CHAPTER 4 EVALUATION OF HAZARDS ASSOCIATED WITH SOIL LIQUEFACTION AND GROUND FAILURES

Soil Liquefaction Hazards

Introduction

Experience from past earthquakes demonstrates the vulnerability of waterfront structures and lifelines to seismically induced ground deformations. In an extensive review of the seismic performance of ports, Werner and Hung (1982) concluded that by far the most significant source of earthquake damage to waterfront structures has been pore pressure build-up in loose to medium-dense, saturated cohesionless soils that prevail in coastal and river environments. This observation has been supported by the occurrence of liquefaction-induced damage at numerous ports in the past decade (e.g., Seed et al. 1990; Chung, 1995; Mejia et al., 1995; Werner and Dickenson, 1996). Components of marine facilities conspicuous for poor performance due to earthquake-induced liquefaction include pile supported structures, sheet-pile retaining walls and bulkheads, and gravity retaining walls founded on or backfilled with loose sandy soils. The observed damage patterns commonly reflect the deleterious effects of (a) poor foundation soils which may have marginal static stability and which also tend to amplify the strong ground motions at these sites, and (b) the combination of high ground water levels and the existence of very loose to medium dense, sandy backfill and foundation soils. Saturated, sandy soils are susceptible to earthquake-induced *liquefaction*, a state wherein excess pore water pressures generated in the soil result in a temporary reduction, or complete loss, of strength and stiffness of the soil.

The liquefaction of a loose, saturated granular soil occurs when the cyclic shear stresses/strains passing through the soil deposit induce a progressive increase in the pore water pressure in excess of hydrostatic. During an earthquake the cyclic shear waves that propagate upward from the underlying bedrock induce the tendency for the loose sand layer to decrease in volume. If undrained conditions during the seismic disturbance are assumed, an increase in pore water pressure and resulting decrease of equal magnitude in the effective confining stress is required to keep the loose sand at constant volume. The degree of excess pore water pressure generation is largely a function of the initial density of the sand layer and the magnitude and duration of seismic shaking. In loose to medium dense sands pore pressures can be generated which are equal in magnitude to the confining stress. At this state, no effective (or intergranular) stress exists between the sand grains, and a complete loss of shear strength is temporarily experienced. The potential for the development of large strain (or flow) behavior is controlled by the initial relative density of the soil. The phenomena associated with the loss of strength of the sandy soils (e.g.; loss of bearing capacity, lateral spread, increase in active lateral earth pressures against retaining walls, loss of passive soil resistance below the dredge line and/or adjacent to anchor systems, and excessive settlements and lateral soil movements) contribute to the excessive deformations of waterfront structures. The large deformations associated with the failure of waterfront retaining walls can result in damage to wharf and backland, adversely affecting the operation of marine oil terminals.

One pertinent example, Figure 4-1, is provided by the observations made at the U.S. Naval Station at Treasure Island after the 1989 Loma Prieta earthquake. Inspection of the acceleration record obtained at the Treasure Island Fire Station shows that at about 15 seconds after the start of recording, the ground motion was subdued; this was probably caused by the occurrence of subsurface liquefaction. Liquefaction occurred after about 4 or 5 "cycles" of shaking, about 5 seconds of strong motion. Sand boils were observed at numerous location and bayward lateral spreading occurred with associated settlements. Ground cracking was visible with individual cracks as wide as 6 inches. Overall lateral spreading of 1 foot was estimated. Ground survey measurements indicate that settlements of 2 to 6 inches occurred variably across the island and that some areas had as much as 10 to 12 inches of settlement. The liquefaction related deformations resulted in damage to several structures and numerous broken underground utility lines, Egan (1991).

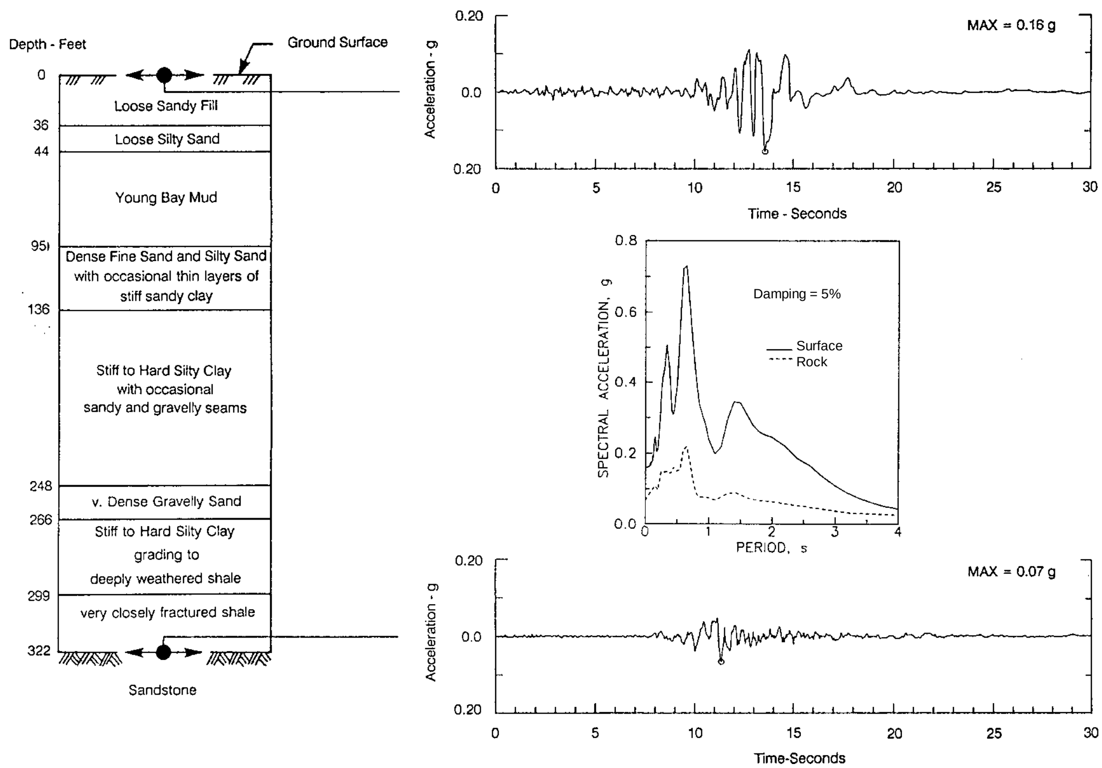


Figure 4-1: Soil Response at Treasure Island During the 1989 Loma Prieta Earthquake (After Seed et al., 1990)

Evaluation of Liquefaction Susceptibility

Experience from liquefaction-induced damage to structures and lifelines during past earthquakes shows that liquefaction hazards can be broadly classified into three general modes: (a) global instability and lateral spreading; (b) localized liquefaction hazard; and (c) failure or excessive deformation of walls and retaining structures. These liquefaction hazards may vary dramatically in scale, and the extent of each of these potential soil failure modes must be evaluated for projects in seismically active regions. The scope of the investigation required will reflect the nature and complexity of the geologic site conditions, the economics of the project, and the level of risk acceptable for the proposed structure or existing facility.

The evaluation of liquefaction hazard is generally performed in several stages that include: (a) preliminary geological/geotechnical site evaluation; (b) quantitative evaluation of liquefaction potential and its potential consequences; and, if necessary (c) development of mitigation and foundation remediation programs. A generalized flow chart for the evaluation of liquefaction hazards to pile supported structures is presented in Figure 4-2. This simplified chart is intended to illustrate the basic procedures involved in evaluating potential liquefaction hazards and developing mitigation programs.

Preliminary Site Investigation A preliminary site evaluation may involve establishing the topography, stratigraphy, and location of the ground water table at the project site. These geologic site evaluations must address the following three basic questions:

a) Are potentially liquefiable soil types present?

b) Are they saturated, and/or may they become saturated at some future date?

c) Are they of sufficient thickness and/or lateral extent as to pose potential risk with respect to major lateral spreading, foundation bearing failure or related settlements, overall site settlements, localized lateral ground movements, or localized ground displacements due to "ground loss"?

