Chapter 7 Commentary

FOUNDATION DESIGN REQUIREMENTS

7.1 GENERAL: The minimum foundation design requirements that might be suitable when any consideration must be given to earthquake resistance are set forth in Chapter 7. It is difficult to separate foundation requirements for minimal earthquake resistance from the requirements for resisting normal vertical loads. In order to have a minimum base from which to start, this chapter assumes compliance with all basic requirements necessary to provide support for vertical loads and lateral loads other than earthquake. These basic requirements include, but are not limited to, provisions for the extent of investigation needed to establish criteria for fills, slope stability, expansive soils, allowable soil pressures, footings for specialized construction, drainage, settlement control, and pile requirements and capacities. Certain detail requirements and the allowable stresses to be used are provided in other chapters of the *Provisions* as are the additional requirements to be used in more seismically active locations.

7.2 STRENGTH OF COMPONENTS AND FOUNDATIONS: The resisting capacities of the foundations must meet the provisions of Chapter 7.

7.2.1 Structural Materials: The strength of foundation *components* subjected to seismic forces alone or in combination with other prescribed loads and their detailing requirements must be as determined in Chapters 8, 9, 10, 11, or 12.

7.2.2 Soil Capacities: This section requires that the building foundation without seismic forces applied must be adequate to support the building gravity load. When seismic effects are considered, the soil capacities can be increased considering the short time of loading and the dynamic properties of the soil.

7.3 SEISMIC DESIGN CATEGORIES A AND B: There are no special seismic provisions for the design of foundations for buildings assigned to Categories A and B.

7.4 SEISMIC DESIGN CATEGORY C: Extra precautions are required for the seismic design of foundations for buildings assigned to Category C.

7.4.1 Investigation: Potential site hazards such as fault rupture, liquefaction, ground deformation, and slope instability should be investigated when the size and importance of the project so warrants. In this section, procedures for evaluating these hazards are reviewed.

Surface Fault Rupture: Fault ruptures during past earthquakes have led to large surface displacements that are potentially destructive to engineered construction. Displacements, which range from a fraction of an inch to tens of feet, generally occur along traces of previously active faults. The sense of displacement ranges from horizontal strike-slip to vertical dip-slip to many combinations of these *components*. The following commentary summarizes procedures to follow or consider when assessing the hazard of surface fault rupture. This commentary is based in large part on Appendix C of California Division of Mines and Geology (CDMG) Special Publication 42, 1988 Revision (Hart, 1988).

Assessment of Surface Faulting Hazard: The evaluation of fault hazard at a given site is based extensively on the concepts of recency and recurrence of faulting along existing faults. The magnitude, sense, and frequency of fault rupture vary for different faults or even along different segments of the same fault. Even so, future faulting generally is expected to recur along pre-existing faults. The development of a new fault or reactivation of a long inactive fault is relatively uncommon and generally need not be a concern. For most engineering applications, a sufficient definition of an active fault is given in CDMG Special Publication 42 (Hart, 1988): "An active fault has had displacement in Holocene time (last 11,000 years)."

As a practical matter, fault investigations should be conducted by qualified geologists and directed at the problem of locating faults and evaluating recency of activity, fault length, and the amount and character of past displacements. Identification and characterization studies should incorporate evaluation of regional fault patterns as well as detailed study of fault features at and in the near vicinity (within a few hundred yards to a mile) of the site. Detailed studies should include trenching to accurately locate, document, and date fault features.

Suggested Approach for Assessing Surface Faulting Hazard: The following approach should be used, or at least considered, in fault hazard assessment. Some of the investigative methods outlined below should be carried out beyond the site being investigated. However, it is not expected that all of the following methods would be used in a single investigation:

1. A review should be made of the published and unpublished geologic literature from the region along with records concerning geologic units, faults, ground-water barriers, etc.
2. A stereoscopic study of aerial photographs and other remotely sensed images should be made to detect fault-related topography, vegetation and soil contrasts, and other lineaments of possible fault origin. Predevelopment air photos are essential to the detection of fault features.
3. A field reconnaissance study generally is required which includes observation and mapping of geologic and soil units and structures, geomorphic features, springs, and deformation of man-made structures due to fault creep. This study should be detailed within the site with less detailed reconnaissance of an area within a mile or so of the site.
4. Subsurface investigations usually are needed to evaluate fault features. These investigations include trenches, pits, or bore holes to permit detailed and direct observation of geologic units and fault features.
5. The geometry of fault structures may be further defined by geophysical investigations including seismic refraction, seismic reflection, gravity, magnetic intensity, resistivity, ground penetrating radar, etc. These indirect methods require a knowledge of specific geologic conditions for reliable interpretation. Geophysical methods alone never prove the absence of a fault and they do not identify the recency of activity.
6. More sophisticated and more costly studies may provide valuable data where geological special conditions exist or where requirements for critical structures demand a more intensive investigation. These methods might involve repeated geodetic surveys, strain measurements, or monitoring of microseismicity and radiometric analysis (^{14}C, K-Ar), stratigraphic correlation (fossils, mineralology) soil profile development, paleomagnetism

(magnetostratigraphy), or other age-dating techniques to date the age of faulted or unfaulted units or surfaces.

The following information should be developed to provide documented support for conclusions relative to location and magnitude of faulting hazards:

1. Maps should be prepared showing the existence (or absence) and location of hazardous faults on or near the site.

2. The type, amount, and sense of displacement of past surface faulting episodes should be documented including sense and magnitude of displacement, if possible.

3. From this documentation, estimates can be made, preferably from measurements of past surface faulting events at the site, using the premise that the general pattern of past activity will repeat in the future. Estimates also may be made from empirical correlations between fault displacement and fault length or earthquake magnitude published by Bonilla et al. (1984) or by Slemmons et al. (1989). Where fault segment length and sense of displacement are defined, these correlations may provide an estimate of future fault displacement (either the maximum or the average to be expected).

There are no codified procedures for estimating the amount or probability of future fault displacements. Estimates may be made, however, by qualified earth scientists. Because techniques for making these estimates are not standardized, peer review of reports is useful to verify the adequacy of the methods used and the estimates reports, to aid the evaluation by the permitting agency, and to facilitate discussion between specialists that could lead to the development of standards.

The following guidelines are given for safe siting of engineered construction in areas crossed by active faults:

1. Where ordinances have been developed that specify safe setback distances from traces of active faults or active fault zones, those distances must be complied with and accepted as the minimum for safe siting of buildings. For example, the general setback requirement in California is a minimum of 50 feet from a well-defined zone containing the traces of an active fault. That setback distance is mandated as a minimum for structures near faults unless a site-specific special geologic investigation shows that a lesser distance could be safety applied (*California Administrative Code,* Title 14, Sec. 3603A).

2. In general, safe setback distances may be determined from geologic studies and analyses as noted above. Setback requirements for a site should be developed by the site engineers and geologists in consultation with professionals from the building and planning departments of the jurisdiction involved. Where sufficient geologic data have been developed to accurately locate the zone containing active fault traces and the zone is not complex, a 50-foot setback distance may be specified. For complex fault zones, greater setback distances may be required. Dip-slip faults, with either normal or reverse motion, typically produce multiple fractures within rather wide and irregular fault zones. These zones generally are confined to the hanging-wall side of the fault leaving the footwall side little disturbed. Setback requirements for such faults may be rather narrow on the footwall side, depending on the quality of the data available, and larger on the hanging wall side of the zone. Some fault zones may

contain broad deformational features such as pressure ridges and sags rather than clearly defined fault scarps or shear zones. Nonessential structures may be sited in these zones provided structural mitigative measures are applied as noted below. Studies by qualified geologists and engineers are required for such zones to assure that building foundations can withstand probable ground deformations in such zones.

Mitigation of Surface Faulting Hazards: There is no mitigative technology that can be used to prevent fault rupture from occurring. Thus, sites with unacceptable faulting hazard must either be avoided or structures designed to withstand ground deformation or surface fault rupture. In general practice, it is economically impractical to design a structure to withstand more than a few inches of fault displacement. Some buildings with strong foundations, however, have successfully withstood or diverted a few inches of surface fault rupture without damage to the structure (Youd, 1989). Well reinforced mat foundations and strongly inter-tied footings have been most effective. In general, less damage has been inflicted by compressional or shear displacement than by vertical or extensional displacements.

