# What Structural Engineers Need to Know About Liquefaction

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## ABSTRACT

Liquefaction is a seismic hazard that must be evaluated for a significant percentage of the developable areas of the state of California. The combination of the presence of active seismic faults, young loose alluvium, and shallow groundwater are the ingredients that are found in many areas of California and other seismically active areas of the world. The state of California, through the Seismic Hazards Mapping Act of 1990, has mandated that liquefaction hazard be determined for new construction. On a parallel track, the Uniform Building Code since 1994 has provisions requiring the determination of liquefaction potential and mitigation for the consequences, such as settlement, flow slides, lateral spreading, ground oscillation, sand boils, and loss of bearing capacity. Fortunately, the state of practice has now evolved that there are field exploration methods and analytical techniques to estimate the liquefaction potential and the possible consequences arising from the occurrence of liquefaction triggering in the soils. There are some areas that still need further research. Mitigation techniques for liquefaction have become more commonplace and confidence in these techniques have been increased based on relatively successful performance of improved sites in several past major earthquakes. The purpose of this paper is to inform the structural engineer about the present state-of-practice in liquefaction analysis and mitigation, and to increase his or her knowledge about the options available to mitigate the liquefaction hazard to structures if potential for liquefaction exists.

## INTRODUCTION

Liquefaction is a process by which sediments below the water table temporarily lose stiffness and strength and behave as a viscous liquid rather than a solid. The types of sediments most susceptible are clay-free deposits of sand and silts; occasionally, gravel liquefies. The actions in the soil which produce liquefaction are as follows: seismic waves, primarily shear waves, passing through saturated granular layers, distort the granular structure, and cause loosely packed groups of particles to progressively densify. Densification increases the pore-water pressure between the grains if drainage cannot occur. If the pore-water pressure rises to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid rather than a solid. Thus, liquefaction has occurred.

The Seismic Hazards Mapping Act of 1990 became California law in 1991. The purpose of the Act is to protect public safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure, or other hazards caused by earthquakes. The Seismic Hazards Mapping Act is a companion and complement to the Alquist-Priolo Earthquake Fault Zoning Act which addresses only surface fault-rupture hazards.

Special Publication 117 (SP 117), published by the California Department of Conservation, Division of Mines and Geology in 1997, presents guidelines for evaluation of seismic hazards other than surface fault-rupture and for recommending mitigation measures. The guidelines in SP 117 provide, among other things, definitions, caveats, and general considerations for earthquake hazard mitigation, including soil liquefaction. It should also be noted that Section 1804.5 of the Uniform Building Code (International Conference of Building Officials, 1994 and 1997) also requires an evaluation of the liquefaction potential of a site for new construction.

SP 117 provides a summary overview of analysis and mitigation of liquefaction hazards. The document also provide guidelines for the review of site-investigation reports by regulatory agencies who have been designated to enforce the Seismic Hazards Mapping Act. However, building officials from both the City and County of Los Angeles desired to have more definitive guidance to aid their agencies in the review of geotechnical investigations that must address seismic hazards and mitigations. Specifically, both agencies sought assistance in the development of recommendations for dealing with earthquake-induced liquefaction and landslide hazards. The City and County of Los Angeles were joined by their counterparts in other southern California counties that include Orange, San Bernardino, San Diego, Riverside, and Ventura Counties.

An "Implementation Committee" was convened under the auspices of the Southern California Earthquake Center (SCEC) at the University of Southern California. It was decided to address the issue of liquefaction first, with the landslide hazards to be addressed after the liquefaction implementation guidelines had been completed. The Liquefaction Implementation Committee has participating members from the practicing professional, academic, and regulatory communities. This Implementation Committee has recently published a technical report through SCEC entitled "Recommended Procedures for Implementation of DMG Special Publication 117 – Guidelines for Analyzing and Mitigating Liquefaction Hazard in California" (Martin and Lew, 1999).

The purpose of the SCEC report was two-fold. The first purpose is to present information that will be useful and informative to Building Officials so that they can properly and consistently review and approve geotechnical reports that address liquefaction hazard and mitigation. The second purpose is to provide a broad-brush survey of some of the most common methods of analyses and mitigation techniques that will be useful to geotechnical engineers, engineering geologists, building officials, and other affected parties including owners, architects, and structural engineers. This paper is intended to summarize that information that should be known by structural engineers about the current state-of-the-practice in liquefaction hazard analysis and mitigation.

## LIQUEFACTION HAZARD ZONES

The State Geologist is required under the Seismic Hazards Mapping Act of 1991 to delineate various "seismic hazard zones," including those for liquefaction. The criteria for delineating Liquefaction Zones were developed by the Seismic Hazards Mapping Act Advisory Committee for the California State Mining and Geology Board in 1993, and will be contained in a revised document entitled "Guidelines For Delineating Seismic Hazard Zones" (CDMG, 1999). Under those criteria, Liquefaction Zones are areas meeting one or more of the following:

1. Areas where liquefaction has occurred during historical earthquakes.
2. Areas of uncompacted or poorly compacted fills containing liquefaction-susceptible materials that are saturated, nearly saturated, or may be expected to become saturated.
3. Areas where sufficient existing geotechnical data and analyses indicate that the soils are potentially susceptible to liquefaction.
4. For areas where geotechnical data are lacking or insufficient, zones are delineated using one or more of the following criteria:
    a) Areas containing soil of late Holocene age (less than 1,000 years old, current river channels and their historical flood plains, marshes, and estuaries) where the groundwater is less than 40 feet deep and the anticipated earthquake peak ground acceleration (PGA) having a 10% probability of being exceeded in 50 years is greater than 0.1g.
    b) Areas containing soils of Holocene age (less than 11,000 years old) where the groundwater is less than 30 feet below the surface and the PGA (10% in 50 years) is greater than 0.2g.
    c) Areas containing soils of latest Pleistocene age (11,000 to 15,000 years before present) where the groundwater is less than 20 feet below the surface and the PGA (10% in 50 years) is greater than 0.3g.

It should be noted that the groundwater levels used for the purposes of zoning are the historically shallowest (highest) groundwater levels using the results of groundwater studies. Sediments deposited on canyon floors are presumed to become saturated during wet seasons and shallow water conditions can occur in narrow stream valleys that can receive an abundance of water runoff from canyon drainages and tributary streams during periods of high precipitation.

Seismic Hazard Zones for potentially liquefiable soils within a region based on these criteria are presented on 7.5-minute quadrangle sheet maps at a scale of 1:24,000. The Seismic Hazard Zone Maps are developed using a combination of historical records, field observations, and computer-mapping technology. These maps may not identify all areas that have potential for liquefaction; a site located outside of a zone of required investigation is not necessarily free from liquefaction hazard. The zones do not always include lateral spread run-out areas.

Seismic Hazard Zone maps are in the process of being released by the California Department of Conservation, Division of Mines and Geology. The maps present zones of identified landslide and liquefaction hazards as determined by the criteria established by the Seismic Hazards Mapping Act Advisory Committee. Currently, these maps are available for limited areas of southern California, mostly for Los Angeles County.

## PRELIMINARY SCREENING FOR LIQUEFACTION

The SP 117 Guidelines state that an investigation of the potential seismic hazards at a site can be performed in two steps: (1) a screening investigation and (2) a quantitative evaluation. The screening investigation should include a review of relevant topographic, geologic and soils engineering maps and reports, aerial photographs, groundwater contour maps, water well logs, agricultural soil survey maps, the history of liquefaction in the area, and other relevant published and unpublished reports. The purpose of the screening investigations for sites within zones of required study is to filter out sites that have no potential or low potential for liquefaction.

