C4. Foundations and Geotechnical Hazards (Systematic Rehabilitation)

C4.1 Scope

The fundamental reason for including consideration of foundations and geotechnical hazards in seismic rehabilitation of existing buildings is to improve the overall performance of the buildings. The geotechnical engineer and engineering geologist should work directly with the structural engineer and the building owner or the owner's representative, when necessary, to achieve the optimum rehabilitation strategy for the desired Rehabilitation Objective.

Typically, foundations have performed reasonably well on sites where ground displacement has not occurred because of surface faulting, landsliding, or liquefaction. Furthermore, modifying foundations to improve their performance during anticipated earthquake loading can be very costly because of the limited working space, as well as the presence of the building. Therefore, it is desirable to undertake costly foundation modifications only when they are essential to meeting seismic Rehabilitation Objectives for the building.

In addition to addressing building foundation capacities and deformations during earthquakes, the guidelines address other potential geologic hazards associated with earthquakes that may affect the performance of buildings on some sites.

C4.2 Site Characterization

In gathering data for site characterization, the following should be included:

- Visual inspection of the structure and its foundation
- Review of geotechnical reports, drawings, test results, and other available documents directly related to the building
- Review of regional or local reports related to geologic and seismic hazards, and subsurface conditions
- Site exploration, including borings and test pits
- Field and laboratory tests

The scope of the documentation program for a building depends upon specific deficiencies and the Rehabilitation Objective. In some cases, the cost of extensive analysis and testing can be justified by producing results that will allow the use of more accurately determined material properties than the conservative default values prescribed by the *Guidelines*.

Geotechnical information will be required to establish the subsurface conditions that exist beneath the building, to describe the building foundations, and to assess potential earthquake-related hazards that may affect the performance of the site. The general procedure for evaluating foundations and geotechnical information is outlined on Figure C4-1. In many instances, existing data may be sufficient to characterize the site. However, a detailed site assessment may be required for:

- Structures that require an enhanced level of seismic performance
- Facilities that are supported upon deep foundations
- Facilities that are located within areas that may be subjected to fault rupture, liquefaction, lateral spreading, differential compaction, and landsliding

Such detailed site assessments may be conducted with existing information or with new subsurface data. The following text discusses data sources that should be reviewed in the site characterization, along with the requirements for defining the subsurface conditions and describing the existing foundations.

Data Sources. Information required to adequately characterize a site will likely be derived from a combination of several sources, including existing data, a site reconnaissance, and site-specific studies. Potential data sources include the following:

- geological maps
- topographical maps
- hazard maps
- geotechnical reports
- design/construction drawings

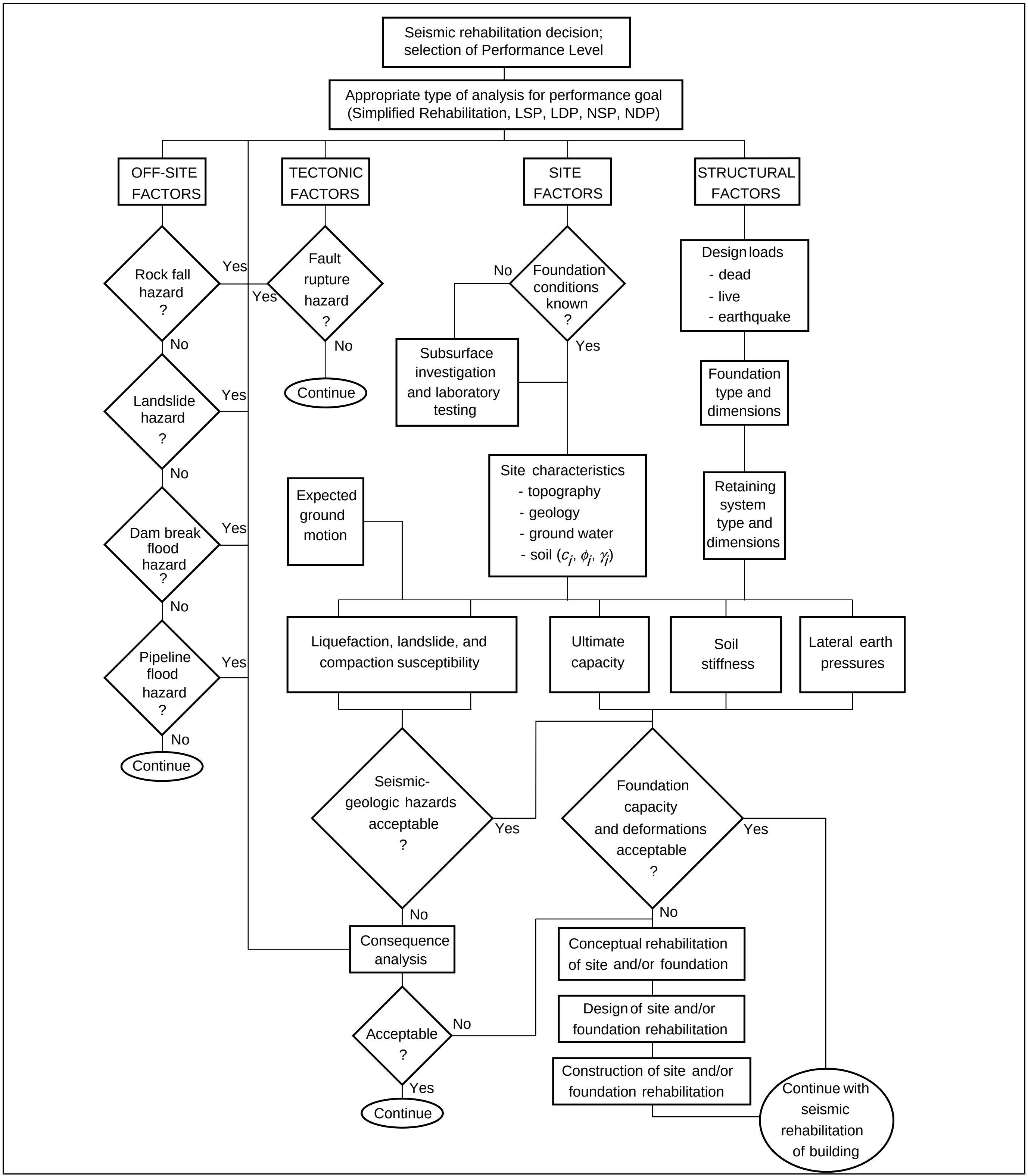

Figure C4-1 General Procedure: Evaluating Foundations and Geotechnical Information

Regional maps—including topographic maps and geologic maps—may be used to provide a general source of information on the conditions in the vicinity of the site. Topographic maps can be useful in assessing the landslide hazard potential that may affect the site. Similarly, geologic maps can provide information on surficial geologic units that may be related to ground stability. Finally, various hazard maps may exist indicating potential earthquake faults, and areas potentially susceptible to liquefaction, landsliding, and flooding or inundation. All of these maps may be used to provide an assessment of the large-scale performance of the site.

On a more local level, site-specific information may be obtained from geotechnical reports and foundation drawings. Relevant site information to be obtained from geotechnical reports includes logs of borings and/or cone penetrometer tests and laboratory tests to determine shear strengths of the subsurface materials, and engineering assessments that may have been conducted addressing geologic hazards, such as faulting, liquefaction, and landsliding. If geotechnical reports are not available for the subject facility, geotechnical reports for adjacent buildings may also provide a basis for developing the engineering assessments of the earthquake performance of the site. Finally, information should be obtained from geological reports or other regional studies regarding potential depths of the groundwater table.

Information contained on existing building drawings should be reviewed for relevant foundation data. This data would include the type, size, and location of all footings and footing design loads.

In addition to gathering existing data, a site reconnaissance should be performed to document the performance of the site and building. The site reconnaissance is conducted to gather information for several purposes. First, the reconnaissance should confirm that the actual site conditions agree with information obtained from the building drawings. Variances from the building drawings should be noted and considered in the evaluation. Such variances include building additions or foundation modifications that are not shown on the existing documentation.

A second purpose is to ascertain the presence of a potentially hazardous condition, such as a nearby steep slope susceptible to landsliding or rock fall, or a stream channel toward which lateral spreading could occur. A third purpose of the site reconnaissance is to document off-site development that may have a potential impact on the building. Such off-site development could include building grading activities that may impose a load or reduce a level of lateral support to the structure under consideration.