Liquefaction: Liquefaction of saturated granular soils has been a major source of building damage during past earthquakes. For example, many structures in Niigata, Japan, suffered major damage as a consequence of liquefaction during the 1964 earthquake. Loss of bearing strength, differential settlement, and differential horizontal displacement due to lateral spread were the direct causes of damage. Many structures have been similarly damaged by differential ground displacements during U.S. earthquakes such as the San Fernando Valley Juvenile Hall during the 1971 San Fernando, California, earthquake and the Marine Sciences Laboratory at Moss Landing, California, during the 1989 Loma Prieta event. Design to prevent damage due to liquefaction consists of three parts: evaluation of liquefaction hazard, evaluation of potential ground displacement, and mitigating the hazard by designing to resist ground displacement, by reducing the potential for liquefaction, or by choosing an alternative site with less hazard.

Evaluation of Liquefaction Hazard: Liquefaction hazard at a site is commonly expressed in terms of a factor of safety. This factor is defined as the ratio between the available liquefaction resistance, expressed in terms of the cyclic stresses required to cause liquefaction, and the cyclic stresses generated by the design earthquake. Both of these stress parameters are commonly normalized with respect to the effective overburden stress at the depth in question.

The following possible methods for calculating the factor of safety against liquefaction have been proposed and used to various extents:

1. Analytical Methods -- These methods typically rely on laboratory test results to determine either liquefaction resistance or soil properties that can be used to predict the development of liquefaction. Various equivalent linear and nonlinear computer methods are used with the laboratory data to evaluate the potential for liquefaction. Because of the considerable difficulty in obtaining undisturbed samples of liquefiable sediment for laboratory evaluation of constitutive soil properties, the use of analytical methods, which rely on accurate constitutive properties, usually are limited to critical projects or to research.

2. Physical Modeling -- These methods typically involve the use of centrifuges or shaking tables to simulate seismic loading under well defined boundary conditions. Soil used in the model is reconstituted to represent different density and geometrical conditions. Because of

difficulties in precisely modeling in-situ conditions at liquefiable sites, physical models have seldom been used in design studies for specific sites. However, physical models are valuable for analyzing and understanding generalized soil behavior and for evaluating the validity of constitutive models under well defined boundary conditions.

3. Empirical Procedures -- Because of the difficulties in analytically or physically modeling soil conditions at liquefiable sites, empirical methods have become a standard procedure for determining liquefaction susceptibility in engineering practice. Procedures for carrying out a liquefaction assessment using the empirical method are given by the National Research Council (1985).

For most empirical methods, the average earthquake-induced cyclic shear stress is estimated from a simple equation or from dynamic response analyses using computer programs such as SHAKE and DESRA. The induced cyclic shear stress is estimated from the peak horizontal acceleration expected at the site using the following simple equation:

$$\frac{\tau}{\sigma_o'} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_o}{\sigma_o'}\right)r_d \qquad \text{(C7.4.1-1)}$$

where (a_{max}/g) = peak horizontal acceleration at ground surface expressed as a decimal fraction of gravity, σ_o = the vertical total stress in the soil at the depth in question, σ_o' = the vertical effective stress at the same depth, and r_d = deformation-related stress reduction factor.

The chart reproduced in Figure C7.4.1-1 is used to estimate r_d.

To determine liquefaction resistance of sandy soils, the induced cyclic stress ratio computed from Eq. C7.4.1-1 is compared to the cyclic stress ratio required to generate liquefaction in the soil in question for a given earthquake of magnitude M. The most common technique for estimating liquefaction resistance is from an empirical relationship between cyclic stress ratio required to cause liquefaction and normalized blow count, $(N_1)_{60}$.

The most commonly used empirical relationship, compiled by Seed et al. (1985), compares $(N_1)_{60}$ from sites where liquefaction did or did not develop during past earthquakes. Figure C7.4.1-2 shows the most recent (1988) version of this relationship for M = 7-1/2 earthquakes. On that figure, cyclic stress ratios calculated for various sites are plotted against $(N_1)_{60}$.

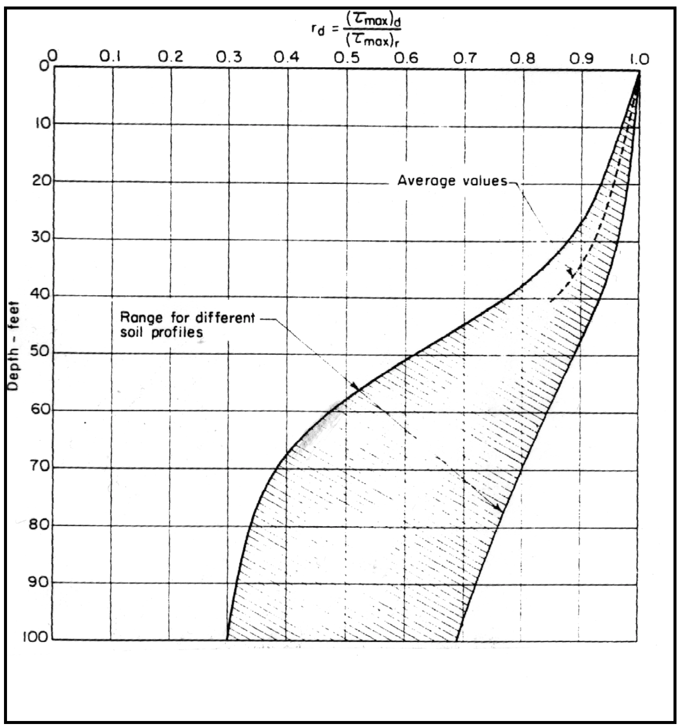


FIGURE C7.4.1-1 Range of values for r_d for different soil properties (after Seed and Idriss, 1971).

Solid dots represent sites where liquefaction occurred and open dots represent sites where surface evidence of liquefaction was not found. Curves were drawn through the data to separate regions where liquefaction did and did not develop. Curves are given for sediments with various fines contents.

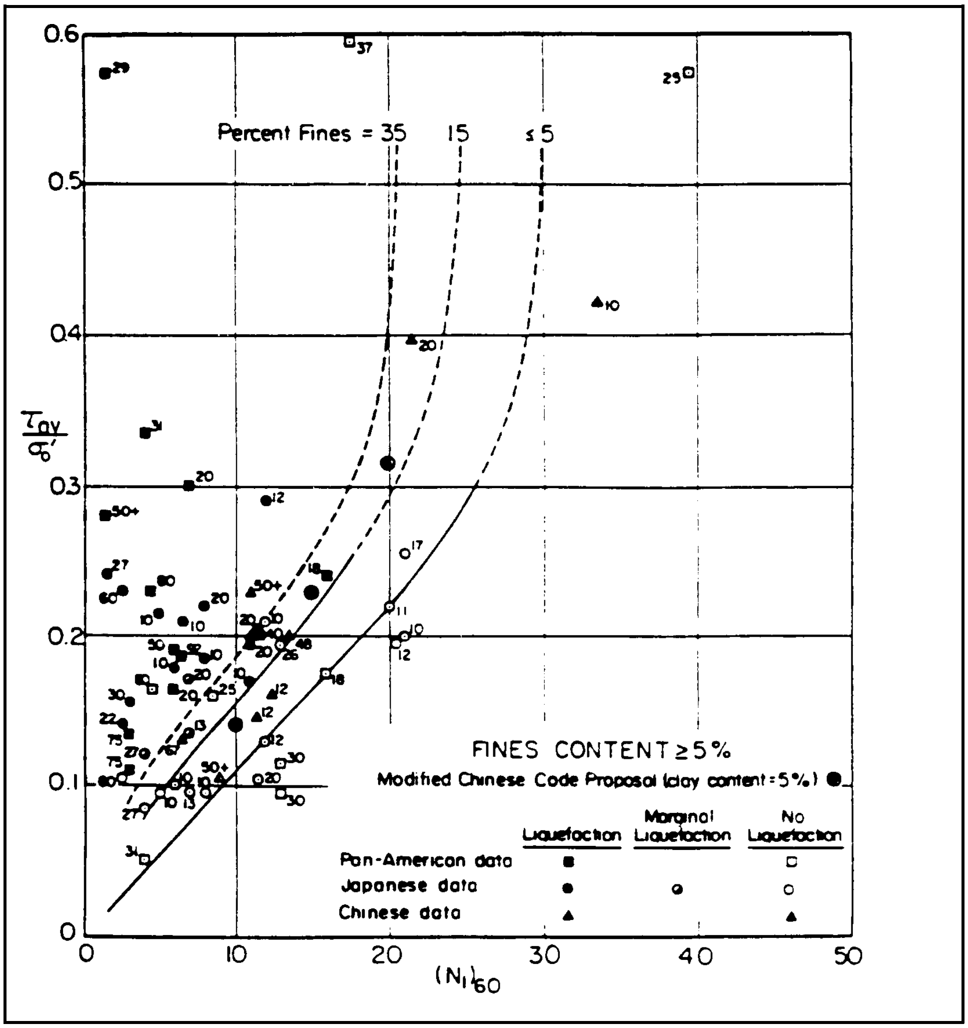


FIGURE C7.4.1-2 Relationship between stress ratios causing liquefaction and N_1 values for silty sands for M = 7-1/2 earthquakes.