The Seismic Hazard Zone maps include Liquefaction Hazard Zones. These maps are based on broad regional studies and do not replace site-specific studies. The fact that a site is located within a Liquefaction Hazard Zone does not mean that there necessarily is a significant liquefaction potential at the site, only that a study should be performed to determine if there is.

The following screening criteria may be applied to determine if further quantitative evaluation of liquefaction hazard potential is not required:

- If the estimated maximum-past-, current-, and maximum-future-ground-water-levels (i.e., the highest ground water level applicable for liquefaction analyses) are determined to be deeper than 50 feet below the existing ground surface or proposed finished grade (whichever is deeper), liquefaction assessments are not required.
- If "bedrock" or similar lithified formational material underlies the site, those materials need not be considered liquefiable and no analysis of their liquefaction potential is necessary.
- If the corrected standard penetration blow count, $(N_1)_{60}$, is greater than or equal to 30 in all samples with a sufficient number of tests, liquefaction assessments are not required. If cone penetration test soundings are made, the corrected cone penetration test tip resistance, $q_{c1N}$, should be greater than or equal to 160 in all soundings in sand materials.
- If clayey soil materials are encountered during site exploration, those materials may be considered non-liquefiable. For purposes of this screening, clayey soils are those that have a clay content (particle size <0.005 mm) greater than 15 percent. However, based on the so-called "Chinese Criteria," (Seed and Idriss, 1982) clayey soils having all of the following characteristics may be susceptible to severe strength loss:
  - Percent finer than 0.005 mm less than 15 percent
  - Liquid Limit less than 35
  - Water Content greater than 0.9 x Liquid Limit

If the screening investigation clearly demonstrates the absence of liquefaction hazards at a project site and the lead agency technical reviewer concurs, the screening investigation will satisfy the site investigation report requirement for liquefaction hazards. If not, a quantitative evaluation will be required to assess the liquefaction hazards.

## FIELD INVESTIGATIONS FOR LIQUEFACTION HAZARD EVALUATION

Field (or geotechnical) investigations are routinely performed for new projects as part of the normal development and design process. Geologic reconnaissance and subsurface explorations are normally performed as part of the field exploration program even when liquefaction does not need to be investigated.

### Geologic Reconnaissance

Geologic research and reconnaissance are important to provide information to define the extent of unconsolidated deposits that may be prone to liquefaction. Such information should be presented on geologic maps and cross sections and provide a description of the formations present at the site that includes the nature, thickness, and origin of Quaternary deposits with liquefaction potential. There also should be an analysis of groundwater conditions at the site that includes the highest recorded water level and the highest water level likely to occur under the most adverse foreseeable conditions in the future.

During the field investigation, the engineering geologist should map the limits of unconsolidated deposits with liquefaction potential. Liquefaction typically occurs in cohesionless silt, sand, and fine-grained gravel deposits of Holocene to late Pleistocene age in areas where the groundwater is shallower than about 50 feet. Common geologic settings include unlithified sediments in coastal regions, bays, estuaries, river floodplains and basins, areas surrounding lakes and reservoirs, and wind-deposited dunes and loess. In many coastal regions, liquefiable sediments occupy back-filled river channels that were excavated during Pleistocene low stands of sea level, particularly during the most recent glacial stage. Among the most easily liquefiable deposits are beach sand, dune sand, and clean alluvium that were deposited following the rise in sea level at the start of the Holocene age, about 11,000 years ago.

Shallow groundwater may exist for a variety of reasons, some of which are of natural and or man-made origin. Groundwater may be shallow because the ground surface is only slightly above the elevation of the ocean, a nearby lake or reservoir, or the sill of a basin. Another concern is man-made lakes and reservoirs that may create a shallow groundwater table in young sediments that were previously unsaturated.

### Subsurface Explorations

Subsurface explorations are routinely performed using borings, with cone penetration tests (CPTs) becoming more commonplace. The scope of the field exploration program will depend on the type of development or building planned. It might be expected that a high-rise building may require an array of closely spaced exploratory borings (and CPTs), whereas a large housing tract will have an array of exploratory borings or pits (or CPTs) that may be less closely spaced.

There are various methods for evaluation of liquefaction potential. The most popular and common methods relate in situ soil indices, such as the standard penetration test (SPT) or the cone penetration test, to observed liquefaction occurrence or non-occurrence during major earthquakes. These indices can generally be routinely and economically obtained. In the case of silts or sandy silts, liquefaction evaluation may require the cyclic testing of soil samples, which can be obtained by high quality sampling techniques during the field exploration program.

The normal field exploration program may need to be expanded to evaluate the potential for liquefaction. Additional and/or deeper SPT-borings and CPTs may be warranted, or the field exploration program may be augmented with other forms of exploration. The exploration program should be planned to determine the soil stratigraphy, groundwater level, and indices that could be used to evaluate the potential for liquefaction by either in situ testing or by laboratory testing of soil samples. Good engineering judgment will need to be exercised in determining the exploration program needed to obtain adequate and sufficient geotechnical information to evaluate the potential for liquefaction. An inadequate exploration program could lead to either overly conservative or unconservative conclusions and actions.

### Depth of Analysis for Liquefaction Evaluation

Traditionally, a depth of 50 feet (about 15 m) has been used as the depth of analysis for the evaluation of liquefaction. The Seed and Idriss EERI Monograph on "Ground Motions and Soil Liquefaction During Earthquakes" (1982) does not recommend a minimum depth for evaluation, but notes 40 feet (12 m) as a depth to which some of the numerical quantities in the "simplified procedure" can be estimated reasonably. Liquefaction has been known to occur during earthquakes at deeper depths than 50 feet (15 m) given the proper conditions such as low-density granular soils, presence of ground water, and sufficient cycles of earthquake ground motion.

Experience has shown that the 50-foot (15 m) depth may be adequate for the evaluation of liquefaction potential in most cases, however, there may be situations where this depth may not be sufficiently deep.

It is recommended that a minimum depth of 50 feet (15 m) below the existing ground surface or lowest proposed finished grade (whichever is lower) be investigated for liquefaction potential. Where a structure may have subterranean construction or deep foundations (e.g., caissons or piles), the depth of investigation should extend to a depth that is a minimum of 20 feet (6 m) below the lowest expected foundation level (e.g., caisson bottom or pile tip) or 50 feet (15 m) below the existing ground surface or lowest proposed finished grade, whichever is deeper.

If, during the investigation, the indices to evaluate liquefaction indicate that the liquefaction potential may extend below that depth, the exploration should be continued until a significant thickness (at least 10 feet or 3 m, to the extent possible) of nonliquefiable soils are encountered.

## Liquefaction Assessment by Use of the Standard Penetration Test (SPT)

One of the most widely used semi-empirical procedures for estimation of liquefaction potential utilizes Standard Penetration Test (SPT) N-values to estimate a soil's liquefaction resistance.

Primarily because of their inherent variability, sensitivity to test procedure, and uncertainty, SPT N-values have the potential to provide misleading assessments of liquefaction hazard, if the tests are not performed carefully. The engineer who wants to utilize the results of SPT N-values to estimate liquefaction potential should become familiar with the details of SPT sampling as given in ASTM D 1586 (ASTM, 1998a) and ASTM D 6066-96 (ASTM, 1998b). Familiarity of these standards will help avoid, or at least reduce, some of the major sources of error.

