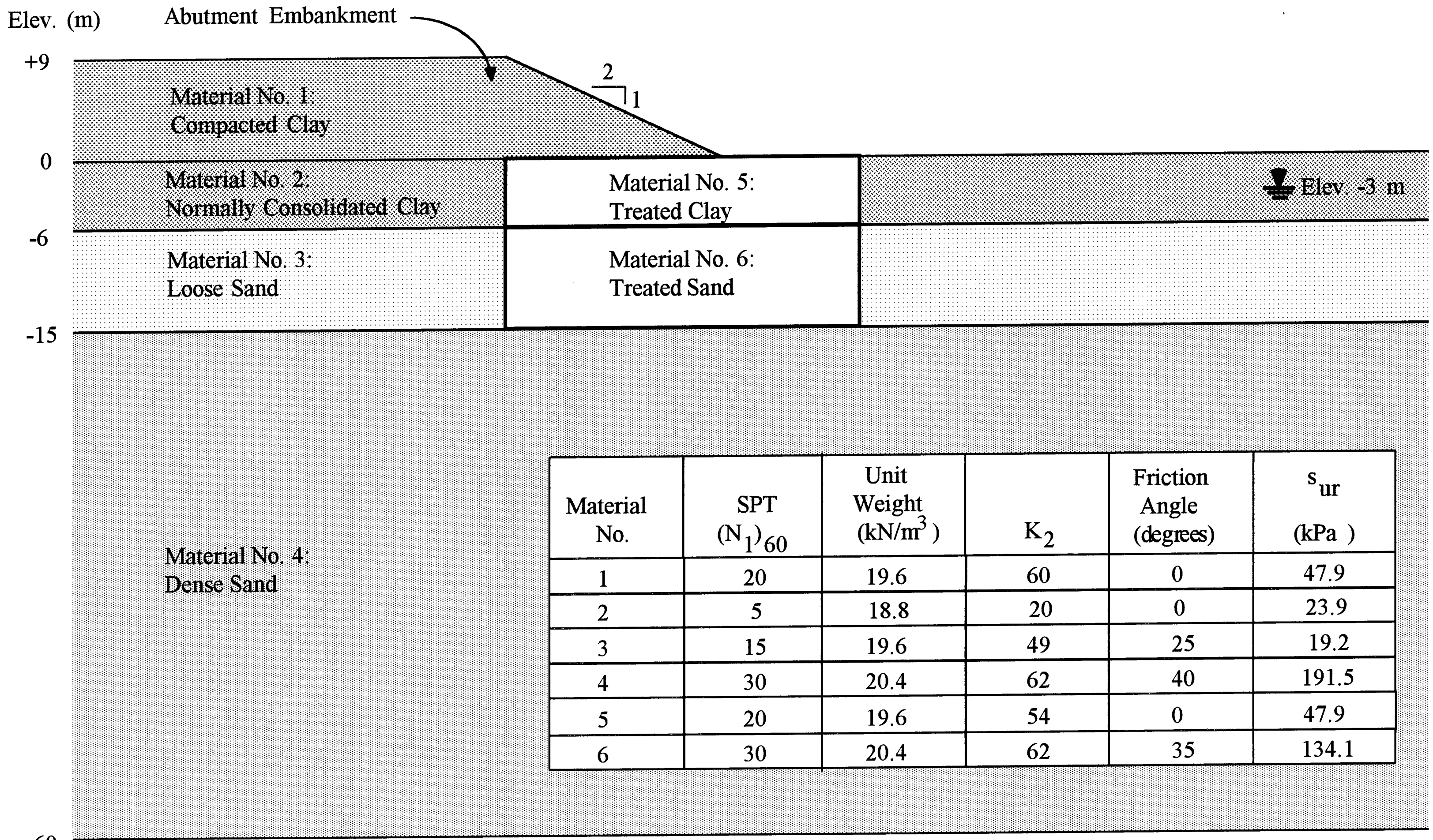
Riemer et al. (1996) studied the effects of improved ground zone size and location on the performance of a section of the Highway I-57/Mississippi River Bridge Abutment, shown in Figure 5-11, by performing dynamic analyses of the zone using base acceleration records from the 1989 Loma Prieta (M=7.1) and 1988 Saguenay (M=6.0) earthquakes having predominant frequencies of about 1 and 10 Hertz, respectively. Both records had strong shaking durations of about 8 to 10 seconds, and were scaled to have maximum accelerations of 0.22g (where g is the acceleration of gravity). These analyses were performed using the finite difference computer code FLAC (Itasca Consulting Group, Inc., 1996). The soil properties used in the analyses are presented in Figure 5-11. Although two sets of analyses were performed using simplified "stiff" and "soft" shear stress-strain models for liquefied sands, with the limiting shear stress being the undrained residual shear strength, the analytical results from only the "stiff" model are presented here.

Figure 5-12 illustrates the effect of improved ground zone width on the predicted maximum lateral displacement of the embankment. As seen from the displacement values (given for earthquakes with predominant frequencies of 1 and 10 Hz) listed in the figure, increasing the distance that the treated zone extends beneath the embankment from the toe results in decreasing lateral displacement due to seismic loading. It can also be seen that although the predicted displacement progressively decreases as the width is progressively increased from 12 m to 36 m, the amount of change in the displacement reduction gets progressively smaller.

The effects of treatment zone locations on lateral displacements of an embankment based on the data of Riemer et al. (1996) are shown in Figure 5-13. The values indicate that a 24-m-wide treatment zone is most effective when it is located beneath the sloping portion of the embankment. As the treated zone location moves outward from beneath the sloping portion, predicted lateral displacements become progressively larger, with the largest displacement occurring when the left side of the treatment zone is located at the toe and the remainder of the zone lies completely outside the embankment.

The results presented by Riemer et al. show quantitatively the effect of treatment zone size and location on the performance of an embankment underlain by a liquefiable soil layer. Although the soil model used in these analyses has not been verified by comparison of predicted displacements with measured values, the results of the analyses provide an indication of the relative effectiveness of treated zone size and location. In particular, it appears that treated zones constructed entirely outside an embankment toe are not very effective in reducing deformations. One reason for the ineffectiveness is that the potential failure surfaces may pass between the toe and the treated zone, or only through a part of the treated zone.

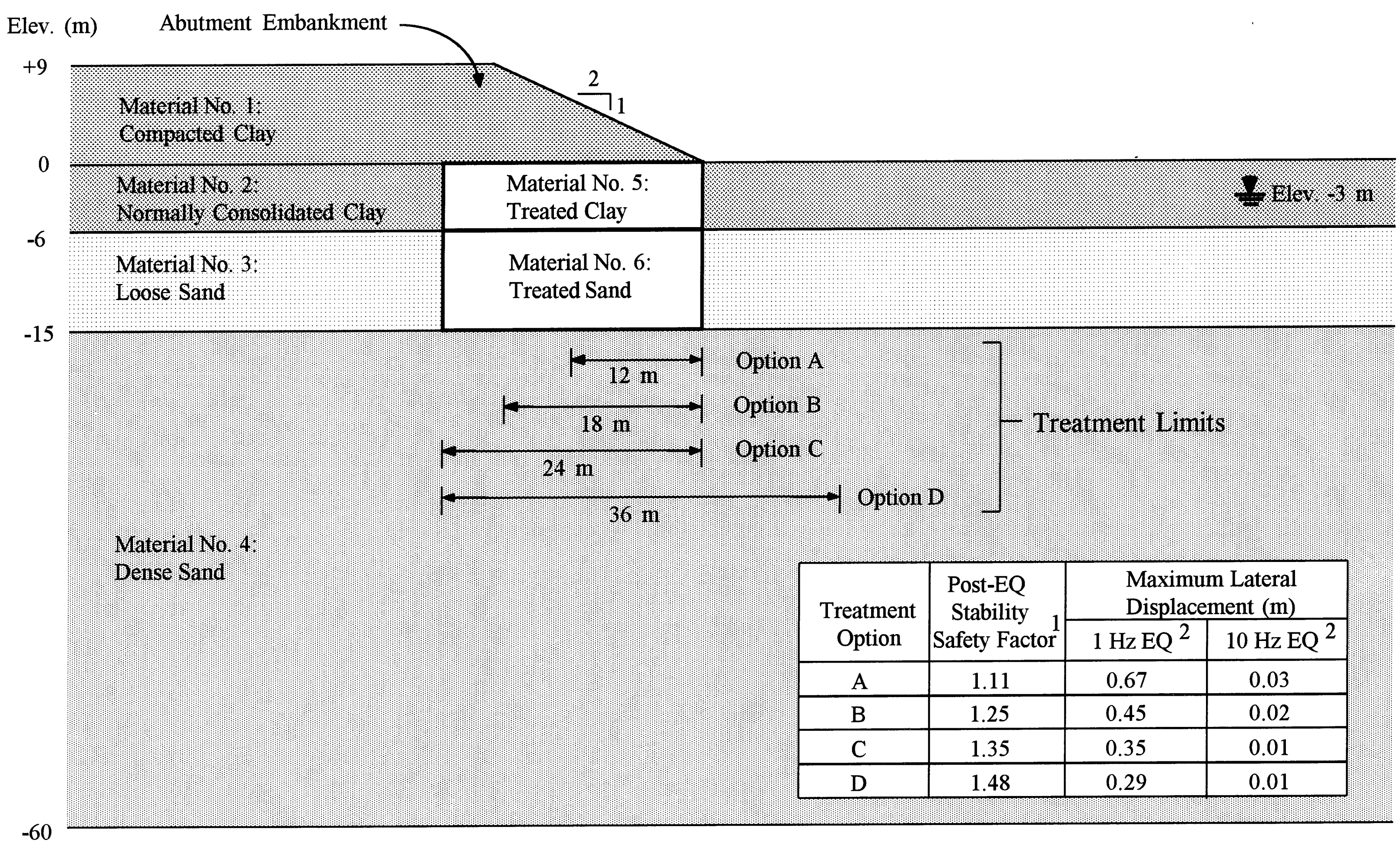
In addition to the effects of the treatment zone size and location, the results in Figures 5-12 and 5-13 indicate the importance that the predominant frequency of the earthquake can have on the induced displacements. A comparison of the displacements given in the figures for the 1 Hz and 10 Hz earthquakes, having scaled peak acceleration amplitudes of 0.22g, indicates that the higher frequency earthquake results in smaller predicted permanent displacements. However, this observation may not be true in cases where the higher frequency motion is closer to the natural frequency of the system than the lower frequency motion.



Material No.	SPT $(N_1)_{60}$	Unit Weight (kN/m^3)	K_2	Friction Angle (degrees)	s_{ur} (kPa)
1	20	19.6	60	0	47.9
2	5	18.8	20	0	23.9
3	15	19.6	49	25	19.2
4	30	20.4	62	40	191.5
5	20	19.6	54	0	47.9
6	30	20.4	62	35	134.1

Notes: 1. K_2 is factor used for calculating maximum shear modulus
2. s_{ur} is the post-seismic shear strength.

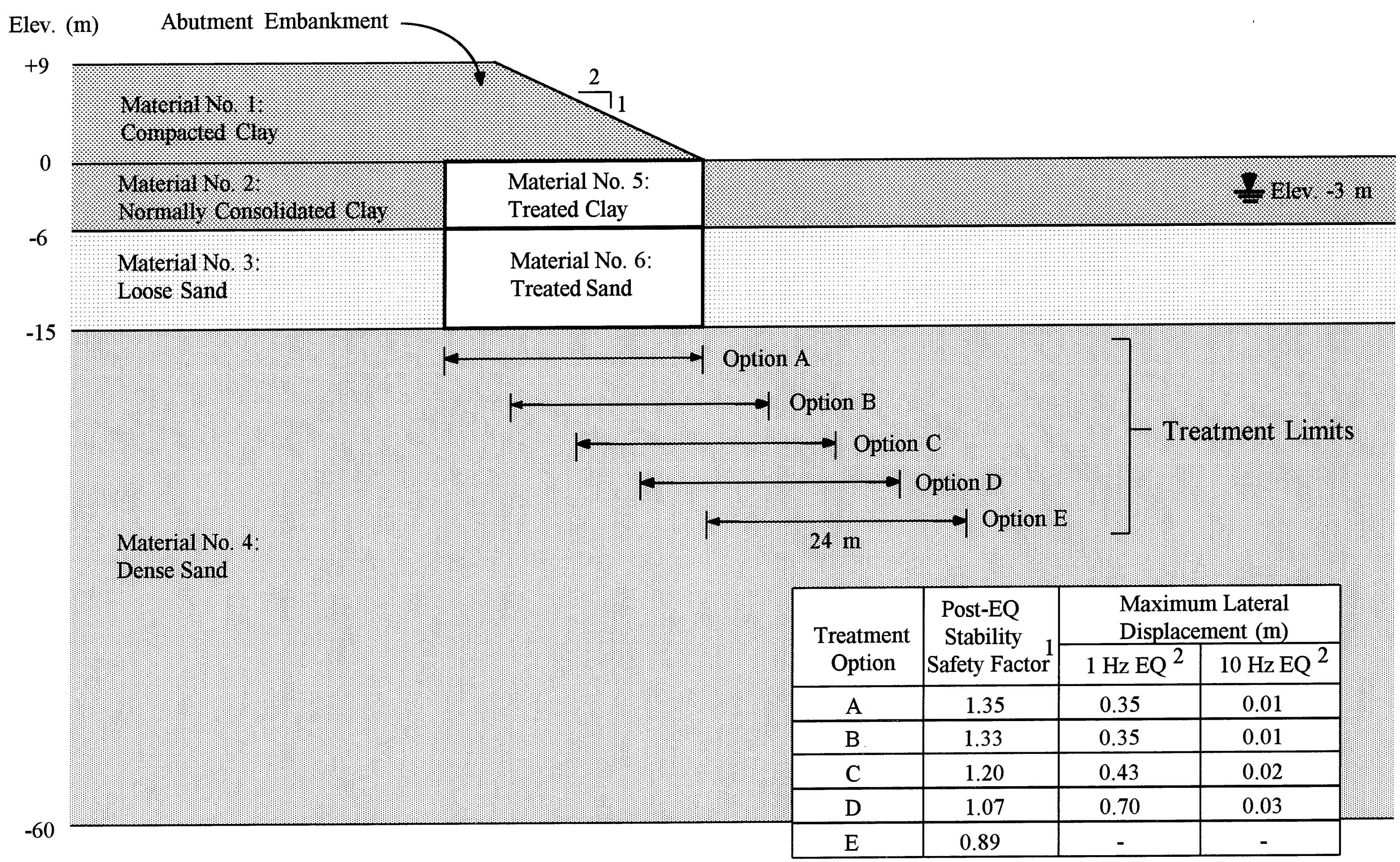
FIGURE 5-11 Soil Profile and Properties for Highway I-57 Bridge Used in Study by Riemer et al. (after Riemer et al., 1996)



Treatment Option	Post-EQ Stability Safety Factor[1]	Maximum Lateral Displacement (m)	
		1 Hz EQ[2]	10 Hz EQ[2]
A	1.11	0.67	0.03
B	1.25	0.45	0.02
C	1.35	0.35	0.01
D	1.48	0.29	0.01

Notes: 1. For no ground treatment the calculated post-earthquake factor of safety for stability was 0.71.
2. Input motions had durations of 8 to 10 seconds and peak amplitude scaled to 0.22g.

FIGURE 5-12 Impact of Treated Zone Size on Predicted Performance of Highway I-57 Bridge Abutment (after Riemer et al., 1996)



Treatment Option	Post-EQ Stability Safety Factor [1]	Maximum Lateral Displacement (m)	
		1 Hz EQ [2]	10 Hz EQ [2]
A	1.35	0.35	0.01
B	1.33	0.35	0.01
C	1.20	0.43	0.02
D	1.07	0.70	0.03
E	0.89	-	-

Notes: 1. For no ground treatment the calculated post-earthquake factor of safety for stability was 0.71.
2. Input motions had durations of 8 to 10 seconds and peak amplitude scaled to 0.22g.

FIGURE 5-13 Impact of Treated Zone Location on Predicted Performance of Highway I-57 Bridge Abutment (after Riemer et al., 1996)

The results in Figures 5-12 and 5-13 also show that trends in the predicted post-earthquake factor of safety for equilibrium (factors of safety presented in figures) for the different ground improvement schemes are consistent with those observed for the displacements, in that the higher the factor of safety, the smaller the deformation. However, the factor of safety can not be used as sole indicator of displacements because of its inability to capture the effects of input motion frequency.

5.5.1.2 Treatment Type

The impact of different treatment types on the performance of an embankment is illustrated by centrifuge test results from Adalier (1996) and summarized in Adalier et al. (1998). In these tests a 4.5-m-high (all quantities presented in prototype scale) clay embankment having side slopes of 1-horizontal to 1-vertical (1H:1V) was constructed on a saturated, 6-m-thick, loose sand layer. The performance of the embankment was investigated with the following types of improvement constructed at the toes of the embankment as illustrated in Figure 5-14: (1) none, (2) 6-m-wide densified zones, (3) 6-m-wide cement treated blocks, (4) 3-m-wide gravel berms, and (5) 0.15-cm-thick steel sheetpiles with tie rods. In all cases the base of each model was successively subjected to three 1.6-Hz sinusoidal acceleration records having magnitudes of 0.09g, 0.18g, and 0.3g. Each of the three records consisted of 10 uniform cycles of acceleration. During the tests, settlements, accelerations, and pore water pressures were monitored at various points in the foundation soils and on the embankment. Deformations were also measured at the end of the final shaking event.