Although the curves drawn by Seed et al. (1985) envelop the plotted data, it is possible that liquefaction may have occurred beyond the enveloped data and was not detected at ground surface. Consequently, a factor of safety of 1.2 to 1.5 is appropriate in engineering design. The factor to be used is based on engineering judgment with appropriate consideration given to type and importance of structure and potential for ground deformation.

The maximum acceleration, a_{max}, commonly used in liquefaction analysis is that which would occur at the site in the absence of liquefaction. Thus, the a_{max} used in Eq. C7.4.1-1 is the estimated rock acceleration corrected for soil site response but with neglect of excess pore-water pressures that might develop. Alternatives for obtaining a_{max} are: (1) from standard peak acceleration attenuation curves valid for comparable soil conditions; (2) from standard peak acceleration attenuation curves for rock, corrected for site amplification or deamplification by means of standard amplification curves or computerized site response analysis such as described in the "Chapter 1 Commentary"; (3) obtaining first the value of effective peak acceleration, A_a, for rock depending on the map area where the site is located and then multiplying this value by a factor between 1 and 3 as discussed in the "Chapter 1 Commentary" to determine a_{max}; (4) from probabilistic maps of a_{max} with or without correction for site amplification or deamplification depending on the rock or soil conditions used to generate the map.

The magnitude, M, needed to determine a magnitude scaling factor from Figure C7.4.1-3 should correspond to the size of the design or expected earthquake selected for the liquefaction evaluation. If Alternative 3 or 4 is selected, the definition of M is not obvious and additional studies and considerations are necessary. In all cases, it should be remembered that the likelihood of liquefaction at the site (as defined later by the factor of safety F_L in Eq. C7.4.1-3) is determined **jointly** by a_{max} and M. Because of the longer duration of strong ground-shaking, large distant earthquakes may generate liquefaction at a site while smaller nearby earthquakes may not generate liquefaction even though a_{max} of the nearer events is larger than that from the more distant events.

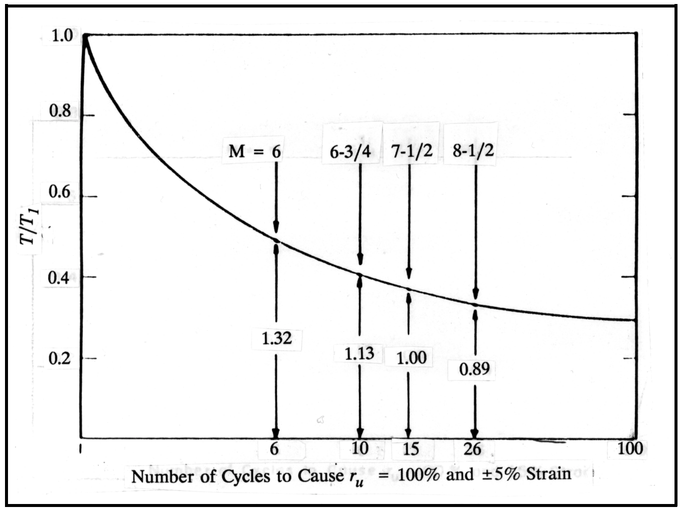


FIGURE C7.4.1-3 Representative relationship between T/T_1 and number of cycles required to cause liquefaction (after Seed et al., 1983).

The corrected blow count, $(N_1)_{60}$, required for evaluation of soil liquefaction resistance is commonly determined from measured standard penetration resistance, N_m, but may also be determined from cone penetration test (CPT) data using standard correlations to estimate N_m values from the CPT measurements. The corrected blow count is calculated from N_m as follows:

$$(N_1)_{60} = C_n \left(\frac{ER_m}{60} \right) N_m \qquad \text{(C7.4.1-2)}$$

where C_n = a factor that corrects N_m to an effective overburden pressure of 1 tsf and ER_m = the rod energy ratio for the type of hammer and release mechanism used in the measurement of N_m.

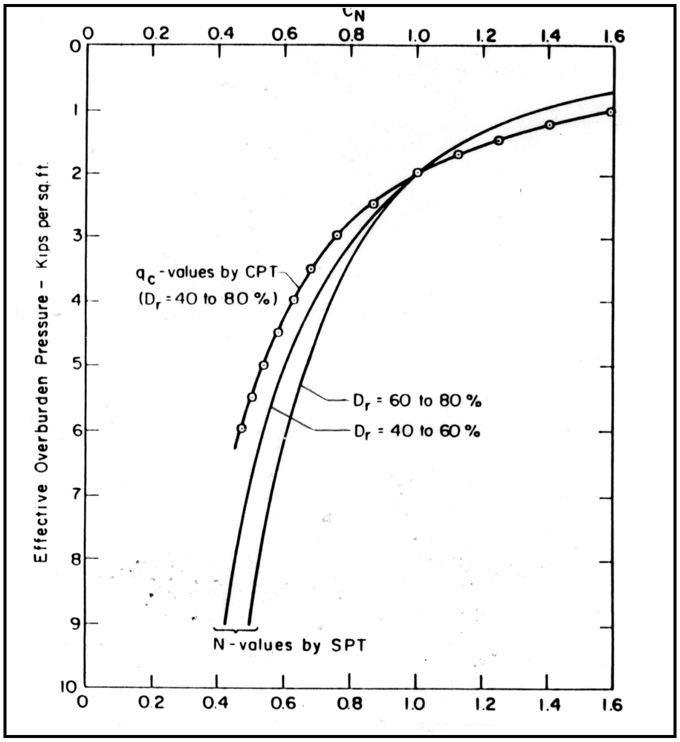


FIGURE C7.4.1-4 Chart for C_n (after Seed et al., 1985).

The curve plotted in Figure C7.4.1-4 is typically used to evaluate C_n. Measured hammer energies or estimates of hammer energies from tabulations such as those in Table C7.4.1 are used to define ER_m. An additional correction should be made to $(N_1)_{60}$ for shallow soil layers where the length of drilling rod is 10 feet or less. In those instances, $(N_1)_{60}$ should be reduced by multiplying by a factor of 0.75 to account for poor hammer-energy transfer in such short rod lengths.

Because a variety of equipment and procedures are used to conduct standard penetration tests in present practice and because the measured blow count, N_m, is sensitive to the equipment and procedures used, the following commentary and guidance with respect to this test is given. Special attention must be paid to the determination of normalized blow count, $(N_1)_{60}$, used in Figure C7.4.1-2. When developing the empirical relation between blow count and liquefaction resistance, Seed and his colleagues recognized that the blow count from SPT is greatly influenced by factors such as the method of drilling, the type of hammer, the sampler design, and the type of mechanism used for lifting and dropping the hammer. The magnitude of variations is shown by the data in Table C7.4.1.

TABLE C7.4.1 Summary of Rod Energy Ratios for Japanese SPT Procedures (after Seed et al., 1985)

Study	Mechanical Trip System (Tonbi)	Rope and Pulley
Nishizawa et al.	80-90	63-72
Decker, Holtz, and Kovacs	76	--
Kovacs and Salomone	80	67
Tokimatsu and Yoshimi	76[a]	--
Yoshimi and Tokimatsu, Yoshimi et al., Oh-Oka	--	--
Adopted for this study	78	67

[a] Equivalent rod energy ratio if rope and pulley method is assumed to have an energy ratio of 67 percent and values for mechanical trip method are different from this by a factor of 1.13.

In order to reduce variability in the measurement of *N*, Seed et al. (1983 and 1985) suggest the following procedures and specifications for the SPT test for liquefaction investigations:

1. The impact should be delivered by a rope and drum system with two turns of the rope around the rotating drum to lift a hammer weighing 140 lb or, more preferably, a drive system should be used for which ER_m has been measured or can be reliably estimated.
2. Use of a hole drilled with rotary equipment and filled with drilling mud. The hole should be approximately 4 in. in diameter and drilled with a tricone or baffled drag bit that produces upward deflection of the drilling fluid to prevent erosion of soil below the cutting edge of the bit.
3. In holes less than 50 feet deep, A or AW rod should be used; N or NW rod should be used in deeper holes.
4. The split spoon sampling tube should be equipped with liners or otherwise have a constant internal diameter of 1-3/8 inch.
5. Application of blows should be at a rate of 30 to 40 blows per minutes. (Some engineers suggest a slower rate of 20 to 30 blows per minute since it is easier to achieve and control and gives comparable results.) The blow count, N_m, is determined by counting the blows required to drive the penetrometer through the depth interval of 6 to 18 in. below the bottom of the hole.