The site reconnaissance also should document the performance of the existing building and the adjacent area to denote signs of poor foundation performance, such as settlement of floor slabs, foundations, or sidewalks. These indicators may suggest structural distress that could affect performance during a future earthquake, as well as indicate the presence of soils that might settle during an earthquake.

The existing site data and information gained from the site reconnaissance may need to be supplemented by additional site explorations where there is a significant potential for the site to be affected by fault rupture, liquefaction, lateral spreading, differential compaction, or landsliding, or where the site has exhibited poor performance as reflected in ground settlement or building settlement. Under these conditions, detailed subsurface information will be required to define the subsurface stratigraphy and the engineering properties of the underlying soils. While the scope and extent of such explorations depends upon the number and type of existing studies that have been conducted at the site, new explorations may be required to augment the existing database. Applicable subsurface exploration procedures include:

- exploration borings
- cone penetrometer tests (CPTs)
- seismic cone penetrometer tests (SCPTs)
- standard penetration tests (SPTs)
- test pits
- laboratory testing

Buildings with shallow foundations often can be evaluated adequately by test pits, particularly if footing dimensions or conditions are unknown. Test pits or borings extending 10–15 feet below the footing often provide adequate geotechnical information. End-driven tube samples should be collected from test pit exposures; shoring of test pit walls must be done to provide safety during sampling and to comply with safety regulations.

Buildings with deep foundations may require borings with SPTs, CPTs, and/or SCPTs to provide adequate geotechnical information on the stratigraphy and material properties of the underlying soils. Explorations must extend below the depth of influence of the foundations. This depth, determined by a geotechnical engineer, depends on the foundation type and the nature of the subsurface materials. SPT sampling should be done at frequent intervals (3–5 feet) within the site borings. Undisturbed sampling should be conducted, where possible, within the underlying soil units to provide suitable samples for laboratory testing to determine unit weight, soil shear strengths, and friction angles of the underlying soil. More detailed stratigraphic information can be obtained from CPTs and SCPTs. Soil stiffnesses may be determined directly from the results of the SCPTs, or indirectly through empirical correlations with static soil properties.

If general information about the site region is known well enough to indicate uniform conditions over the dimensions of the building, then one boring, sounding, or test pit may be adequate. However, two or more borings, soundings, test pits, or a combination of the subsurface investigation techniques will be needed to increase confidence that the site is being adequately characterized. The adequate number of subsurface investigation locations depends on the size of the site, the complexity of the site geology, and the importance of the structure.

C4.2.1 Foundation Soil Information

It is necessary to define subsurface conditions at each building location in sufficient detail so as to assess the ultimate capacity of the building foundations and to determine if the site may be potentially affected by an earthquake-related hazard, such as earthquake-induced landsliding, lateral spreading, and liquefaction. The level to which subsurface conditions need to be defined depends on the Rehabilitation Objective for the facility and the specific foundations and subsurface conditions.

As a minimum, the site stratigraphy must be defined to establish the materials that underlie the foundations. This assessment must include information on the material composition (sand/clay) and the consistency or relative density of the underlying soil units. The consistency or the relative density of the underlying soil may be assumed from empirical correlations of SPT N-values. Additionally, the definition of the site subsurface conditions must include an assessment of the location of the water table beneath the structure and any seasonal fluctuations of the water table. Fluctuations of the water table may affect the ultimate bearing capacity of the building foundations and the potential for liquefaction.

With this minimum amount of information, presumptive or prescriptive procedures may be used to determine the ultimate bearing capacity of the foundations. However, additional information is required for site-specific assessments of foundation bearing capacity and stiffness. Acquiring this additional information involves determining unit weights, shear strength, friction angle, compressibility characteristics, soil moduli, and Poisson’s ratio.

The site characterization also requires information defining the type, size, and location of the foundation elements supporting the structure. Types of foundations include spread footings, mats, driven pile foundations, cast-in-place piles, and drilled piers. Other required information includes the size of the foundation elements, locations of the base of the footings or the tips of the piles, the pile cap elevations, foundation material composition (i.e., wood, steel, or concrete piles), and pile installation methods (i.e., opened- or closed-end piles, driven or jetted). The design drawings may also indicate information regarding the allowable bearing capacity of the foundation elements. This information can be used directly in a presumptive or prescriptive evaluation of the foundation capacity. Construction records may also be available indicating ultimate pile capacities if load tests were performed. Finally, information on the existing loads on the structure is relevant to determining the amount of overload that the foundations may be capable of resisting during an earthquake.

C4.2.2 Seismic Site Hazards

Earthquake-related site hazards—including fault rupture, liquefaction, differential compaction, landsliding, or flooding—can affect the ability of a structure or building to meet the desired seismic Performance Level. In some instances, the probability of occurrence of these hazards is small enough that they may be neglected, depending on the Rehabilitation Objectives for a specific project.

The *Guidelines* provide information on evaluation of site hazards. An initial assessment for each hazard can be conducted based on readily available data. This initial assessment might result in an indication that further consideration of a specific hazard is unnecessary. For example, on hillside sites with slopes of less than some prescribed value, landsliding need not be a design consideration. If a specific hazard cannot be eliminated from further consideration, the *Commentary* provides resources for more detailed investigations.

The result of the detailed investigation of site hazards will be to predict the nature and magnitude of ground movement for use by a structural engineer in the rehabilitation design. The events causing these movements must be consistent, in a probabilistic sense, with the chosen Performance Levels for the rehabilitation. It makes no sense to rehabilitate a structure to remain operational after a 500-year earthquake if a landslide with a much greater chance of occurrence could cause its collapse.

C4.2.2.1 Fault Rupture

Ground displacements generally are expected to recur along preexisting faults. The development of a new fault or reactivation of a very old (pre-Quaternary) fault is uncommon and generally need not be a concern for typical buildings. In general, the more recent and frequent the displacement is along a fault, the greater the probability of future faulting. The evaluation of future fault-rupture hazards involves careful application of skills and techniques not commonly used in other engineering geologic investigations (e.g., detailed examination of trench exposures and radiometric dating of geologic materials). Many active faults are complex, consisting of multiple breaks that may have originated during different surface-faulting earthquakes. To accurately evaluate the potential hazards of surface fault rupture, the engineering geologist must determine:

- The locations of fault traces
- The nature and amount of near-surface fault deformations (shear displacements and folding or warping)
- The history of the deformations

Key parameters are the age of the most recent displacement and the recurrence interval between successive displacements. Guidelines for evaluating surface fault rupture hazards have been developed in California and Utah (California Division of Mines and Geology, 1975; Slosson, 1984; Utah Section of the Association of Engineering Geologists, 1987). Maps showing the location of faults that have been active during Quaternary time (the most recent 1.8 million years of earth history) have been prepared for a number of regions (e.g., Nakata et al., 1982; Jennings, 1992; Hecker, 1993) and local areas (e.g., Hart et al., 1981; Bell, 1984; Personius and Scott, 1990).

Buildings found to straddle active faults must be assessed to determine if any rehabilitation is warranted—possibly to reduce collapse potential of the structure, given the likely amount and direction of fault displacement. Fault rupture is generally treated differently from seismic hazards related to ground motion. Active faults are considered capable of rupturing the ground surface on the basis of deterministic reasoning. Ground motion and the secondary hazards caused by it (liquefaction and landsliding) are evaluated with probabilistic reasoning. Thus, a site susceptible to liquefaction under ground motion considered to be less likely than 10%/50 years may be judged to have an acceptable risk, and seismic rehabilitation may proceed. However, a site straddling a fault considered to have displaced the ground surface two feet during the past 10,000 years may be judged to have an unacceptable risk, and rehabilitation may be abandoned. It is generally considered unacceptable for a new building to be situated straddling the trace of an active fault. However, policy has yet to be developed regarding the value and utility of an existing building that straddles an active fault.