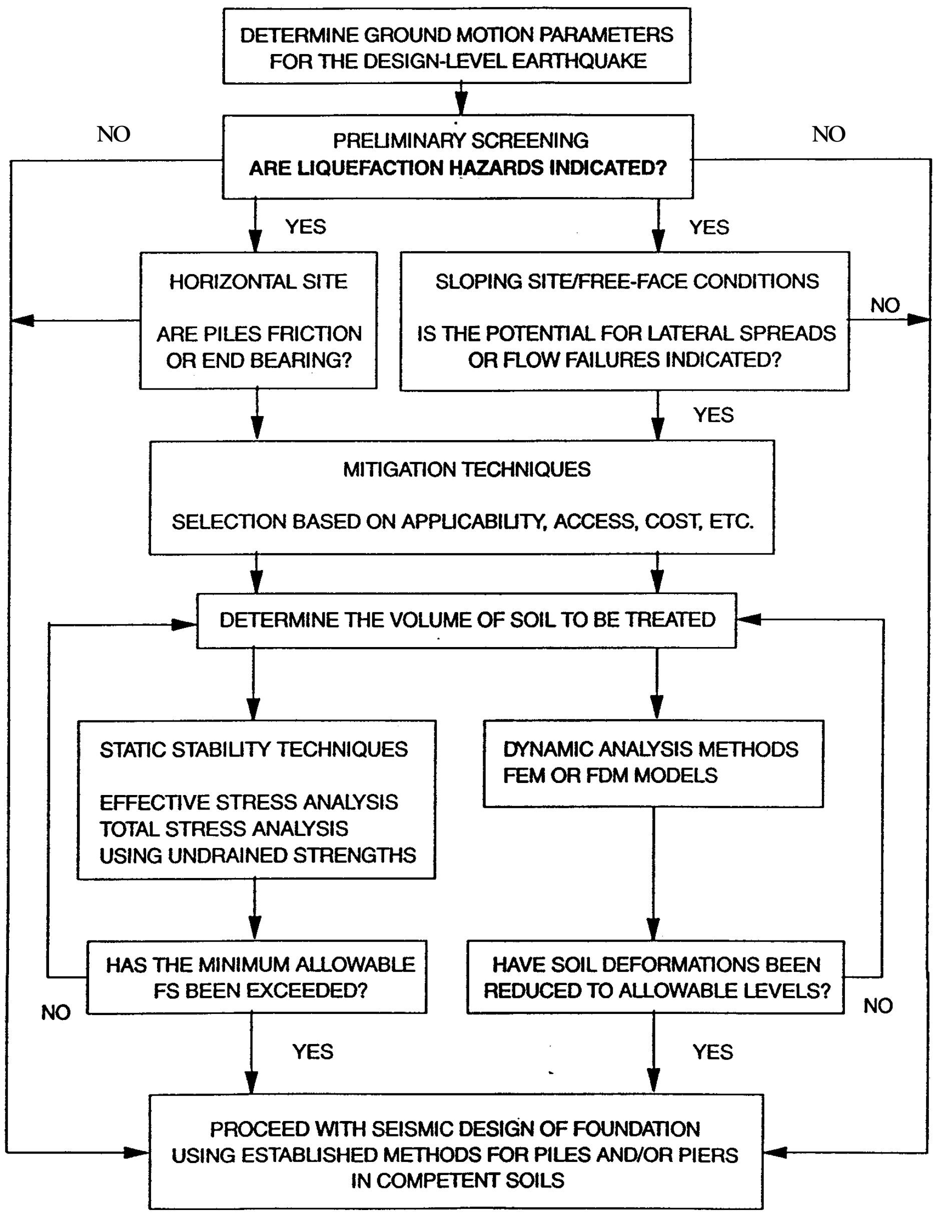


Figure 4-2: A Flow Chart For The Evaluation Of Liquefaction Hazards To Pile Supported Structures

The general geologic information, combined with ground motion data related to the design-level seismic events, can be used to provide a preliminary indication of the potential liquefaction susceptibility of soils at the site. This methodology has been developed by Youd and his co-workers, and is described in a recent state-of-the-art paper on the mapping of earthquake-induced liquefaction (Youd, 1991). In addition, California Department of Conservation, Division of Mines and Geology has established guidelines for mapping areas which might be susceptible to the occurrence of liquefaction. These zones establish where site-specific geotechnical investigations must be conducted to assess liquefaction potential and, if required, provide the technical basis to mitigate the liquefaction hazard. The following is taken directly from their criteria:

Liquefaction Hazard Zones are areas meeting one or more of the following criteria:

1. Areas known to have experienced liquefaction during historic earthquakes. Field studies following past earthquakes indicate liquefaction tends to recur at many sites during successive earthquakes

2. All areas of uncompacted fills containing liquefaction susceptible material 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 liquefiable. The vast majority of liquefaction hazard areas are underlain by recently deposited sand and/or silty sand. These deposits are not randomly distributed, but occur within a narrow range of sedimentary and hydrologic environments. Geologic criteria for assessing these environments are commonly used to delineate bounds of susceptibility zones evaluated from other criteria, such as geotechnical analysis (Youd, 1991) . Ground water data should be compiled from well logs and geotechnical borings. Analysis of aerial photographs of various vintages may delineate zones of flooding, sediment accumulation, or evidence of historic liquefaction. The Quaternary geology should be mapped and age estimates assigned based on ages reported in the literature, stratigraphic relationships and soil profile descriptions. In many areas of Holocene and Pleistocene deposition, geotechnical and hydrologic data are compiled. Geotechnical investigation reports with Standard Penetration Test (SPT) and/or Cone Penetration Test (CPT) and grain size distribution data can be used for liquefaction resistance evaluations.

4. Areas where geotechnical data are insufficient. The correlation of Seed et al. (1985) , and the $(N_1)_{60}$ data can be used to assess liquefaction susceptibility. Since geotechnical analyses are usually made using limited available data the susceptibly zones should be delineated by use of geologic criteria. Geologic cross sections, tied to boreholes and/or trenches, should be constructed for correlation purposes. The units characterized by geotechnical analyses are correlated with surface and subsurface units and extrapolated for the mapping project.

CDMG criteria uses the minimum level of seismic excitation for liquefaction hazard zones to be that level defined by a magnitude 7.5-weighted peak ground surface acceleration for UBC S2 soil conditions with a 10 percent probability of exceedance over a 50-year period.

In areas of limited or no geotechnical data, susceptibility zones are identified by CDMG geologic criteria as follows:

(a) Areas containing soil deposits of late Holocene age (current river channels and their historic floodplains, marshes and estuaries), where the magnitude 7.5-weighted peak acceleration that has a 10 percent probability of being exceeded in

50 years is greater than or equal to 0.10 g and the water table is less than 40 feet below the ground surface; or

(b) Areas containing soil deposits of Holocene age (less than 11,000 years), where the magnitude 7.5-weighted peak acceleration that has a 10 percent probability of being exceeded in 50 years is greater than or equal to 0.20 g and the historic high water table is less than or equal to 30 feet below the ground surface; or

(c) Areas containing soil deposits of latest Pleistocene age (between 11,000 years and 15,000 years), where the magnitude 7.5-weighted peak acceleration that has a 10 percent probability of being exceeded in 50 years is greater than or equal to 0.30 g and the historic high water table is less than or equal to 20 feet below the ground surface.

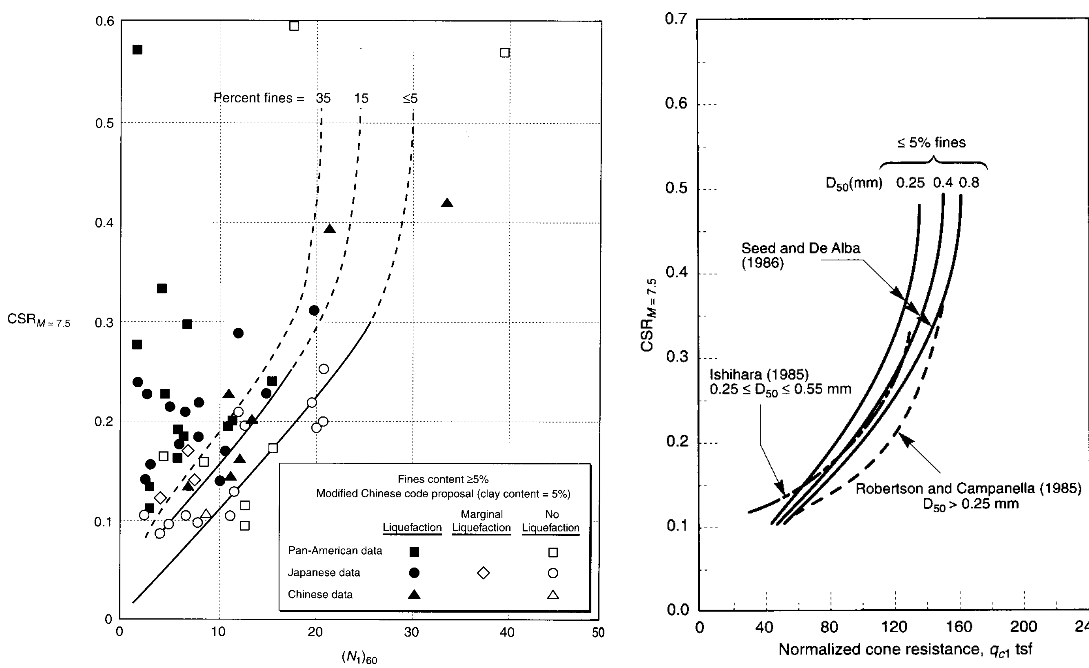
According to CDMG, the Quaternary geology may be taken from existing maps, and hydrologic data should be compiled. Application of this criteria permits development of liquefaction hazard maps which definite regions requiring detailed investigation, allowing concentration of sampling and testing in areas requiring most delineation.