Failure to follow these standard guidelines introduces large uncertainties into liquefaction estimates.

The curves in Figure C7.4.1-2 were developed from data for magnitude 7.5 earthquakes and are only valid for earthquakes of that magnitude. For larger or smaller earthquakes, the cyclic stress ratios determined from Figure C7.4.1-2 are corrected for magnitude by multiplying the determined cyclic stress ratio by a magnitude scaling factor taken from Figure C7.4.1-3. As the magnitude increases, the scaling factor decreases. For example, for an $(N_1)_{60}$ of 20, a clean sand (fines content < 5 percent) and an earthquake magnitude of 7.5, the CSRL determined from Figure C7.4.1-2 is 0.22. For the same site conditions but for a magnitude 8.0 earthquake, a CSRL of 0.20 is obtained after applying the magnitude scaling factor of 0.89 determined from Figure C7.4.1-3.

Soils composed of sands, silts, and gravels are most susceptible to liquefaction while clayey soils generally are immune to this phenomenon. The curves in Figure C7.4.1-2 are valid for soils composed primarily of sand. The curves should be used with caution for soils with substantial amounts of gravel. Verified corrections for gravel content have not been developed; a geotechnical engineer, experienced in liquefaction hazard evaluation, should be consulted when gravelly soils are encountered. For soils containing more than 35 percent fines, the curve in Figure C7.4.1-2 for 35 percent fines should be used provided the following criteria developed by Seed et al. (1983) are met (i.e, the weight of soil particles finer than 0.005 mm is less than 15 percent of the dry weight of a specimen of the soil, the liquid limit of soil is less than 35 percent, and the moisture content of the in-place soil is greater than 0.9 times the liquid limit.

In summary, the procedure for evaluation of liquefaction resistance for a site is as follows: First, from a site investigation determine the measured standard penetration resistance, N_m, the percent fines, the percent clay (> 0.005 mm), the natural moisture content, and the liquid limit of the sediment in question. Check the measured parameters against the fines content and moisture criteria listed above to assure that the sediment is of a potentially liquefiable type. If so, correct N_m to $(N_1)_{60}$ using Eq. C7.4.1-2 and use Figure C7.4.1-2 to determine the cyclic stress ratio required to cause liquefaction for a magnitude 7.5 earthquake. Then correct that value using the appropriate magnitude scaling factor. That product is the cyclic stress ratio required to cause liquefaction in the field (CSRL). Next, calculate the cyclic stress ratio (CSRE) that would be generated by the expected earthquake using Eq. C7.4.1-1. Then compute the factor of safety, F_L, against liquefaction from the equation:

$$F_L = \frac{CSRL}{CSRE} \tag{C7.4.1-3}$$

If F_L is greater than one, then liquefaction should not develop. If at any depth in the sediment profile, F_L is equal to or less than one, then there is a liquefaction hazard. As noted above, a factor of safety of 1.2 to 1.5 is appropriate for building sites with the factor selected depending on the importance of the structure and the potential for ground displacement at the site.

Evaluation of Potential for Ground Displacements: Liquefaction by itself may or may not be of engineering significance. Only when liquefaction is accompanied by loss of ground support and/or ground deformation does this phenomenon become important to structural design. Loss of bearing capacity, flow failure, lateral spread, ground oscillation, and ground settlement are ground failure mechanisms that have caused structural damage during past earthquakes. These types of ground failure are described by the National Research Council (1985). The type of failure and amount of ground displacement are a function of several parameters including the thickness and extent of the liquefied layer, the thickness of unliquefied material overlying the liquefied layer, the ground slope, and the nearness of a free face. Criteria are given by Ishihara (1985) for evaluating the influence of thickness of layers on surface manifestation of liquefaction effects (ground fissures and sand boils) for level sites. These criteria may be used for noncritical or nonessential structures on level sites. Additional analysis should be required for critical or essential structures.

Loss of Bearing Strength: Loss of bearing strength is not likely for light structures with shallow footings founded on stable, nonliquefiable materials overlying deeply buried liquefiable layers, particularly if the liquefiable layers are relatively thin. General guidance for how deep or how thin the layers must be has not yet been developed. A geotechnical engineer, experienced in liquefaction hazard assessment, should be consulted to provide such guidance. Although loss of bearing strength may not be a hazard for deeply buried liquefiable layers, liquefaction-induced ground settlements or lateral-spread displacements could still cause damage and should be evaluated.

Ground Settlement: Tokimatsu and Seed (1987) published an empirical procedure for estimating ground settlement. It is beyond the scope of this commentary to outline that procedure which, although explicit, has several rather complex steps. For saturated or dry granular soils in a loose condition, their analysis suggests that the amount of ground settlement could approach 3 to 4 percent of the thickness of the loose soil layer. The Tokimatsu and Seed technique is recommended for estimating earthquake-induced ground settlement at sites underlain by granular soils and can be applied whether liquefaction does or does not occur.

Horizontal Ground Displacement: Only primitive analytical and empirical techniques have been developed to date to estimate ground displacement, and no single technique has been widely accepted or verified for engineering design. Analytical techniques generally apply Newmark's analysis of a rigid body sliding on an infinite or circular failure surface with ultimate shear resistance estimated from the residual strength of the deforming soil. Alternatively, nonlinear finite element methods have been used to predict deformations. Empirical procedures use correlations between past ground displacement and site conditions under which those displacements occurred. The liquefaction severity index (LSI) correlation of Youd and Perkins (1987) provides a conservative upper bound for displacement for most natural soils (Figure C7.4.1-5; curves noted for various earthquakes are calculated from the equation on the figure). In this procedure, maximum horizontal displacement of lateral spreads in late Holocene fluvial deposits are correlated against earthquake magnitude and distance for the seismic source. The data are from the western United States and the correlation is valid only for that region. Because maximum displacements at very liquefiable sites were used in the LSI analysis, displacements predicted by that technique are conservative in that they predict an upper bound displacement for most natural deposits. Displacements may be greater, however, on uncompacted fill or extremely loose natural deposits.

The ground motions to be primarily considered in evaluating liquefaction potential are consistent with the *design earthquake* motions used in structural design. The structural design should be consistent with liquefaction-induced deformations resulting from those ground motions.

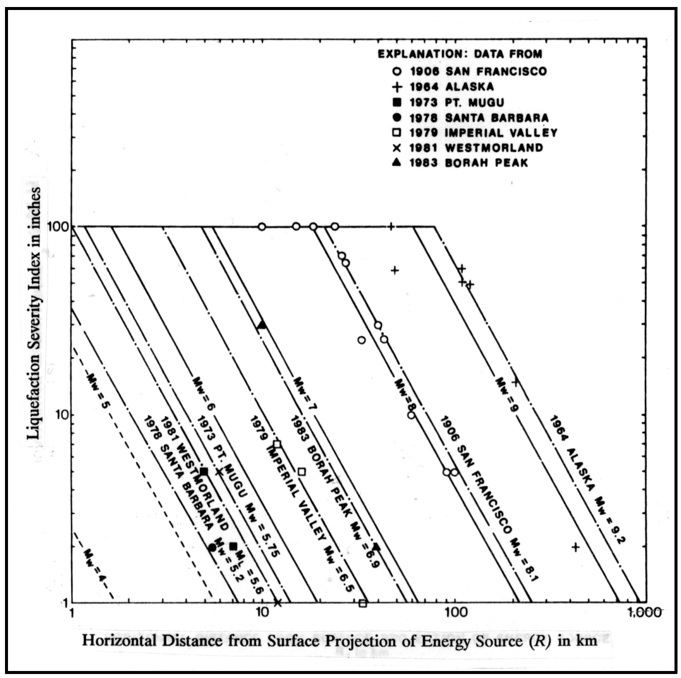


FIGURE C7.4.1-5 LSI from several western U.S. and Alaskan earthquakes plotted against horizontal distance from seismic energy sources (after Youd and Perkins, 1987).

Liquefaction-induced deformations are not directly proportional to ground motions and may be more than 50 percent higher for maximum considered earthquake ground motions. The liquefaction potential and resulting deformations for ground motions consistent with the maximum considered earthquake should also be evaluated and, while not required in the *Provisions*, should be used by the registered design professional in checking for building damage that may result in collapse. In addition, Seismic Use Group III structures should be designed to retain a significant margin against collapse following liquefaction-induced deformations resulting from maximum considered earthquake ground motions.

The following further information is given for general guidance for ground conditions and range of displacements commonly associated with liquefaction-induced ground failures (National Research Council, 1995; Barlett and Youd, 1995):

1. Flow failures generally develop in loose saturated sands or silts on slopes greater than 3 degrees (5 percent) and may displace large masses of soil tens of meters. Standard limit equilibrium slope stability analyses may be used to assess flow failure potential with the residual strength used as the strength parameter in the analyses. The residual strength may be determined from empirical correlations such as that published by Seed and Harder (1989).