The semi-empirical procedures that relate SPT N-values to liquefaction resistance use an SPT blow count that is normalized to an effective overburden pressure of 100 KPa (or 1.044 ton per square foot). This normalized SPT blow count is denoted as $N_1$, which is obtained by multiplying the uncorrected SPT blow count by a depth correction factor, $C_N$. A correction factor may be needed to correct the blow count for an energy ratio of 60%, which has been adopted as the average SPT energy for North American geotechnical practice. Additional correction factors may need to be applied to obtain the corrected normalized SPT N-value, $(N_1)_{60}$. It has been suggested that the corrections should be applied according to the following formula:

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

Where $N_M$ = measured standard penetration resistance

$C_N$ = depth correction factor

$C_E$ = hammer energy ratio (ER) correction factor

$C_B$ = borehole diameter correction factor

$C_R$ = rod length correction factor

$C_S$ = correction factor for samplers with or without liners

A useful reference, which discusses energy delivery and the SPT, is Seed et al. (1985). A summary of the recommended procedure for performing the SPT is given in Table 1. Table 2 presents the correction factors for the field SPT "N" values.

The SPT tests should be performed to investigate the liquefaction potential of the soils to the minimum depths recommended earlier. However, if the SPT tests indicate that there is a potential for liquefaction to extend below the minimum depth, SPT tests should be continued until a significant thickness of nonliquefiable soils are encountered. This thickness is recommended to be at least 10 feet or 3 meters.

## SPT Testing in Gravel Deposits

SPT tests are difficult, at best, to perform in gravel deposits. Because of the coarse size of the particles, as compared to the size of the sampler, those deposits have the potential to provide misleadingly high N-values.

An alternative in gravel deposits is to obtain Becker Hammer blow counts, which have been correlated to the standard penetration test blow count. Another alternative would be to measure the shear wave velocities of the gravel deposits to determine the liquefaction potential.

**Table 1. Recommended SPT Procedure**

| | |
|---|---|
| Borehole size | 66 mm < Diameter < 115 mm |
| Borehole support | Casing for full length and/or drilling mud |
| Drilling | Wash boring; side discharge bit<br>Rotary boring; side or upward discharge bit<br>Clean bottom of borehole* |
| Drill rods | A or AW for depths of less than 15 m<br>N or NW for greater depths |
| Sampler | Standard 51 mm O.D. +/- 1 mm<br>35 mm I.D. +/- 1 mm<br>>457 mm length |
| Penetration Resistance | Record number of blows for each 150 mm;<br>N = number of blows from 150 to 450 mm penetration |
| Blow count Rate | 30 to 40 blows per minute |

* Maximum soil heave within casing <70 mm

**Table 2. Corrections to Field SPT N-Values (modified from Youd and Idriss, 1997)**

| Factor | Equipment Variable | Term | Correction |
|---|---|---|---|
| Overburden Pressure | | $C_N$ | $(P_a / \sigma'_{vo})^{0.5}$;<br>$0.4 \leq C_N \leq 2$ * |
| Energy Ratio | Safety Hammer<br>Donut Hammer<br>Automatic Trip Hammer | $C_E$ | 0.60 to 1.17<br>0.45 to 1.00<br>0.9 to 1.6 |
| Borehole Diameter | 65 mm to 115 mm<br>150 mm<br>200 mm | $C_B$ | 1.0<br>1.05<br>1.15 |
| Rod Length** | 3 m to 4 m<br>4 m to 6 m<br>6 m to 10 m<br>10 m to 30 m<br>>30 m | $C_R$ | 0.75<br>0.85<br>0.95<br>1.0<br><1.0 |
| Sampling Method | Standard Sampler<br>Sampler without liners | $C_S$ | 1.0<br>1.2 |

* The Implementation Committee recommends using a minimum of 0.4.
** Actual total rod length, not depth below ground surface

## Liquefaction Assessment by Use of the Cone Penetration Test (CPT)

This section presents suggested minimum requirements for Cone Penetration Test or CPT-based liquefaction evaluation.

The primary advantages of the CPT method are:

1. The method provides an almost continuous penetration resistance profile that can be used for stratigraphic interpretation.
2. The repeatability of the test is very good.
3. The test is fast and economical compared to drilling and laboratory testing of soil samples.

The limitations of the method are:

1. The method does not routinely provide soil samples for laboratory tests.
2. The method provides approximate interpreted soil behavior types and not the actual soil types according to ASTM Test Methods D 2488 (Visual Classification) or D 2487 (USCS Classification) [ASTM, 1998].
3. The test cannot be performed in gravelly soils and sometimes the presence of hard/dense crusts or layers at shallow depths makes penetration to desired depths difficult.

The CPT method should be performed in general accordance with ASTM D 3441 (ASTM, 1998).

The recent proceedings from the January 1996 NCEER workshop (Youd and Idriss, 1997) on the evaluation of liquefaction resistance of soils represent the most up-to-date consensus among some of the foremost experts in the liquefaction field. That document will likely set the standard of practice for liquefaction potential evaluation for the next several years.

Historically, CPT-based liquefaction evaluations typically use a CPT-SPT correlation to estimate the SPT blow count values from CPT data. This method of liquefaction evaluation is also considered acceptable according to the NCEER report (Youd and Idriss, 1997). However, direct use of CPT may have supplanted these procedures.

The NCEER report identifies the CPT as a prime candidate for reconnaissance exploration and indicates that the CPT can be used to develop preliminary soil and liquefaction resistance profiles for site investigations. These preliminary profiles should always be checked by the use of selected boring samples retrieved during site investigations.

In practice, site investigations are seldom performed solely for the purpose of evaluating liquefaction potential. Soil samples (and therefore, soil borings), both disturbed and "relatively undisturbed," are usually needed to perform laboratory tests for typical geotechnical studies. Therefore, typically CPT alone will not be sufficient to provide the geotechnical consultant with all the information needed to prepare a complete geotechnical report.

The following suggestions on the use of CPT soundings for liquefaction study are made:

- CPT soundings should be extended to the minimum depth needed for proper evaluation of liquefaction potential (i.e., the same minimum depth recommendations used for the SPT evaluation should be met).

  The minimum recommended depth of investigation is 50 feet (15 m). When a structure may have subterranean construction or deep foundations, the depth should extend to a minimum of 20 feet (6 m) below the lowest expected foundation level (bottom of caisson or pile) or 50 feet (15 m) below the ground surface, whichever is deeper. If there is a potential for liquefaction to extend below the minimum depth, CPTs should be continued until a significant thickness (at least 10 feet or 3 m) of nonliquefiable soils are encountered. The CPT tip resistance in that zone should exceed a corrected value of 160 tsf (16 MPa) in coarse-grained soils or the soils should be demonstrated to be nonliquefiable.

- As a minimum, one boring used for sampling and testing (for providing other geotechnical recommendations) should be performed next to one of the CPT soundings to check that the CPT-soil behavior type interpretations are reasonable for the project site. The boring and CPT sounding should not be spaced so closely that stress relief would significantly affect the results; therefore, consideration should be given to the sequence of the explorations. This boring should be extended to at least the same depth as the CPT sounding. Soil samples should be taken at least every 2½ or 3 feet using SPT, Modified California Drive, or other appropriate samplers, or at changes in soil stratigraphy. Blow-counts from the Modified California or other samplers should not be relied upon. Any differences between the SPT and CPT should be reconciled before proceeding with liquefaction analyses.