The vertical and lateral deformations observed in each of the models after the third shaking event are illustrated in Figure 5-15. Table 5-2 presents approximate ratios of improved to unimproved (embankment) peak crest accelerations, crest settlements, and lateral toe displacements for the different remediation schemes based on data presented in Adalier (1996) and Adalier et al. (1998). In addition, the ratios of the peak embankment crest accelerations to input base accelerations are presented.

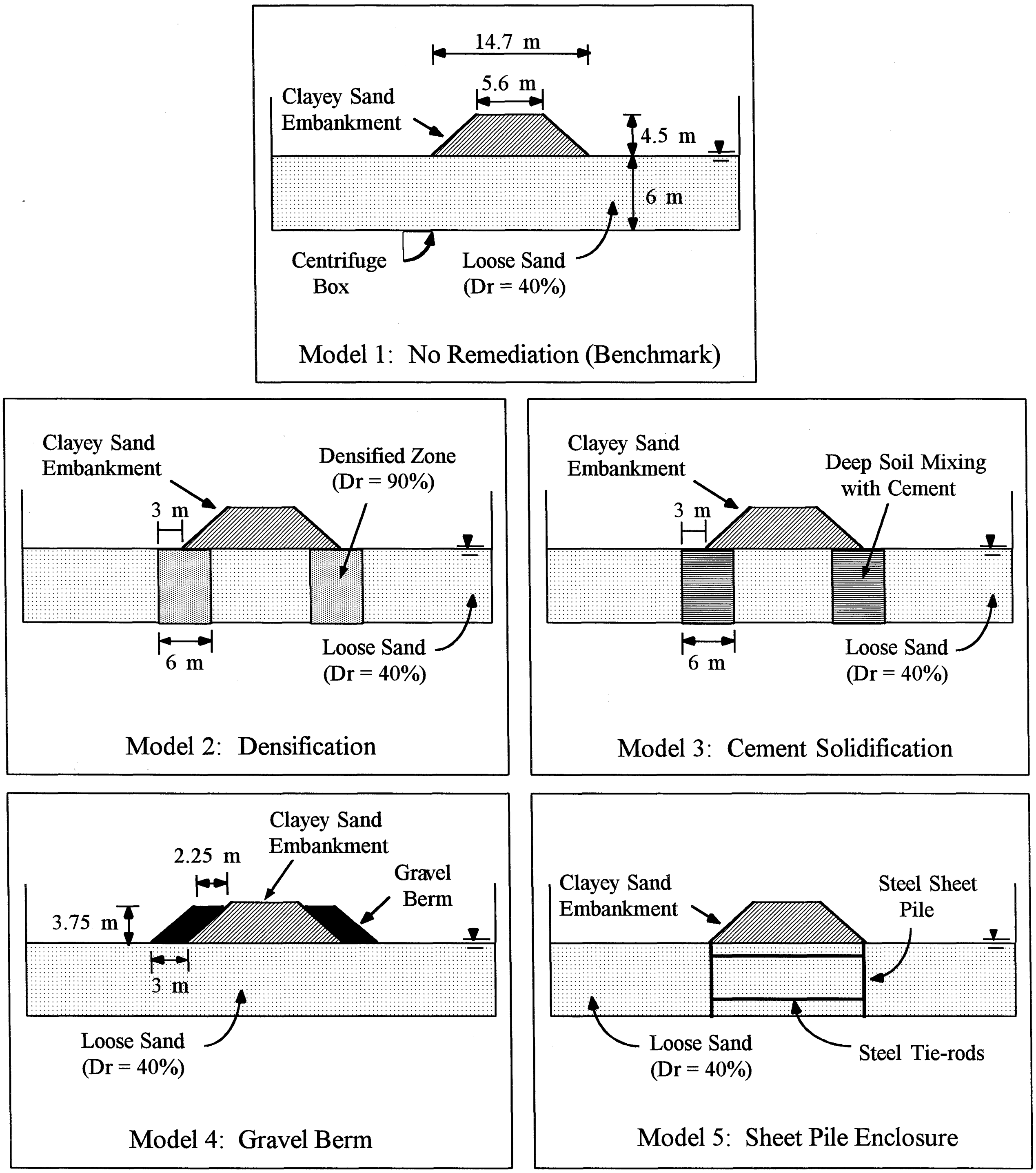


FIGURE 5-14 Improvement Schemes for Embankment Modeled in Centrifuge Tests by Adalier (after Adalier et al., 1998; dimensions in prototype scale)

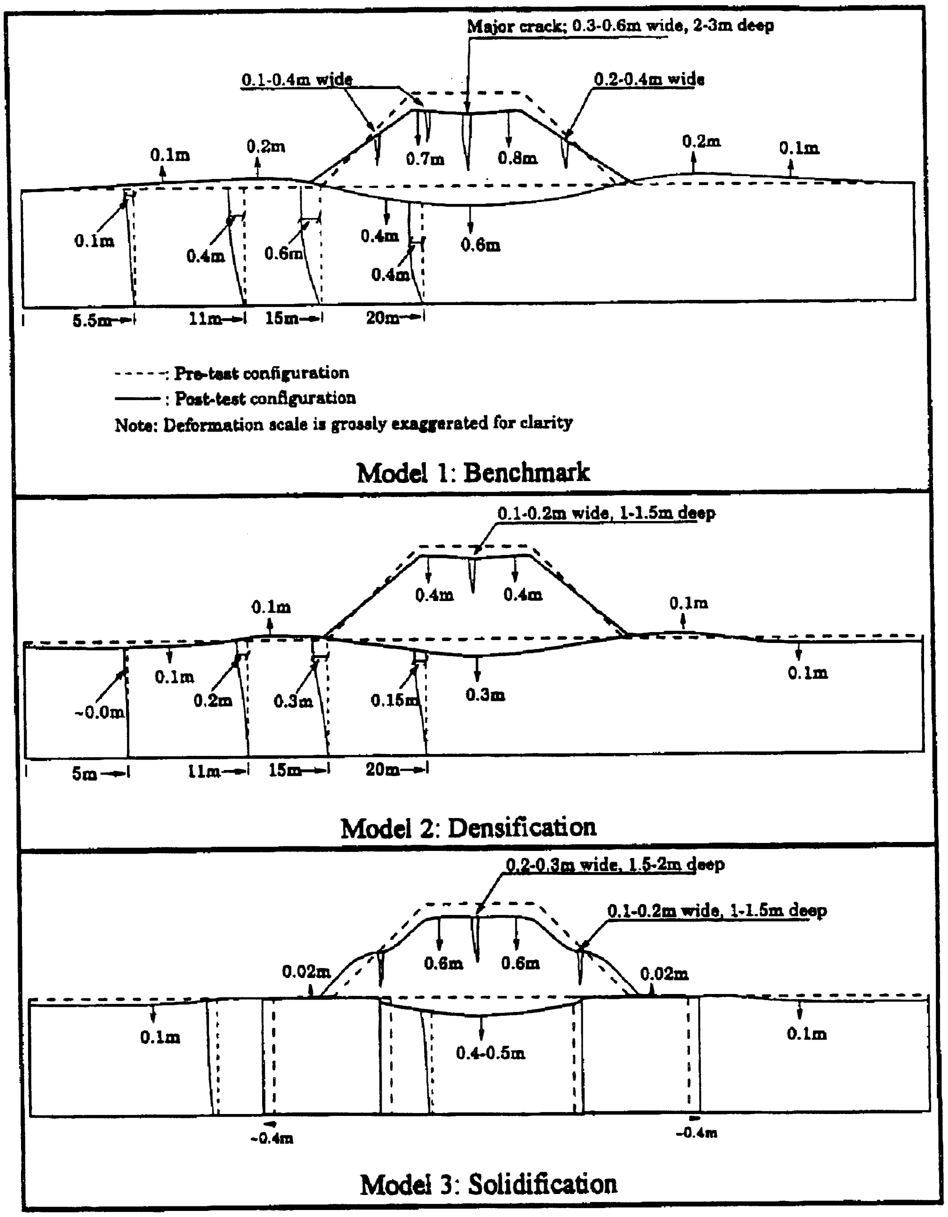


FIGURE 5-15 Ground Deformations (in Prototype Scale) Observed in Centrifuge Tests By Adalier (from Adalier et al., (1998). "Foundation Liquefaction Countermeasures for Earth Embankments," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 124, No. 6. Reprinted by permission of ASCE)

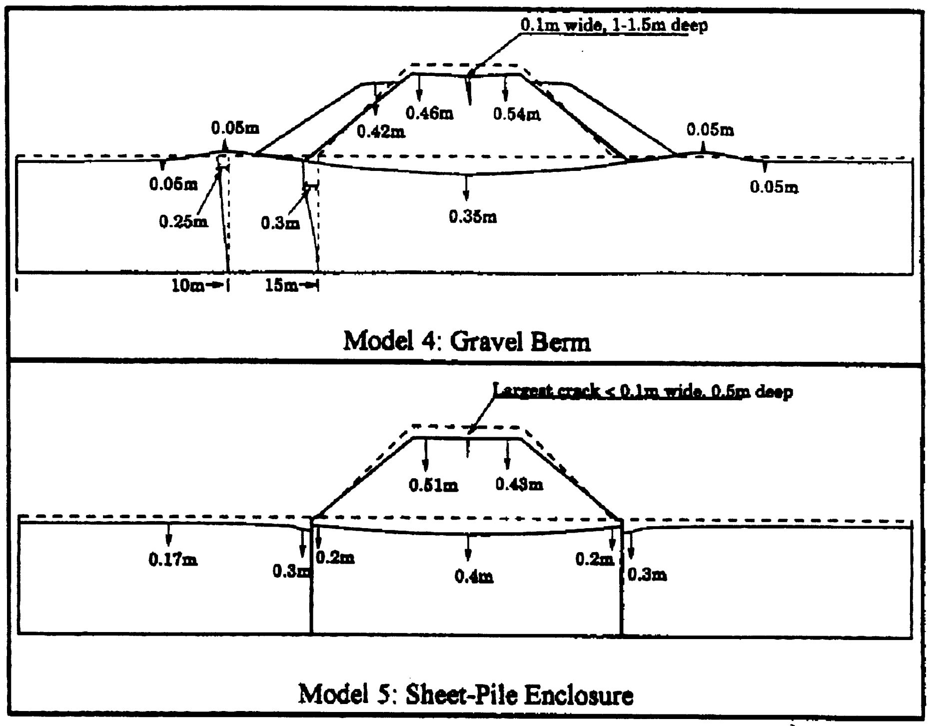


FIGURE 5-15 (cont.) Ground Deformations (in Prototype Scale) Observed In Centrifuge Tests by Adalier (from Adalier et al., (1998). "Foundation Liquefaction Countermeasures for Earth Embankments," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 124, No. 6. Reprinted by permission of ASCE)

TABLE 5-2 Comparison of Improved and Unimproved Embankment Response for Adalier et al. (1998) Centrifuge Tests

Improvement Type	Ratio of Crest to Base[1] Peak Acceleration[2]	Ratio of Improved to Unimproved Embankment		
		Peak Crest Acceleration[2]	Average Crest Settlement[3]	Lateral Toe Displacement[3]
None	0.9	-	-	-
Densified Zones	1.3	1.4	0.5	0.5
Cemented Blocks	1.5	1.6	0.8	NA[4]
Gravel Berms	1.1	1.2	0.7	0.5
Sheet Piles	0.7	0.7	0.6	0.0

Notes: 1. Third shaking event used with centrifuge container base acceleration amplitude of 0.3g.
2. Ratios of accelerations were computed using peak accelerations scaled from records for third shaking event presented in Adalier et al. (1998)
3. Ratios are for total deformations after all three shakes.
4. Large lateral deformations observed at toe but measured data not available (NA).

Adalier et al. (1998) make several observations about the effectiveness of the various ground treatment methods on improving the performance of the embankment. They note that if the intent of the ground improvement is to minimize lateral displacement and vertical settlement of the embankment, then the methods in order of decreasing effectiveness are: (1) sheetpiles with tie rods, (2) densification or gravel buttresses (the overall effectiveness of both were judged to be about the same), and (3) cement-treated blocks.

Adalier et al. (1998) note that the steel sheetpile enclosure (Model 5) resulted in the smallest cracking and slumping of the embankment, with the embankment basically retaining its original shape. This good deformation performance occurred despite the fact there was a sharp rise in the pore water pressure beneath the embankment during shaking. It can largely be attributed to the reinforcement and confinement provided by the sheetpiles which prevented lateral spreading. The peak acceleration measured at the embankment crest was the smallest (about 70 percent of the base input acceleration amplitude) with this improvement scheme. The low crest acceleration is attributed by Adalier et al. to liquefaction of the foundation soils directly beneath the embankment between the sheetpiles resulting in dynamic isolation of the embankment.

Densification (Model 2) was the most effective improvement method for reducing settlement of the embankment and limiting the development of excess pore water pressures beneath the embankment. However, measured peak accelerations at the embankment crest were about 1.3 times the base input acceleration amplitude.

The gravel berm (Model 4) improvement helped to improve embankment performance by providing lateral support for the embankment and increasing stress levels. Adalier et al (1998) also attribute the improved performance to the longer distance that liquefied soil beneath the embankment had to move to reach the free ground surface. Part of the improved performance was also attributed to the experimental effect of the reduced free space between the berm and centrifuge box wall, as compared to the cases without the berm. Despite the improved lateral deformation performance, the berm treatment case had higher embankment settlements relative to other improvement methods due to the berm weight. Measured peak accelerations at the embankment crest were about 1.1 times those of the base input amplitude.

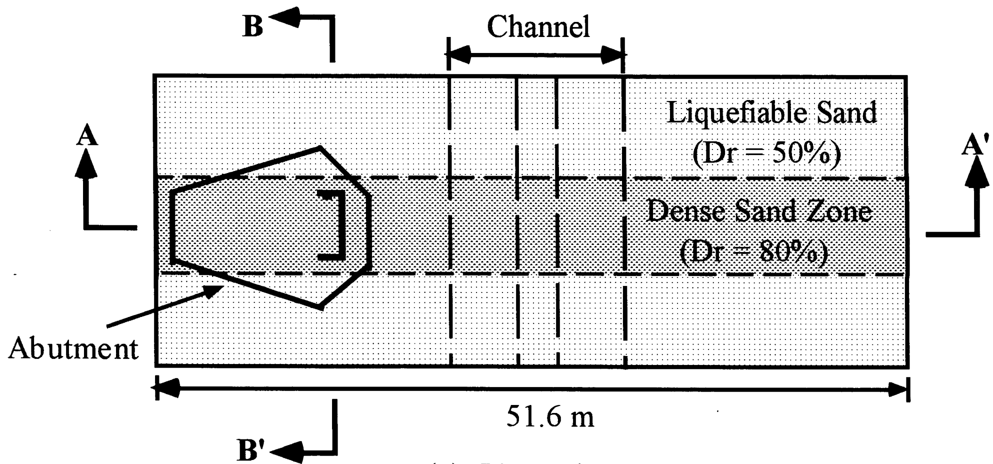
The use of cement treated blocks (Model 3) was reported to have a negligible effect on embankment deformations in comparison to the case with no improvement. The large lateral deformations observed were attributed to the high excess pore water pressures and low soil strength that developed beneath the embankment during shaking. However, Adalier et al. postulate that the performance with the cemented blocks would be better if the bottom of the blocks were embedded in an underlying firm soil layer as opposed to resting on the base of the centrifuge model box. The measured "peak" accelerations of the embankment crest with the cement block zones were about 1.5 times those of the base input amplitude of 0.3g.

Based on the results presented above, the deformation performance of an abutment and approach embankment supported on liquefiable soils will improve in relation to the degree of confinement provided by a given ground improvement method. The degree of confinement increases as the stiffness of the confining system increases. Confining systems installed at the toe of the embankment appear to be effective provided the system is rigid and there is no opportunity for the liquefied soil to flow around the system. As noted in the previous section, ground improvement becomes less effective when it is performed outside the embankment. Accelerations at the top of an embankment can be reduced if high pore water pressures develop in the soil within the confining system, but this also results in larger settlements.

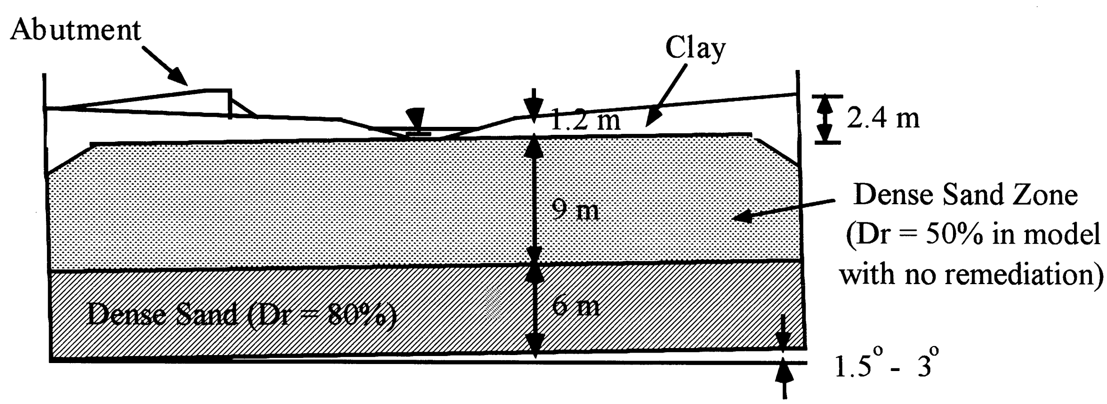
Although the improvement methods and configurations used in the centrifuge tests did improve the model embankment performance, as illustrated by the ratios presented in Table 5-2, it is important to note that the measured deformations and accelerations were still relatively large (refer to values given in Figure 5-15). These data illustrate that reducing liquefaction-induced ground deformations to acceptable levels may require more than one improvement type. The data also indicate that: (1) there may be a limit to the reduction in deformations and accelerations that can be achieved and (2) a particular type of improvement may be effective for improving one ground response (i.e. - acceleration, deformation, or pore water pressure) but less effective in improving others.