Quantitative Evaluation of Liquefaction Resistance If the results of the preliminary site evaluation indicate that more in-depth studies are warranted, then additional geotechnical characterization of the soils will be necessary. Guidelines for the analysis and mitigation of liquefaction hazards have been presented by the CDMG (Special Publication No. 117). In the context of a factor of safety, the occurrence of liquefaction can be thought of as the capacity of the soil to resist the development of excess pore pressures versus the demand imposed by the seismic ground motions. In practice, the quantitative evaluation of liquefaction resistance is usually based on in-situ Standard Penetration Test (SPT) data (Seed and Idriss, 1982; Seed and De Alba, 1986) and/or Cone Penetration Test (CPT) data (Robertson et al., 1992). The liquefaction resistance of a sand is related to the penetration resistance obtained by either the SPT (N in blows/30 cm (or foot)) or CPT (q_c in kg/cm^2 (or tsf)). The penetration values measured in the field are corrected to account for confining stresses and normalized to obtain a value which corresponds to the N-value that would be measured if the soil was under a vertical effective stress of 1 kg/cm^2 (1 tsf). Additional corrections may be required for SPT N-values depending on the type of drive hammer and release system, length of drill stem, and other factors as outlined by Seed and Harder (1990). The corrected N-value used in the liquefaction analysis is designated $(N_1)_{60}$.

The earthquake-induced cyclic stresses in the soil can be estimated by (a) the simplified evaluation procedure developed by Seed and Idriss (1982), or (b) performing a dynamic soil response analysis. The Seed and Idriss technique yields an estimate of the ratio of the average cyclic shear stress on a horizontal plane in the soil to the initial vertical effective stress on that plane. Correlations of the penetration resistance of soils and cyclic stress ratio (CSR) at sites which did or did not liquefy during recent earthquakes have been established for level ground conditions. In practice, the cyclic stress ratio (CSR) required for the “triggering” of liquefaction can be determined once the penetration resistance of the soil has been obtained by the use of liquefaction boundary curves, Figure 4-3). The cyclic stress ratio developed during an earthquake is given by the equation:

$$(\frac{\tau_{av}}{\sigma_{v'}}) \approx 0.65 \cdot \frac{a_{\max}}{g} \cdot \frac{\sigma_v}{\sigma_{v'}} \cdot r_d \cdot r_{MSF}$$

where τ_{av} is the average cyclic shear stress, σ_v' is the vertical effective stress, σ_v is the total vertical stress, a_{max} is the maximum horizontal ground surface acceleration, g is the acceleration of gravity, r_d is a stress reduction factor which accounts for the fact that the soil column above the soil element behaves as a deformable body, and r_{MSF} is the magnitude scaling factor which is used to convert the CSR required for liquefaction due to a magnitude 7.5 earthquake (the basis for the boundary curves in Figure 4-3 to the magnitude of interest.



a) b)

Figure 4-3:Liquefaction Boundary Curves;
A) Seed Et Al., 1975, B) After Mitchell And Tseng, 1990

B)

Several practical points should be noted regarding these plots:

a) The N-values used in the development of the relationship were obtained using an ASTM standard sampler driven by a 64 kg (140 lb) weight falling 76 cm (30 in). In light of the variety of soil samplers and driving mechanisms (e.g.; safety hammers, donut hammers, slip-jars, etc.) commonly used in practice, the engineer should realize that correlations between N-values obtained by various methods are tenuous at best. Appropriate caution should be exercised when interpreting potential liquefaction behavior based on N-values from non-standard techniques.

b) The boundaries between liquefaction and non-liquefaction are based on case histories for the surface-evidence of liquefaction. It is possible that sites classified as not exhibiting liquefaction experienced the development of significant excess pore pressures that were not manifested at the surface. The boundaries are therefore not intended to delimit the definitive occurrence or nonoccurrence of liquefaction, but rather an approximate indication of whether ground failures may be experienced.

c) The intensity of the ground motions are accounted for in the formulation of the CSR. The number of load cycles is also an important parameter. To represent the effects of the number of cycles, a magnitude scaling factor has been included. Recent studies have served to enhance the magnitude scaling factors originally derived by Seed and Idriss (1982). The results of these studies are presented in Figure 4-4 and indicate that the current scaling factors are overconservative at earthquake magnitudes less than roughly 7.0 (Arango, 1996).

d) This plot is appropriate for horizontal sites only. Approximate corrections can be made to the cyclic stress ratio required to cause liquefaction (CSR_l) for sloping sites (Seed and Harder, 1990).

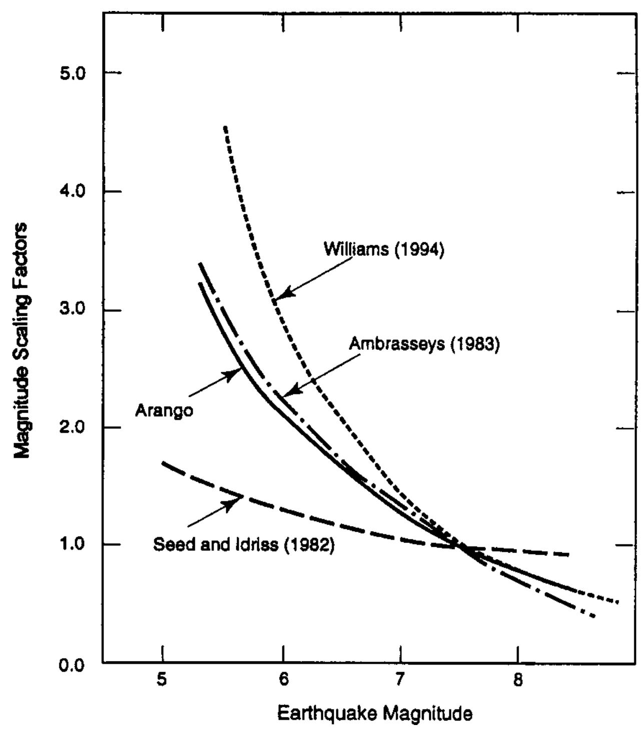


Figure 4-4: Comparison of Earthquake Magnitude Scaling Factors from Various Sources (Arango, 1996)

e) The methods for evaluating the liquefaction resistance of soils have been developed for free-field conditions at horizontal sites where there are no static shear stresses on horizontal planes. This technique is applicable for sites of new construction, yet liquefaction hazard studies may also be warranted for existing facilities. Foundation loads imposed by structures (particularly around the perimeter of the foundation) induce static shear stresses on horizontal planes in the soil. These additional stresses will affect the liquefaction susceptibility of the soil, increasing the hazard in loose soils (relative density ≤ 40%) and potentially decreasing the liquefaction susceptibility in medium dense and dense soils (relative density ≥ 50%). The influence of foundation stresses on the liquefaction behavior of soils has been investigated by Rollins and Seed (1990).

The simplified N-based method of liquefaction hazard evaluation has been shown to provide reasonable estimates for the occurrence of liquefaction during numerous recent earthquakes. This agreement with the field case histories has led to its widespread use in engineering practice.

If the Seed and Idriss method clearly demonstrates the absence of liquefaction hazard, then this investigation may be sufficient. However, if the occurrence of liquefaction is predicted, additional seismic hazards should be addressed. These include:

Hazards associated with lenses of liquefiable soil or by potentially liquefiable layers which underlie resistant, nonliquefiable capping layers. In situations where few, thin lenses of liquefiable soil are identified, the interlayering of liquefiable and resistant soils may serve to minimize structural damage to light, ductile structures. It may be determined that "life safety" and/or "serviceability" requirements may be met despite the existence of potentially liquefiable layers. Ishihara (1985) developed an empirical relation which provides approximate boundaries for liquefaction-induced surface damage for soil profiles consisting of a liquefiable layer overlain by a resistant, or protective, surface layer, Figure 4-5. This relation has been validated by Youd and Garris (1995) for earthquakes with magnitudes between 5.3 and 8. In light of the heterogeneous nature of most soil deposits and the uncertainties inherent in the estimation of ground motion parameters, it is recommended that this method of evaluation be considered for noncritical structures only.