Active faults differ in degree of activity and amount and character of displacement. Major active faults exhibit large amounts of displacement, which can be concentrated on a single trace, or several relatively closely spaced traces. Minor active faults exhibit small amounts of displacement on individual traces and can have a moderate amount of displacement distributed across an area. Active faults have caused strike-slip, normal-slip, and reverse-slip displacement (Figure C4-2a, b, c, respectively). Examples are the 1992 Landers earthquake in California, the 1983 Borah Peak earthquake in Idaho, and the 1971 San Fernando earthquake in California, respectively. In some geologic environments, surface fault rupture is oblique-slip (strike plus normal or reverse). Active faults commonly display a variety of characteristic landforms attesting to geologically youthful displacements. Figure C4-3

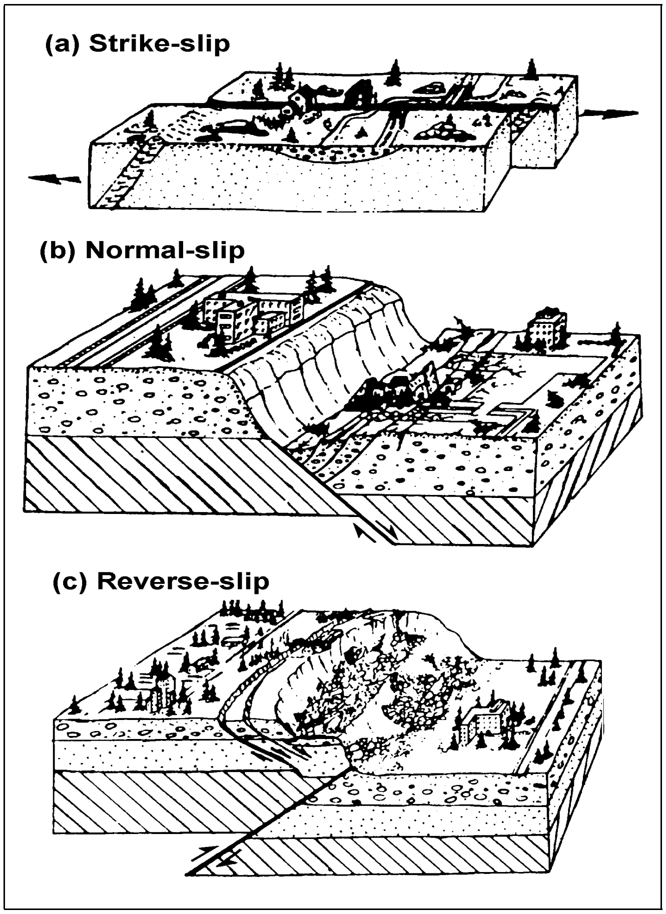

Figure C4-2 Schematic Diagrams of Surface Fault Displacement (modified from Slemmons, 1977)

illustrates some geomorphic features along active strike-slip faults.

C4.2.2.2 Liquefaction

Soil liquefaction is a phenomenon in which a soil below the groundwater table loses a substantial amount of strength due to strong earthquake ground shaking. Recently deposited (i.e., geologically young) and relatively loose natural soils and uncompacted or poorly compacted fill soils are potentially susceptible to liquefaction. Loose sands and silty sands are particularly susceptible; loose silts and gravels also have potential for liquefaction. Dense natural soils and well-compacted fills have low susceptibility to liquefaction. Clay soils are generally not susceptible, except for highly sensitive clays found in some geographic regions.

The *Guidelines* provide criteria that facilitate screening sites that do not have a significant liquefaction hazard. In addition to these criteria, if the site is located in an area where a regional mapping of liquefaction potential has been carried out by the USGS or other governmental agency, then such mapping might also be used to screen for a liquefaction hazard. Generally, sites located in areas characterized as having a low or very low liquefaction hazard can be screened out. However, definitions used in regional liquefaction potential zonations vary, and the definitions, bases, uncertainty, and qualifications associated with the zonation should be carefully reviewed before relying on regional maps.

The following paragraphs provide guidelines for evaluating liquefaction potential for cases where the hazard cannot be screened out. The occurrence of liquefaction by itself does not necessarily imply adverse consequences to a structure. Potential consequences of liquefaction include lateral spreading and flow slides, bearing capacity failure, settlements, increased lateral pressures on retaining walls, and flotation of buried structures. It is essential to assess the consequences of liquefaction and their effects on the structure. Thus, guidelines for such assessment are also presented below. Measures that may be considered to mitigate liquefaction hazards are discussed in Section C4.3.2.

In assessing liquefaction potential, available geotechnical data on the local geology (particularly the age of the geologic units) and the subsurface soil and groundwater conditions should be examined. Often, sufficient data are available from prior geotechnical investigations. If not, supplemental borings can be made or other subsurface investigation techniques (e.g., CPTs) can be used. Simplified, empirically-based procedures using blow count data from soil borings (or CPT data) generally can be used to evaluate liquefaction susceptibility. Occasionally, when dealing with soil types for which empirical correlations are less applicable, such as silts and gravels, it may be necessary to conduct special field and/or laboratory investigations.

Seed-Idriss Procedure for Evaluating Liquefaction Potential. The potential for liquefaction to occur may be assessed by a variety of available approaches (National Research Council, 1985). The most commonly utilized approach is the Seed-Idriss simplified empirical procedure—presented by Seed and Idriss (1971, 1982) and updated by Seed et al. (1985) and Seed and Harder (1990)—that utilizes SPT blow

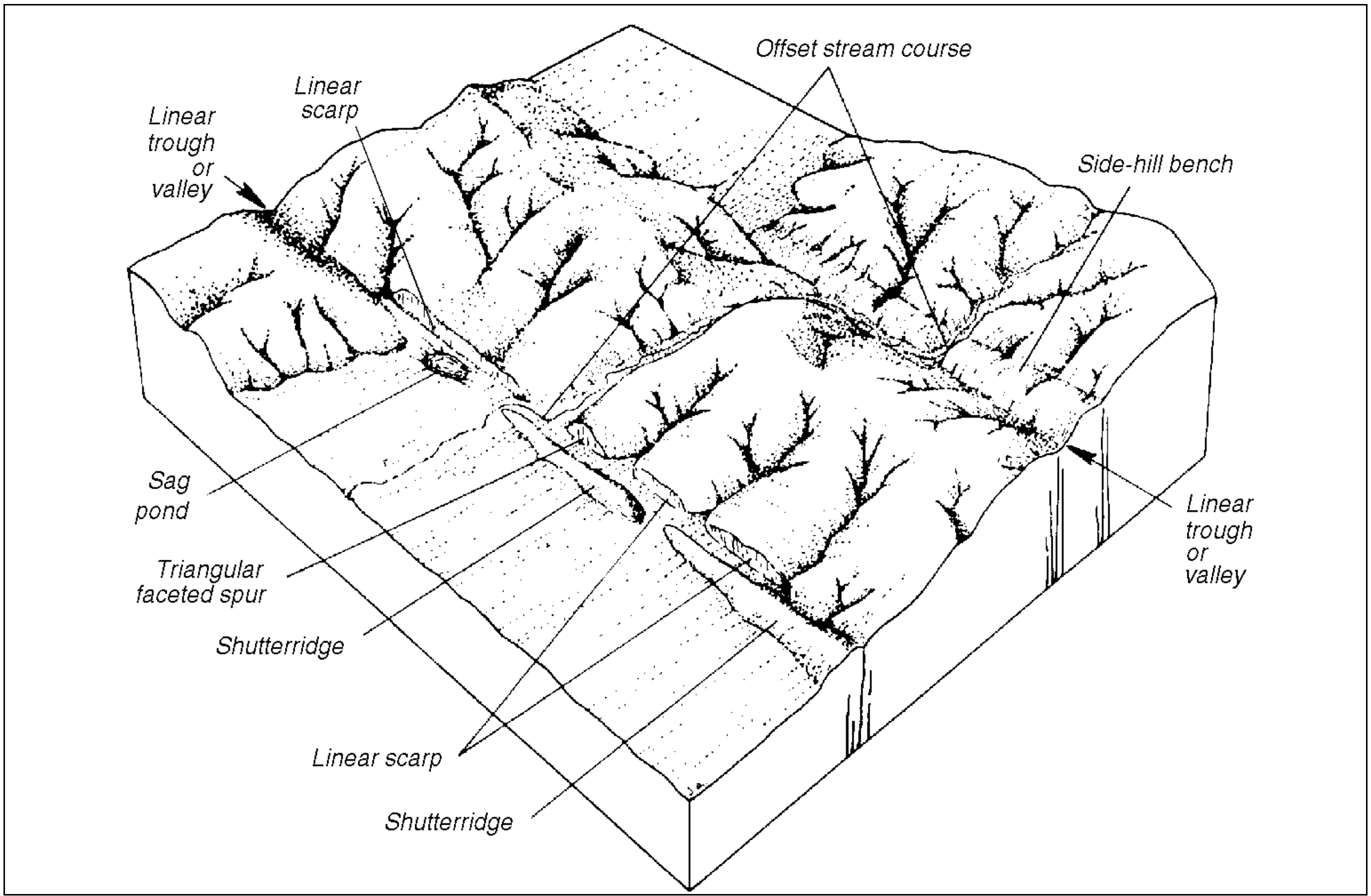

Figure C4-3 Features Commonly Found along Active Strike-Slip Faults (modified from Slemmons, 1977)

count data. Using SPT data to assess liquefaction potential due to an earthquake is considered a reasonable engineering approach (Seed and Idriss, 1982; Seed et al., 1985; National Research Council, 1985), because many of the factors affecting penetration resistance affect the liquefaction resistance of sandy soils in a similar way, and because these liquefaction potential evaluation procedures are based on actual performance of soil deposits during worldwide historical earthquakes.