5.5.1.3 Effects of Non-Liquefiable Surface Layer

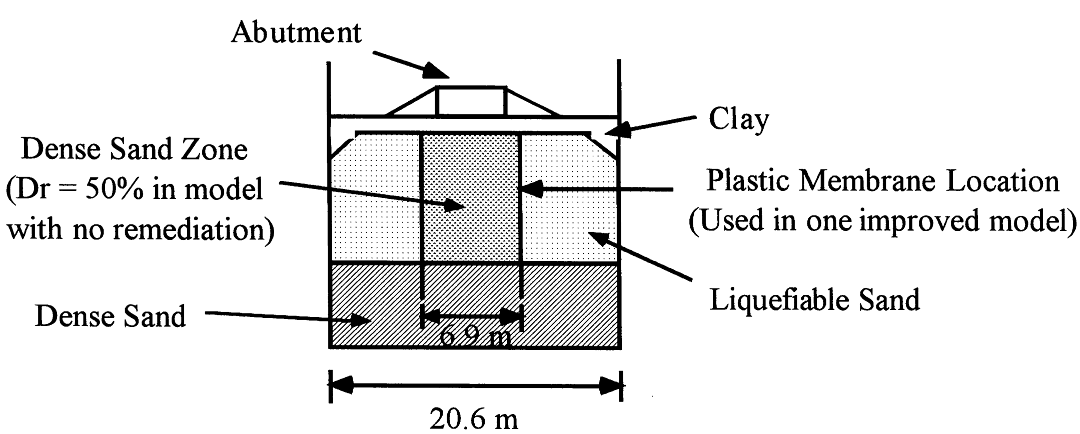
Balakrishnan et al. (1998) performed a series of five centrifuge tests involving liquefaction and lateral spreading. The surficial soil in the model consisted of a non-liquefiable clay layer sloping up to 3 degrees toward a channel in the middle of each model. The clay layer was underlain by liquefiable, saturated fine sand (Nevada sand at 50 percent relative density) over dense sand (Nevada sand at 80 percent relative density), as shown schematically in Fig. 5-16. In some of the models, abutments were constructed on top of the clay layer, on one or both sides of the channel, with the ground surface on the abutment side having little to no slope. Three of the models had no improvement within the liquefiable sand and two of the models had dense sand zones (Nevada sand at 80 percent relative density) within the liquefiable zone. The dense sand zone extended the full depth of the liquefiable zone and the entire length of the model box, running under the abutments and slopes on both sides of the channel. The dense zone width was only slightly larger than the abutment width. In one of the models, the dense sand zone was enclosed in a plastic membrane to reduce pore water pressure migration into the zone. The models were subjected to



(a) Plan View



(b) Section A-A'



(c) Section B-B'

Note: Dimensions in prototype scale

Figure 5-16 Schematic of Centrifuge Test with Improved (Dense) Zone by Balakrishnan et al. (after Balakrishnan et al., 1998)

a series of simulated earthquakes, primarily by applying scaled records of the ground motion measured at 82 meter depth on Port Island during the 1995 Kobe earthquake in Japan. Peak acceleration amplitudes of the records ranged from approximately 0.15g to 0.7g.

According to Balakrishnan et al. (1998), the models with the improved zones within the liquefiable soil layer generally had reduced settlements, lateral deformations, and pore water pressures in comparison to the models with no improvement. They report that the improved zone settlements were reduced by a factor of 2 to 4 compared to the zones where there was no improvement. Lower settlements (20 to 40 percent lower) were observed in the model where the dense zone improvement was enclosed in a plastic membrane as compared to the case where no plastic membrane was used. The potential reason indicated for the lower settlements with the membrane was the reduction of pore pressure migration into the dense zone.

Ground improvement using a dense zone was less effective in reducing the lateral deformations (treated zone deformations were about 25 percent less than untreated zones) measured in the centrifuge tests. The reduced effectiveness of the ground improvement in mitigating lateral deformations in these tests was attributed by Balakrishnan et al. (1998) to the improvement not extending through the overlying clay, thereby leaving a continuous clay layer over the treated and untreated zones (Figure 5-16b and c). This configuration resulted in the clay over the treated zone being dragged along with the portion over the untreated zone. They conclude this indicates a need to tie an unliquefiable surficial soil layer to the underlying improved zone to prevent it from moving with the surrounding laterally spreading soil. In addition they note that although a narrow width of treatment was effective in reducing settlements, it likely needed to be wider to reduce the lateral deformations.

5.5.2 Shallow Foundation Performance

5.5.2.1 Treatment Width

Hatanaka et al. (1987) investigated the effect of the width of a densified sand zone within a liquefiable sand on the settlement of a supported structure by performing shaking table tests. The basic model configuration is illustrated in Figure 5-17. The model structures consisted of concrete blocks having widths of 10, 20, or 30 cm. These structures were supported on densified sand zones (D_r = 90 percent) having a thickness of 15 cm and varying width, which were constructed at the surface of a 40-cm-thick liquefiable sand layer. The models were then subjected to a 2-Hz sinusoidal acceleration record having a uniform peak amplitude of approximately 0.1g for a total duration of about 10 seconds.

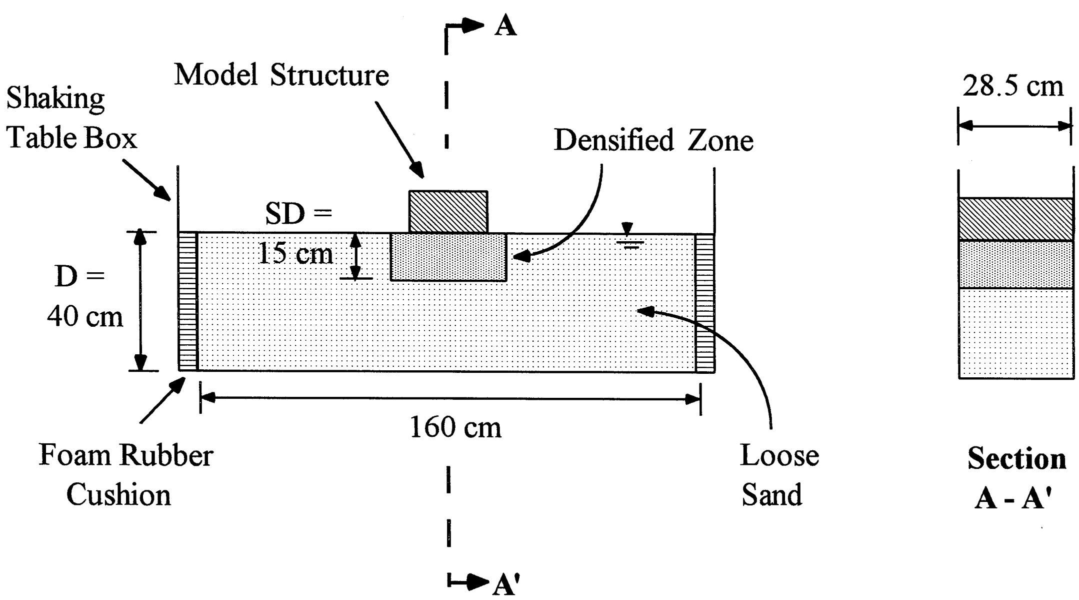


FIGURE 5-17 Sections Showing Structure on Densified Zone Modeled in Shaking Table Tests by Hatanaka et al. (after Hatanaka et al., 1987)

Figure 5-18 developed by Hatanaka et al. from the test results shows that, for a given structure width, increasing the ratio of the treatment zone width (SW) to the structure width (BW) results in a decrease in the settlement ratio of the structure (measured settlement of structure, S, divided by the liquefiable sand layer thickness, D). As noted by Hatanaka et al. and shown in Figure 5-18, for a given SW to BW ratio the settlement ratio is smaller for a wider structure due to the increased confinement provided by a wider structure.

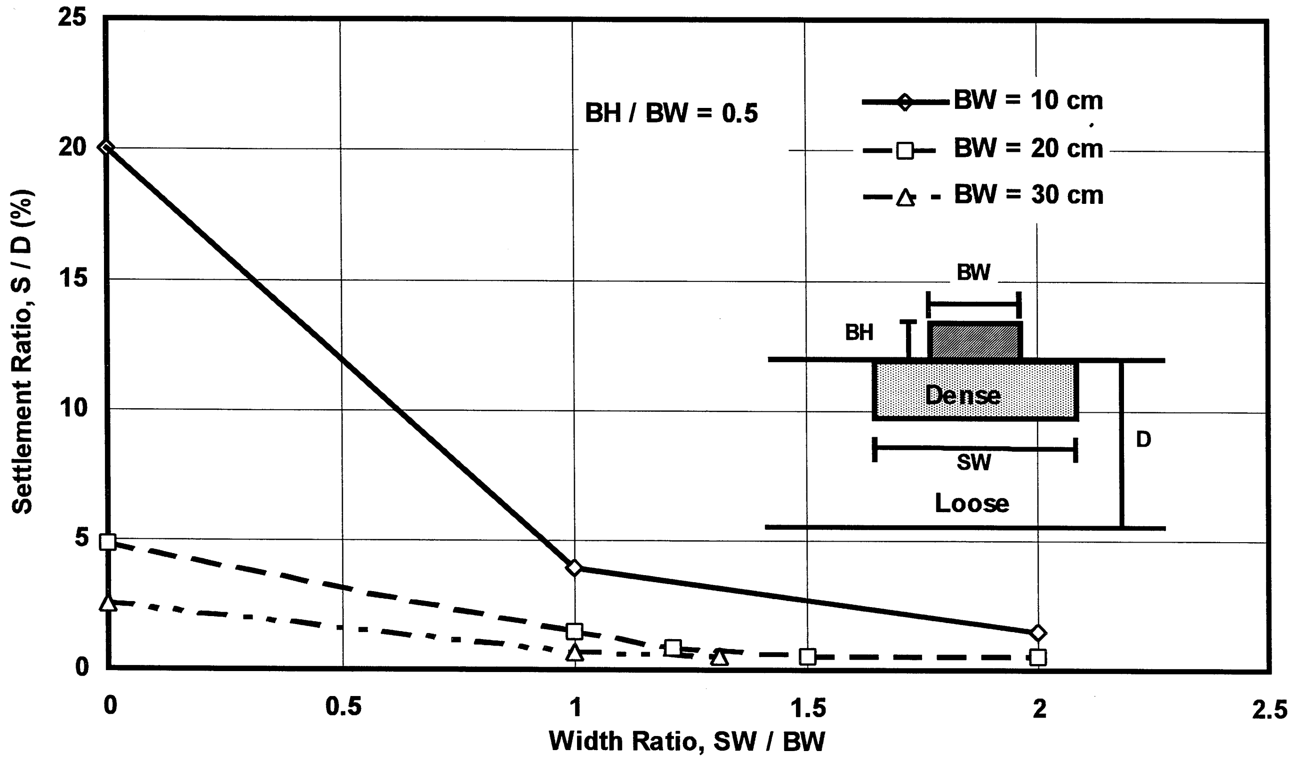


FIGURE 5-18 Relationship Between Settlement and Width Ratios for Improved Zone in Shaking Table Tests (after Hatanaka et al., 1987)

Figure 5-19 presents the settlement ratio of the structure, S/D, versus the ratio of the treatment distance beyond the edge of the structure, SL, to the treatment zone depth, SD. The observed trends in this figure are similar to those in Figure 5-18. Both Figures 5-18 and 5-19 indicate there may be an optimum treatment width beyond which the decrease in structure settlement is minimal. Figure 5-19 indicates that this optimal treatment width may be on the order of 0.5 times the treatment depth or greater, and is dependent in part on the width of the supported structure.

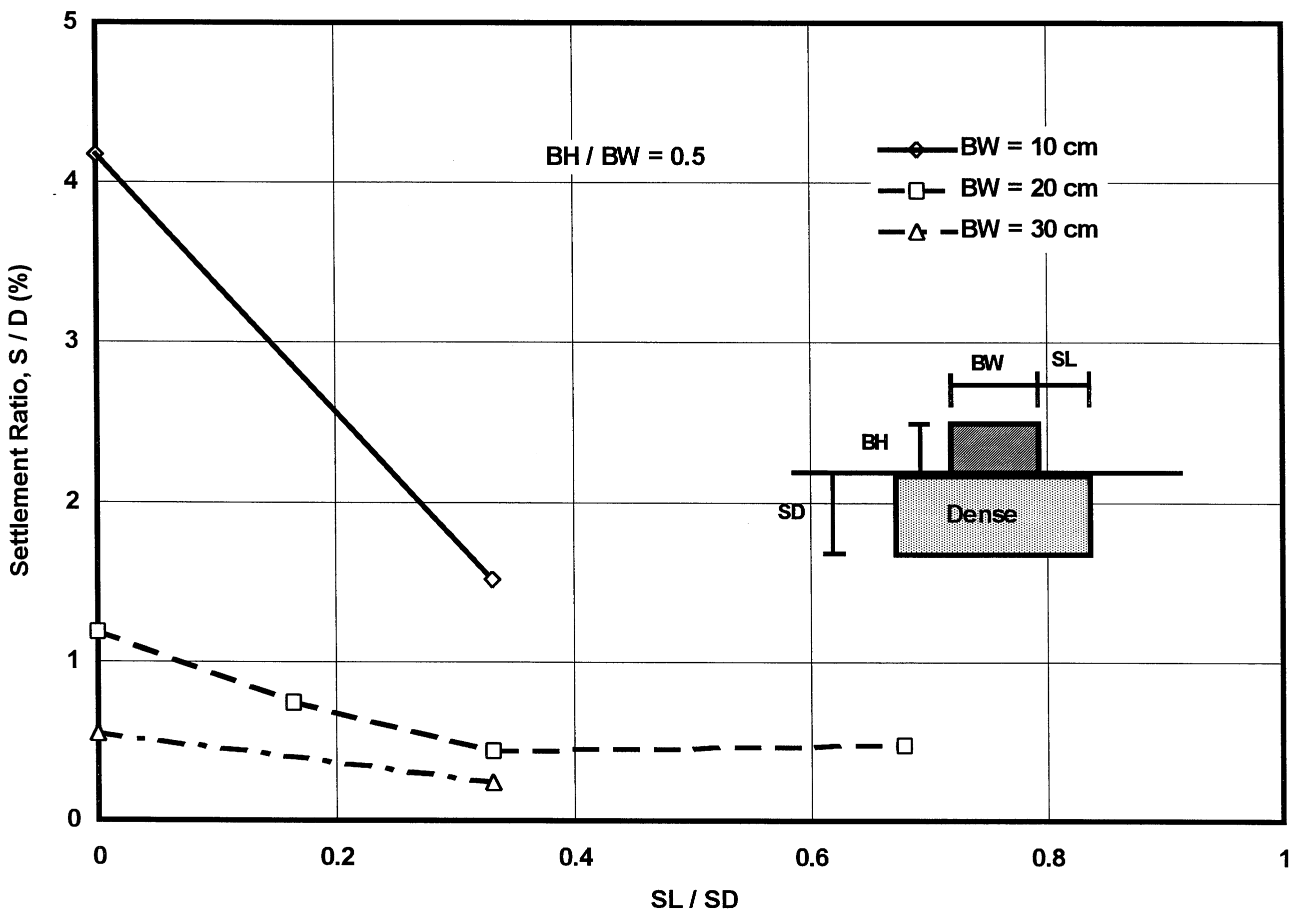


FIGURE 5-19 Relationship Between Settlement Ratio and Ratio of Treatment Width Beyond Structure to Treatment Depth for Improved Zone (after Hatanaka et al., 1987)

Hatanaka et al. also investigated the effect of structure height (BH) to width (BW) ratio on settlement ratio for cases with no ground densification. Figure 5-20, which they developed based on experimental results, indicates that as the height to width ratio, BH/BW, increases the settlement ratio for the structure also increases. Such an effect may also be applicable to structures supported on improved ground, although no experimental evidence is provided in the paper to directly support this hypothesis.