The potential for lateral ground movements and the effects on foundations and buried structures (Bartlett and Youd, 1995).

The effects of changes in lateral earth pressures on retaining structures (Ebeling and Morrison, 1993; Power et al., 1986).

Estimated total and differential settlements at the ground surface due to liquefaction and subsequent densification of the soils (Ishihara and Yoshimine, 1992; Tokimatsu and Seed, 1987).

Several of these liquefaction hazards are addressed in the following section.

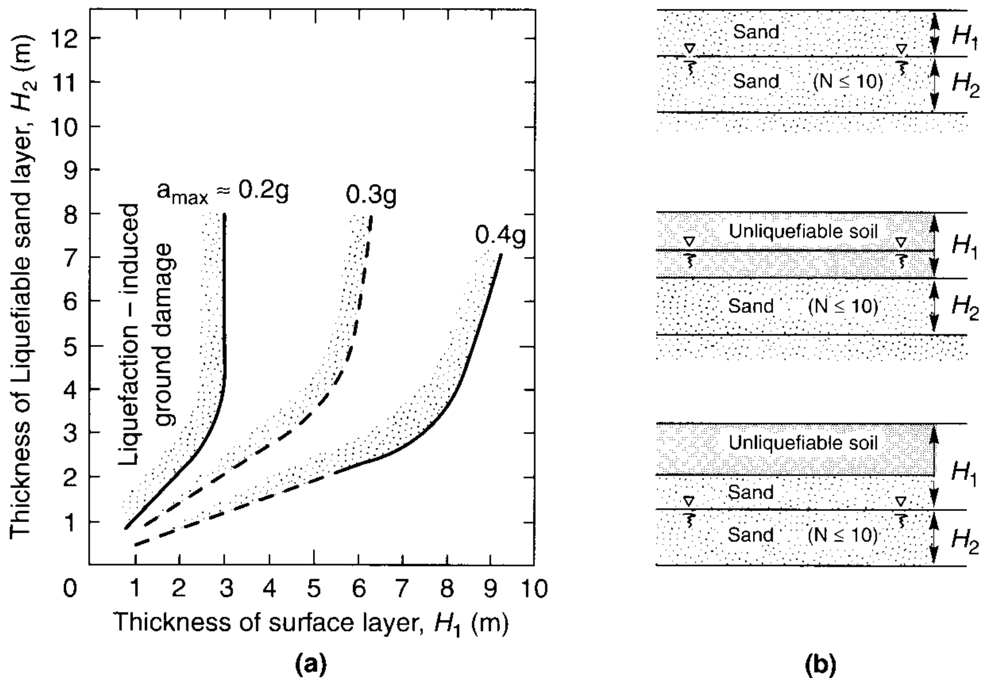


Figure 4-5: (A) Relationship between thickness of liquefiable layer and thickness of overlying layer at sites for which surface manifestation of liquefaction has been observed, and (B) guides to evaluation of respective layer thicknesses (after Ishihara, 1985)

Post-Liquefaction Behavior of Sandy Soils

Post-Liquefaction Volume Change of Sandy Soils The densification of partially-saturated or saturated loose sandy soils due to cyclic loading can result in damaging differential settlement. This phenomena was graphically demonstrated at Port Island (Port of Kobe) after the 1995 Hyogoken Nanbu earthquake where settlements over much of the island averaged 50 cm, with maximum settlements of over 1 m in many places. Several methods have been developed for estimating the magnitude of earthquake-induced settlements in sandy soils (e.g., Ishihara and Yoshimine, 1992; Tokimatsu and Seed, 1987). A simple chart for estimating the volumetric strain in sandy soils during earthquakes is shown in Figure 4-6..

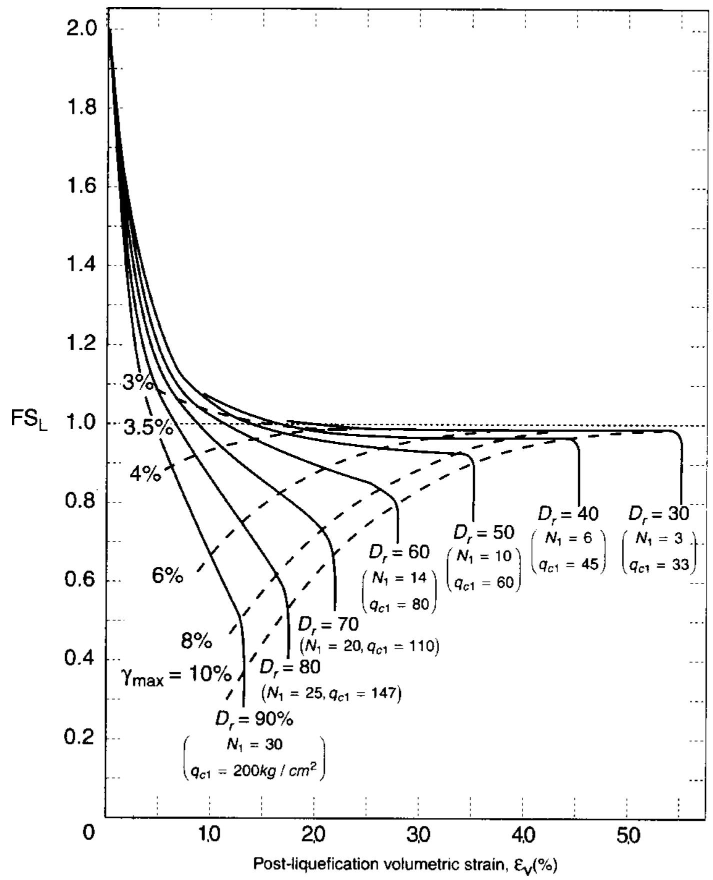


Figure 4-6: Post Volumetric Shear Strain for Clean Sands (Ishihara and Yoshimine, 1992)

Post-Liquefaction Shear Strength In order to evaluate the stability of slopes and embankments, waterfront retaining structures, and other structures underlain by liquefied soils, the strength of the liquefied material must be estimated. Although the condition of initial liquefaction is often defined as the state at which the effective stress (and therefore the shear strength) is equal to zero, the soil will mobilize a residual shear strength if it undergoes large shear strains. In two recent studies, the undrained shear strength of the liquefied sand has been back-calculated from a number of documented slope failures (Seed and Harder, 1990; Stark and Mesri, 1992). These reports describe methods for estimating the residual undrained shear strength of sands based on the SPT penetration resistance of the soil prior to the earthquake, Figure 4-7. The undrained strength values obtained from these relations are used in standard total stress stability analyses equivalent to those commonly performed when evaluating the short term stability of embankments or foundations on saturated clayey soils.

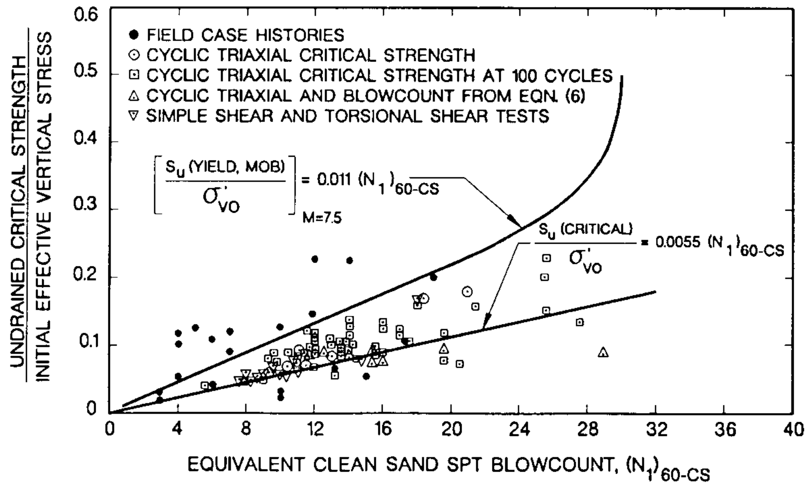


Figure 4-7. Relationship between Undrained Critical Strength Ratio and Equivalent Clean Sand Blow Count (Stark and Mesri, 1992)

Liquefaction-Induced Ground Failures The use of the Seed and Idriss method of evaluating liquefaction hazard constitutes what may be termed a “triggering” analysis. This term is used to indicate that the method identifies the CSR required for the surface manifestation of liquefaction. Soils beneath slopes of as little as 0.2% may experience flow failures subsequent to the onset of initial liquefaction. In order to evaluate the seismic performance of pile supported structures, breakwaters, pipelines and other structures near slopes, it is necessary to estimate the lateral deformation that is likely to occur during the design level earthquakes. Both empirical and numerical techniques have been developed for this analysis. Youd and his co-workers have developed simplified procedures for estimating the magnitude of lateral displacements based on data from numerous case histories (Youd and Perkins, 1987; Bartlett and Youd, 1995). A regression analysis of field data has resulted in the development of an equation for predicting the lateral deformation for free-field sites (i.e., in the absence of piles and structures) on gentle slopes or adjacent to free-faces such as stream banks or dredged channels.