The basic correlation used in the Seed-Idriss evaluation procedure is shown in Figure C4-4. The plot relates the cyclic stress ratio, τ_{av}/σ'_o, required to cause liquefaction to the normalized blow count obtained from SPT measurements in soil borings. In Figure C4-4, $(N_1)_{60}$ refers to SPT blow count values obtained using a standard 60% hammer energy efficiency and normalized to an effective overburden pressure of 2 ksf. Seed and Idriss (1982) and Seed et al. (1985) provide procedures to convert actual SPT blow counts measured in soil borings to $(N_1)_{60}$ values. Using the simplified procedure of Seed and Idriss (1971), values of τ_{av}/σ'_o induced in the soils by the earthquake ground shaking can be calculated and compared with the values of τ_{av}/σ'_o required to cause liquefaction as determined by the site measurements $(N_1)_{60}$ and Figure C4-4. The simplified procedure equation for calculating the induced cyclic stress ratio is:

$$\frac{\tau_{av}}{\sigma'_o} = 0.65\frac{PGA}{g}\frac{\sigma_o}{\sigma'_o}r_d \qquad \text{(C4-1)}$$

where

τ_{av}/σ'_o = Induced cyclic stress ratio

PGA = Peak ground acceleration (g units)

σ_o = Total overburden pressure at a depth of interest

σ'_o = Effective overburden pressure at a depth of interest

r_d = Stress reduction factor that decreases from a value of 1.0 at the ground surface to a value of 0.9 at a depth of about 35 feet

As an alternative to comparing the induced cyclic stress ratios with those required to cause liquefaction, critical values of $(N_1)_{60}$ can be determined from Figure C4-4 for the induced cyclic stress ratios obtained using Equation C4-1; these critical $(N_1)_{60}$ values can then be compared with the actual $(N_1)_{60}$ values for the site. For

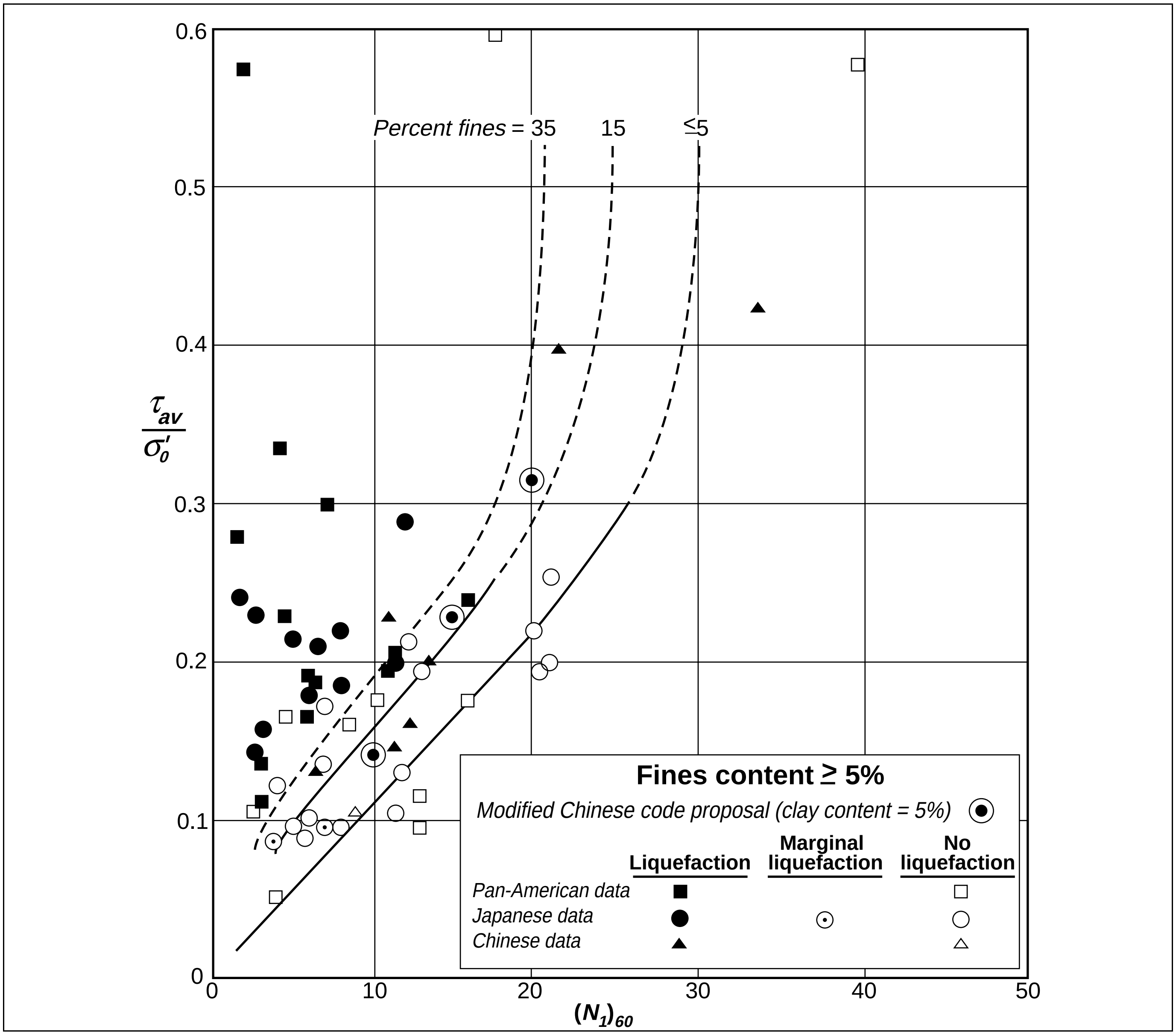

Figure C4-4 ***Relationship Between Cyclic Stress Ratio Causing Liquefaction and $(N_1)_{60}$ values for M = 7.5 Earthquakes (from Seed et al., 1985)***

example, Figure C4-5 illustrates a comparison between the critical $(N_1)_{60}$ line obtained based on the site peak ground acceleration and Figure C4-4, and the actual $(N_1)_{60}$ data for a site. In this illustration, the critical

$(N_1)_{60}$ line exceeds most of the site $(N_1)_{60}$ values, indicating liquefaction is likely to occur in this case. It should be recognized that the Seed-Idriss simplified procedure is based on average $(N_1)_{60}$ values at site; Fear and McRoberts (1995) conducted a reinterpretation of the catalogue of case histories that provided the basis for the Seed-Idriss simplified procedure systematically using minimum $(N_1)_{60}$, and pointed out the excess conservatism that could arise from treating the $(N_1)_{60}$ values from the Seed-Idriss curves as representing threshold (minimum) values.

CPT data may also be utilized with the Seed-Idriss approach by conversion to equivalent SPT blow counts, using correlations developed among cone tip resistance Q_c, friction ratio, soil type, and Q_c/N in which N is the SPT blow count (Seed and DeAlba, 1986; Robertson and Campanella, 1985). Direct correlations of CPT data with liquefaction potential have also been developed (Robertson and Campanella, 1985; Mitchell and Tseng, 1990; Robertson et al., 1992), but to date these are not as widely used as the Seed-Idriss correlation with $(N_1)_{60}$ blow count as shown in Figure C4-4.