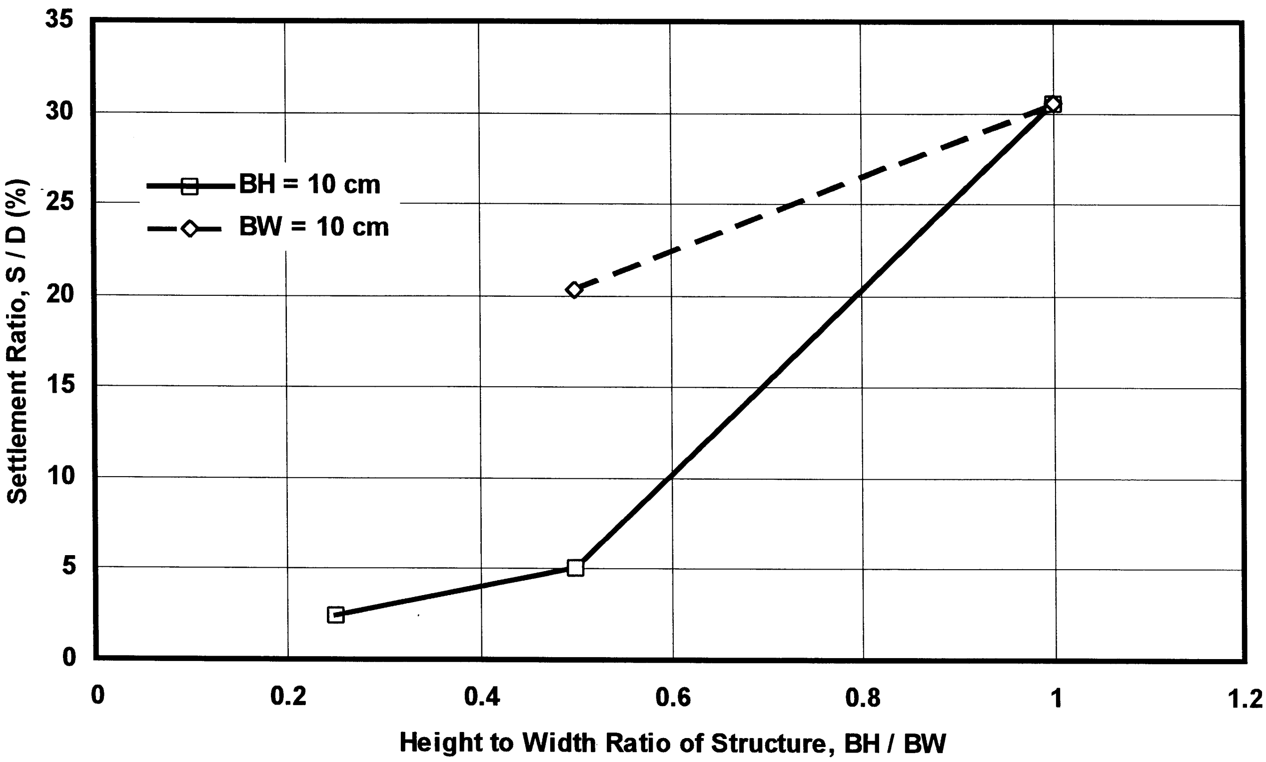
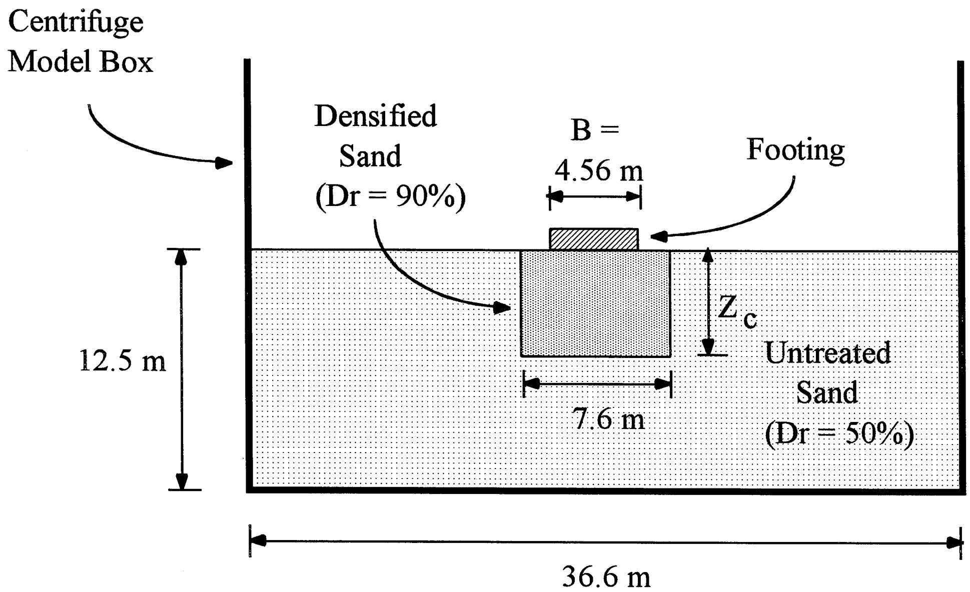


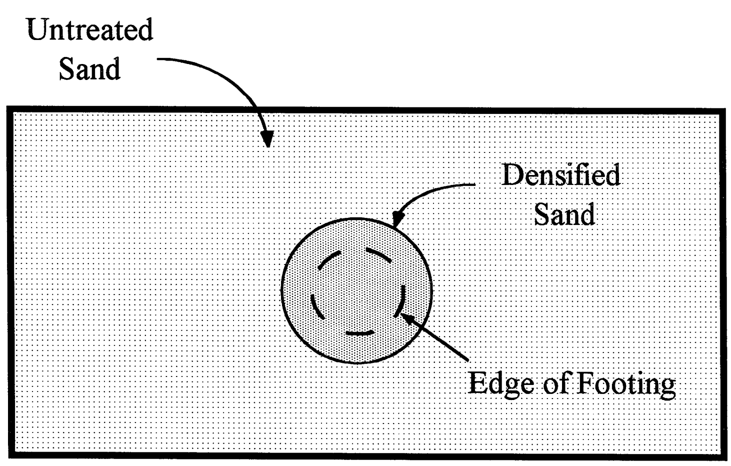
FIGURE 5-20 Relationship Between Settlement Ratio and Height to Width Ratio of Structure on Unimproved Soil (after Hatanaka et al., 1987)

5.5.2.2 Treatment Depth

The effect of treatment depth on the response of a footing supported on a densified sand zone within a liquefiable sand deposit was investigated by Liu and Dobry (1997) using centrifuge tests. In the tests a circular steel footing having a height of 0.8 m (all quantities presented in prototype scale), diameter of 4.56 m, and exerting a bearing pressure of 100 kilopascals, was supported on the surface of a circular densified sand zone (D_r of approximately 90 percent) having a diameter of 7.3 m (1.6 times the footing diameter) and varying thickness. The footing and densified zone were located within a 12.5-m-thick liquefiable sand layer having a relative density of about 50 percent and a permeability of approximately 0.168 centimeters per second. The models were subjected to a 10-cycle, 1.5 Hz uniform sinusoidal acceleration record having an amplitude of 0.2g applied at the base. A schematic of the model set up is shown in Figure 5-21.



Profile



Plan View

FIGURE 5-21 Centrifuge Model of Footing on Densified Zone Tested by Liu and Dobry (after Liu and Dobry, 1997; dimensions in prototype scale)

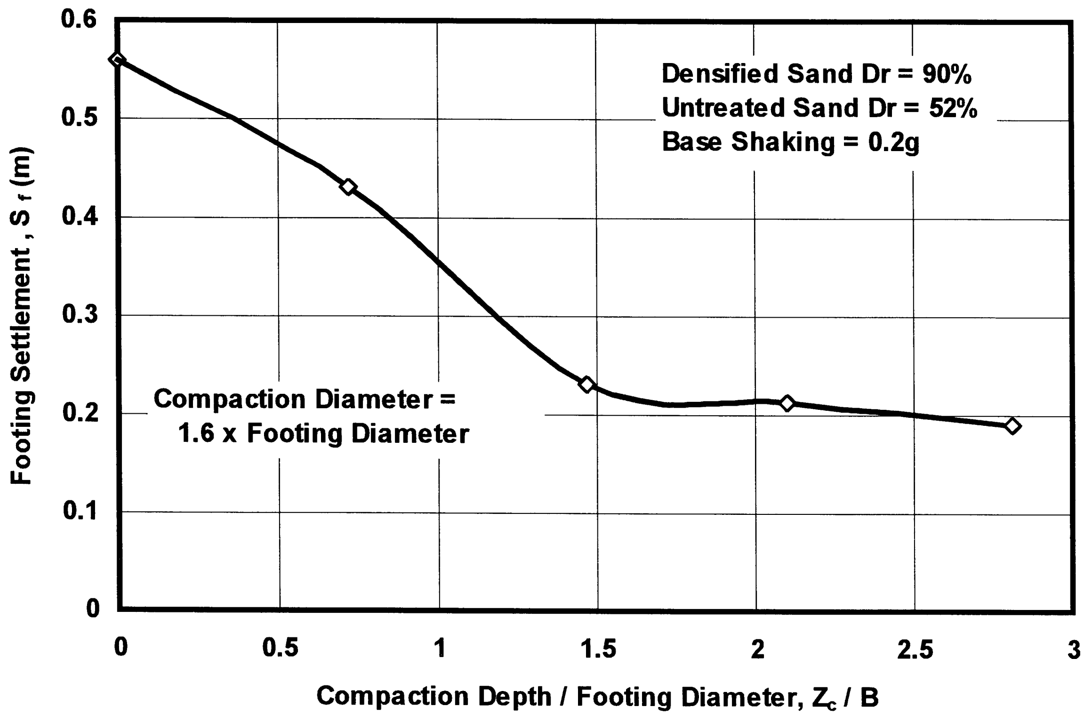
Figure 5-22(a) by Liu and Dobry presents the measured footing settlement, S_f, versus the ratio of the compaction depth, Z_c, to footing diameter, B. The results in this figure indicate that the magnitude of footing settlement can be reduced by increasing the depth of treatment beneath the footing. This conclusion is also supported by Figure 5-22(b) where the footing settlement has been normalized by the settlement of the surrounding soil in the free-field, S_s, after liquefaction and dissipation of excess pore water pressures. Both Figures 5-22(a) and 5-22(b) suggest that there may be an optimum depth of treatment beneath a footing beyond which the reduction of the footing settlement is relatively minor. Liu and Dobry note that most of the observed footing settlement occurred during shaking and is mostly due to the inertial load acting on the footing.

Although increasing the depth of densification beneath the footing resulted in decreased settlement of the footing, it also had the adverse effect of increasing the peak footing acceleration. Figure 5-23 by Liu and Dobry, which presents the normalized footing acceleration (for both the peak footing acceleration and the footing acceleration at the sixth cycle) versus the normalized compaction depth, shows this trend.

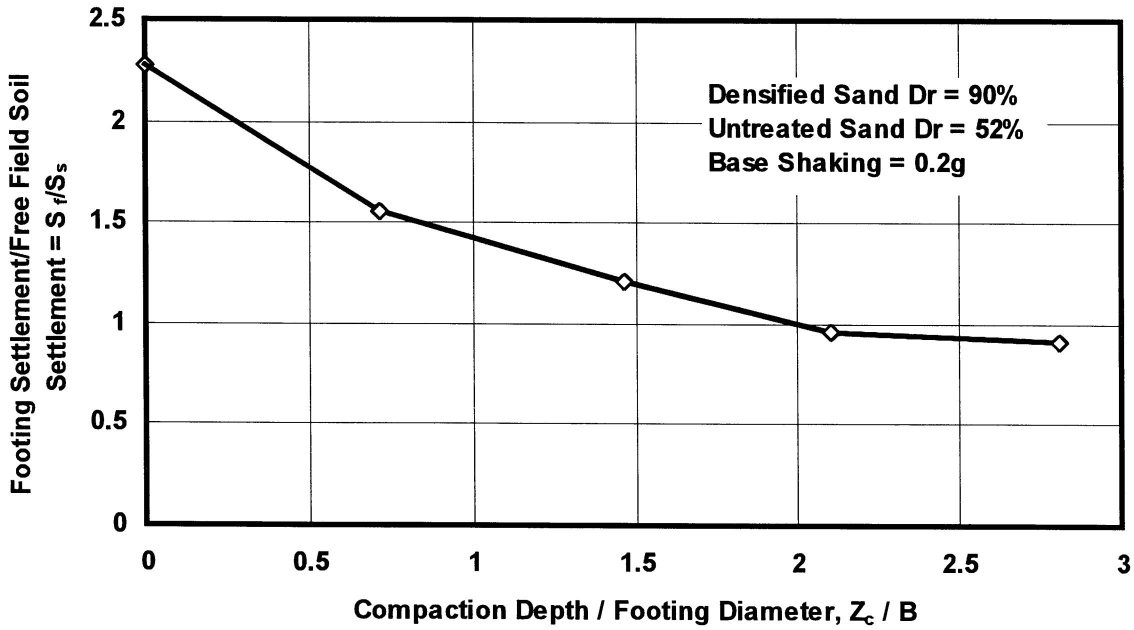
The variation of footing settlement and acceleration with densification depth suggests it may be necessary to optimize the treatment depth to reduce the footing settlement to an acceptable level while at the same time ensuring that the footing acceleration is tolerable.

5.5.3 Deep Foundation Performance

Many bridge piers and abutments are supported on deep foundations that extend completely through liquefiable soil deposits Munfakh et al. (1998) note that pile-supported structures have performed well in recent earthquakes (e.g. 1989 Loma Prieta and 1995 Kobe earthquakes) in areas where liquefaction occurred. However, there are also cases from past earthquakes where the structures have performed well, but the deep foundations have been damaged, and where both the structures and deep foundations have been damaged. Damage to bridges on deep foundations has been particularly high in areas where lateral spreading has occurred, as supported by case history information from several earthquakes (Ross et al., 1969; Bartlett and Youd, 1992; Youd, 1993; Berrill et al, 1997).



a) Settlement Not Normalized



b) Settlement Normalized

FIGURE 5-22 Foundation Settlement versus Normalized Compaction Depth for Centrifuge Test (after Liu and Dobry, 1997)

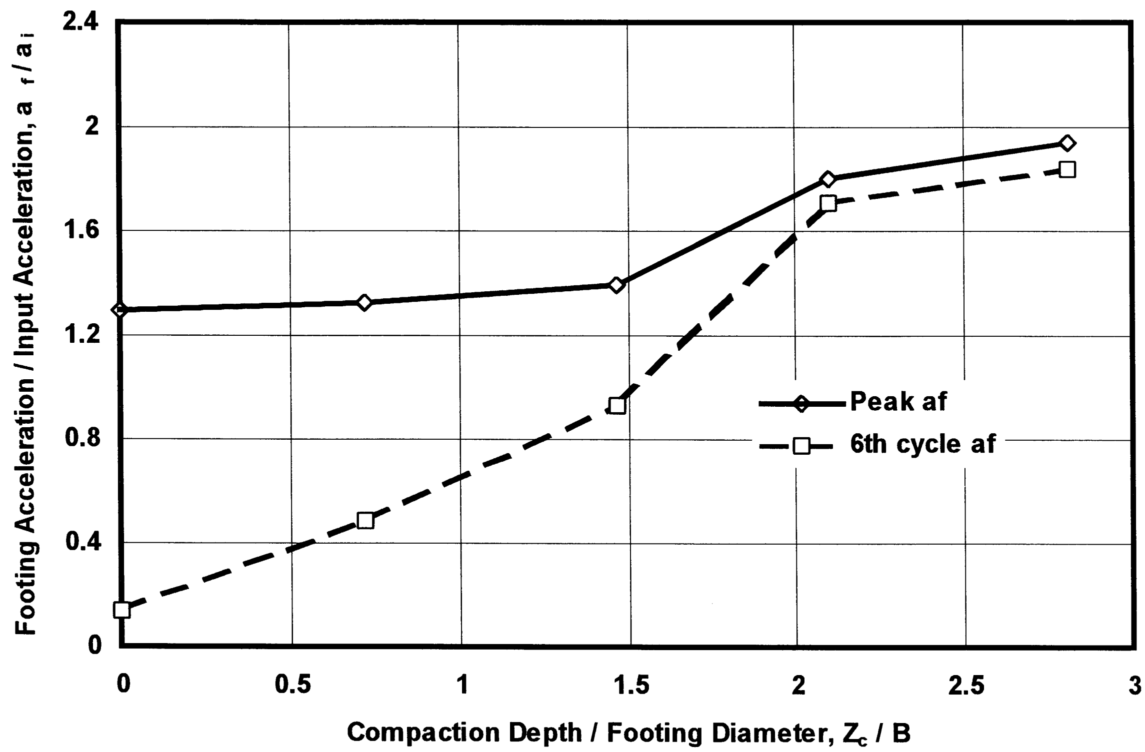


FIGURE 5-23 Normalized Footing Acceleration versus Normalized Compaction Depth Illustrating Amplification Effects (after Liu and Dobry, 1997)

Centrifuge model and field case history information are not available on the performance of deep foundations in improved ground zones during earthquake loading and liquefaction. However, there are a few examples of the use and potential performance of deep foundations in improved zones. One example of improved ground design for piles is briefly described by Jackura and Abghari (1994). Stone columns were used at a new bridge in San Diego, CA, to create "islands" of densified soil approximately 30 meters in diameter to provide support (and prevent failure) during an earthquake for bridge pier piles extending through liquefiable sands, as shown schematically in Figure 5-5. The piles were evaluated for design loads assuming the densified sand had a friction angle of 30 degrees. The principal mechanisms cited for improved support of the piles are the densification and improved drainage effects of the stone columns. Since little to no lateral spreading was anticipated at the site, Jackura and Abghari state that any forces exerted by laterally moving soil were assumed to be absorbed by the perimeter of the improved zone; therefore they were not considered in design.

Fukutake et al. (1996) evaluated the seismic performance of a hypothetical pile foundation (supporting an upright cylinder structure) in a densified zone or surrounded by a soil-cement wall within a liquefiable soil stratum. In their evaluation they used three-dimensional finite element analyses and an effective stress soil model. Their results indicate a substantial decrease in the shear force, bending moments, and displacements of the piles when compared to analyses performed where no improved soil was used.

Further research is needed into the performance of deep foundations in liquefiable soil deposits and the benefits of using ground improvement to reduce the risk of damage to deep foundations, particularly in areas prone to lateral spreading.