- Additional confirmation borings may be necessary if the site is large or the subsurface conditions vary significantly within the site. If an additional boring(s) is performed for other geotechnical design purposes, it may serve as confirmation boring(s). The need for and the number of additional borings shall be determined by the project geotechnical consultant, subject to the review of the appropriate regulatory agencies.

### Liquefaction Assessment Using Other In Situ Indices

As data and correlations are being developed and verified with other in situ indices, alternative methods of assessment may become available. A limited amount of data have been collected and correlated to relate the liquefaction potential to shear wave velocities (Youd and Idriss, 1997). In particular, the shear wave velocity approach may be an alternative method to the Becker Hammer method (Youd and Idriss, 1997) for evaluating the liquefaction potential of gravelly deposits.

### Overburden Corrections For Differing Water Table Conditions

To perform analyses of liquefaction triggering, liquefaction settlement, seismically induced settlement, and lateral spreading, it is necessary to develop a profile of SPT blow-counts or CPT $q_c$-values that have been normalized using the effective overburden pressure. That normalization should be performed using the effective stress profile that existed at the time the SPT or CPT testing was performed. Then, those normalized values are held constant throughout the remainder of the analyses, regardless of whether or not the analyses are performed using higher or lower water-table conditions. Although the possibility exists that softening effects due to soil moistening can influence SPT or CPT results if the water table fluctuates, it is commonly assumed that the only effect that changes in the water table have on the results are due to changes in the effective overburden stress.

Raw, field N-values (or $q_c$-values) obtained under one set of groundwater conditions should not be input into an analysis where they are then normalized using $C_N$ correction factors based on a new (different) water table depth.

## GROUND MOTIONS FOR LIQUEFACTION ANALYSES

To perform analyses of liquefaction triggering, liquefaction settlement, seismically induced settlement, and lateral spreading, an earthquake magnitude, a peak horizontal ground acceleration, and a distance are needed. To obtain those values, consultants can perform either a site-specific seismic hazard analysis or they can use the moderately detailed CDMG seismic hazard maps. For some analyses, the CDMG seismic hazard maps may be sufficient, however, a site-specific hazard hazard analysis may provide better estimation of the ground motions at this point in time.

There are two basic approaches for calculating site-specific design ground motions: deterministic and probabilistic. In the deterministic approach, a specific scenario earthquake is selected (i.e., with a particular magnitude and location) and the ground motion is computed using applicable attenuation relations. Even when the earthquake is specified in terms of its magnitude and distance to the site, there is still a large range of potential ground motions that could occur at the site. This variability of the ground motions can be characterized by the standard deviation of the attenuation relation. Traditionally, in deterministic analyses, either the median (50th percentile) or median-plus-one-standard-deviation (84th percentile) ground motion is selected for use as design ground motion.

In the probabilistic approach, multiple potential earthquakes are considered. That is, all of the magnitudes and locations believed to be applicable to all of the presumed sources in an area are considered. Thus, the probabilistic approach does not consider just one scenario, but all of the presumed possible scenarios. For a normal probabilistic analysis, the rate of earthquake occurrence (how often each scenario earthquake occurs) and the probabilities of earthquake magnitudes, locations, and rupture dimensions, also are considered. Also, rather than just considering a median or 84th percentile ground motion, the probabilistic approach considers all possible ground motions for each earthquake and their associated probabilities of occurring based on the variability of the ground motion attenuation relation. In addition, more elaborate probabilistic analyses can be performed using logic tree or Monte Carlo simulations to consider modeling uncertainty.

To lessen the burden of performing site-specific probabilistic seismic hazard analyses for some analyses pursuant to the Seismic Hazards Mapping Act, the use of a set of standardized ground-motion maps may be considered as a procedure to estimate ground motion for liquefaction analyses. To facilitate that procedure, the State has developed a series of moderately detailed earthquake ground motion maps on a quadrangle by quadrangle basis (Petersen et al., 1996). Ground motions shown on the maps are expressed as peak ground accelerations (PGA) having a 10% probability of being exceeded in a 50 year period (corresponding to a 475-year return period) in keeping with the UBC-level of hazard.

The ground motion maps prepared by the State of California are contained in the evaluation reports that correspond to each seismic hazard zone quadrangle map. Color images of seismic hazard zone maps, and the text of associated evaluation reports are accessible at the CDMG web site found at the address: http://www.consrv.ca.gov/.

## EVALUATION OF LIQUEFACTION HAZARDS

### Liquefaction Potential

The most basic procedure used in engineering practice for assessment of site liquefaction potential is that of the "Simplified Procedure" originally developed by Seed and Idriss (1971, 1982) with subsequent refinements by Seed et al. (1983), Seed et al. (1985), Seed and De Alba (1986), and Seed and Harder (1990). That procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude).

Values of CRR were originally established from empirical correlations using extensive databases for sites that did or did not liquefy during past earthquakes where values of $(N_1)_{60}$ could be correlated with liquefied strata. The current version of the baseline chart defining values of CRR as a function of $(N_1)_{60}$ for moment magnitude 7.5 earthquakes is shown on Figure 1. That chart was recently established by a consensus at the 1996 NCEER Workshop, which convened a group of experts to review new developments (Youd and Idriss, 1997). A corresponding chart documenting revised magnitude scaling factors was also developed, and is shown on Figure 2. Note that there are significant increases in scaling factors for moment magnitudes less than 7.5, compared to the original values. The new scaling factors supersede those in previous documents, for example: Seed et al. (1985).

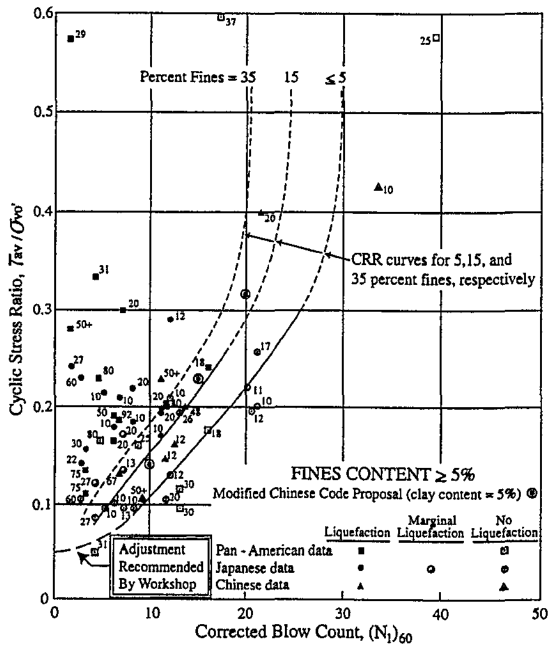


**Figure 1.** Simplified Base Curve Recommended for Determination of CRR from SPT Data for Moment Magnitude 7.5 Along with Empirical Liquefaction Data (after Youd and Idriss, 1997)