The empirical techniques have been augmented by the results of several numerical studies. The numerical studies are largely based on the “sliding block” technique (described in Section 4.6) wherein coherent blocks of soil are modeled as moving over a liquefied layer. In this case the undrained residual (or “steady state”) strength will control the behavior of the soil mass. The seismic stability of slopes underlain by potentially liquefiable soils can be assessed using estimated values for the undrained shear strength of the sandy soils.

The transition from the static shear strength to the undrained residual strength requires several cycles of loading, and it would be advantageous to account for this progressive strength loss in ground response analyses. Also, the flow behavior of the soil and subsequent reconsolidation should, theoretically, be modeled in analyses involving soil liquefaction. In light of this stress-strain-strength behavior, the phenomena of liquefaction-induced ground failure is complex, and the numerical models used for the evaluation of this hazard have relied on simplifying assumptions. The sliding block method has been used in conjunction with residual undrained strengths for sands to predict the seismic performance of slopes and earth structures (Baziar et al., 1992; Byrne et al., 1994).

Code Provisions and Factors Of Safety Against Liquefaction

In general building codes do not give extensive guidance for liquefaction apart for the need for investigating a site for geologic hazards. The AASHTO Standard Specification For Highway Bridges (1992) suggests the factor of safety of 1.5 is desirable to establish a reasonable measure of safety against liquefaction in cases of important bridge sites. While not specifically stated it is presumed that this is to be used in conjunction with their acceleration maps which give a 10 percent probability of exceedance in 50 years. Similar recommendations have been provided in the CDMG Guidelines for Evaluation and Mitigating Seismic Hazards in California (1997). The geotechnical panel assembled for the preparation of the CDMG report recommended that "If the screening investigation does not conclusively eliminate the possibility of liquefaction hazards at a proposed project site (a factor of safety of 1.5 or greater), then more extensive studies are necessary."

Techniques for Mitigating Liquefaction Hazards

If hazard evaluations indicate that there are potentially liquefiable soils of such extent and location that an unacceptable level of risk is presented, then soil improvement methods should be considered for the mitigation of these hazards. Mitigation must provide suitable levels of protection with regard to the three general types of liquefaction hazard previously noted: (a) potential global translational site instability; (b) more localized problems; and (c) failure of retaining structures.

Liquefaction remediation must address the specifics of the problem on a case by case basis. These specifics include the local site conditions, the type of structure, and the potential for flows and settlements. When liquefaction occurs, there can be a potential for extensive lateral flow slides which can affect a large area, a global site instability. Also there can be local soil settlements and bearing failures which affect a structure on a local level. Specific types of structures can have specific associated problems. Buried structures can become buoyant. Retaining structures where the backfill has liquefied can experience increased lateral loading and deformation.

Potentially suitable methods of mitigation may include the following: removal and replacement, dewatering, in-situ soil improvement, containment or encapsulation structures, modification of site geometry, deep foundations, structural systems and, if possible, alternate site selection. In general, soil improvement methods reduce the liquefaction susceptibility of sandy soils by increasing the relative density, providing conduits for the dissipation of excess pore pressures generated during earthquakes, and/or providing a cohesive strength to the soil. The effectiveness and economy of any method, or combination of methods, will depend on geologic and hydrologic factors (e.g.; soil stratigraphy, degree of saturation, location of ground water table, depth of improvement, volume of soil to be improved, etc.) as well as site factors (e.g.;

accessibility, proximity to existing structures, etc.). An overview of the available liquefaction remediation measures is provided in Table 4-1.

Significant advances have been made in the field of soil improvement over the past several decades. These developments have been made on two fronts: (a) an increased understanding of geotechnical hazards; and (b) the development of innovative construction techniques by specialty contractors. Practical information which addresses recent advances in the mitigation of liquefaction hazards to foundations can be found in numerous recent publications (Borden et al., 1992; Hryciw, 1995; Kramer and Siddharthan, 1995). Several techniques of improvement for liquefiable soils referenced in the geotechnical literature include: (a) densification (e.g.; vibro-methods, dynamic compaction, deep blast

Table 4-1. Liquefaction Remediation Measures (after Ferritto, 1997B)

Method	Principle	Most Suitable Soil Conditions or Types	Maximum Effective Treatment Depth	Relative Costs
1) Vibratory Probe a) Terraprobe b) Vibrorods c) Vibrowing	Densification by vibration; liquefaction-induced settlement and settlement in dry soil under overburden to produce a higher density.	Saturated or dry clean sand; sand.	20 m routinely (ineffective above 3-4 m depth); > 30 m sometimes; vibrowing, 40 m.	Moderate
2) Vibrocompaction a) Vibrofloat b) Vibro-Composer system.	Densification by vibration and compaction of backfill material of sand or gravel.	Cohesionless soils with less than 20% fines.	> 20 m	Low to moderate
3) Compaction Piles	Densification by displacement of pile volume and by vibration during driving, increase in lateral effective earth pressure.	Loose sandy soil; partly saturated clayey soil; loess.	> 20 m	Moderate to high
4) Heavy tamping (dynamic compaction)	Repeated application of high-intensity impacts at surface.	Cohesionless soils best, other types can also be improved.	30 m (possibly deeper)	Low
5) Displacement (compaction grout)	Highly viscous grout acts as radial hydraulic jack when pumped in under high pressure.	All soils.	Unlimited	Low to moderate
6) Surcharge/buttress	The weight of a surcharge/buttress increases the liquefaction resistance by increasing the effective confining pressures in the foundation.	Can be placed on any soil surface.	Dependent on size of surcharge/buttress	Moderate if vertical drains are used
7) Drains a) Gravel b) Sand c) Wick d) Wells (for permanent dewatering)	Relief of excess pore water pressure to prevent liquefaction. (Wick drains have comparable permeability to sand drains). Primarily gravel drains; sand/wick may supplement gravel drain or relieve existing excess pore water pressure. Permanent dewatering with pumps.	Sand, silt, clay.	Gravel and sand > 30 m; depth limited by vibratory equipment; wick, > 45 m	Moderate to high
8) Particulate grouting	Penetration grouting-fill soil pores with soil, cement, and/or clay.	Medium to coarse sand and gravel.	Unlimited	Lowest of grout methods

Method	Principle	Most Suitable Soil Conditions or Types	Maximum Effective Treatment Depth	Relative Costs
9) Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate.	Medium silts and coarser.	Unlimited	High
10) Pressure injected lime	Penetration grouting-fill soil pores with lime	Medium to coarse sand and gravel.	Unlimited	Low
11) Electrokinetic injection	Stabilizing chemical moved into and fills soil pores by electro-osmosis or colloids in to pores by electrphoresis.	Saturated sands, silts, silty clays.	Unknown	Expensive
12) Jet grouting	High-speed jets at depth excavate, inject, and mix a stabilizer with soil to form columns or panels.	Sands, silts, clays.	Unknown	High
13) Mix-in-place piles and walls	Lime, cement or asphalt introduced through rotating auger or special in-place mixer.	Sand, silts, clays, all soft or loose inorganic soils.	> 20 m (60 m obtained in Japan)	High
14) Vibro-replacement stone and sand columns a) Grouted b) Not grouted	Hole jetted into fine-grained soil and backfilled with densely compacted gravel or sand hole formed in cohesionless soils by vibro techniques and compaction of backfilled gravel or sand. For grouted columns, voids filled with a grout.	Sands, silts, clays.	> 30 m (limited by vibratory equipment)	Moderate
15) Root piles, soil nailing	Small-diameter inclusions used to carry tension, shear, compression.	All soils.	Unknown	Moderate to high
16) Blasting	Shock waves an vibrations cause limited liquefaction, displacement, remolding, and settlement to higher density.	Saturated, clean sand; partly saturated sands and silts after flooding.	> 40 m	Low

densification, compaction grouting, and compaction piles); (b) drainage to allow rapid dissipation of excess pore pressures (e.g.; vibro-replacement and stone columns); and (c) chemical modification/cementation (e.g.; permeation grouting, jet grouting, and deep mixing). Additional possible methods of increasing the liquefaction resistance of soils include permanent dewatering, and removal of loose soils and replacement at a suitable compactive effort.