5.6 Implications for Design

As indicated in the discussion above, only certain types of ground and foundation improvement will be suitable for particular bridges. When selecting potential improvement methods consideration must be given to the conditions that exist at a particular bridge and the potential impact of the improvement method on the bridge during construction. These factors may render certain improvement types unsuitable.

The type and configuration of the improvement will have a large impact on the resulting performance of a bridge during earthquake shaking and liquefaction. Results from numerical analyses, shaking table tests, and centrifuge tests clearly indicate that the type, size, and location of the improvements affect the performance of the supported structure. The experimental data also indicate there may be a limit to the change in ground and supported structure performance that can be achieved using available remediation methods. Additionally, a treatment method may be effective in improving one aspect of bridge performance, such as lateral movement, but not as effective in improving another aspect, such as settlement. Minimizing the effect of one factor, such as treated ground displacement, can potentially have the adverse effect of maximizing another factor, such as the supported pier acceleration. Therefore, remedial measure designs will need to strike a balance in satisfying the various criteria important to the bridge performance. Adoption of a logical design procedure that recognizes these needs and incorporates them in the design process is critical, as discussed in the next section.

SECTION 6
REMEDIAL MEASURE DESIGN CONSIDERATIONS

6.1 Introduction

Once the need for ground or foundation improvement to mitigate liquefaction effects at an existing highway bridge has been established using the procedures discussed in Section 4, selection and design of potentially feasible improvement methods are necessary. A wide variety of remedial measures for liquefaction mitigation at existing bridges are available, as indicated in Section 5. However, only a few of these methods will likely be feasible for mitigation at a bridge based on the various constraints that exist. A well-defined, logical design procedure is needed to proceed from (1) the determination of improvement measures that should be considered for remediation to (2) design evaluations and finally to (3) selection of a preferred improvement scheme. Such a procedure is presented below, along with a discussion of the critical design issues and some general design guidelines.

6.2 Design Issues

6.2.1 Stability and Deformations

As discussed in Section 3, the stability and deformations of the foundations supporting an existing bridge under earthquake and post-earthquake conditions are critical to the bridge performance. For this reason, foundation stability and deformations are key issues in the design of remedial measures to support bridges within liquefiable soil deposits. Any improvement method must provide adequate support such that the bridge foundations will be sufficiently stable and experience movements which are compatible with the desired performance of the bridge. In addition, the supported bridge must be capable of withstanding accelerations transmitted by the improved ground or foundations.

Several factors influence the stability, deformation, and acceleration of improved ground zones and supported structures during and after an earthquake. These factors must be explicitly or implicitly incorporated in the design of improved ground zones for liquefaction mitigation such that reliable predictions of the improved ground and supported structure performance are obtained. The factors, which are illustrated schematically in Figure 6-1, include:

- Pore pressure migration (Fig. 6-1a): During and after an earthquake, pore water pressures will be higher in a liquefiable zone than in an adjacent improved zone. The difference in pressures between the two zones induces flow into the treated zone, causing an increase in pore water pressure and a potential decrease in strength.

- Ground motion amplification (Fig. 6-1b): The earthquake-induced ground motion at the base of an improved ground block may be amplified as shear waves travel upward through the block, resulting in more severe loading on the supported structure.

- Inertial force phasing (Fig. 6-1c): During an earthquake, differences may exist between the times when the maximum inertial forces act on the improved ground and supported structure. The phasing which develops is a complex problem in soil-structure interaction.

- Dynamic fluid pressures (Fig. 6-1d): Movement of a relatively stiff body, such as an improved ground zone, within a fluid medium, such as liquefied soil, may result in a dynamic component of fluid pressure acting on the body in addition to the static fluid pressure.

- Influence of structure: Placement of a structure on an improved ground zone alters the stress state within the zone and can thereby effect the dynamic response and stress-strain behavior of the improved soil.

- Lateral spreading forces (Fig. 6-1e): In areas of sloping ground, additional pressures/forces can be exerted on the improved ground zone as surrounding unimproved soil undergoes lateral spreading and moves relative to the treated zone.

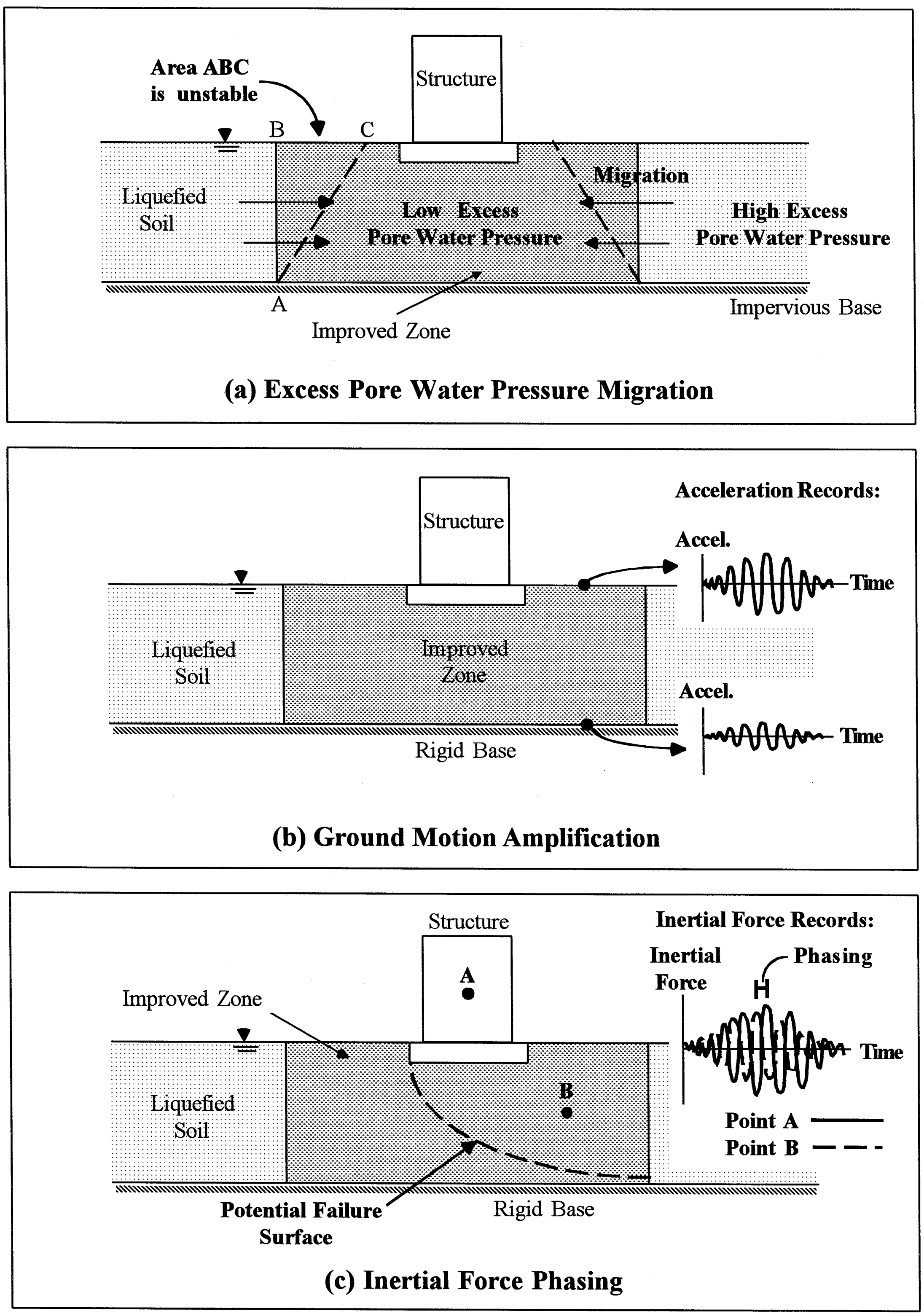
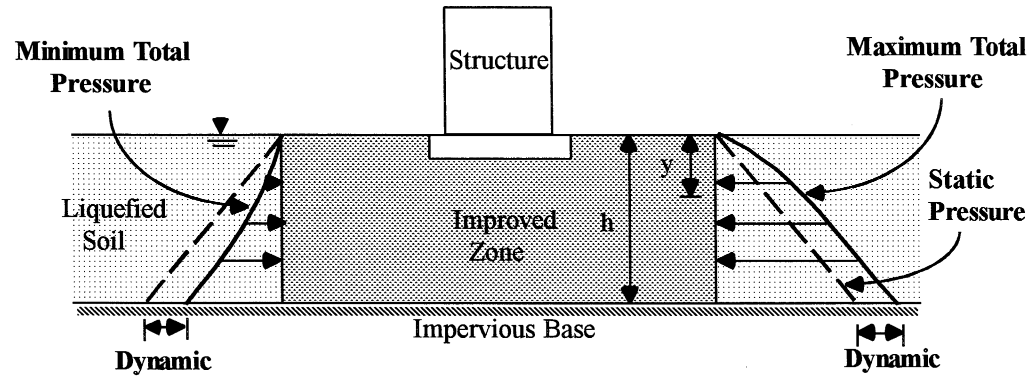


FIGURE 6-1 Factors Affecting Improved Ground Performance (after Mitchell et al. (1998), "Design Considerations in Ground Improvement for Seismic Risk Mitigation," Geotechnical Earthquake Engineering and Soil Dynamics III, Geotechnical Special Publication No. 30. Reprinted by Permission of ASCE.)

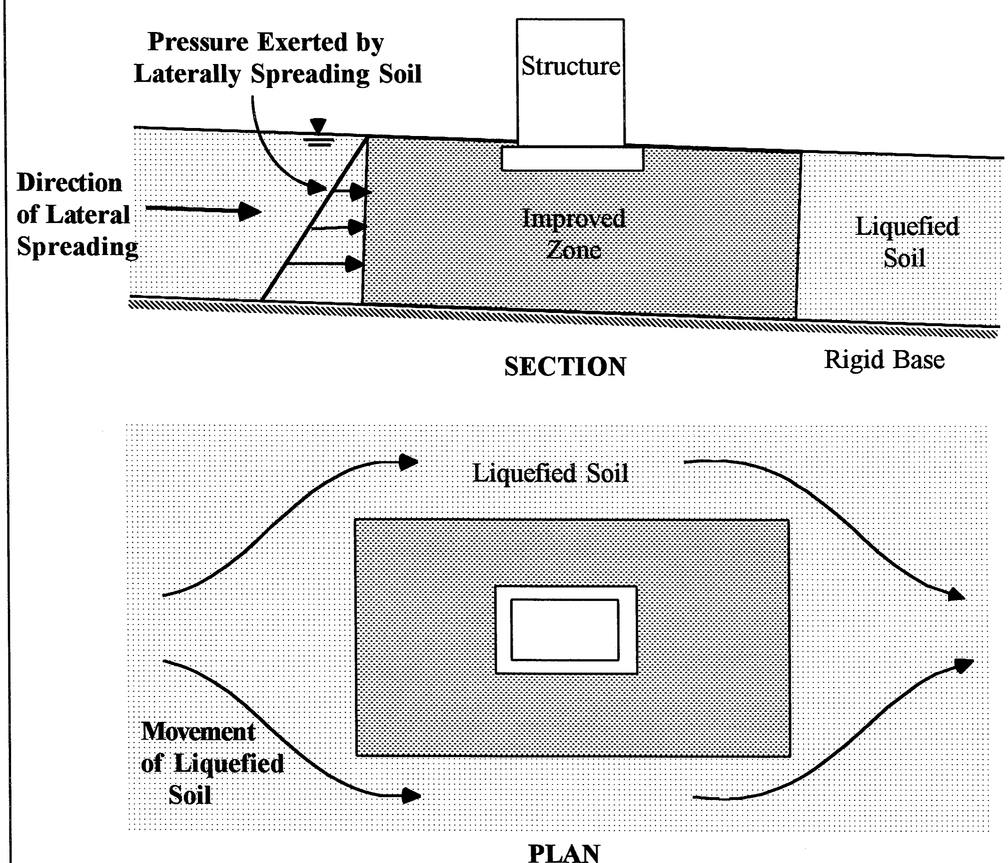


Static Pressure = γ h

Dynamic Pressure = $\frac{7}{8}\frac{a}{g}\gamma\,(y\,h)^{0.5}$

where: a = maximum acceleration
g = gravitational acceleration
γ = unit weight of liquefied soil

(d) Dynamic Fluid Pressure



(e) Lateral Spreading Forces

FIGURE 6-1 (cont.) Factors Affecting Improved Ground Performance

Further discussion of these related factors and their incorporation in the design of improved ground zones is presented in Mitchell et al. (1998) and will be included in a forthcoming MCEER Technical Report on the design and evaluation of improved ground zones. In many cases, particularly for non-critical bridges, the incorporation of some of these factors is done implicitly in design, rather than explicitly. However, the engineer should be aware of them and their potential effects on remediation performance.

6.2.2 Soil Parameters

The manner in which soil parameters are determined for unimproved and improved ground has a significant effect on remediation design. Parameter values that are too conservative can result in an uneconomical or unfeasible remediation plan. On the other hand, parameter values that are unconservative may result in inadequate performance of the supported bridge.

At first glance, assigning properties for treated zones to be used in analyses would appear more straightforward than for untreated, liquefiable soils because the treated zones are designed to be improved to specific criteria. However, the potential variation of the improvement, particularly at locations in the improved soil mass between the actual treatment locations, makes this task challenging. Based on knowledge of the variation in improvement that typically occurs in different soil conditions, the engineer must select soil parameters which adequately quantify the behavior of the improved soil as a whole.

Assigning realistic soil properties to untreated, liquefiable zones to be used in analyses typically requires a great deal of judgment. Loose, saturated granular soils, such as clean sands, will tend to collapse and have low residual undrained shear strengths at large strains. Guidance regarding the selection of residual undrained shear strengths can be found in Seed and Harder (1990). It is important to note, however, that recent studies by Dobry and Abdoun (1998) indicate even loose, saturated sands may exhibit dilative behavior under cyclic loading, making the use of undrained residual strengths somewhat conservative in some cases. Further research is needed in this area (Dobry and Abdoun, 1998).

In all cases, the strength properties used for soils should be compatible with the expected strains in the soil mass. In addition, the reduction of soil shear resistance due to excess pore water pressure migration into zones of lower pore water pressure from zones of higher pore water pressure should be taken into account.

6.2.3 Two Level Design

As discussed in Section 4.4, one of the recommendations of ATC-18 is the adoption of a two level approach to seismic design, particularly for important structures. In this approach, a structure is designed for "less severe" and "more severe" seismic events, with different performance criteria established for each case. Such an approach can be utilized for the design of liquefaction remediation measures where deemed appropriate. The remedial measures would be designed for two different seismic events, with different limits set for the stability factor of safety and tolerable deformations for the two cases.

6.2.4 Analytical Methods

Analytical methods for design of liquefaction mitigation measures are not well-established. These methods can range from very crude approximations to very detailed, time-consuming analyses. Some of the approaches discussed in Section 3 can be used. A detailed discussion of analytical tools for designing liquefaction mitigation measures will be presented in a forthcoming MCEER Technical Report on design and evaluation of improved ground zones. The design procedure discussed in Section 6.4 implicitly assumes that appropriate analytical methods will be used at various stages of designing improvement schemes for liquefaction mitigation.

6.3 Design Guidelines

Although analytical procedures for design of liquefaction remediation schemes are not well established, information from studies conducted to date and presented in the literature provide the basis for the design guidelines and observations of improved ground given below.

1. Improvement of liquefiable soil beneath most structures generally extends to the bottom of the liquefiable material. In many cases, the lateral extent of treatment beyond the edge of the structure is a distance equal to the depth of treatment beneath the structure, although some data suggest the distance could be as small as one-half the depth.

2. Remediation for reducing the lateral deformations of existing embankments is generally more effective when the foundation soils are treated in a zone between the crest and toe of the embankment. Improvement schemes providing rigid confinement of soils that liquefy beneath the embankment typically result in better performance with respect to deformations.

3. The perimeter of zones improved by densification generally become unstable during earthquake shaking and liquefaction. The portion of the densified zone enclosed by the vertical boundary of the treated zone and a line drawn at an angle of 30° to 35° to the vertical from the bottom of the boundary (zone ABC in Figure 6-1(a)) can be treated as liquefied material for design purposes (Iai et al., 1988; Akiyoshi et al., 1994).

4. Positive pore water pressures may develop in a densified soil zone during and after an earthquake that consist of the pressures which develop internally within that zone due to induced shear strains and migration of pore water pressures from the surrounding liquefied soil. The rate of pore water pressure migration will be dependent primarily on the permeability and compressibility properties of the treated soil, with faster migration occurring for higher permeability and lower compressibility. In cases where the rate of

pore pressure migration is rapid and the earthquake duration is long, maximum pore pressures can be estimated using steady-state seepage analyses.

5. Lateral spreading forces exerted by an unliquefied, surficial crust on an improved zone can be estimated using passive earth pressure theory. Where there is crust of significant thickness, the lateral spreading force from the crust will likely be substantially greater than that from the underlying liquefied soil.

6. Inertial forces acting on an improved soil mass should be included when evaluating the stability of an improved ground zone supporting a structure.

7. Field performance data suggest that liquefaction effects on structures will be minor when the supporting ground is improved to the "no liquefaction" side of the liquefaction potential curves recommended for use in the "simplified method" for liquefaction potential assessment (NCEER, 1997).

These guidelines and observations provide the engineer with some simple "rules of thumb" that can be used in the preliminary design process. Refinements and verification of the designs can then be performed using analytical procedures carried out as part of the design procedure discussed in Section 6.4.