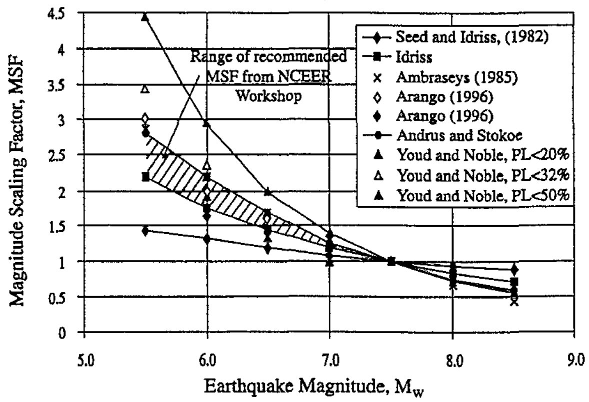


**Figure 2.** Magnitude Scaling Factors Derived by Various Investigators (After Youd and Idriss, 1997)

For estimating values of the earthquake-induced cyclic stress ratio, CSR, the NCEER Workshop recommended essentially no change to the original simplified procedure (Seed and Idriss, 1971), where the use of a mean $r_d$ factor defining the reduction in CSR with depth is usually adopted for routine engineering practice, as shown in Figure 3. As an alternative, a site-specific response analysis of the ground motions can be performed. Then values of CRR and CSR once established for a soil stratum at a given depth, allow a factor of safety against liquefaction, CRR/CSR, to be computed.

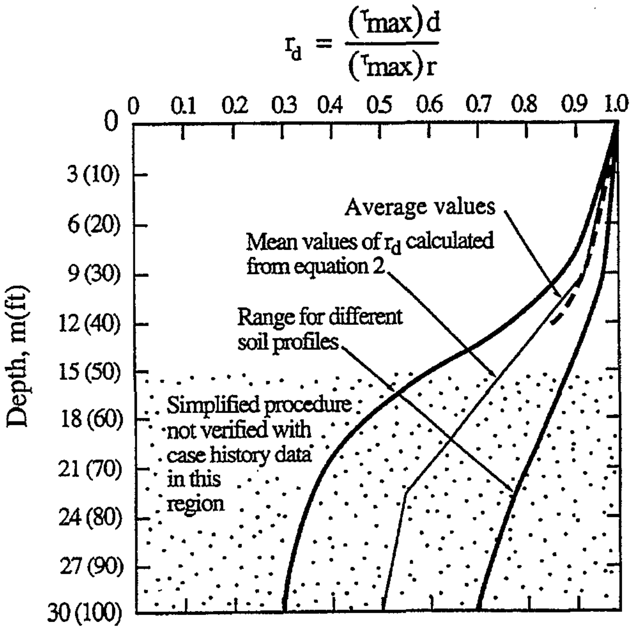


**Figure 3.** $r_d$ Versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean Value Lines (After Youd and Idriss, 1997)

The above procedure should be regarded as the minimum requirement for evaluating site liquefaction potential, where SPT data are used as a basis for determining liquefaction strengths. However, the use of the CPT is now recognized as one of the preferred investigation tools to estimate liquefaction strengths. It has the advantage of providing continuous data with depth, and the relatively low cost of performing multiple soundings over a site enable continuity of liquefiable strata to be assessed. The latter advantage is particularly important in determining the potential for lateral spreads and significant differential post-liquefaction settlements.

Historically, in using CPT data to establish liquefaction strengths, CPT data have been converted to equivalent SPT blow counts using procedures such as described by Martin (1992). With such an approach, confirmation of correlations is essential using at least one SPT borehole (needed anyway for laboratory classification tests) adjacent to a CPT sounding. An example of such a verification study is illustrated in Figure 4. SPT blow counts at 5 foot intervals and corrected for fines content (using the procedure described by Seed et al. [1985]), are compared to CPT-derived blow count data derived using the correlation chart described by Martin et al. (1991). In general, the CPT-derived SPT data are seen to be in reasonable agreement with the measured SPT data. However, note that the five-foot sampling interval used for the SPT lacks the ability to pick up the significant variations in blow counts with depth, typical of interbedded sedimentary stratigraphy.

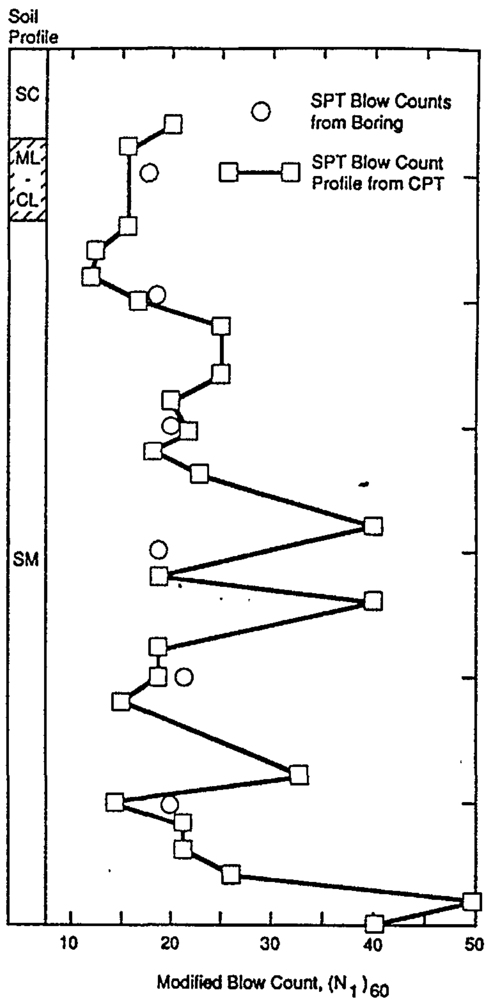


**Figure 4.** Comparison of Blow Counts from SPT and Those Derived from CPT Soundings (After Martin et al., 1991)

As discussed in the NCEER Workshop Proceedings, increased field performance data have become available at liquefaction sites investigated with CPT in recent years. Those data have facilitated the development of CPT-based liquefaction resistance correlations. These correlations allow direct calculation of CRR, without the need to convert CPT measurements to equivalent SPT blow counts and then applying SPT criteria.

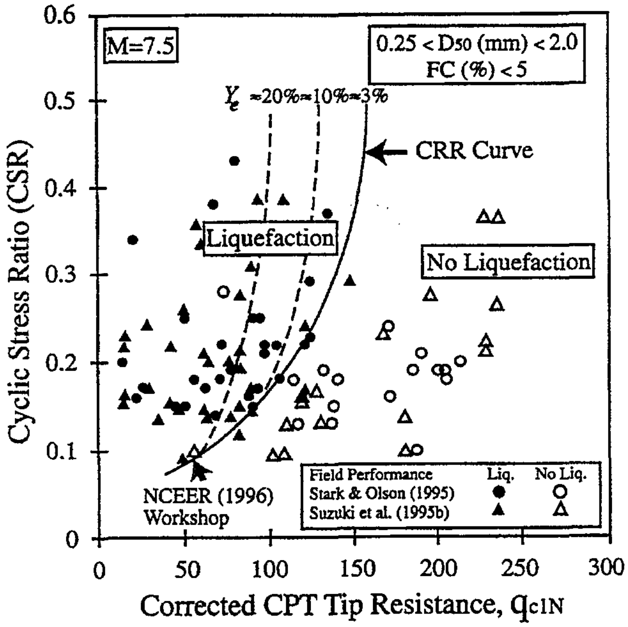


**Figure 5.** Curve Recommended for Determination of CRR from CPT Data Along with Empirical Liquefaction Data (After Robertson and Wride, 1997)

Figure 5 shows a chart developed by Robertson and Wride (Youd and Idriss, 1997) for determining liquefaction strengths for clean sands (fines content, FC, less than or equal to 5%) from CPT data. The chart, which is only valid for magnitude 7.5 earthquakes, shows calculated cyclic stress ratios plotted as a function of corrected and normalized CPT resistance, $q_{c1N}$, from sites where liquefaction effects were or were not observed following past earthquakes. A curve separates regions of the plot with data indicative of liquefaction from regions indicative of nonliquefaction. Dashed curves showing approximate cyclic shear strain potential, $\gamma_c$, as a function of $q_{c1N}$ are shown to emphasize that cyclic shear strain and ground deformation potential of liquefied soils decrease as penetration resistance increases.