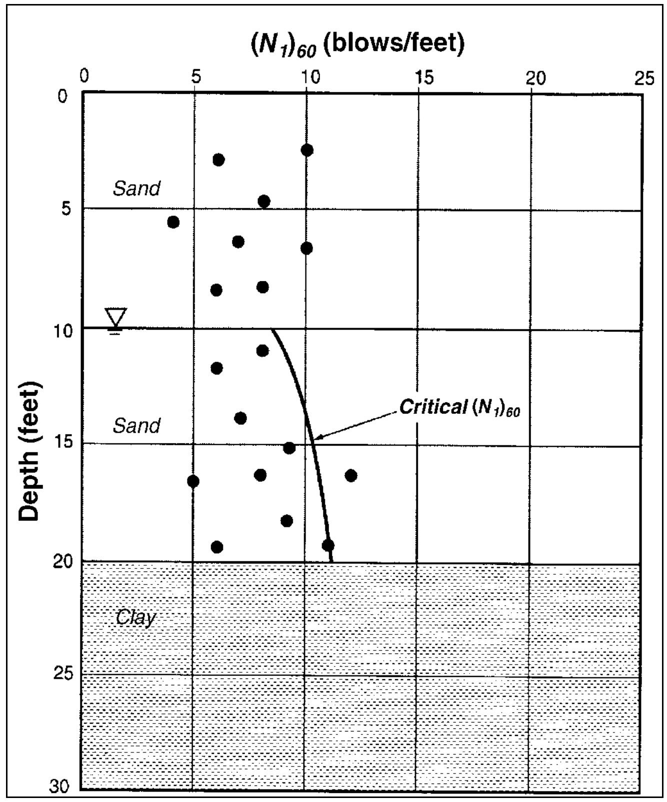

Figure C4-5 ***Comparing Site $(N_1)_{60}$ Data from Standard Penetration Tests with Critical $(N_1)_{60}$ Values Calculated using the Seed-Idriss Procedure***

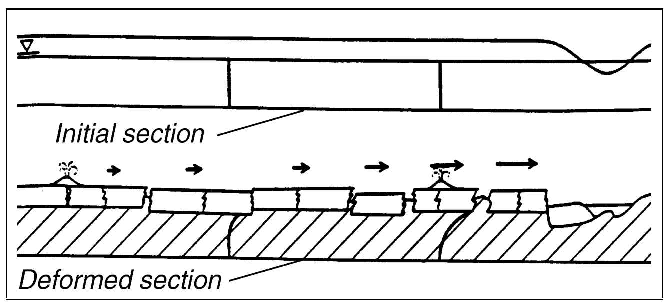

Figure C4-6 ***Lateral Spread Before and After Failure (from Youd, 1984)***

Evaluating Potential for Lateral Spreading. Lateral spreads are ground-failure phenomena that can occur on gently sloping ground underlain by liquefied soil. Earthquake ground-shaking affects the stability of sloping ground containing liquefiable materials by seismic inertia forces within the slope and by shaking-induced strength reductions in the liquefiable materials. Temporary instability due to seismic inertia forces is manifested by lateral "downslope" movement that can potentially involve large land areas. For the duration of ground shaking associated with moderate to large earthquakes, there could be many such occurrences of temporary instability, producing an accumulation of "downslope" movement. The resulting movements can range from a few inches or less to tens of feet, and are characterized by breaking up of the ground and horizontal and vertical offsets. A schematic of lateral spreading is illustrated in Figure C4-6.

Various relationships for estimating lateral spreading displacement have been proposed, including the Liquefaction Severity Index (LSI) by Youd and Perkins (1978), a relationship incorporating slope and liquefied soil thickness by Hamada et al. (1986), a modified LSI approach presented by Baziar et al. (1992), and a relationship by Bartlett and Youd (1992), in which they characterize displacement potential as a function of earthquake and local site characteristics (e.g., slope, liquefaction thickness, and grain size distribution). The relationship of Bartlett and Youd (1992), which is empirically based on analysis of case histories where lateral spreading did and did not occur, is relatively widely used, especially for initial assessments of the hazard. More site-specific analyses can also be made based on slope stability and deformation analysis procedures using undrained residual strengths for liquefied sand (Seed and Harder, 1990; Stark and Mesri, 1992), along with either Newmark-type simplified displacement analyses (Newmark, 1965; Franklin and Chang, 1977; Makdisi and Seed, 1978; Yegian et al., 1991) or more complex deformation analysis approaches.

Evaluating Potential for Flow Slides. Flow generally occurs in liquefied materials found on steeper slopes and may involve ground movements of hundreds of feet or more. As a result, flow slides can be the most catastrophic of the liquefaction-related ground-failure phenomena. Fortunately, flow slides occur much less commonly than lateral spreads. Whereas lateral spreading requires earthquake inertia forces to create instability for movement to occur, flow movements occur when the gravitational forces acting on a ground slope exceed the strength of the liquefied materials within the slope. The potential for flow sliding can be assessed by carrying out static slope stability analyses using undrained residual strengths for the liquefied materials.

Evaluating Potential for Bearing Capacity Failure. The occurrence of liquefaction in soils supporting foundations can result in bearing capacity failures and large plunging-type settlements. In fact, the buildup of pore water pressures in a soil to less than a complete liquefaction condition will still reduce soil strength and may threaten bearing capacity if the strength is reduced sufficiently. Figure C4-7 illustrates how excess pore water pressures relate to the factor of safety against liquefaction, where the factor of safety is the stress ratio required to cause liquefaction (for example, from Figure C4-4) divided by the stress ratio induced in the soils by the earthquake ground shaking. If the factor of safety is less than about 1.5, excess pore pressure development may become significant. The amount of excess pore water pressure development may be evaluated using data such as shown in Figure C4-7.

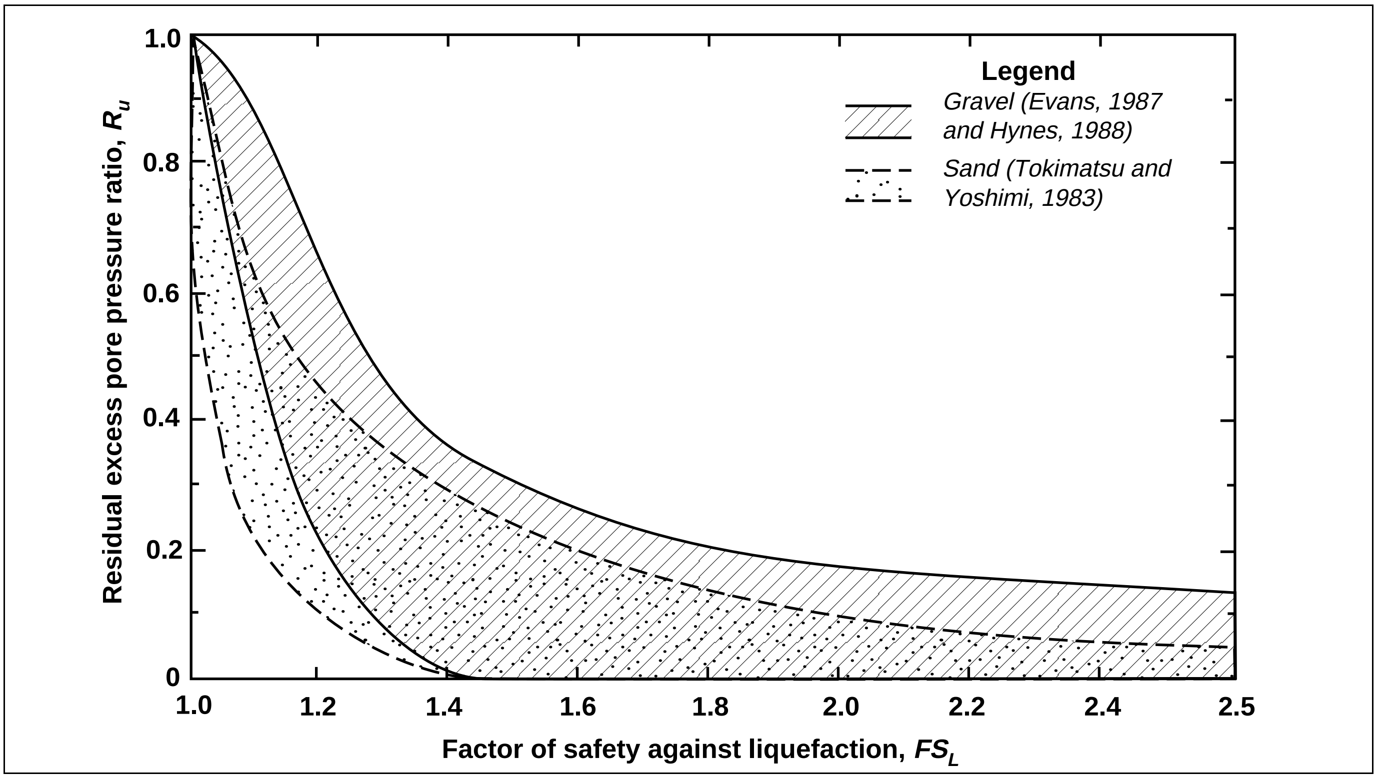

Figure C4-7 Typical Relationships for Sand and Gravel (from Marcuson and Hynes, 1990)