6.4 Design Procedure

The design procedure used for selecting a remedial measure scheme for mitigating liquefaction effects at an existing highway bridge can be broken down into the series of steps shown in Figure 6-2. These steps provide a sequence of evaluations that proceed from simple to more detailed, allowing a preferred method of improvement to emerge by the end of the design process. At any point during the process, the designer may decide to return to an earlier step and perform another iteration of previously performed steps, perhaps based on new information or the need for further refinement in the analyses. Detailed descriptions of the steps given in Figure 6-2 are provided in Sections 6.4.1 to 6.4.6.

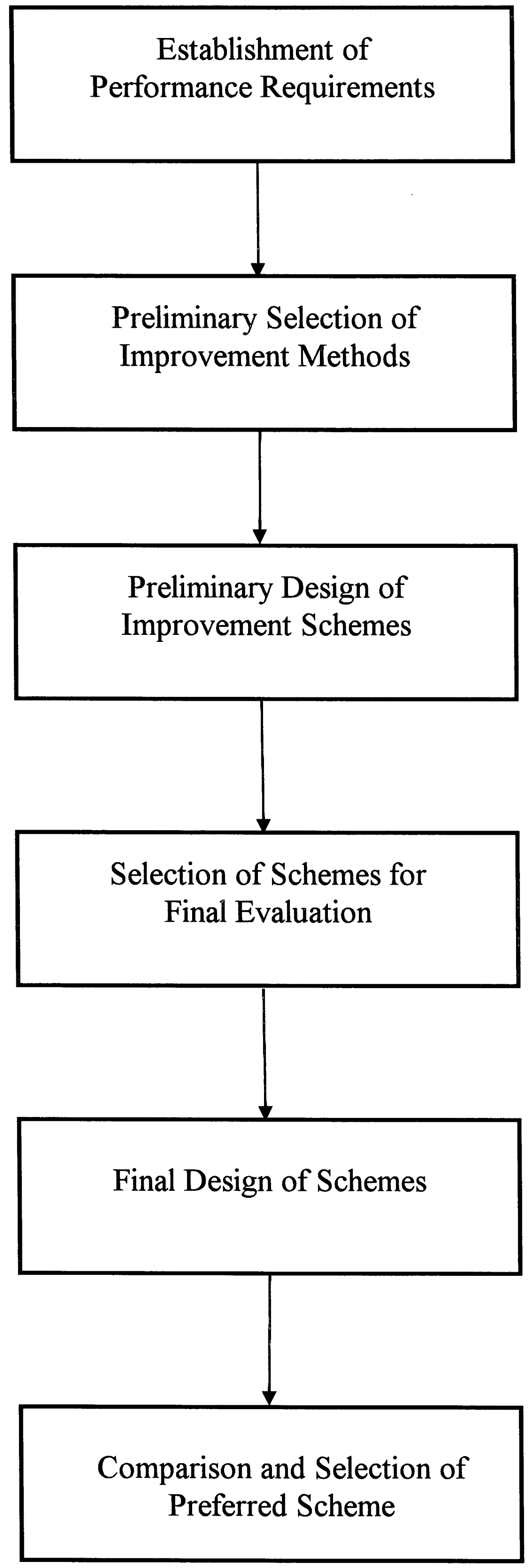


Figure 6-2 Remedial Measure Design Procedure

6.4.1 Performance Requirements

Prior to selecting and assessing potential improvement schemes, the performance requirements must be established for the bridge being evaluated. As discussed in sections 3.4 and 6.2.1, these requirements generally consist of deformation limits for the bridge foundation, as well as some minimum factor of safety for stability. The requirements should account for deformations and stability both during and immediately after an earthquake. Stability and deformations during construction of the remedial measures must also be evaluated and included in the assessment of bridge performance.

A literature review of stability and deformation performance criteria for bridges is presented in sections 3.4.1 and 3.4.2, respectively. Although the criteria presented in these sections serve as a good starting point, performance criteria for bridges should generally be established by the engineer on a case by case basis. For some projects, two sets of performance requirements may be necessary to evaluate performance under both "less severe" and "more severe" seismic events, as discussed in Section 6.2.3.

In addition to stability and deformation criteria, there may be limits on acceptable improved ground or foundation accelerations for the bridge structure. Such limits will likely be established by the bridge engineer based on the structural details and expected dynamic response of the bridge.

6.4.2 Preliminary Selection

Once performance criteria have been established for a bridge, preliminary selection of potentially feasible ground or foundation improvement methods can be made. This selection is generally made on a qualitative basis, taking into consideration the limitations imposed by the bridge site and subsurface conditions, as well as a general knowledge of the level of improvement achievable with different treatment methods in comparison to the bridge performance requirements. Table

5-1 in Section 5 of this report provides one guide that can be used in the preliminary selection process.

The primary objective of the preliminary selection step is to eliminate improvement methods which are clearly not feasible for liquefaction remediation at a particular bridge. When making this evaluation, the designer should not only consider the potential of using an improvement method by itself, but also the possibility of combining that method with one or more other methods to form a remediation scheme. Only methods that are not feasible for use either by themselves, or in conjunction with other methods, should be eliminated in the preliminary selection stage.

6.4.3 Preliminary Design

In the preliminary design phase, the improvement methods identified as being potentially feasible for liquefaction remediation are further investigated and developed into different improvement schemes.

Development of ground improvement schemes requires that some preliminary design analyses be performed to estimate the size, location, and level of improvement needed for the treatment zones at the abutments and piers of the bridge. For non-critical, simple structures the analyses typically involve confirming that the factor of safety against liquefaction is adequate. In cases where densification improvement is used, this would generally involve using the anticipated SPT $(N_1)_{60}$ or CPT q_{c1} values for the improved soil to obtain the liquefaction resistance from standard liquefaction potential curves (NCEER, 1997) which, along with the anticipated cyclic stress ratio, is used to compute the liquefaction safety factor. The empirical guidelines given in Section 6.3 can then be used to establish the sizes and locations of the treated areas, and rough estimates made of the factor of safety for stability and of the expected deformations. On the other hand, preliminary design for a critical or complex structure will likely involve performing some preliminary dynamic finite element or finite difference analyses of the proposed remedial measures and supported structure to evaluate the effect of different treatment zone sizes, locations, and properties on stability, deformations, and perhaps ground accelerations. More details regarding analytical procedures for evaluating stability and deformations of improved ground will be provided in an MCEER Technical Report on the design and evaluation of improved ground zones.

Preliminary design of foundation improvement options will involve a preliminary determination of the location, type, size, capacity, and depth of new foundation elements to be added to the existing foundation system.

6.4.4 Selection for Final Evaluation

After development of preliminary designs for different improvement schemes, the schemes are compared to each other to determine which appear to be more desirable. Prior to making this comparison rough estimates of the costs of the various schemes should be developed. The schemes can then be compared to each other in terms of expected performance, cost, ease of construction, and any other factors the engineer deems relevant for the project. Based on this comparison one or more schemes are selected for final design.

6.4.5 Final Design

In the final design phase, complete detailed designs of the selected improvement schemes are developed. This work includes more refined stability and deformation analyses to finalize the size, location and extent of ground or foundation improvements. Recommendations are being developed for inclusion in a future MCEER Technical Report on design and evaluation of improved ground zones.

In addition to performing more refined analyses, detailed improvement plans and cost estimates are developed during final design. For ground improvement options such as grouting or vibro-replacement, the detailed plans can range anywhere from specifying the overall dimensions and locations of improved zones to establishing the location and depth of holes for treatments. For foundation improvement options, such as underpinning with mini-piles, construction details will be developed including pile layout, tip elevations, and tie in mechanisms to existing pile caps.

The nature of the detailed plans will depend in part on the type of specifications used for the project, as discussed in Section 7.3. For large projects, field tests should be conducted to verify the effectiveness of proposed improvement methods and the compatibility with the design assumptions. The proposed improvement schemes should be modified, as necessary, on the basis of the field test results to provide a system that will meet the performance requirements established for the project. Further discussion of the use of field tests for final design is presented in Section 7.2.

6.4.6 Comparison and Selection

Based on the final design plans and cost estimates, the preferred scheme is selected for reducing liquefaction-induced damage to the existing bridge. This selection is not only based on cost, but also on constructability, the expected performance of the bridge, and other factors relevant to the project. Construction specifications are then developed for the selected improvement scheme. Further discussion of specifications is provided in Section 7, which deals with improvement implementation.

SECTION 7
CONSIDERATIONS IN IMPROVEMENT IMPLEMENTATION

7.1 Introduction

The ultimate goal of liquefaction remediation for an existing highway bridge is to provide the improvements necessary to limit liquefaction-induced damage during an earthquake to acceptable levels. After design of a liquefaction remediation scheme is completed, the final phase of work to achieve this goal consists of successfully implementing those improvements in the field. There are a number of important steps which contribute to the successful construction of a remediation scheme including:

- performance of field tests to verify that the proposed remediation method can achieve the desired improvement;
- development of specifications which provide the necessary information and requirements for implementation of the proposed improvements in the field; and
- construction quality assurance and control (QA/QC) to verify, through observations and testing performed during and after construction, that the specified improvements have been achieved.

The effort expended on each of these items will be dependent on the size and importance of the particular project.

7.2 Field Tests

The purposes of field tests are primarily to (1) verify that the planned improvements can be constructed in the field using the proposed construction methods and (2) validate design assumptions regarding the level of improvement that can be achieved. The nature and timing of the tests will be dependent primarily on the size of the project, as well as the potential for cost savings.

7.2.1 Small Projects

For many small projects, field tests will likely be performed at the beginning of construction rather than as part of the final design phase. The primary reason is the cost of doing a detailed field test will often exceed any potential savings from refinements in the remediation scheme resulting from the test. It is likely to be more economical to design the ground improvement conservatively without the benefit of information from field tests than to conduct extensive tests as part of final design. The tests can be conducted at the beginning of construction to verify that the proposed improvement methods and techniques are adequate. Adjustments to the construction procedures can then be made, if necessary.

7.2.2 Large Projects

Field tests should be conducted during the final design phase of liquefaction remediation for large projects due to the potential savings that could result from refinements in the remediation scheme based on the test information. In this case the tests would typically involve varying the treatment parameters to see the effect on the improvement achieved. These parameters often include such things as the pattern, spacing, and sequence of treatment and the consistency of materials used in the treatment.

The performance of field tests during final design may also be necessary for unusual soil conditions. In this case, several different improvement methods or combination of methods may have to be tested before a successful treatment scheme is developed.

7.3 Specifications

The specifications for performing liquefaction remediation are essential documents for ensuring that the planned remedial measures are constructed as designed. Therefore, careful consideration should be given to the type of specifications adopted for a particular project and the manner in which they are written.

The specifications for a project are either method based, performance based, or a combination of both. The choice of which type to adopt will depend on the particular project and the type of ground or foundation improvement being used, as well as consideration of local practice.

7.3.1 Method Specifications

Method specifications, as used herein, refers to specifications where the treatment method, detailed treatment locations (i.e. layout of stone columns, mixed-in-place walls, etc.), and construction equipment and procedures are specified. This type of specification provides the engineer with control over the construction techniques used by the contractor. Potential limitations of using method specifications include (1) they do not allow for contractor innovation and (2) large cost overruns can occur if the required improvement can not be achieved using the method specified.

Method specifications are probably best suited for improvement projects where the expected performance is dependent on improvement elements being constructed at specific locations and in a particular sequence. For instance, the performance of ground improvement methods that rely on reinforcement of the ground (i.e. mixed-in-place columns or walls, stone columns, root piles) or drainage (i.e. gravel drains) is often dependent on a specific pattern and spacing of reinforcing or drainage elements. Likewise, the use of foundation improvements, such as underpinning, requires that the new foundation components be constructed at specific locations and tied into the existing foundations in a particular way.

7.3.2 Performance Specifications

Performance specifications, as used herein, refers to specifications where the desired end product for liquefaction remediation is specified, but the means to achieve the end product is largely left up to the contractor. This type of specification can generally be used for cases where the overall size, location, and material properties of treated zones are the only criteria which need to be provided to ensure adequate performance. Examples of remediation methods where performance specifications may be appropriate are those which involve densification or grouting of the soil.

A performance specification has the advantage of encouraging innovation and efficiency on the part of the contractor, which can result in more economical construction. However, the engineer must be careful to clearly define in the specifications the performance requirements and how the meeting of those requirements will be assessed. In addition, requirements for the bridge performance during construction of the improvements must be clearly defined.

7.3.3 Combined Specification

A combined specification incorporates features of both method and performance specifications. This specification allows the engineer to specify both the construction techniques used by the contractor and the requirements for the level of improvement achieved using those techniques. The engineer probably has the most control over construction using the combined specification. However, the required improvement must be achievable by the contractor with reasonable effort using the methods specified or else the engineer will be faced with large construction claims from the contractor.

7.4 Quality Assurance and Control (QA/QC)

QA/QC methods utilized during and after construction of liquefaction remediation measures are critical for evaluating whether the required improvements have been achieved in the field. The QA/QC procedures used will be dependent on the type of treatment methods used in the improvement project. Based on the observations and measurements made as part of the QA/QC procedures, deficiencies in the remediation work can be identified early during construction and adjustments made in the implementation methods so that the project requirements are satisfied.

7.4.1 Construction Observations

Construction observations are useful for a preliminary evaluation of the uniformity and effectiveness of the remediation work being performed. Although these observations can not be used as a sole indicator of the improvement achieved, they do provide a preliminary indication of areas where the improvement was or was not successful, which is then verified by selective construction testing. Construction observations commonly made for different improvement methods are summarized below.

7.4.1.1 Grouting

Observations commonly made for grouting are the volumes of different grout mixes injected into the ground and the locations, depths, pressures, and duration of grout injection. Survey measurements of the ground surface elevation before, during, and after grouting are also obtained to check for heave (or settlement) of the ground and structure. All of this information gives a rough indication of where the grout is going and the effectiveness of grout penetration (for permeation grouting) or densification (for compaction grouting) of the soil mass. Grout mix samples for strength testing are taken on a regular basis for all grouting methods.

As part of the QA/QC for jet grouting, prototype columns constructed during the field tests are typically excavated to verify the diameter of the columns and properties of the grouted materials. During construction of production jet-grouted columns, observations are made of the rate of rotation and removal of the grout pipe and material consumption, as well as the other observations cited above for other grouting methods, to ensure consistency with the prototype column construction.

7.4.1.2 Vibro Systems

Important observations when vibro systems are used, particularly vibro-compaction and vibro-replacement, are: (1) the location of probe holes; (2) detailed records of the volume of stone, gravel, or sand used to backfill the probe holes at different depths; and (3) detailed records of vibroflot energy and time expended in densifying the backfill as a function of probe elevation. These observations, along with monitoring of ground surface and structure settlement, give indications of the densification achieved.

7.4.1.3 Surcharge/Buttress Fills

The primary observations made during the construction of surcharge or buttress fills are the overall size, location and density of the fill. It is also important to monitor the quality of the fill material for buttresses because the strength of the fill is also being relied upon to provide resistance against failure.

7.4.1.4 Mixed-in-Place Walls and Columns

During installation of mixed-in-place walls and columns by deep soil mixing, records are maintained of the quantity of admixture used in the process, the depth and rate of penetration and withdrawal of the mixing augers, and the quantity of spoil produced. These observations provide an indication of the relative quantities of soil and admixture in the column or wall and the degree of mixing achieved. Grab samples of the mixed-in-place material are taken and cured for laboratory strength testing.

7.4.1.5 Root Piles and Mini-Piles

The physical integrity of root piles and mini-piles is monitored by measurement of the material quantities used in construction and subsequent comparison to the design quantities to verify that the pile has not been compromised by "squeezing" of the ground into the hole. In addition, observations are made of the drilling depths, times, difficulties and spoils to provide additional information regarding the adequacy of the constructed piles. Samples of the grout used in the piles are obtained and cured for laboratory strength testing.

If post-grouting (described in Appendix A, Section A.6) of the mini-piles is performed then records are also maintained of the additional grout injected into the ground, as well as the injection duration and pressures.

7.4.1.6 Vertical Drains

Construction observations made during installation of vertical drains include monitoring the location and depth of drains, as well as backfill materials used. When gravel and sand drains are constructed in auger holes, it is particularly important to monitor the quantity of backfill used during auger withdrawal to ensure that "necking" of the drain does not occur. The quality of the gravel or sand backfill should also be closely monitored, with periodic grab samples taken for laboratory grain size analyses and permeability testing.

7.4.2 Evaluation of Treated Ground

The purpose of evaluating treated ground is to provide definitive evidence that the desired improvements in soil/material properties for liquefaction remediation have been achieved in the field. The evaluation should complement construction observations to provide confidence that all zones where ground or foundation improvement have been performed meet the design requirements.

Treated ground evaluation should be an integral part of the construction process, being performed as sections of improvement are completed. The results from the testing will provide on-going information regarding deficiencies in the treatment so that adjustments can be made in construction procedures.

It is critical that the properties controlling the remediation scheme design are the ones checked during treated ground evaluation. Methods used for checking these properties can include in-situ and laboratory testing. When using in-situ testing, suitable correlations must be selected for the soil properties being evaluated. Properties of concern for ground improvement generally include the liquefaction resistance, relative density (for densification methods), strength, and stiffness of the treated soil and any reinforcing elements. For foundation improvement methods, such as underpinning with mini-piles, the main concern is the strength of the new foundation elements.