The NCEER Workshop Proceedings provide an explicit commentary on how the new Robertson and Wride CPT procedure should be used for liquefaction evaluations. Because there is not complete consensus about this procedure, it is recommended that the method be used with care. A parallel borehole immediately adjacent to one CPT sounding should be drilled to check soil classification, particularly for clayey silts where the Chinese liquefaction criteria may be applicable.

## Hazard Assessment

The report on liquefaction assessment at a given site should include drill hole logs, field and corrected SPT blow counts, and classification test results, if SPT tests are performed. If CPT tests are performed, field and normalized CPT data (tip resistance, sleeve friction, and friction ratio) should be provided. The CPT data also should be interpreted to estimate soil behavior types. Values of $(N_1)_{60}$ and/or $q_{c1N}$ required to resist liquefaction for a factor of safety equal to 1.0 should be determined. The site liquefaction potential should be evaluated for a specific design earthquake magnitude and peak ground acceleration.

In using such data to evaluate mitigation needs and to establish appropriate factors of safety for analyses, four principal liquefaction-related potential hazards need to be considered:

1. Flow slides or large translational or rotational site failures mobilized by existing static stresses (i.e., the site static factor of safety drops below unity [1.0] due to low strengths of liquefied soil layers).
2. Limited lateral spreads of the order of feet or less triggered and sustained by the earthquake ground shaking.
3. Ground settlement.
4. Surface manifestation of underlying liquefaction.

Each of those hazards and their potential should be addressed in the site report, along with mitigation options, if appropriate. In evaluating the need to address the above hazards, an acceptable factor of safety needs to be chosen. Often the acceptable factor of safety is chosen arbitrarily. The CDMG guidelines (Special Publication 117) suggest a minimum factor of safety of 1.3 when using the CDMG ground motion maps, with a caveat that if lower values are calculated, the severity of the hazard should be evaluated. Clearly, no single value can be cited in a guideline, as considerable judgment is needed in weighing the many factors involved in the decision. Several of those factors are noted below:

1. The type of structure and its vulnerability to damage. In some situations, structural mitigation solutions may be more economical than ground remediation.
2. Levels of risk accepted by the owner or governmental regulations associated with questions related to design for life safety, limited structural damage, or essentially no damage.
3. Damage potential associated with the particular liquefaction hazards. Clearly flow failures or major lateral spreads pose more damage potential than differential settlement. Hence, factors of safety could be adjusted accordingly.
4. Damage potential associated with design earthquake magnitude. Clearly a magnitude 7.5 event is potentially far more damaging than a 6.5 event.
5. Damage potential associated with SPT values, i.e., low blow counts have a greater cyclic strain potential than higher blow counts.
6. Uncertainty in SPT- or CPT- derived liquefaction strengths used for evaluations. Note that a change in silt content from 5 to 15% could change a factor of safety from 1.0 to 1.25.
7. For high levels of design ground motion, factors of safety may be indeterminant. For example, if $(N_1)_{60} = 20$, $M = 7.5$ and fines content = 35%, liquefaction strengths cannot be accurately defined due to the vertical asymptote on the empirical strength curve.

Factors of safety in the range of about 1.1 may be acceptable for single family dwellings for example, where the potential for lateral spreading is very low and differential settlement is the hazard of concern, and where post-tensioned floor slabs are specified. On the other hand, factors of safety of 1.3 may be more appropriate for assessing hazards related to flow failure potential for large magnitude earthquake events.

The final choice of an appropriate factor of safety must reflect the particular conditions associated with a specific site and the vulnerability of site related structures. Considering the high levels of seismicity in California, Table 3 provides a generalized guide that reflects many of the factors noted above.

These factors of safety remain open for discussion. Within the Implementation Committee, there was not a complete consensus on these factors of safety; a minority position favors setting the factors of safety in the range between 1.25 and 1.5.

**Table 3. Suggested Factors of Safety for Liquefaction Hazard Assessment**

| Consequence of Liquefaction | $(N_1)_{60}$ (clean sand) | Factor of Safety |
|---|---|---|
| Settlement | $\leq 15$ | 1.1 |
| | $\geq 30$ | 1.0 |
| Surface Manifestation | $\leq 15$ | 1.2 |
| | $\geq 30$ | 1.0 |
| Lateral Spread | $\leq 15$ | 1.3 |
| | $\geq 30$ | 1.0 |

## MITIGATION OF LIQUEFACTION HAZARDS

In the presence of strong ground motion, liquefaction hazards are likely to occur in saturated cohesionless soils. Densification methods, modifications leading to improving the cohesive properties of the soil (hardening or mixing), removal and replacement, or permanent dewatering can reduce or eliminate liquefaction potential. Other methods such as reinforcement of the soil or the use of shallow or deep foundations designed to accommodate the occurrence of liquefaction and associated vertical and horizontal deformations may also achieve an acceptable level of risk.

Often a mitigation measure may involve the implementation of a combination of techniques or concepts such as densification, reinforcement, and mixing. Shallow or deep foundations may also be designed to work with partial ground improvement techniques in order to reduce cost while achieving an acceptable level of risk.

As stated in SP 117, mitigation should provide suitable levels of protection with regard to potential large lateral spread or flow failures, and more localized problems including bearing failure, settlements, and limited lateral displacements.

The choice of mitigation methods will depend on the extent of liquefaction and the related consequences. Also, the cost of mitigation must be considered in light of an acceptable level of risk. Youd (1998) has suggested that structural mitigation for liquefaction hazards may be acceptable where small lateral displacements (say less than 1 foot or 0.3 meter) and vertical settlement (say less than 4 inches or 10 centimeters) are predicted. Youd cites evidence that houses and small buildings with reinforced perimeter footings and connected grade beams have performed well in Japan, and similar performance should be expected in the United States.

### Performance Criteria

Liquefaction mitigation and performance criteria vary according to the acceptable level of risk for each structure type and human occupation considerations.

Implementation of mitigation measures should be designed to either eliminate all liquefaction potential or to allow partial improvement of the soils, provided the structure in question is designed to accommodate the resulting liquefaction-induced vertical and horizontal deformations. In some cases, engineers may decide to design mitigation measures to prevent liquefaction of certain soil types and allow limited deformations in others (i.e., allow some liquefaction).

During the initial site investigation and liquefaction evaluation, the engineer will determine the extent of liquefaction and potential consequences such as bearing failure, and vertical and/or horizontal deformations. Similarly, the engineer will determine the liquefaction hazard in terms of depth and lateral extent affecting the structure in question. The lateral extent affecting the structure will depend on whether there is potential for large lateral spreads toward or away from the structure and the influence of liquefied ground surrounding mitigated soils within the perimeter of the structure. Large lateral spread or flow failure hazards may be mitigated by the implementation of containment structures, removal or treatment of liquefiable soils, modification of site geometry, or drainage to lower the groundwater table.