2. Lateral spreads generally develop on gentle slopes between 0.5 and 3 degrees (0.1 and 5 percent) and may induce up to several feet of lateral displacement. Empirical correlations have been developed by Bartlett and Youd (1995) to estimate lateral ground displacement due to liquefaction. Analytical procedures using appropriately reduced (residual) strengths of soils also are available to estimate displacements. These procedures range from simplified Newmark-type sliding block methods (e.g., Newmark, 1985; Makdisi and Seed, 1978) to more sophisticated finite element analyses. In general, the empirical correlations are simple to apply, do not require data beyond the commonly compiled engineering site investigations, and are usually adequate for routine engineering applications.

3. Ground oscillation occurs on nearly flat surfaces where the slope is too gentle to induce permanent horizontal displacement. During an earthquake, however, ground oscillation generates transient vertical or horizontal displacements that may range up to a few feet. For example, ground oscillation caused the rather chaotic pattern of ground displacements that offset pavements, thrust sidewalks over curbs, etc., in San Francisco's Marina District following the 1989 Loma Prieta earthquake.

Mitigation of Liquefaction Hazard: With respect to liquefaction hazard, three mitigative measures might be considered: design the structure to resist the hazard, stabilize the site to reduce the hazard, or choose an alternative site. Structural measures that are used to reduce the hazard include deep foundations, mat foundations, or footings interconnected with ties as discussed in Sec. 7.4.3. Deep foundations have performed well at level sites of liquefaction where effects were limited to ground settlement and ground oscillation with no more than a few inches of lateral displacement. Deep foundations, such as piles, may receive very little soil support through the liquefied layer and may be subjected to transient lateral displacements across the layer. Well reinforced mat foundations also have performed well at localities where ground displacements were less than 1 foot although releveling of the structure has been required in some instances (Youd, 1989). Strong ties between footings also should provide increased resistance to damage where differential ground displacements are less than a few inches.

Evaluations of structural performance following two recent Japanese earthquakes, 1993 Hokkaido Nansei-Oki (M = 8.2) and 1995 (Kobe) Hyogo-Ken Nanbu (M = 7.2), indicate that small *structures* on shallow foundations performed well in liquefaction areas. Sand boil eruptions and open ground fissures in these areas indicate minor effects of liquefaction, including ground oscillation and up to several tenths of a meter of lateral spread displacement. Many small *structures* (mostly houses, shops, schools, etc.) were structurally undamaged although a few tilted slightly. Foundations for these *structures* consist of reinforced concrete perimeter wall footings with reinforced concrete interior wall footings tied into the perimeter walls at intersections. These foundations acted as diaphragms causing the soil to yield beneath the foundation which prevented fracture of foundations and propagation of differential displacements into the superstructure.

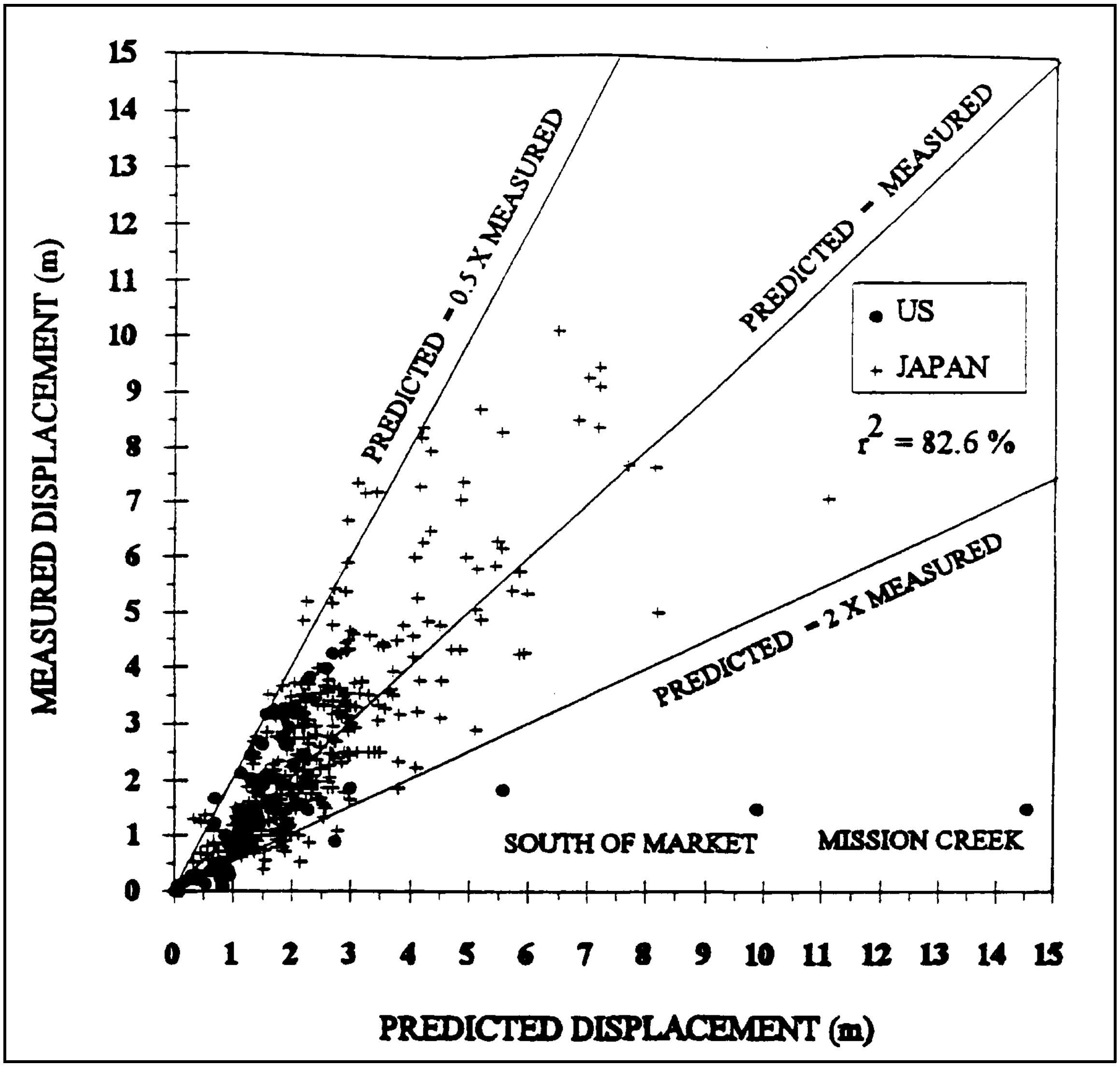


FIGURE C7.4.1-5 Measured displacements plotted against predicted displacements for U.S. and Japanese case-history data (after Bartlett and Youd, 1995).

Similarly, well reinforced foundations that would not fracture could be used in U.S. practice as a mitigative measure to reduce structural damage in areas subject to liquefaction but with limited potential for lateral (< 0.3 m) or vertical (< 0.05 m) ground displacements. Such strengthening also would serve as an effective mitigation measure against damage from other sources of limited ground displacement including fault zones, landslides, and cut fill boundaries. Where slab-on-grade or basement slabs are used as foundation elements, these slabs should be reinforced and tied to the foundation walls to give the structure adequate strength to resist ground displacement. Although strengthening of foundations, as noted above, would largely mitigate damage to the structure, utility connections may be adversely affected unless special flexibility is built into these nonstructural *components*.

Another possible consequence of liquefaction to structures is increased lateral pressures against basement walls. A common procedure used in design for such increased pressures is to assume that the liquefied material acts as a dense fluid having a unit weight of the liquefied soil. The wall then is designed assuming that hydrostatic pressure for the dense fluid acts along the total subsurface height of the wall. The procedure applies equivalent horizontal earth pressures that are greater than typical at-rest earth pressures but less than passive earth pressures. As a final consideration, to prevent buoyant rise as a consequence of liquefaction, the total weight of the structure should be greater than the volume of the basement or other cavity times the unit weight of liquefied soil. (Note that structures with insufficient weight to counterbalance buoyant effects could differentially rise during an earthquake.)

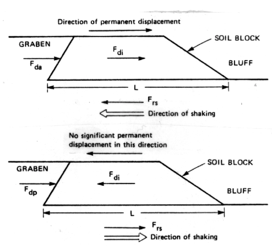
At sites where expected ground displacements are unacceptably large, ground modification to lessen the liquefaction or ground failure hazard or selection of an alternative site may be required. Techniques for ground stabilization to prevent liquefaction of potentially unstable soils include removal and replacement of soil; compaction of soil in place using vibrations, heavy tamping, compaction piles, or compaction grouting; buttressing; chemical stabilization with grout; and installation of drains. Further explanation of these methods is given by the National Research Council (1985).