The most common methods for remediation of liquefaction hazards at open, undeveloped sites include (in order of increasing cost): (a) deep dynamic compaction; (b) vibro-compaction; (c) vibro-replacement, excavation and replacement; and (c) grouting methods. Each of these techniques results in a significant displacement of soil. On projects involving foundation remediation adjacent to existing structures or buried utilities, ground movements must be minimized to avoid architectural and structural damage. Several projects with these constraints have utilized grouting techniques to stabilize potentially liquefiable soils.

Several methods of densification have been used including vibroprobe, vibro-compaction, dynamic compaction, compaction grouting, and compaction piles. Substitution or replacement of soil to improve drainage has been used including vibro-replacement and stone columns. Techniques like stone columns achieve their effectiveness by replacing liquefiable cohesionless soils with stiffer columns of gravel and rock which improves strength and promotes drainage. Cement grouting, jet grouting and deep mixing have been used as chemical means of eliminating/reducing liquefaction potential. Surcharging a site increases liquefaction resistance by increasing the effective confining pressures. Table 4 presents a summary of methods used for remediation and their relative cost as reported by Professor Whitman (NRC 1985). Navy facilities on Treasure Island during the 1989 Loma Prieta earthquake can attest to the effectiveness of remediation. Areas where remediation was done performed well while other areas suffered settlements of 6 to 8 inches and lateral spreads. Observation of damage during the 1995 Hyogoken Nanbu (Kobe) Earthquake again confirmed the performance of improved sites. Preloading, sand drains, sand compaction piles, and vibro-compaction were shown to be effective.

Method	Vertical Settlement	
	Range (cm)	Average (cm)
Untreated	25 to 95	42
Preloading	15 to 60	30
Sand drains	0 to 40	15
Sand drains & preloading	0 to 25	12
Vibro-compaction	0 to 5	near 0
Sand compaction piles	0 to 5	near 0

Generally costs increase from dynamic compaction to vibro-compaction to replacement The measure of effectiveness of a remediation undertaking is the increase in minimum soil density and specifications usually measure this by the improvement in penetration resistance or laboratory testing. Engineering practice tends to be conservative and factors of safety from 1.5 to 2.0 against liquefaction are often specified. These values may be harder to achieve at the waterfront in regions of high seismicity.

The contract specifications for ground improvement must include a program for the verification of soil improvement. Ground improvement specifications typically require that a minimum soil density (as related to a penetration resistance) be achieved. A level of risk can be assessed by the factor of safety against liquefaction -- which is defined as the ratio of the CSR required for liquefaction of the improved soil to the CSR induced by the design earthquake. Factors of safety between 1.5 and 2.0 are commonly specified for non-critical structures. It should be noted that the penetration resistance of recently deposited, or modified, soils increases with time after treatment. This phenomena, termed “aging” must be accounted for when specifying the time between soil improvement and the in-situ verification testing. There is also evidence that the SPT and CPT tests are not sufficiently sensitive to detect the minor changes in soil fabric that can significantly increase the liquefaction resistance of the soil. This has led to the incorporation of lab testing programs in addition to field tests for verification of soil improvement for liquefaction hazards.

There are currently very few references on the volume of soil that should be improved to mitigate liquefaction hazards to structures (JPHRI, 1997; Dickenson and McCullough, 1998). At horizontal sites, general recommendations for buildings and pile-supported structures may call for soil improvement to the base of the liquefiable deposit (or to a maximum depth of 15 to 18 meters) and over a lateral extent equal to the thickness of the layer, plus an additional increment based on judgment. The uncertainty associated with general recommendations like these is compounded for sloping sites and areas adjacent to waterfront retaining structures. In these situations, the volume of soil that may be involved in a lateral spread is difficult to ascertain, therefore existing recommendations tend to be very conservative. Based on shake table tests, Iai (1992) has proposed tentative guidelines for the extent of soil improvement required to mitigate liquefaction hazards behind caissons. These recommendations have been incorporated into the guidelines for soil improvement adjacent to waterfront retaining structures developed by the Japan Port and Harbor Research Institute (JPHRI, 1997). Several examples for soil improvement adjacent to waterfront retaining structures are contained in Figure 4-8.

Very few case studies exist for the seismic performance of improved soil sites. The effectiveness of the soil improvement at limiting deformations adjacent to foundations and retaining structures will be a function of the strength and duration of shaking. In only a limited number of cases have the improved sites been subjected to design-level ground motions. The cases that have been documented demonstrate a substantial reduction in liquefaction-induced ground failures and ground deformations due to the ground treatment (e.g., Iai et al., 1994; Ohsaki, 1970; Mitchell et al., 1995; Yasuda et al., 1996). However, as noted in several of these papers, the ground deformations were not reduced to imperceptible levels. This observation is especially germane when establishing allowable deformation limits for waterfront retaining structures, where adjacent gantry cranes and other sensitive components may be damaged by lateral movements of the walls. In several cases (e.g., flexible retaining structures such as anchored sheet pile bulkheads and cellular sheet pile bulkheads), these structures may deform when subjected to strong ground motions despite the utilization of ground treatment. In cases such as this, the ground treatment would serve to preclude catastrophic failure of the retaining structures.

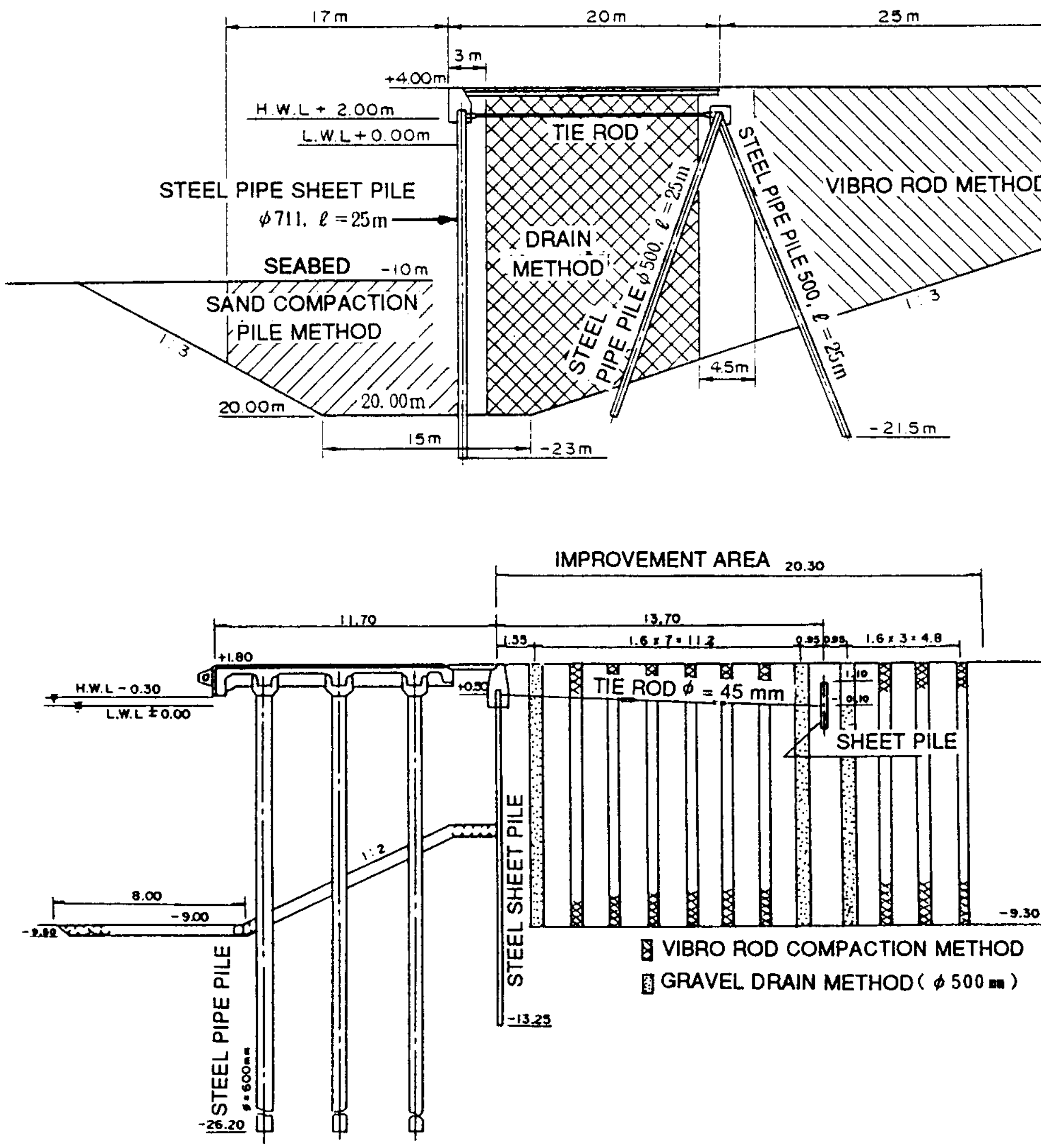


Figure 4-8. Examples of Soil Improvement for Waterfront Retaining Structures (PHRI, 1997)