7.4.2.1 In-Situ Tests

In-situ tests, particularly the standard penetration test (SPT) and cone penetration test (CPT), are commonly used to assess the properties of soils improved by densification methods such as compaction grouting or vibro-systems. The SPT and CPT soundings are performed midway between the actual treatment locations within the improved zone to evaluate the "lower bound" of improved soil properties. The corrected SPT blow count, $(N_1)_{60}$, or the corrected CPT tip resistance, q_{c1}, are then used with appropriate correlations to estimate the desired properties. Recommended correlations are presented in Table 4-2.

An alternative or supplementary method of evaluating in-situ properties of densified soils is to measure the shear wave velocity, v_s, and use available correlations, such as those recommended in Table 4-2. Shear wave velocity tests provide a direct measurement of the maximum shear modulus and, when performed properly, can be used with great confidence for estimating this property. In the past several years there have been improvements in shear wave velocity correlations for other properties, particularly liquefaction resistance (NCEER, 1997). However, the experience with these correlations is still not as extensive as those for the CPT or SPT. Therefore, the use of v_s to estimate properties other than the maximum shear modulus should be done cautiously, and probably in conjunction with other in-situ tests having more firmly established correlations, such as the SPT or CPT. Another limitation of shear wave velocity tests is that measurements made in improved ground may be difficult to interpret due to the heterogeneous nature of the treated soil. Although methods have been proposed for taking v_s

measurements in soils improved by some treatment methods, such as vibro-replacement (Baez and Martin, 1995), certain conditions may still pose problems and limit the ability to use these types of measurements.

When using correlations to estimate the properties of improved soils, the engineer should be aware of the limitations of the correlations. Only those correlations that are appropriate for the soil type and conditions should be used.

For cases involving grouted or mixed-in-place zones or elements, in-situ tests, such as the SPT, CPT, or geophysical tests, can be used to establish that the limits of the treated zone conform to the design requirements. However, the use of the SPT or CPT may be limited by the potential for damage to the testing equipment if the treated soils have high strength. The actual strength of grouted or mixed-in-place materials is typically evaluated using laboratory tests.

7.4.2.2 Laboratory Tests

Laboratory tests are typically used for establishing that the strength of soils improved by permeation grouting, jet grouting, or deep soil mixing is adequate. Likewise, they are used to provide quality control on the strength of grout in mini-piles or root piles. Although it would be preferable to obtain "undisturbed" samples of the improved materials for laboratory testing after they have cured in the ground, such sampling may not always be possible or feasible. In such cases representative batch samples of freshly grouted material can be taken and cured under appropriate conditions for laboratory strength testing.

Permeability and grain size analyses are other laboratory tests which may be used to provide quality control on ground improvement projects. Permeability tests would typically be performed to check the hydraulic conductivity of materials used for drainage elements, such as gravel or sand drains. Grain size analyses can be used for quality control of material gradation, particularly for improvements involving the use of gravel or sand drains or buttress fills.

As indicated in Table 4-2, many other soil properties can be measured using laboratory tests. However, laboratory testing for evaluation of these properties for improved soils is both expensive and time consuming, requiring "undisturbed" samples from the field. Therefore, the use of in-situ tests is preferred for determining many of these properties, although many times a combination of both in-situ and laboratory tests will be needed.

SECTION 8
CONCLUSION

The focus of this report has been (1) evaluating the need for liquefaction remediation at existing highway bridges, (2) selecting and designing remedial measure schemes, and (3) implementing the proposed treatment plans in the field. The overall objective is to ensure that the stability and deformations of an existing highway bridge located in liquefiable soil deposits meet the performance requirements for that bridge when subjected to the design earthquake. Key points presented in the report relevant to the procedures for assessment of needs and design of remedial measures are summarized below.

8.1 Assessment of Needs for Liquefaction Remediation

A flow chart procedure has been presented that focuses on evaluating the potential for damage to existing bridges due to full or partial liquefaction of supporting soils, and the need for remedial measures. The essence of the procedure is to first use simple evaluation methods to assess the potential for liquefaction. If the conditions indicate liquefaction, then the procedure progresses to more complex and detailed analytical methods for evaluating ground stability and deformations. The main components of the evaluation procedure for a bridge are: (1) a preliminary evaluation of site and earthquake conditions, (2) an assessment of liquefaction potential, (3) gross estimates of stability and deformations, and (4) refined estimates of deformations. Based on the results of analyses recommended in these flow charts, a decision can ultimately be made regarding whether ground or foundation improvement are deemed necessary at a bridge to ensure adequate performance during the design earthquake.

Stability and deformations are both evaluated as part of the recommended procedure for assessing the potential for liquefaction-induced damage to a bridge, although it is deformations that ultimately govern the performance. Methods for estimating stability and deformations range from simple to complex.

For footings and slopes, simplified methods of evaluating the stability generally consist of pseudostatic analyses. The Newmark method can be used to evaluate deformations for cases where there is no lateral spreading provided the strength of the liquefied soil is accounted for adequately (including any dilative behavior). For cases where lateral spreading occurs, ground deformations can be estimated using the empirical procedure recommended by Bartlett and Youd (1995).

A simplified method for estimating the lateral stability and deformations of piles embedded in liquefiable ground, where no lateral spreading occurs, consists of superimposing the deformations from ground response analyses and the results of p-y analyses of the piles for loads imposed by the structure. For cases where lateral spreading occurs around the piles, simplified methods of estimating pile deformations and stresses developed by O'Rourke et al. (1994) or Ishihara and Cubrinovski (1998) can be used. The axial performance of piles must also be evaluated taking into account loads from the structure, as well as additional vertical loads on the piles due to downdrag imposed by settlement of the liquefied soil and overlying deposits. The impact of the loss of lateral and axial resistance due to liquefaction and cyclic degradation of soils must be incorporated into these analyses.

Complex analyses of ground and foundation deformations are performed using finite element or finite difference formulations. The analyses typically use an effective stress analysis in conjunction with a non-linear, elastic-plastic constitutive soil model.

Seismic performance criteria for bridges consist of a minimum factor of safety for stability of the foundations and maximum allowable deformations. Typical factors of safety for stability under seismic loading range between 1 and 1.5. Vertical and lateral deformation limits for maintaining serviceability are on the order of 25 to 100 mm. The acceptable factors of safety and deformations should be established on a bridge by bridge basis by the engineer based on performance requirements.

8.2 Remedial Measure Design

Ground and foundation improvement methods considered potentially feasible for remediation of liquefiable soils at existing highway bridges include:

- Grouting - compaction, permeation, jet;
- Vibro systems - vibratory probe, vibro-compaction, vibro-replacement (stone columns);
- Surcharge and buttress fills;
- Reinforcement and containment - root piles, mixed-in-place walls and columns;
- Vertical drains; and
- Underpinning - mini-piles.

The applicability of these improvement methods to a particular bridge remediation project is dependent on space and geometry limitations and subsurface conditions at the site, as well as the anticipated post-treatment performance of the bridge and the potential for construction-induced movements and vibrations.

Key criteria for evaluating the adequacy of a liquefaction remediation scheme are the stability and deformations of the improved ground/foundation and supported structure. In addition, the accelerations generated within the improved ground zone will also likely be of importance. An effective remedial measure design will strike a balance in satisfying stability, deformation and acceleration requirements.

Three factors that have a significant effect on the performance of a remedial measure scheme are the types, sizes, and locations of the treated zones. Recent model and analytical studies on the performance of embankments supported on improved ground zones have revealed the following regarding their behavior under seismic loading:

- treated zones beneath embankments appear to be most effective when located in the foundation soils between the toe and crest of the embankment slope and least effective when located entirely beyond the toe;
- wider improved zones constructed beneath the slope of an embankment result in smaller embankment deformations, but the amount of change in the deformation reduction gets progressively smaller as the width is increased;
- stiffer treatment zones constructed at the toe of an embankment result in less lateral deformation of the embankment due to the increased confinement provided for soils which may liquefy beneath the embankment;
- higher pore water pressures beneath an embankment will generally result in less crest acceleration but larger crest settlements; and
- in areas of sloping ground with a non-liquefiable surface layer, it may be necessary to tie the portion of the non-liquefiable layer over the improved area to the improved zone, or isolate it from the surrounding untreated ground.

Model studies on the performance of footings/structures supported on improved zones indicate that:

- increasing the treatment width relative to the width of the supported footing/structure or depth of treatment beneath the structure results in decreasing footing/structure settlements, but the change in the settlement reduction gets progressively smaller as the width gets larger (optimum treatment width beyond the edge of the structure appears to be at least 0.5 times the treatment depth and preferably equal to the treatment depth);
- the larger the depth of treatment beneath a footing/structure the smaller the settlement, but the larger the acceleration, of the footing/structure;
- the wider the structure supported on an improved ground zone the smaller the settlement of the structure provided the distance of treatment beyond the edge of the structure and the treatment depth is the same; and
- structures with larger height to width ratios experience larger settlements.

Numerical modeling indicates that densifying soils or constructing soil-cement zones around piles in liquefiable soils substantially decreases the shear forces, bending moments, and displacements of the piles compared to no improvement.

There are several important factors which must be incorporated in the design of ground improvement for liquefaction mitigation either explicitly or implicitly. These factors include: (1) excess pore water pressure migration into the improved zone, (2) amplification of the ground motions, (3) phasing of inertial forces acting on the structure and improved ground, (4) dynamic fluid pressure acting on the improved ground sides, (5) soil-structure interaction, (6) and forces exerted by laterally spreading ground.

A design procedure for liquefaction remediation measures was presented in Section 6.4 of this report. The procedure consists of a series of steps which proceed from simple to more detailed evaluations of potential improvement measures. Steps in the procedure include: (1) establishment of bridge performance requirements, (2) preliminary selection of improvement methods, (3) preliminary design of improvement schemes, (4) selection of schemes for final evaluation, (5) final design of schemes, and (6) comparison and selection of the preferred scheme. The procedure is designed to allow a preferred method of improvement for a particular bridge to emerge by the end of the design process. Some guidelines for preliminary design of improved ground zones were presented in Section 6.3 based on laboratory and field observations.

8.3 Remedial Measure Construction

Construction of remedial measure schemes involves performing field tests of the proposed improvement method, preparing specifications for implementation of the scheme, and conducting observations and testing during construction to verify the improvements achieved satisfy design requirements.

Field tests are an important part of verifying that the proposed treatment methods are adequate to achieve the desired improvements. The tests will likely be performed during final design for large projects and at the beginning of construction for small projects.

Method specifications, performance specifications, or a combination of both can be used for implementation of liquefaction remediation at bridges. The type of specifications selected for a particular project is dependent on the degree of control that must be exercised over the construction procedures to ensure that the remediation is successfully performed.

During construction, sufficient observations and testing must be performed to verify that the remedial measures in the field meet design requirements. In-situ testing is preferred for evaluating most improved ground properties due to the difficulties and expense associated with obtaining "undisturbed" samples for laboratory testing.

The systematic approaches recommended in this report for assessing the need for liquefaction remediation, selecting and designing potential improvement measures, and implementing the design provide a means for successful liquefaction mitigation at existing highway bridges.

SECTION 9
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APPENDICES

APPENDIX A
IMPROVEMENT METHODS

More detailed descriptions are given below of the improvement methods discussed in Section 5.

A.1. Grouting

A.1.1 Compaction Grouting

Compaction grouting consists of injecting a stiff, low-slump (less than 50 millimeters) soil-cement mixture into the ground forming expanding bulbs which displace and densify the adjacent soil, as shown schematically in Figure A-1. The grout is injected at high pressures (typically 3500 to 7000 kilopascals) through the end of a pipe but does not penetrate the soil pores because of its stiff consistency. Grout bulbs up to 1 meter in diameter are common.

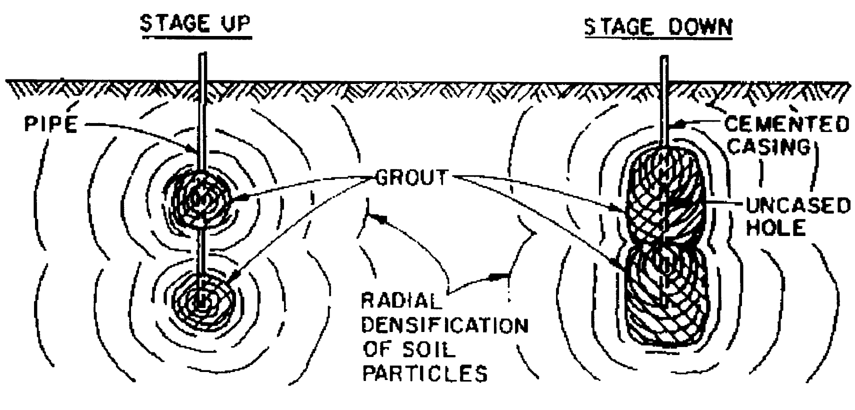

FIGURE A-1 Compaction Grouting Densification Mechanism (from Graf (1992), "Compaction Grout, 1992," Proc., Grouting, Soil Improvement and Geosynthetics, Vol. II, Geotechnical Special Publication No. 30, ASCE. Reprinted by permission of ASCE)

Compaction grouting is usually performed in a series of primary, secondary, and sometimes tertiary holes, with final horizontal spacings of 1.5 to 3.5 meters typical and generally no restriction on depth. The smaller horizontal spacings are generally required for grouting at depths of 2.5 meters or less due to the lower pressures used to reduce the potential for ground surface heave. Each hole is grouted in vertical stages ranging from about 0.3 to 1 meter, with grouting proceeding either from top to bottom or bottom to top. Grout holes can be inclined, but Salley et al. (1987) suggest that the inclination be limited to 20 degrees from vertical to avoid surface cracking problems which developed when they used inclinations of 45 degrees.

A summary of compaction grouting practice over 30 years is given by Warner (1982), and recent developments are discussed in ASCE's ten year update on ground improvement practice (ASCE, 1997). More detailed explanations of hole spacing and staging for grouting; pumping rates and pressures; equipment; testing; quality control; and applications can be found in these two references.

A.1.2 Permeation Grouting (Particulate and Chemical)

Permeation grouting is performed by injecting material into the ground, in slurry form, to permeate and fill voids within the soil structure. The primary objective of this grouting is to fill the space between soil particles rather than rearranging the particles themselves. Particulate or chemical grouts are used to perform permeation grouting.

Particulate grouts consist of a mixture of water and one or more particulate solids such as cement, fly ash, clays, or sands. Cement or a cement-bentonite mixture is commonly used for liquefaction remediation to replace the water in the soil voids with material that binds the particles together, thus eliminating the mechanisms causing liquefaction. The water:cement ratio of the grout can range from 0.5:1 to 6:1 by weight depending on the gradation of the soil being grouted and the strength required. Particulate grouts can only be used for grouting soil deposits consisting of medium sands or coarser material.

Chemical grouts consist of a mixture of chemical solutions which are injected into the ground where they react and harden. The most commonly used chemical grouts are silicates, which are typically formed in a one-shot process by injecting sodium silicate and catalysts into the ground at the same time. Soils consisting of silts or sands can be chemically grouted. Silicate concentrations of 40 to 60 percent are used for high strength applications.

Horizontal hole spacings of 0.5 to 2.5 meters are typical in permeation grouting, with the grouting being done in primary, secondary, and sometimes tertiary stages. There is generally no limitation on depth. Each hole is generally grouted either using an open-ended pipe or a pipe with sleeved ports (tube-a-manchette). Grouting pressures are usually limited to approximately the overburden pressure, except under heavy structures where use of higher pressures may be possible.

Detailed discussions of permeation grouting can be found in Mitchell (1981) and Xanthakos et al. (1994). Recent developments in permeation grouting and its uses are presented in ASCE's ten year update on soil improvement (ASCE, 1997)

A.1.3 Jet Grouting

Jet grouting consists of using high pressure jets to erode soil and then mixing or replacing it with grout to form columns of improved material, as shown in Figure A-2. The process begins by drilling a small diameter hole (typically 70 to 150 mm) to the desired depth of treatment. In a single stage system (double and triple stage systems are also used) grout is then pumped under high pressure (20 to 60 megapascals) through a rotating nozzle at the bottom of the drill rod as it is slowly raised, such that a grout-soil or grout column is left in place. Any excess waste material is washed up to the surface during the process.

In jet grouting improvement a series of improved columns are installed, generally in an overlapping pattern. Column diameters can range from 0.3 to 4.0 meters depending on the system used, soils encountered, and cutting pressures.

A thorough explanation of jet grouting systems and improvements is given by Xanthakos et al. (1994).

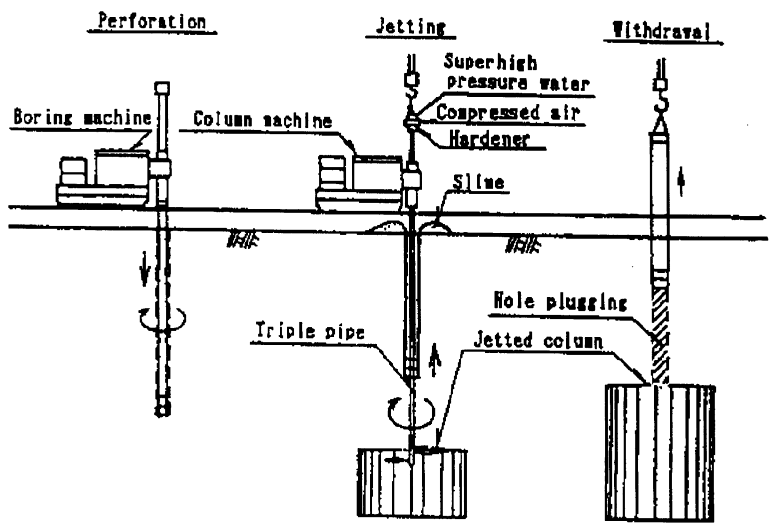

FIGURE A-2 Jet Grouting Process (from Ichibashi et al., (1992), "Jet Grouting in Airport Construction," Proc., Grouting, Soil Improvement and Geosynthetics, Geotechnical Special Publication No. 30, ASCE. Reprinted by permission of ASCE)

A.2 Vibro Systems

A.2.1 Vibratory Probe

The vibratory probe method consists of penetrating a long tube, typically about 0.75 meters in diameter, into the ground using a vibrator attached to the top of the tube. This process of penetration and withdrawal is repeated at regularly spaced locations to provide densification of the in-situ cohesionless soils within the specified treatment area. The method is generally effective for densifying sands having a fines content (percent by weight passing No. 200 sieve) less than 15 percent. Densification can be achieved to depths of up to about 12 meters.