Slope Instability: The stability of slopes composed of dense (nonliquefiable) or nonsaturated sandy soils or nonsensitive clayey soils can be determined using standard procedures.

For initial evaluation, the pseudostatic analysis may be used. (The deformational analysis described below, however, is now preferred.) In the pseudostatic analysis, inertial forces generated by earthquake shaking are represented by an equivalent static horizontal force acting on the slope. The seismic coefficient for this analysis should be the peak acceleration, a_{max}, or A_a. The factor of safety for a given seismic coefficient can be estimated by using traditional slope stability calculation methods. A factor of safety greater than one indicates that the slope is stable for the given lateral force level and further analysis is not required. A factor of safety of less than one indicates that the slope will yield and slope deformation can be expected and a deformational analysis should be made using the techniques discussed below.

Deformational analyses yielding estimates of slope displacement are now accepted practice. The most common analysis uses the concept of a frictional block sliding on a sloping plane or arc. In this analysis, seismic inertial forces are calculated using a time history of horizontal acceleration as the input motion. Slope movement occurs when the driving forces (gravitational plus inertial) exceed the resisting forces. This approach estimates the cumulative displacement of the sliding mass by integrating increments of movement that occur during periods of time when the driving forces exceed the resisting forces. Displacement or yield occurs when the earthquake ground accelerations exceed the acceleration required to initiate slope movement or yield acceleration. The yield acceleration depends primarily on the strength of the soil and the gradient and height and other geometric attributes of the slope. See Figure C7.4.1-6 for forces and equations used in analysis and Figure C7.4.1-7 for a schematic illustration for a calculation of the displacement of a soil block toward a bluff.

The cumulative permanent displacement will depend on the yield acceleration as well as the intensity and duration of ground-shaking. As a general guide, a ratio of yield acceleration to maximum acceleration of 0.5 will result in slope displacements of the order of a few inches for typical magnitude 6.5 earthquakes and perhaps several feet of displacement for magnitude 8 earthquakes. Further guidance on slope displacement is given by Makdisi and Seed (1978).



F_{da} = drifing force due to active soil pressure

F_{di} = driving force due to earthquake inertia

F_{rs} = resisting force due to soil shear strength

F_{dp} = resisting force due to passive soil pressure

$$F_{di} = K_{\max} W$$

where K_{max} = maximum seismic coefficient and W = weight of soil block

$$F_{rs} = S_u L$$

where S_u = average undrained shear strength of soil and L = length of soil block

Yield seismic coefficient:

$$K_y - \frac{F_{rs} - F_{da}}{W}$$

FIGURE C7.4.1-6 Forces and equations used in analysis of translatory landslides for calculating permanent lateral displacements from earthquake ground motions (National Research Council, 1985; from Idriss, 1985).

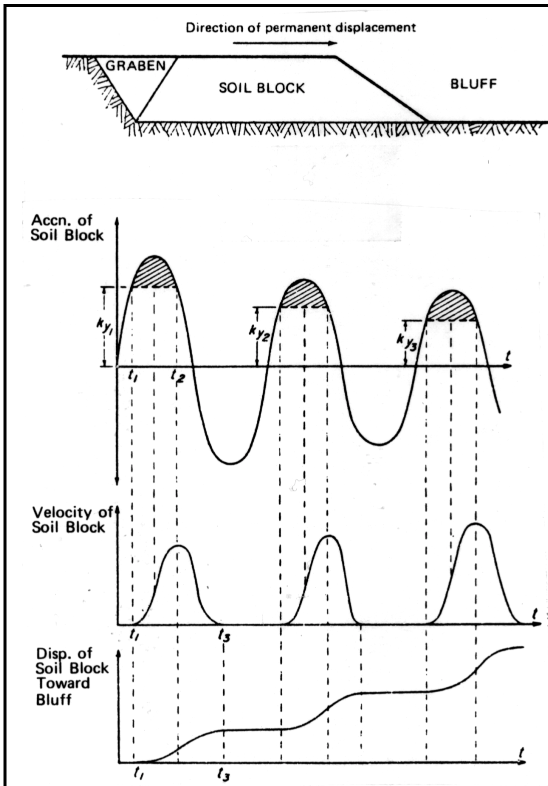


FIGURE C7.4.1-7 Schematic illustration for calculating displacement of soil block toward the bluff (National Research Council, 1985; from Idriss, 1985, adapted from Goodman and Seed, 1966).

Mitigation of Slope Instability Hazard: With respect to slope instability, three general mitigative measures might be considered: design the structure to resist the hazard, stabilize the site to reduce the hazard, or choose an alternative site. Ground displacements generated by slope instability are similar in destructive character to fault displacements generating similar senses of movement: compression, shear, extension or vertical. Thus, the general comments on structural design to prevent damage given under mitigation of fault displacement apply equally to slope displacement. Techniques to stabilize a site include reducing the driving forces by grading and drainage of slopes and increasing the resisting forces by subsurface drainage, buttresses, ground anchors, or chemical treatment.

7.4.2 Pole-Type Structures: The use of pole-type structures is permitted. These structures are inherently sensitive to earthquake motions. Bending in the poles and the soil capacity for lateral resistance of the portion of the pole embedded in the ground should be considered and the design completed accordingly.

7.4.3 Foundation Ties: One of the prerequisites of adequate performance of a building during an earthquake is the provision of a foundation that acts as a unit and does not permit one column or wall to move appreciably with respect to another. A common method used to attain this is to provide ties between footings and pile caps. This is especially necessary where the surface soils are soft enough to require the use of piles or caissons. Therefore, the pile caps or caissons are tied together with nominal ties capable of carrying, in tension or compression, a force equal to $C_a/4$ times the larger pile cap or column load.

A common practice in some multistory buildings is to have major columns that run the full height of the building adjacent to smaller columns in the basement that support only the first floor slab. The coefficient applies to the heaviest column load.

Alternate methods of tying foundations together are permitted (e.g., using a properly reinforced floor slab that can take both tension and compression). Lateral soil pressure on pile caps is not a recommended method because the motion is imparted from soil to structure (not inversely as is commonly assumed), and if the soil is soft enough to require piles, little reliance can be placed on soft-soil passive pressure to restrain relative displacement under dynamic conditions.

If piles are to support structures in the air or over water (e.g., in a wharf or pier), batter piles may be required to provide stability or the piles may be required to provide bending capacity for lateral stability. It is up to the foundation engineer to determine the fluidity or viscosity of the soil and the point where lateral buckling support to the pile can be provided (i.e., the point where the flow of the soil around the piles may be negligible).

7.4.4 Special Pile Requirements: Special requirements for concrete or composite concrete and steel piles are given in this section. The piles must be connected to the pile caps with dowels.

Although unreinforced concrete piles are common used in certain areas of the country, their brittle nature when trying to conform to ground deformations makes their use in earthquake-resistant design undesirable. Nominal longitudinal reinforcing is specified to reduce this hazard. The reinforcing steel should be extended into the footing to tie the elements together and to assist in load transfer at the top of pile to the pile cap. Experience has shown that concrete piles tend to hinge or shatter immediately below the pile cap so tie spacing is reduced in this area to better contain the concrete. In the case of the metal-cased pile, it is assumed that the metal casing provides containment and also a nominal amount of longitudinal reinforcement in the lower portion of the pile.

Bending stresses in piles caused by transfer of seismic motions from ground to structure need not be considered unless the foundation engineer determines that it is necessary. It has been a convenient analytical assumption to assume that earthquake forces originate in the building and are transmitted into and resisted by the ground. Actually the force or motion comes from the ground--not the structure. This makes the necessity of interconnecting footings more important, but what is desired is stability--not the introduction of forces.