The potential for bearing capacity failure beneath a spread footing depends on the depth of the liquefied (or partially liquefied) layer below the footing, the size of the footing, and the load. If lightly-loaded small footings are located sufficiently above the depth of liquefied materials, bearing capacity failure may not occur. The foundation bearing capacity for a case where a footing is located some distance above a liquefied layer can be assessed by evaluating the strength of the liquefied (excess pore pressure ratio = 1.0), partially liquefied (excess pore pressure ratio <1.0, Figure C4-7), and nonliquefied strata, then applying bearing capacity formulations for layered systems (Meyerhof, 1974; Hanna and Meyerhof, 1980; Hanna, 1981). The capacity of friction pile or pier foundations can be similarly assessed, based on the strengths of the liquefied, partially liquefied, and nonliquefied strata penetrated by the foundations.

Evaluating Potential for Liquefaction-Induced Settlements. Following the occurrence of liquefaction, over time the excess pore water pressures built up in the soil will dissipate, drainage will occur, and the soil will densify, manifesting at the ground surface as settlement. Differential settlements occur due to lateral variations in soil stratigraphy and density. Typically, such settlements are much smaller and tend to be more uniform than those due to bearing capacity failure. They may range from a few inches to a few feet at the most where thick, loose soil deposits liquefy.

One approach to estimating the magnitude of such ground settlement, analogous to the Seed-Idriss simplified empirical procedure for liquefaction potential evaluation (i.e., using SPT blow count data and cyclic stress ratio), has been presented by Tokimatsu and Seed (1987); the relationships they presented are shown on Figure C4-8. Relationships presented by Ishihara and Yoshimine (1992) are also available for assessing settlement.

Evaluating Increased Lateral Earth Pressures on Retaining Walls. Behind a retaining wall, the buildup of pore water pressures during the liquefaction process increases the pressure on the wall. This pressure is a static pressure, which reduces with time after the earthquake as pore pressures dissipate. The increased lateral pressures due to either partial or complete liquefaction of the backfill are readily calculated using conventional static earth pressure formulations. For the case of complete liquefaction, the total earth pressures are those of a fluid having a unit weight equal to the total unit weight of the soil.

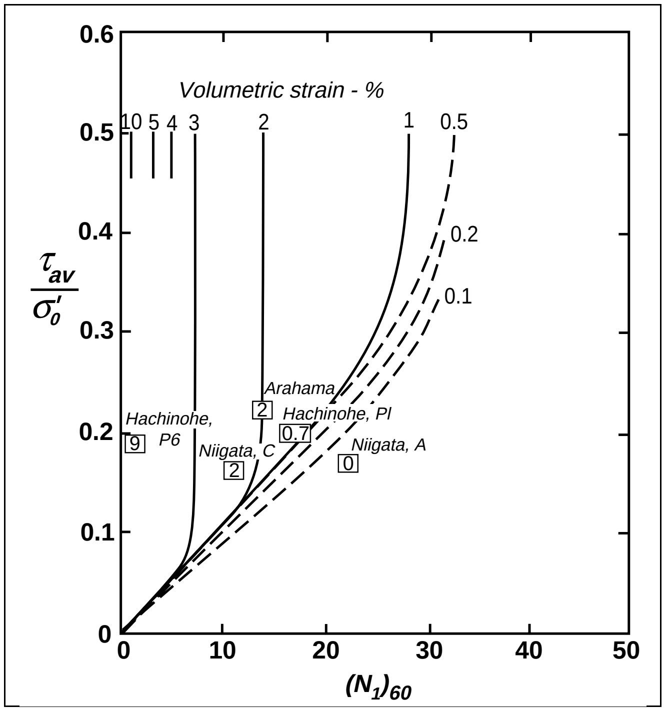

Figure C4-8 ***Relationship among Cyclic Stress Ratio, $(N_1)_{60}$, and Volumetric Strain for Saturated Clean Sands (from Tokimatsu and Seed, 1987)***

Evaluating Potential for Flotation of Buried Structures. A common phenomenon accompanying liquefaction is the flotation of tanks or structures that are embedded in liquefied soil. A building with a basement surrounded by liquefied soil can be susceptible to either flotation or bearing capacity failure, depending on the building weight and the structural continuity (i.e., whether the basement acts as an integral unit). The potential for flotation of a buried or embedded structure can be evaluated by comparing the total weight of the buried or embedded structure with the increased uplift forces occurring due to the buildup of liquefaction-induced pore water pressures.

C4.2.2.3 Differential Compaction

A procedure to evaluate settlement associated with post-liquefaction densification of soils below the water table was just discussed in Section C4.2.2.2. Loose cohesionless soils above the water table will also tend to densify during the period of earthquake ground shaking, as the earthquake-induced shear strains cause the soil particles to shift into a denser state of packing. Procedures described by Tokimatsu and Seed (1987) may be used to estimate settlements of cohesionless soils above the groundwater table. The simplified procedures they described may be used to estimate the shear strains induced by the ground shaking. The graph in Figure C4-9, which is based on laboratory unidirectional cyclic tests, may then be used to estimate volumetric strains (percent settlements) as a function of the induced shear strains and the normalized SPT blow counts of the soils. The graph in Figure C4-9 is for 15 cycles of shaking corresponding to a magnitude 7.5 earthquake; Tokimatsu and Seed provide scaling factors for other magnitude earthquakes. The graph is also for one horizontal component of ground motion. As described by Tokimatsu and Seed (1987), research by Pyke et al. (1975) indicated that volumetric strains due to multidirectional shaking are about twice those due to unidirectional shaking. Therefore, the settlement obtained using Figure C4-9 should be doubled to estimate field settlements.

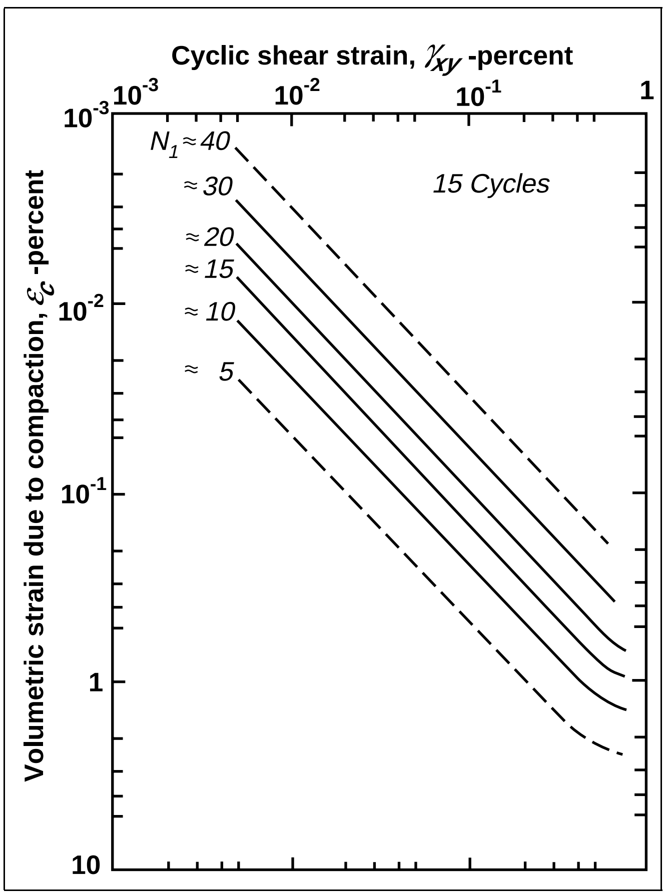

Figure C4-9 ***Correlation for Volumetric Strain, Shear Strains, and $(N_1)_{60}$ (from Tokimatsu and Seed, 1987)***

Situations most susceptible to differential compaction include heavily graded areas where deep fills have been placed to create building sites for development. If the fills are not well compacted, they may be susceptible to significant settlements, and differential settlements may occur above variable depths of fill placed in canyons and near the transitions of cut and filled areas.