One particularly relevant case study involving the waterfront retaining structures at the Kushiro Port in Japan has been documented by Iai and his co-workers at the Port and Harbour Research Institute (Iai et al., 1994). Kushiro Port is located on the east coast of the island of Hokkaido, a region that is prone to large subduction zone earthquakes. Prior to 1993, the port had been subjected to at least two damaging earthquakes (the M_{JMA} 8.1 1952 Tokachi-Oki Earthquake and the M_{JMA} 7.4 1973 Nemuro-Hanto-Oki Earthquake) and these experiences appear to have influenced the seismic design criteria subsequently adopted at the port. In the older portions of the port, the sandy backfill soil adjacent to retaining structures had been left in its original loose state while, the in the newer sections of the port (constructed as late as 1992), a soil improvement program was implemented to reduce the liquefaction susceptibility of the fills near retaining walls. The remediation project called for the use of various types and degrees of soil improvement in the waterfront. In 1993, the port was subjected to strong ground motions generated during the M_{JMA} 7.8 Kushiro-Oki Earthquake. It is significant to note that while the waterfront areas at which soil improvement was implemented performed successfully, thereby allowing the port to continue operation immediately after the earthquake, the waterfront retaining structures founded in loose soils experienced dramatic liquefaction-induced failures.

Slope Instability Hazards

Introduction

Sloping ground conditions exist throughout ports as natural and engineered embankments (e.g., river levees, sand or rock dikes, etc., and dredged channel slopes). On-shore and submarine slopes at ports have been found to be vulnerable to earthquake induced deformations. High water levels and weak foundation soils common at most ports can result in slopes which have marginal static stability and which are very susceptible to earthquake induced failures. In addition to waterfront slopes, several recent cases involving failures of steep, natural slopes along marine terraces located in backland areas have resulted in damage to coastal ports. Large scale deformations of these slopes can impede shipping and damage adjacent foundations and buried structures thereby limiting port operations following earthquakes.

The most commonly used methods for analysis of slope stability under both static and dynamic conditions are based on standard rigid body mechanics and limit equilibrium concepts that are familiar to most engineers. The development and application of these techniques are introduced in most geotechnical engineering textbooks and in numerous design manuals. Therefore, they will not be described here. Instead, this section will introduce the strengths and limitations of these analysis techniques as applied at port facilities.

Pseudostatic Methods of Analysis

Standard, limit equilibrium methods for analyzing the static stability of slopes are routinely used in engineering practice. The use of these design tools have several advantages in practice: (a) the techniques are familiar to most engineers; (b) requisite input includes standard geotechnical parameters that are obtained during routine foundation investigations; and (c) the methods have been coded in very straightforward, efficient computer programs that allow for sensitivity studies to be made of various design options.

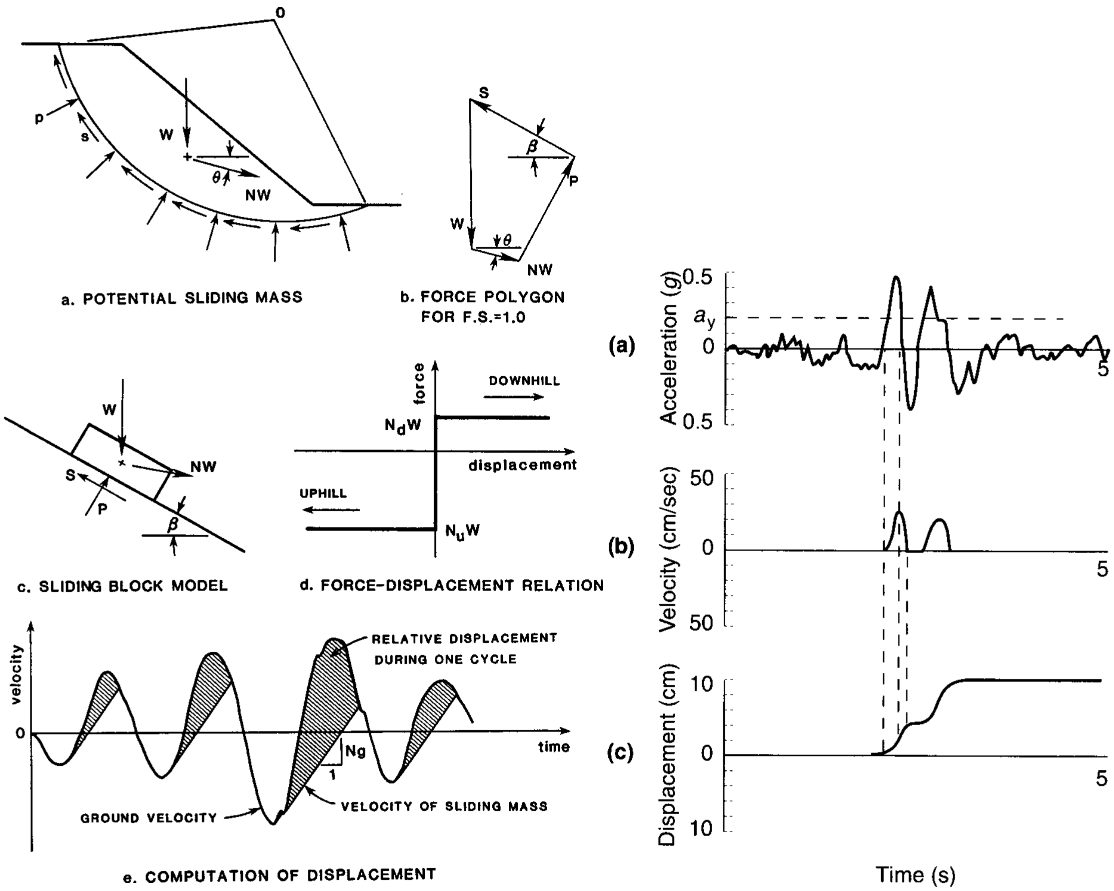
For use in determining the seismic stability of slopes, limit equilibrium analyses are modified slightly with the addition of a permanent lateral body force which is the product of a *seismic coefficient* and the mass of the soil bounded by the potential slip. The seismic coefficient (usually designated as k_h, N_h) is specified as a fraction of the peak horizontal acceleration, due to the fact that the lateral inertial force is applied for only a short time interval during transient earthquake loading. Seismic coefficients are commonly specified as roughly $^1/_3$ to $^1/_2$ of the peak horizontal acceleration value (Seed, 1979; Marcuson, et al., 1992).

In most cases involving soils which do not exhibit considerable strength loss after the peak strength has been mobilized, common pseudostatic rigid body methods of evaluation will generally suffice for evaluating the stability of slopes. These methods of evaluation are well established in the technical literature (Kramer, 1996). Although these methods are useful for indicating an approximate level of seismic stability in terms of a factor of safety against failure, they suffer from several potentially important limitations. The primary disadvantages of pseudostatic methods include: (a) they do not indicate the range of slope deformations that may be associated with various factors of safety; (b) the influence of excess pore pressure generation on the strength of the soils is incorporated in only a very simplified, "decoupled" manner; (c) progressive deformations that may result due to cyclic loading at stresses less than those required to reduce specific factors of safety to unity are not modeled; (d) strain softening behavior for liquefiable soils or sensitive clays is not directly accounted for; and (e) important aspects of soil-structure interaction are not evaluated.

Limited Deformation Analysis

In most applications involving waterfront slopes and embankments, it is necessary to estimate the permanent slope deformations that may occur in response to the cyclic loading. Allowable deformation limits for slopes will reflect the sensitivity of adjacent structures, foundations and other facilities to these soil movements. Enhancements to traditional pseudostatic limit equilibrium methods of embankment analysis have been developed to estimate embankment deformations for soils which do not lose appreciable strength during earthquake shaking (Ambraseys and Menu, 1988; Makdisi and Seed, 1978; Jibson, 1993).