Possibly the simplest illustration is shown in Figure C7.4.4. Consider a small structure subjected to an external force such as wind; the piles must resist that force in lateral pressure on the lee side of the piles. However, if the structure is forced to move during an earthquake, the wave motion is transmitted through the firmer soils, causing the looser soils at the surface and the building to move. For most structures, the structure weight is negligible in comparison to the weight of the

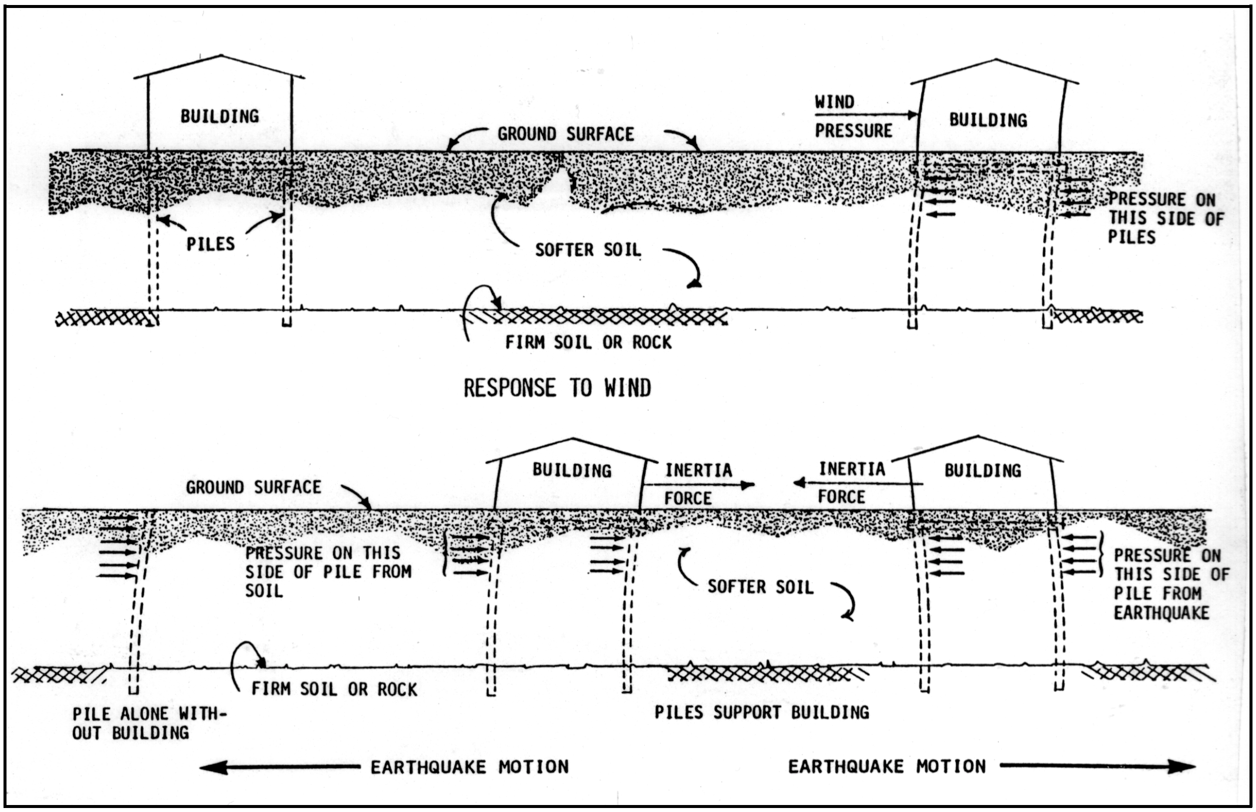


FIGURE C7.4.4 Response to earthquake.

surrounding surface soils. If an unloaded pile were placed in the soil, it would be forced to bend similar to a pile supporting a building.

The primary requirement is stability, and this is best provided by piles that can support their loads while still conforming to the ground motions and, hence, the need for ductility.

7.5 SEISMIC DESIGN CATEGORIES D , E, AND F: For Category D, E, or F construction, all the preceding provisions for Categories A, B, and C apply for the foundations, but the earthquake detailing is more severe and demanding. Adequate pile ductility is required and provision must be made for additional reinforcing to ensure, as a minimum, full ductility in the upper portion of the pile.

7.5.1 Investigation: In addition to the potential site hazard discussed in *Provisions* Sec. 7.4.1, consideration of lateral pressures on earth retaining structures shall be included in investigations for Seismic Design Categories D, E, and F.

Earth Retaining Structures: Increased lateral pressures on retaining structures during earthquakes have long been recognized; however, design procedures have not been prescribed in U.S. model building codes. Waterfront structures often have performed poorly in major earthquake due to excess pore water pressure and liquefaction conditions developing in relatively loose, saturated granular soils. Damage reports for structures away from waterfronts are generally limited with only a few cases of stability failures or large permanent movements (Whitman, 1991). Due to the apparent conservatism or overstrength in static design of most walls, the complexity of nonlinear dynamic soil-structure interaction, and the poor understanding of the behavior of retaining structures with cohesive or dense granular soils, Whitman (1991) recommends that "engineers must rely primarily on a sound understanding of fundamental principles and of general patterns of behavior."

Seismic design analysis of retaining walls is discussed below for two categories of walls: "yielding" walls that can move sufficiently to develop minimum active earth pressures and "nonyielding" walls that do not satisfy this movement condition. The amount of movement to develop minimum active pressure is very small. A displacement at the top of the wall of 0.002 times the wall height is typically sufficient to develop the minimum active pressure state. Generally, free-standing gravity or cantilever walls are considered to be yielding walls (except massive gravity walls founded on rock), whereas building basement walls restrained at the top and bottom are considered to be nonyielding.

Yielding Walls: At the 1970 Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, Seed and Whitman (1970) made a significant contribution by reintroducing and reformulating the Monobe-Okabe (M-O) seismic coefficient analysis (Monobe and Matsuo, 1929; Okabe, 1926), the earliest method for assessing the dynamic lateral pressures on a retaining wall. The M-O method is based on the key assumption that the wall displaces or rotates outward sufficiently to produce the minimum active earth pressure state. The M-O formulation is expressed as:

$$P_{AE} = (1/2)\gamma H^2 (1 - k_v) K_{AE} \qquad (7.5.1\text{-}1)$$

where: P_{AE} is the total (i.e., static + dynamic) lateral thrust, (γ is unit weight of backfill soil, H is height of backfill behind the wall, k_V is vertical ground acceleration divided by gravitational acceleration, and K_{AE} is the static plus dynamic lateral earth pressure coefficient which is dependent on (in its most general form) angle of friction of backfill, angle of wall friction, slope of backfill surface, and slope of back face of wall, as well as horizontal and vertical ground acceleration. The formulation for K_{AE} is given in textbooks on soil dynamics (Prakash, 1981; Das, 1983; Kramer, 1996) and discussed in detail by Ebeling and Morrison (1992).

Seed and Whitman (1970), as a convenience in design analysis, proposed to evaluate the total lateral thrust, P_{AE}, in terms of its static component (P_A) and dynamic incremental component (ΔP_{AE}):

$$P_{AE} = P_A + \Delta P_{AE} \tag{7.5.1-2}$$

or

$$K_{AE} = K_A + \Delta K_{AE} \tag{7.5.1-2a}$$

or

$$\Delta P_{AE} = (1/2)\gamma H^2 \Delta K_{AE} \tag{7.5.1-2b}$$

Seed and Whitman (1970), based on a parametric sensitivity analysis, further proposed that for practical purposes:

$$\Delta K_{AE} \sim (3/4)K_h \tag{7.5.1-3}$$

$$\Delta P_{AE} \sim (1/2)\gamma H^2 (3/4)k_h \sim (3/8)k_h \gamma H^2 \tag{7.5.1-3a}$$

where k_h is horizontal ground acceleration divided by gravitational acceleration. It is recommended that k_h be taken equal to the site peak ground acceleration that is consistent with design earthquake ground motions as defined in *Provisions* Sec. 7.5.3(i.e., $k_h = S_{DS}/2.5$). Equation 7.5.1-3 and 7.5.1-3a generally are referred to as the simplified M-O formulation.

Since its introduction, there has been a consensus in geotechnical engineering practice that the simplified M-O formulation reasonably represents the dynamic (seismic) lateral earth pressure increment for yielding retaining walls. For the distribution of the dynamic thrust, ΔP_{AE}, Seed and

Whitman (1970) recommended that the resultant dynamic thrust act at 0.6H above the base of the wall (i.e., inverted trapezoidal pressure distribution).

Using the simplified M-O formulation, a yielding wall may be designed using either a limit-equilibrium force approach (conventional retaining wall design) or an approach that permits movement of the wall up to tolerable amounts. Richards and Elms (1979) introduced a method for seismic design analysis of yielding walls considering translational sliding as a failure mode and based on tolerable permanent displacements for the wall. There are a number of empirical formulations for estimating permanent displacements under a translation mode of failure; these have been reviewed by Whitman and Liao (1985). Nadim (1980) and Nadim and Whitman (1984) incorporated the failure mode of wall tilting as well as sliding by employing coupled equations of motion, which were further formulated by Siddharthan et al. (1992) as a design method to predict the seismic performance of retaining walls taking into account both sliding and tilting. Alternatively, Prakash and others (1995) described design procedures and presented design charts for estimating both sliding and rocking displacements of rigid retaining walls. These design charts are the results of analyses for which the backfill and foundation soils were modeled as nonlinear viscoelastic materials. A simplified method that considers rocking of a wall on a rigid foundation about the toe was described by Steedman and Zeng (1996) and allows the determination of the threshold acceleration beyond which the wall will rotate. A simplified procedure for evaluating the critical threshold accelerations for sliding and tilting was described by Richards and others (1996).