A.2.2 Vibro-Compaction

In vibro-compaction a vibratory probe, with the vibrator enclosed near the bottom and oscillating on a vertical axis, is penetrated into the ground to the desired treatment depth with the aid of water or air jets. The hole created by the probe is then backfilled with sand or gravel as the probe is withdrawn and repenetrated in increments of about 1 meter. This process results in densification of the in-situ cohesionless soil. A schematic of the process is shown in Fig. A-3a.

Densification within a specified zone of improvement is achieved by performing the vibro-compaction procedure using probe treatment locations spaced at approximately 1.5 to 3.5 meters. Treatment depths up to 30 meters are possible.

The vibro-compaction procedure is generally effective for sands having fines contents less than 15 to 20 percent.

A.2.3 Vibro-Replacement (Stone Columns)

Vibro-replacement using stone columns is generally the same as vibro-compaction, with the exception that all of the holes created by the penetrating probe are backfilled with compacted crushed rock or gravel. Once the probe has reached the desired treatment depth, the backfill is added and compacted in lifts of 0.5 to 0.7 meters, as shown in Fig. A-3b. Each lift is penetrated several times with the vibrator to compact it thoroughly and densify any surrounding sands and silts. When treatment at a probe location is completed, a densified column of stone typically ranging in diameter from 0.5 to 1 meter, or more, is left in place. The center-to-center spacing between columns generally ranges from 1.5 to 3.5 meters.

Stone columns can be installed using wet or dry feed methods and top or bottom feed methods. The bottom feed, dry method is generally preferred, when conditions allow, because of the greater control afforded in placing the stone and the elimination of the need for large quantities of water.

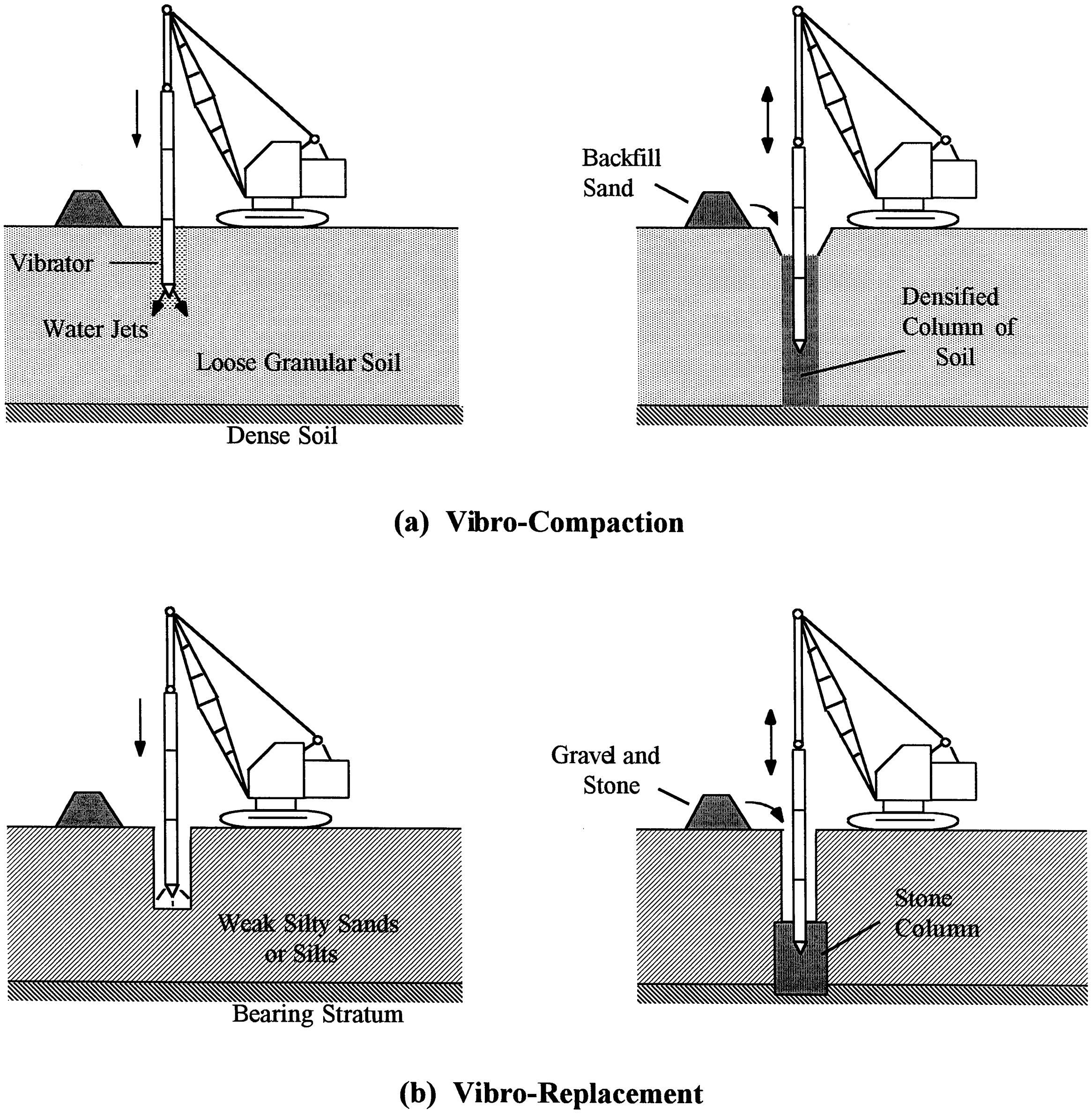

FIGURE A-3 Vibro-Compaction and Vibro-Replacement Processes (after Hausmann, 1990)

Vibro-replacement is generally effective for densifying silty sands having up to 25 percent fines, including those where the fines are slightly cohesive. In addition to densifying the sand, the stone columns provide reinforcement and drainage for the soil mass. The method can also be used for improvement of cohesive soils because the stone columns will act as stiff reinforcing elements which can reduce the load on the soft soils. Improvement of mixed soil deposits consisting of both cohesionless and cohesive soils is achieved by a combination of densification, reinforcement, and drainage.

A.3 Surcharge and Buttress Fills

A.3.1 Surcharge

A surcharge is fill material placed over a surface area primarily to increase the effective stress in the ground beneath the fill. The fill will generally be at least a few meters thick and cover an area of sufficient size to increase the effective stress in the specified zone within the ground to the desired level. Miscellaneous soil fill can generally be used for the surcharge and only minimal compaction is necessary. Surcharges for liquefaction remediation are permanent.

A.3.2 Buttress

A buttress is similar to a permanent surcharge except its primary purpose is to provide additional weight and shearing resistance (by lengthening the failure surface) to help prevent a sliding/slope failure from developing in the ground. A secondary effect of the buttress is the increase in effective stress within the ground which provides additional shear resistance to reduce deformations. An example of a buttress placed to improve the stability at an approach embankment and stub abutment is shown in Figure 5-7.

If the only function of a buttress is to provide additional resistance against failure by its weight, miscellaneous soil fill is suitable for use and minimal compaction is needed. However, if additional resistance is to be provided by the strength of the buttress material, then select soil fill should be compacted to meet specified requirements.

A.4 Containment and Reinforcement

A.4.1 Mixed-In-Place Columns and Walls

A mixed-in-place column consists of a shaft constructed with a mixture of in-situ soil and one or more stabilizing materials. The shaft is constructed by the deep soil mixing (DSM) method in which the the ground is penetrated with an auger, while at the same time stabilizers are injected through the auger shaft and mixed with the in-situ soils. A common stabilizer is cement.

Typical mixed-in-place columns are approximately 1 meter in diameter, although diameters ranging from 0.6 to 3 meters can be constructed. The column lengths generally range from about 8 to 30 meters.

Although columns can be installed individually using a set horizontal spacing, it is more common to install overlapping shafts, which form a panel, using installation equipment having a series (typically three) of overlapping augers. Series of overlapping panels can be constructed to form mixed-in-place walls or cells of different shapes and sizes. Figure A-4 shows an example of a series of cells constructed using overlapping panels.

A.4.2 Root Piles

A root pile consists of a small-diameter hole (typically 75 to 250 mm) drilled in the ground into which reinforcing steel and grout are placed. The hole is typically formed by installing casing using rotary or percussion methods, along with circulating a fluid to remove drilling spoil. After the desired tip elevation is reached and the hole flushed to remove remaining spoil, the reinforcing steel is inserted and a neat cement or cement-soil grout is tremied into place. Additional grout is added under pressure as the casing is withdrawn (unless the design

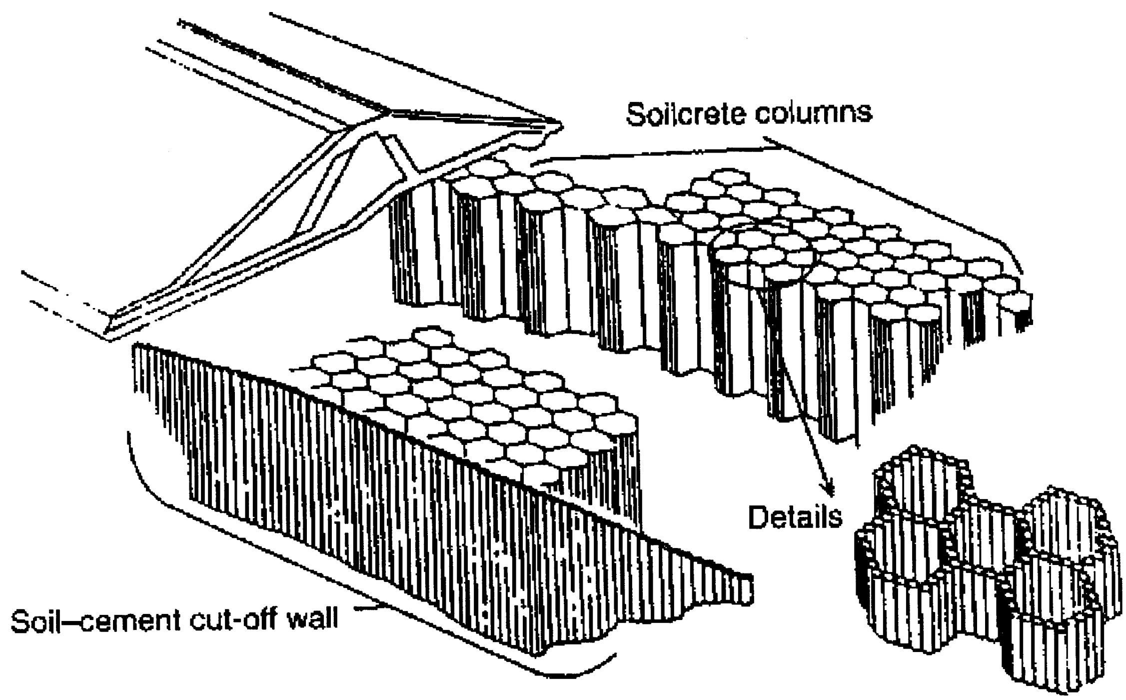

FIGURE A-4 Example Detail of Mixed-in-Place Columns Used to Form Containment Cells at Jackson Lake Dam Modification Project (Taki and Yang, (1991), "Soil-Cement Mixed Wall Technique," Proc., Geotechnical Engineering Congress 1991, Geotechnical Special Publication No. 27, ASCE. Reprinted by permission of ASCE.)

specification states that the casing be left in place) such that the hole remains filled with grout at all times.

Root pile lengths commonly range between 3 and 30 meters. The piles can be installed vertically or battered.

Root piles typically act as structural reinforcing within the ground and therefore are designed to resist compression, tension and shear forces, as well as bending moments. Groups of piles can be installed to reinforce a soil mass.

A.5 Drains

A.5.1 Gravel and Sand Drains

Gravel and sand drains are commonly installed by advancing casing equipped with auger flights and a closed bottom into the ground. During penetration of the casing, the water level inside should be maintained at or above the groundwater level. When the desired tip elevation of the drain is reached, sand or gravel is placed into the casing as it is extracted, thus leaving a continuous column of sand or gravel in place. Ono et al. (1991) describe a system used in Japan where the gravel is compacted in lifts using a rod during casing extraction, as illustrated in Figure A-5. This figure also serves to illustrate the general installation process for drains where compaction is not used.

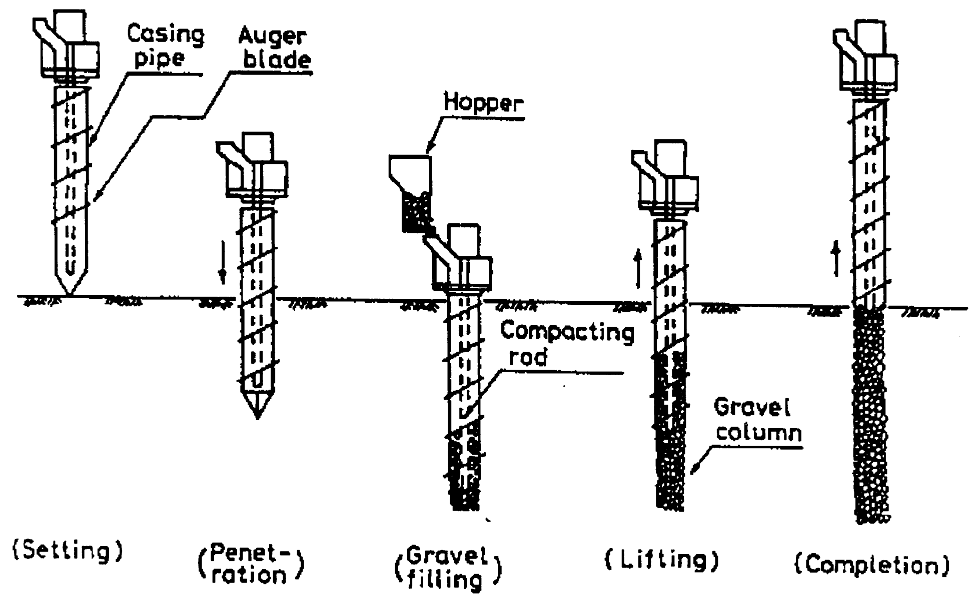

Figure A-5 Gravel Drain Installation (from Ono et al. (1991), "Efficient Installation of Gravel Drains," Proc., 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Vol. 1, University of Missouri, Rolla.)

Gravel drain spacings of 0.8 to 1.5 meters have been used. Drain diameters are typically 0.4 to 0.8 meter. Installation depths generally range up to about 20 meters.

Four case histories in Japan where gravel drains were used behind quay walls for liquefaction mitigation are summarized by Andrus and Chung (1995).

A.5.2 Pre-Fabricated (Wick) Drains

Pre-fabricated (or wick) drains are drainage elements typically constructed of a plastic inner core and covered by a geotextile jacket. These drains are usually installed vertically into the ground from the ground surface. Water enters the drain by passing through the geotextile jacket and then flows upward along its length to a drainage blanket constructed at the ground surface. The drain cross-section commonly has a width ranging from 95 to 100 millimeters and thickness of 3 to 4 millimeters.

Installation of pre-fabricated drains is accomplished by pushing them into the ground using a crane equipped with a large boom and a mandrel. The mandrel is a long metal sleeve which completely covers the drain as it is being pushed into the ground, thus protecting it from damage. A metal anchor is attached to the bottom of each drain so that the drain will be anchored in the soil after it has been pushed to the desired tip elevation and the mandrel is withdrawn. Any void left by withdrawal of the mandrel can be backfilled with "clean" sand having a permeability greater than the in-situ soil

Prefabricated drain lengths up to 25 to 30 meters are common, but drains greater than 60 meters long have been installed. Drains are normally installed in square or triangular patterns with spacings ranging from 1 to 3 meters.

A.6 Underpinning with Mini-piles

Mini-/pin piles have become popular for underpinning purposes because they can be installed in areas with limited access and work space, the installation procedure generally does not produce large vibrations, and in some cases they can be installed through the existing foundation. These piles are the same as the root piles described above, except they are designed to carry the load of an existing foundation down to a deeper, more competent bearing stratum rather than to tie a soil mass together. Pile capacities typically range from 100 to 300 kilonewtons. Figure 5-10 shows a schematic illustrating the use of mini-piles at an existing bridge pier.

In some cases mini-piles are pressure grouted one or more times after initial construction and grout set up. This procedure is commonly referred to as "post-grouting" and is performed through a tube installed in the center of the pile and extending along its length. At specified depths the tube has holes in its side which are covered with a flexible sleeve. Grout is injected into the post-grouting tube at pressures high enough to fracture the grout adjacent to the holes, and force it's way into the soil. The objective of this procedure is to increase the capacity of the pile.

Extensive information on the design and construction of mini/pin piles is given by Xanthakos et al. (1994).

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