C4.2.2.4 Landsliding

Earthquake-induced landslides represent a significant hazard to the seismic performance of facilities located on steep slopes in marginally stable areas. Landslides may affect a structure by directly undermining a facility, resulting in structural damage. Alternatively, off-site landslides could develop above a structure, and the debris from the landslide (avalanche, rock fall, or debris torrent) could impinge upon a structure and lead to undesirable performance. Thus, consideration of landslide effects should include both on-site and off-site sources. Sites that are more likely to be affected by earthquake-induced landslides include locations with slopes of 18 degrees or greater, or a history of rock falls, avalanches, or debris torrents.

Stability analysis shall be performed for all sites located on slopes steeper than three horizontal to one vertical (approximately 18 degrees), and the stability analysis should consider the following factors:

- Slope geometry
  - slope inclination
  - slope height
- Subsurface conditions
  - stratigraphy (material type and bedding)
  - material properties (unit weight, friction angle, and cohesion)
  - groundwater conditions (level, perched locations, and hydrostatic pressures)
- Level of ground shaking

Pseudo static analyses may be used to evaluate landsliding potential. Such analyses should be used only in instances where liquefaction would not develop and where the underlying materials would not suffer major strength degradation as a result of earthquake ground shaking (i.e., soft, sensitive clays). The analyses should be conducted using a seismic coefficient equal to one-half the peak ground acceleration for the site area. A safety factor of at least 1.0 should be obtained. The pseudo-static analysis is conservative because it is performed with a continuously applied horizontal force acting in the downhill direction. A static factor of safety of 1.0 is considered acceptable for this type of analysis. Safety factors of 1.5 are appropriate for static vertical load conditions, which the slopes must meet independently.

If the results from the pseudo-static analyses indicate a safety factor of less than 1.0, sliding block analyses such as Newmark's (1965) method should be conducted. The Newmark analyses may consider the potential effects of both on-site and off-site stability. The advantage of the Newmark procedure is that it provides an evaluation of the permanent ground deformation that may occur as a result of earthquake ground shaking. This evaluation of deformation may be used in developing structural strengthening to withstand this level of deformation (see Sections 4.3 and 4.6).

Earthquake-induced rock fall hazards exist only if a cliff or steep slope with blocks available to fall is located in close proximity upslope from the building site. Where this is the case, blocks of rock often fall from such cliffs or slopes without earthquake shaking, and boulders (often used for landscaping) commonly are present on the site and in the immediate vicinity. Falling rock starts from an at-rest condition, achieves a maximum velocity, and comes to rest again. Blocks of rock that have come to rest beyond the site indicate that such rocks had kinetic energy as they passed over the building site. The amount of energy at the building site can be estimated with the aid of the Colorado Rock Fall Simulation Program (Pfeiffer and Higgins, 1991).

If no blocks of rock are present at the site, but a cliff or steep slope is located nearby, then the likely performance of the cliff under earthquake loading should be evaluated. The earthquake loading condition for cliff performance must be compatible with the earthquake loading condition selected for the Rehabilitation Objective for the building.

Some sites may be exposed to hazards from major landslides moving onto the site from upslope, or retrogressive removal of support from downslope. Such conditions should be identified during site characterization, and may pose special challenges if adequate investigation requires access to adjacent property.

C4.2.2.5 Flooding or Inundation

Flooding hazards originating off-site may adversely affect a building being considered for seismic rehabilitation. Tsunami and seiche can be triggered by earthquakes, causing wave impact and inundation damage at building sites located near shorelines. Failure of reservoirs, aqueducts, and canals upslope from building sites can cause site flooding.

Some buildings may be located in potential flood paths in the event that a dam or pipeline fails during an earthquake. Individual states are responsible for dam safety inspections, and specific information should be available for all high-hazard dams. Pipeline rupture and resulting flood or severe erosion typically has not been addressed. Given the cost of rehabilitation, it may be prudent to consider the consequences of such hazards under earthquake loading compatible with the desired Performance Level for the building.

In low-lying coastal areas, tsunami or seiche processes can be significant for buildings meeting Life Safety or Immediate Occupancy Performance Levels. Historical records of wave run-up should be reviewed, or coastal engineering evaluations of potential wave run-up should be performed as a guide. The return period of the tsunami or seiche should be the same as the earthquake ground motion that serves as the basis for building rehabilitation.

C4.3 Mitigation of Seismic Site Hazards

C4.3.1 Fault Rupture

No commentary is provided for this section.

C4.3.2 Liquefaction

Figure C4-10 illustrates conceptual schemes to mitigate the hazard of liquefaction-induced bearing capacity reduction or settlements due to liquefaction-induced soil densification beneath a building. As stated in the *Guidelines*, the schemes fall into three different categories—modify either the structure, the foundation, or the soil conditions. Figure C4-11 illustrates conceptual schemes to resist liquefaction-induced lateral spreading. The soil may be stabilized beneath the building and, if needed, sufficiently beyond the buildings that liquefaction and spreading of the surrounding areas will not cause significant spreading beneath the building, as illustrated by the stabilized "soil island" concept in Figure C4-11A. Alternatively, a buttress of stabilized ground can be constructed beyond the building to prevent significant lateral spreading behind the buttress, as illustrated in Figure C4-11B. The buttress approach does not prevent settlement from occurring beneath the building, but if bearing capacity failures are not expected (due to lightly loaded footings a sufficient distance above the liquefied zone) and densification settlements are tolerable for the structure (considering the Rehabilitation Objective), then the buttressing approach, by eliminating potentially large spreading-type movements beneath the structure, may be effective.

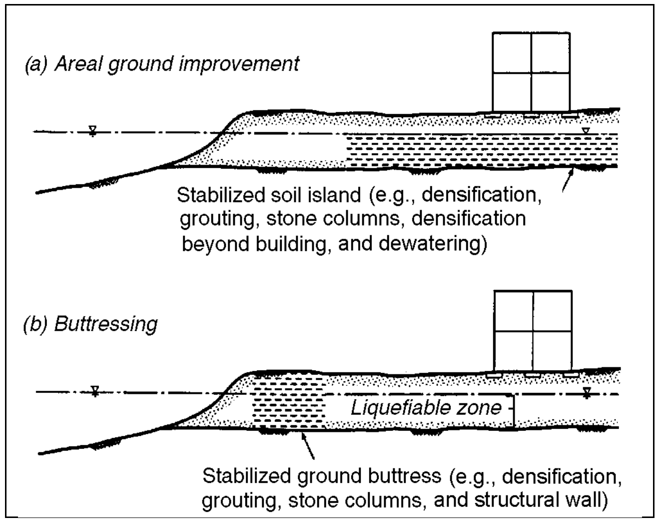

Figure C4-11 Conceptual Schemes to Resist Liquefaction-Induced Lateral Spreading

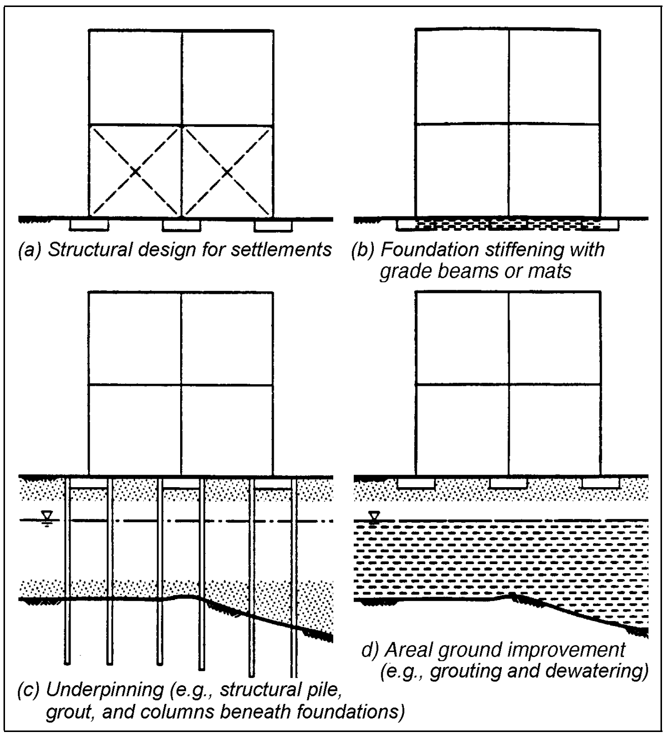

Figure C4-10 Conceptual Schemes to Resist Liquefaction-Induced Settlement or Bearing Capacity Reductions

Ground improvement techniques that can be considered to be used beneath an existing structure include soil grouting, installation of drains, and installation of permanent dewatering systems. In general, ground modification techniques that involve vibratory densification of soils to reduce their liquefaction potential (e.g., vibrocompaction or vibroreplacement) cannot be implemented beneath existing buildings because of the settlements induced during the process.