Application of methods for evaluating tilting of yielding walls have been limited to a few case studies and back-calculation of laboratory test results. Evaluation of wall tilting requires considerable engineering judgement. Because the tilting mode of failure can lead to instability of a yielding retaining wall, it is suggested that this mode of failure be avoided in the design of new walls by proportioning the walls to prevent rotation and displace only in the sliding mode.

Nonyielding Walls: Wood (1973) analyzed the response of a rigid nonyielding wall retaining a homogeneous linear elastic soil and connected to a rigid base. For such conditions, Wood established that the dynamic amplification was insignificant for relatively low-frequency ground motions (i.e., motions at less than half of the natural frequency of the unconstrained backfill), which would include many or most earthquake problems.

For uniform, constant k_h applied throughout the elastic backfill, Wood (1973) developed the dynamic thrust, ΔP_E, acting on smooth rigid nonyielding walls as:

$$\Delta P_E = F k_h \gamma H^2 \tag{7.5.1-4}$$

The value of F is approximately equal to unity (e.g., Whitman, 1991) leading to the following approximate formulation for a rigid nonyielding wall on a rigid base:

$$\Delta P_E = k_h \gamma H^2 \tag{7.5.1-5}$$

As for yielding walls, the point of application of the dynamic thrust is taken typically at a height of 0.6*H* above the base of the wall.

It should be noted that the model used by Wood (1973) does not incorporate any effect on the pressures of the inertial response of a superstructure connected to the top of the wall. This effect may modify the interaction between the soil and the wall and thus modify the pressures from those calculated assuming a rigid wall on a rigid base. The subject of soil-wall interaction is addressed in the following sections. This section also provides further discussion on the applicability of the Wood and the M-O formulations.

Soil-Structure-Interaction Approach And Modeling for Wall Pressures: Lam and Martin (1986), Soydemir and Celebi (1992), Veletsos and Younan (1994a and 1994b), and Ostadan and White (1998), among others, argue that the earth pressures acting on the walls of embedded structures during earthquakes are primarily governed by soil-structure interaction (SSI) and, thus, should be treated differently from the concept of limiting equilibrium (i.e., M-O method). Soil-structure interaction includes both a kinematic component--the interaction of a massless rigid wall with the adjacent soil as modeled by Wood (1973)-- and an inertial component--the interaction of the wall, connected to a responding superstructure, with the adjacent soil. Detailed SSI analyses incorporating kinematic and inertial interaction may be considered for the estimation of seismic earth pressures on critical walls. Computer programs that may be utilized for such analyses include FLUSH (Lysmer et. al, 1975) and SASSI (Lysmer et al., 1981).

Ostadan and White (1998) have observed that for embedded structures subjected to ground shaking, the characteristics of the wall pressure amplitudes vs. frequency of the ground motion were those of a single-degree-of-freedom (SDOF) system and proposed a simplified method to estimate the magnitude and distribution of dynamic thrust. Results provided by Ostadan and White (1998) utilizing this simplified method, which were also confirmed by dynamic finite element analyses, indicate that, depending on the dynamic properties of the backfill as well as the frequency characteristics of the input ground motion, a range of dynamic earth pressure solutions would be obtained for which the M-O solution and the Wood (1973) solution represent a “lower” and an “upper” bound, respectively.

Chang and others (1990) have found that dynamic earth pressures recorded on the wall of a model nuclear reactor containment building were consistent with dynamic pressures predicted by the M-O solution. Analysis by Chang and others indicated that the dynamic wall pressures were strongly correlated with the rocking response of the structure. Whitman (1991) has suggested that SSI effects on basement walls of buildings reduce dynamic earth pressures and that M-O pressures may be used in design except where structures are founded on rock or hard soil (i.e., no significant rocking). In the latter case, the pressures given by the Wood (1973) formulation would appear to be more applicable. The effect of rocking in reducing the dynamic earth pressures on basement walls also has been suggested by Ostadan and White (1998). This condition may be explained if it is demonstrated that the dynamic displacements induced by kinematic and inertial *components* are out of phase.

Effect of Saturated Backfill on Wall Pressures: The previous discussions are limited to backfills that are not water-saturated. In current (1999) practice, drains typically are incorporated in the design to prevent groundwater from building up within the backfill. This is not practical or feasible, however, for waterfront structures (e.g., quay walls) where most of the earthquake-induced failures have been reported (Seed and Whitman, 1970; Ebeling and Morrison, 1992; ASCE-TCLEE, 1998).

During ground shaking, the presence of water in the pores of a backfill can influence the seismic loads that act on the wall in three ways (Ebeling and Morrison, 1992; Kramer, 1996): (1) by altering the inertial forces within the backfill, (2) by developing hydrodynamic pressures within the backfill and (3) by generating excess porewater pressure due to cyclic straining. Effects of the presence of water in cohesionless soil backfill on seismic wall pressures can be estimated using formulations presented by Ebeling and Morrison (1992).

A soil liquefaction condition behind a wall may under the design earthquake have a pronounced effect on the wall pressures during and for some time after the earthquake. At present (1999), there is no general consensus established for estimating lateral earth pressures for liquefied backfill conditions. One simplified and probably somewhat conservative approach is to treat the liquefied backfill as a heavy viscous fluid exerting a hydrostatic pressure on the wall. In this case, the viscous fluid has the total unit weight of the liquefied soil. If unsaturated soil is present above the liquefied soil, it is treated as a surcharge that increases the fluid pressure within the underlying liquid soil by an amount equal to the thickness times the total unit weight of the surcharge soil. In addition to these "static" fluid pressures exerted by a liquefied backfill, hydrodynamic pressures can be exerted by the backfill. The magnitude of any such hydrodynamic pressures would depend on the level of shaking following liquefaction. Hydrodynamic effects may be estimated using formulations presented by Ebeling and Morrison (1992).

7.5.2 Foundation Ties: The additional requirement is made that spread footings on soft soil profiles should be interconnected by ties. The reasoning explained above under Sec. 7.4.3 also applies here.

7.5.4 Special Pile and Grade Beam Requirements: Additional pile reinforcing over that specified for Category C buildings is required. The reasoning explained above under Sec. 7.4.4 applies here.

Special consideration is required in the design of concrete piles subject to significant bending during earthquake shaking. Bending can become crucial to pile design where portions of the foundation piles may be supported in soils such as loose granular materials and/or soft soils that are susceptible to large deformations and/or strength degradation. Severe pile bending problems may result from various combinations of soil conditions during strong ground shaking.

For example:

1. Soil settlement at the pile-cap interface either from consolidation of soft soil prior to the earthquake or from soil compaction during the earthquake can create a free-standing short column adjacent to the pile cap.

2. Large deformations and/or reduction in strength resulting from liquefaction of loose granular materials can cause bending and/or conditions of free-standing columns.

3. Large deformations in soft soils can cause varying degrees of pile bending. The degree of pile bending will depend upon thickness and strength of the soft soil layer(s) and/or the properties of the soft/stiff soil interface(s).

Such conditions can produce shears and/or curvatures in piles that may exceed the bending capacity of conventionally designed piles and result in severe damage. Analysis techniques to evaluate pile bending are discussed by Margason and Holloway (1977) and these effects on concrete piles are further discussed by Shepard (1983). For homogeneous, elastic media and assuming the pile follows the soil, the free-field curvature (soil strains without a structure present) can be estimated by dividing the peak ground acceleration by the square of the shear wave velocity of the soil although considerable judgment is necessary in utilizing this simple relationship in a layered, inelastic profile with pile-soil interaction effects. Norris (1994) discusses methods to assess pile-soil interaction with regard to pile foundation behavior.

The designer needs to consider the variation in soil conditions and driven pile lengths in providing for pile ductility at potential high curvature interfaces. Interaction between the geotechnical and structural engineers is essential.

It is prudent to design piles to remain functional during and following earthquakes in view of the fact that it is difficult to repair foundation damage. The desired foundation performance can be accomplished by proper selection and detailing of the pile foundation system. Such design should accommodate bending from both reaction to the building's inertial loads and those induced by the motions of the soils themselves. Examples of designs of concrete piles include:

1.Use of a heavy spiral reinforcement and

2.Use of exterior steel liners to confine the concrete in the zones with large curvatures or shear stresses.

These provide proper confinement to ensure adequate ductility and maintenance of functionality of the confined core of the pile during and after the earthquake.

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