Provided the potential for lateral spreads is addressed and level ground conditions exist, the extent of lateral mitigation beyond the structure footprint is related to bearing capacity and seepage conditions during and after the earthquake event (Port and Harbor Research Institute, 1997). Because liquefaction mitigation is likely to treat the ground underneath the structure to a sufficient depth, in most cases the bearing capacity reduction due to liquefiable ground outside the structure is not likely to govern the design. Instead, the propagation of excess pore pressures from liquefied to improved ground tends to determine the lateral extent of improvement required. Studies by Iai et al. (1988) indicate that in the presence of liquefiable clean sands an area of softening due to seepage flow occurs to a distance beyond the improved ground on the order of two-thirds of the liquefiable thickness layer.

The performance criteria for liquefaction mitigation, established during the initial investigation, may be in the form of a minimum, or average, penetration resistance value associated with a soil type (fines content, clay fraction, USCS classification, CPT soil behavior type index $I_c$, normalized CPT friction ratio), or a tolerable liquefaction settlement. Soils meeting the discussed Chinese criteria can be excluded from vertical deformation calculations, but they should be carefully considered for loss of strength and potential bearing failure or lateral deformations.

## Soil Improvement Options

Soil liquefaction improvement options can be characterized as densification, drainage, reinforcement, mixing, or replacement. As noted before, the implementation of these techniques may be designed to fully, or partially, eliminate the liquefaction potential, depending on input forces and the amount of deformation that the structure in question can tolerate. With regards to drainage techniques for liquefaction mitigation, only permanent dewatering works satisfactorily. The use of gravel or prefabricated drains, installed without soil densification, is unlikely to provide pore pressure relief during strong earthquakes and may not prevent excessive settlement. Their use should be evaluated with extreme caution. The following soil improvement methods have demonstrated successful performance in past earthquakes.

### Densification Techniques

The most widely used techniques for in-situ densification of liquefiable soils are vibro-compaction, vibro-replacement (also known as vibro-stone columns), deep dynamic compaction, and compaction (pressure) grouting (Hayden and Baez, 1994).

Vibro-compaction and vibro-replacement techniques use similar equipment, but use different backfill material to achieve densification of soils at depth. In vibro-compaction a sand backfill is generally used, whereas in vibro-replacement stone is used as backfill material. Vibro-compaction is generally effective if the soils to be densified are sands containing less than approximately 10 percent fine-grained material passing the No. 200 sieve. Vibro-replacement is generally effective in soils containing less than 15 to 20% fines. However, recent experience (Luehring et al., 1998) has verified that even non-plastic sandy silts can be densified by a combination of vibro-replacement and vertical band (wick) drains. In such a case, the vertical band drains are installed at the midpoint of stone column locations prior to installation of vibro-replacement. Due to the usual variation of liquefiable soil types in a given profile and economy of the system, vibro-replacement is typically the most widely used liquefaction countermeasure used in North America (Hayden and Baez, 1994).

Deep dynamic compaction involves the use of impact energy on the ground surface to densify and compact subsurface soils. Weights typically ranging from 10 to 30 tons are lifted with standard, modified, or specialty machines and dropped from about 50 to 120 feet heights. Free-fall impact energy is controlled by selecting the weight, drop height, number of drops per point and the spacings of the grid. Empirical relationships are available to design deep dynamic compaction programs to treat specific site requirements and reconstitute liquefiable soils to a denser condition (Lukas, 1986). In general, treatment depths of up to 35 feet may be achievable in granular soils. If surficial saturated cohesive soils are present or the groundwater table is within 3 to 5 feet of the surface, a granular layer is often needed to limit the loss of impact energy and transfer the forces to greater depths. The major limitations of the method are vibrations, flying matter, and noise. For these reasons, work often requires 100 to 200 feet clearance from adjacent occupied buildings or sensitive structures.

Displacement or compaction grouting involves the use of low slump, mortar-type grout pumped under pressure to densify loose soils by displacement. Compaction grouting pipes are typically installed by drilling or driving steel pipes of 2-inch internal diameter or greater. Injection of the stiff, 3-inch or less slump, cement grout is accomplished with pressures generally ranging from 100 to 300 pounds per square inch (psi). Refusal pressures of 400 to 500 psi are common in most granular soil projects where liquefaction is the problem. Grout pipes are installed in a grid pattern that usually ranges from 5 to 9 feet. The use of primary spacing patterns with secondary or tertiary intermediate patterns infilled later is effective to achieve difficult densification criteria. Grouting volumes can typically range from 3 to 12 percent of the treated soil volume in granular soils, although volumes up to 20 percent have been reported for extremely loose sands or silty soils. Inadequate compaction is likely to occur when sufficient vertical confinement (less than 8 to 10 feet of overburden) is not present. Theory and case histories on this technique can be found in Graf (1992), Baez and Henry (1993), and Boulanger and Hayden (1995), among others.

### Hardening (Mixing) Techniques

Hardening and/or mixing techniques seek to reduce the void space in the liquefiable soil by introducing grout materials either through permeation, mixing mechanically, or jetting. These techniques are known as permeation grouting, soil mixing, or jet grouting.

Permeation grouting involves the injection of low viscosity liquid grout into the pore spaces of granular soils. The base material is typically sodium silicate or microfine cements where the $D_{15}$ of the soil should be greater than 25 times $D_{85}$ of the grout for permeation.

With successful penetration and setting of the grout, a liquefiable soil with less that approximately 12 to 15 percent fine-grained fraction becomes a hardened mass. Use of this method in North America has been limited to a few projects such as the bridge pier in Santa Cruz, California (Mitchell and Wentz, 1991), and a tunnel horizon in downtown San Francisco. Design methodology and implementation of this technique are described in detail by Baker (1982) and Moseley (1993).

Jet grouting forms cylindrical or panel shapes of hardened soils to replace liquefiable, settlement sensitive, or permeable soils with soil-cement having strengths up to 2,500 psi. The method relies on up to 7,000 psi water pressure at the nozzle to cut soils, mix in place cement slurry and lift spoils to the surface. Control of the drill rotation and pull rates allows treatment of variable soils as described by Moseley (1993). Lightweight drill systems can be used in confined spaces such as inside existing buildings that are found to be at risk of liquefaction after construction.

Deep soil-mixing is a technique involving mixing of cementitious materials using a hollow-stem-auger and paddle arrangement. Gangs of 1 to 5 shafts with augers up to 3 feet or more in diameter are used to mix to depths of 100 feet or more. As the augers are advanced into the soil, the hollow stems are used as conduits to pump grout and inject into the soil at the tip. A trencher device has also been used successfully in Japan. Confining cells are created with the process as the augers are worked in overlapping configurations to form walls. Liquefaction is controlled by limiting the earthquake induced shear strains, and re-distributing shear stresses from soils within the confining cells to the walls. As with jet grouting, treatment of the full range of liquefiable soils is possible and shear strengths of 25 psi or more can be achieved even in silty soils. The method has been used for liquefaction remediation in only a few cases in North America, including Jackson Lake dam in Wyoming (Ryan and Jasperse, 1989). However, the method has found more extensive use in Japan (Schaefer, 1997).