Rigid body, "sliding block" analyses, which assume the that soil behaves as a rigid, perfectly plastic material, can be used to estimate limited earthquake-induced deformations. The technique, developed by Newmark (1965) and schematically illustrated in Figure 4-9, is based on simple limit equilibrium stability analysis for determining the critical, or yield, acceleration which is required to bring the factor of safety against sliding for a specified block of soil to unity. The second step involves the introduction of an acceleration time history. When the ground



a) **b)**

Figure 4-9. Elements of Sliding Block Analysis, A) Hynes-Griffin and Franklin, 1984, B) after Wilson and Keefer, 1985

motion acceleration exceeds the critical acceleration (a_{crit}, a_y) the block begins to move down slope. By double integrating the area of the acceleration time history that exceeds a_{crit}, the relative displacement of the block is determined. A simple spreadsheet routine can be used to perform this calculation (Jibson, 1993). Numerical studies based on this method of analysis have lead to the development of useful relationships between ground motion intensity and the seismically-induced deformations (e.g. Ambraseys and Menu, 1988; Makdisi and Seed, 1978; Jibson, 1993). The relationship proposed by Makdisi and Seed is shown in Figure 4-10.

The amount of permanent displacement depends on the maximum magnitude and duration of the earthquake. The ratio of maximum acceleration to yield acceleration of 2.0 will result in block displacements of the order of a few inches for a magnitude 6 1/2 earthquake and several feet for a magnitude 8 earthquake. It should be noted that significant pore pressure increases may be induced by earthquake loading in saturated silts and sands. For these soils a potential exists for a significant strength loss. For dense saturated sand, significant undrained shear strength can still be mobilized even when residual pore pressure is high. For loose sands, the residual undrained strength which can be mobilized after high pore pressure build-up is very low and is often less than the static undrained shear strength. This may result in flow slides or large ground deformations.

Given that the sliding block analyses are based on limit equilibrium techniques, they suffer from many of the same deficiencies previously noted for pseudostatic analyses. One of the primary limitations with respect to their application for submarine slopes in weak

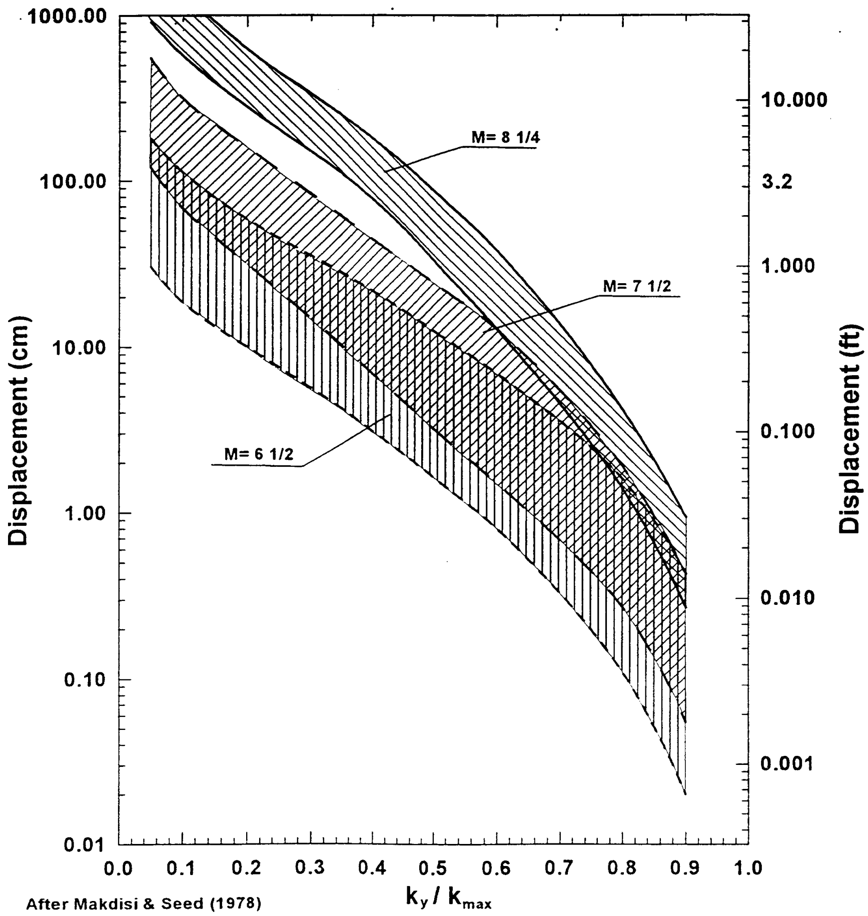


Figure 4-10. Empirical Relationships between Permanent Displacement of sliding Block and ratios of accelerations (after Makdisi and Seed, 1978)

soils is that strain softening behavior is not directly accounted incorporated in the analysis. The sliding block methods have, however, been applied for liquefiable soils by using the post-liquefaction undrained strengths for sandy soils. A recent method proposed by Byrne et al. (1994) has been developed for contractive, collapsible soils which are prone to liquefaction.

Advanced Numerical Modeling of Slopes

In situations where the movement of a slop impacts adjacent structures, such as pile supported structures embedded in dikes, buried lifelines and other soil-structure interaction problems, it is becoming more common to rely on numerical modeling methods to estimate the range of slope deformations which may be induced by design level ground motions (Finn, 1990). The numerical models used for soil-structure interaction problems can be broadly classified based on the techniques that are used to account for the deformations of both the soil and the affected structural element. In many cases the movement of the soil is first computed, then the response of the structure to these deformations is determined. This type of analysis is termed *uncoupled*, in that the computed soil deformations are not affected by the existence of embedded structural components. A common enhancement to this type of uncoupled analysis includes the introduction of an iterative solution scheme which modifies the soil deformations based on the response of the structure so that compatible strains are computed. An example of this type of analysis is drag loading on piles due to lateral spreading. In an uncoupled analysis the ground deformations would be estimated using either an empirical relationship (e.g. Bartlett and Youd 1995) or a sliding block type evaluation (e.g. Byrne et al., 1994) as discussed in Section 0 and 0. Once the pattern of ground deformations has been established a model such as LPILE (Reese and Wang, 1994) can be used to determine the loads in the deformed piles. In addition, modifications can be made to the p-y curves to account for the reduced stiffness of the liquefied soil (O'Rourke and Meyerson, 1994). The lateral spread displacement is forced onto the p-y spring (i.e., drag loading).

In a *coupled* type of numerical analysis the deformations of the soil and structural elements are solved concurrently. Two-dimensional numerical models such as FLUSH (Lysmer et al., 1975), FLAC (Itasca, 1995), DYSAC2 (Muraleetharan et al., 1988), and LINOS (Bardet, 1990) have been used to model the seismic performance of waterfront components at ports (e.g., Finn, 1990; Roth et al., 1992; Werner, 1986; Wittkop, 1994). The primary differences in these numerical analyses include; (a) the numerical formulation employed (e.g., FEM, FDM, BEM), (b) the constitutive model used for the soils, and (c) the ability to model large, permanent deformations. Each of the methods listed have been useful in evaluating various aspects of dynamic soil-structure interaction.

Advanced numerical modeling techniques are recommended for soil-structure interaction applications, such as estimating permanent displacements of slopes and embankments with pile supported wharves. The primary advantages of these models include: (a) complex embankment geometries can be evaluated, (b) sensitivity studies can be made to determine the influence of various parameters on the seismic stability of the structure, (c) dynamic soil behavior is much more realistically reproduced, (d) coupled analyses which allow for such factors as excess pore pressure generation in contractive soils during ground shaking and the associated reduction of soil stiffness and strength can be used, (e) soil-structure interaction and permanent deformations can be evaluated. Disadvantages of the numerical analyses methods include: (a) the engineering time required to construct the numerical model can be extensive, (b) numerous soil parameters are often required, thereby increasing laboratory testing costs (the number of soil properties required is a function of the constitutive soil model employed in the model), (c) very few of the available models have been validated with well documented case studies of the seismic performance of actual retaining structures, therefore the level of uncertainty in the analysis is often unknown.

Mitigation of Seismic Hazards Associated with Slope Stability

Remedial strategies for improving the stability of slopes have been well developed for both onshore and submarine slopes. Common techniques for stabilizing slopes include: (a) modifying the geometry of the slope; (b) utilization of berms; (c) soil replacement (key trenches with engineered fill); (d) soil improvement; and (e) structural techniques such as the installation of piles adjacent to the toe of the slope. Constraints imposed by existing structures and facilities, and shipping access will often dictate which of the methods, or combinations of methods, are used.

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