### Structural Options

In some cases, structural mitigation for liquefaction effects may be more economical than soil improvement mitigation methods. However, structural mitigation may have little or no effect on the soil itself and may not reduce the potential for liquefaction. With structural mitigation, liquefaction and related ground deformations will still occur. A competent licensed structural engineer that is familiar with seismic design principles and has an understanding of liquefaction effects should design the structural mitigation. The structural mitigation should be designed to protect the structure from liquefaction-induced deformations, recognizing that the structural solution may have little or no improvement on the soil conditions that cause liquefaction. The appropriate means of structural mitigation may depend on the magnitude and type of soil deformation expected because of liquefaction. If liquefaction-induced flow slides or significant lateral spreading is expected, structural mitigation may not be practical or feasible in many cases. However, if the soil deformation is expected to be primarily vertical settlement, structural mitigation may be economically and technically feasible.

Where the structure is small (in building footprint) and light in weight, such as in typical single family residential houses, a post-tensioned slab foundation system may be beneficial. A post-tensioned slab should have sufficient rigidity to span over voids that may develop under the slab due to differential soil settlement. Light buildings also may be supported on continuous spread footings having isolated footings interconnected with grade beams. For heavier buildings with a low profile and relatively uniform mass distribution, a mat foundation may be feasible. The mat should be designed to bridge over local areas of settlement.

Piles or caissons extending to unliquefiable soil or bedrock below the potentially liquefiable soils may be feasible. Such designs should take into account the possible downdrag forces on the foundation elements due to settlement within the liquefiable and upper soils. Design must also accommodate seismic lateral forces that must be transmitted from the structure to the supporting soils and displacement demand, due to lateral ground deformations. As there may be a considerable loss of lateral soil stiffness and capacity, the piles or caissons will have to transmit the lateral loads to the deeper supporting soils. Experience from recent earthquakes (Benuska, 1990) have shown that battered piles are not effective in seismic conditions and should not be used in general. Floor slabs on grade should be expected to undergo settlements in sympathy with the liquefaction-induced settlements of the ground. If such floor settlements are not acceptable, the floor slabs could be structurally supported on the pile or caisson system.

Subterranean wall structures retaining potentially liquefiable soils may be subjected to substantially greater than normal active or at-rest lateral soil pressures. An evaluation should be made to determine the appropriate lateral earth pressures and structural design for this condition.

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

## Quality Assurance

Soil improvement techniques generally use specialized equipment and require experienced personnel. As such, they should be implemented by specialty construction companies with a minimum of 5 years experience in similar soils and job conditions as those considered for the project in question. Minimum quality assurance requirements will vary significantly depending on the technique being implemented.

For dynamic compaction, measurement of energy being delivered to the ground, sequence and timing of drops, as well as ground response in the form of crater depth and heave of the surrounding ground are important quality control parameters. Similarly, the location of the water table and presence of surface "hard pans" could greatly affect the quality and outcome of the densification process. Pore water pressures of an area recently treated should be allowed to dissipate before secondary treatments are implemented.

Vibro compaction and vibro replacement are generally performed with electric or hydraulic powered depth vibrators. When electric vibrators are used, the "free hanging" amperage as well as the amperage developed during construction are strong indicators of the likely success of the densification effort. The equipment should be capable of delivering the appropriate centrifugal force to cause densification. Stone backfill materials should be generally clean and hard with minimum durability index of about 40 (Caltest method 229). When the engineer relies on the stone backfill material to provide reinforcement for vertical or horizontal deformations, the stone should be crushed and have a suitable angle of internal friction. In some cases, computer data acquisition systems may be desired to monitor the depth of the vibrator, stone usage, and amperage developed.

Compaction grouting requires the verification of slump and consistency of the mix, as well as careful monitoring of grout volumes, injection pressures, and ground movement at the surface or next to sensitive structures. Critical projects also monitor pore water pressure and deep ground heave (borros points) development during the compaction grouting procedures. Because grout is typically injected in stages from the bottom up, at each stage a stopping criteria of grout volume, pressure, or heave is followed before proceeding with the next stage. Usage of grout casing with less than 2 inches in internal diameter should be avoided as it could cause detection of high back pressures before sufficient grout is injected. Over injection of grout in a primary phase may lead to early ground heave and may diminish densification effectiveness. Spacing and sequence of the grout points may also affect the quality of densification or ground movement achieved.

In general, the geotechnical engineer of record or his/her representatives conducts on-site inspection of all the procedures mentioned above. Testing locations are selected at random and tend to be located in the middle of a grid pattern formed by the densification locations. This is somewhat conservative and more realistic average results can be obtained by testing closer to the densification points. To permit pore pressure relaxation, a minimum of 48 to 72 hours after soil improvement is implemented should be allowed for prior to testing.

Soil mixing and jet grouting are also constructed with specialized equipment capable of rate of rotation and lifting rate of the injection ports. The grout or binder may include cement, fly ash, quicklime, or other components and additives designed to obtain the desired strength properties of the mixed soil. The binders are controlled for quality by checking consistency as measured by specific gravity. This is generally checked with mud balance or hydrometer devices. Pumping pressures and rates are designed to achieve production and strength requirements of the product. Installed columns are usually tested by wet sampling, coring with a minimum 3-inch core, CPT, pressuremeter, or seismic devices. Variation in quality and strength should be expected in the final product.

## REPORTING OF RESULTS

The report should be prepared under the direction of and signed by a competent registered professional civil (or geotechnical) engineer with the aid of a certified engineering geologist, having competence in the field of liquefaction hazard evaluation and mitigation. The geotechnical report should contain site-specific evaluations of the liquefaction hazard affecting the "project," and should identify portions of the site affected by the liquefaction hazard. The contents of the report should include, but shall not be limited to, the following:

1. Project description.
2. A description of the geologic and geotechnical conditions at the site, including an appropriate site location map. The descriptions should also include information regarding the site and near-site topography; topographic maps, geologic maps, and cross sections may be helpful.
3. Evaluation of the site-specific liquefaction hazard based on the geological and geotechnical conditions, in accordance with the current standards of practice.
4. Recommendations for appropriate mitigation measures.
5. Logs of field explorations. Detailed description of field test procedures, such as SPT and CPT should be given.
6. A description of laboratory tests conducted on soil/rock samples and summary of test results.
7. A summary of the assumptions used in analysis. Calculations should be submitted to facilitate review.

The report should contain a complete description of the test procedures used to evaluate liquefaction potential and the method of analysis used to evaluate the site-specific hazard. Assumptions should be clearly presented as well as supporting reference data.

## CONCLUDING REMARKS

A broad overview of the practice of liquefaction analysis, evaluation, and mitigation techniques has been presented. The state of the practice continues to evolve and advance at an ever-increasing pace and it is a certainty that new methodologies in liquefaction geotechnical engineering will develop.

It is the hope of the authors that this paper will provide structural engineers with more knowledge and understanding about the current state-of-the-practice of liquefaction analysis, evaluation, and mitigation. With this knowledge and understanding, structural engineers will be able to make better informed decisions regarding the available options to eliminate or reduce the effects of liquefaction hazards.


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The members of the Implementation Committee were: Geoffrey R. Martin and Marshall Lew, co-chairs, K. Arulmoli, Juan I. Baez, Thomas F. Blake, Johnnie Earnest, Fred Gharib, J. Goldhammer, David Hsu, Steve Kupferman, Jim O'Tousa, Charles R. Real, Wes Reeder, Ebrahim Simantob, and T. Leslie Youd.

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