Different types of grouting are illustrated schematically in Figure C4-12. Compaction grouting, permeation grouting, and jet grouting may have application for mitigation of liquefaction hazard beneath an existing building.

Compaction grouting involves pumping a mixture of soil, cement, and water into the ground to form bulbs of grouted material. The formation of these bulbs compresses and densifies the surrounding soil and increases the lateral earth stresses, thus reducing its liquefaction potential. Effects may be somewhat nonuniform, depending on the spatial pattern of grout bulb formation. The amount of densification that can be achieved may be limited because static compression is less effective than vibration in densifying sands. Compaction grouting must be done carefully to avoid creating unacceptable heaving or lateral displacements during the grouting process.

Permeation grouting involves injecting chemical grout into liquefiable sands to essentially replace the pore water and create a nonliquefiable solid material in the grouted zone. The more fine-grained and silty the sands, the less effective is permeation grouting. If soils are suitable for permeation grouting, this technique can potentially eliminate liquefaction potential.

Jet grouting is a technique in which high-velocity jets cut and mix a stabilizing material such as cement into the soil.

In addition to their use to stabilize entire volumes of soil beneath a building, these grouting techniques can also be used locally beneath individual footings to form stabilized columns of soil, which will transfer vertical foundation loads to deeper nonliquefiable strata.

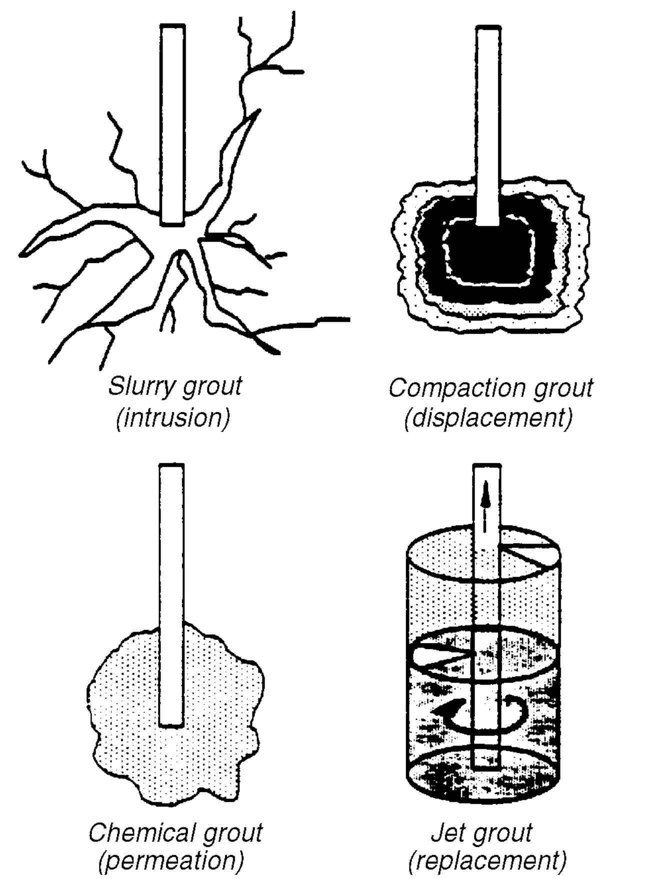

Figure C4-12 Schematic Diagram of Types of Grouting (from notes taken during a 1989 GKN Hayward Baker, Inc., Ground Modification Seminar)

Drain installation (e.g., stone or gravel columns) involves creating closely spaced vertical columns of permeable material in the liquefiable soil strata. Their purpose is to dissipate soil pore water pressures as they build up during the earthquake shaking, thus preventing liquefaction from occurring.

Permanent dewatering systems lower groundwater levels below liquefiable soil strata, thus preventing liquefaction. Because lowering the water table increases the effective stresses in the soil, the potential for causing consolidation in any underlying compressible soil deposits should be evaluated when considering permanent dewatering systems. The dewatering process may also cause settlements in the liquefiable deposits, although in sands these would tend to be small. This alternative also involves an ongoing cost for operating the dewatering system.

Ground stabilization methodologies are discussed in a number of publications, including Mitchell (1981), Ledbetter (1985), National Research Council (1985), Mitchell et al. (1990), and Mitchell (1991). Additional information on these techniques is also available from contractors who specialize in ground modification.

C4.3.3 Differential Compaction

The conceptual mitigation schemes and techniques discussed in Section C4.3.2 can be considered for mitigating the hazard of differential compaction caused by either liquefaction or densification of loose soils above the water table.

C4.3.4 Landslide

The stability of hillside slopes may be improved using a variety of schemes. These range from grading, drainage, buttressing, and soil improvement to structural schemes—retaining walls (gravity, tieback, soil nail, mechanically stabilized earth), barriers, and building options such as grade beams and shear walls. Selection of an appropriate remediation scheme depends on the desired Performance Level for the facility, the size of the potential landslide, and the costs and consequences associated with the earthquake-induced ground movement. Mitigation schemes should be evaluated for acceptable performance using both pseudo-static and dynamic analysis techniques.

C4.3.5 Flooding or Inundation

No commentary is provided for this section.

C4.4 Foundation Strength and Stiffness

The *Guidelines* utilize a stiffness and ultimate capacity approach to evaluating the adequacy of foundations and structures to withstand the imposed static plus seismic loads. In general, soils have considerable ductility unless they degrade significantly in stiffness and strength under cyclic action or large deformations. Degrading soils include cohesionless soils that are predicted to liquefy or build up large pore pressures, and sensitive clays that may lose considerable strength when subject to large strains. Soils not subject to significant degradation will continue to mobilize load, but with increasing deformations after reaching ultimate soil capacity.

The amount of acceptable deformations for foundations in such soils depends primarily on the effect of the deformation on the structure, which in turn depends on the desired Structural Performance Level. However, it should be recognized that foundation yield associated with mobilization at ultimate capacity during earthquake loading may be accompanied by progressive permanent foundation settlement during continued cyclic loading, albeit in most cases this settlement probably would be less than a few inches. In general, if the real loads transmitted to the foundation during earthquake loading do not exceed ultimate soil capacities, it can be assumed that foundation deformations will be relatively small.

If calculated foundation loads exceed twice (m = 2.0) the ultimate foundation capacities, two alternatives for evaluating the effects on structural behavior are presented. One alternative is to perform the NSP or NDP, because the nonlinear load-deformation characteristics of the foundations can be directly incorporated in these analyses (Section 4.4.2). Parametric analyses to cover uncertainties in the load-deformation characteristics are recommended. In the static analysis, a somewhat conservative interpretation of the results is recommended because cyclic loading effects cannot be directly incorporated.

For the alternative of a linear procedure using linear foundation springs, wide parametric variations in spring stiffnesses are recommended because of additional uncertainties associated with the linearization of the foundation behavior. This approach is not recommended for the Immediate Occupancy Performance Level.

One of the major changes in traditional seismic design procedures in the *Guidelines* is the direct inclusion of geotechnical and foundation material properties in the Analysis Procedures. In order to accomplish this improvement, the engineer must quantify foundation capacity, stiffness, and displacement characteristics. Considering the multitude of foundation types and soils materials that may be encountered, the authors have concentrated on techniques that may be adapted by qualified experts to generate information for specific projects. For example, a classical general expression for soil bearing capacity is:

$$Q_c = cN_c\zeta_c + \gamma DN_q\zeta_q + \frac{1}{2}\gamma BN_\gamma\zeta_\gamma \qquad \text{(C4-2)}$$

where

c	=	Cohesion property of the soil
N_c	=	Cohesion bearing capacity (see Figure C4-13)
N_q	=	Surcharge bearing capacity factor (see Figure C4-13)
N_γ	=	Density bearing capacity factor (see Figure C4-13)
ζ_c, ζ_q, ζ_γ	=	Footing shape factors (see Table C4-1)
γ	=	Soil density
D	=	Depth of footing
B	=	Width of footing

Table C4-1 Shape Factors for Shallow Foundations (after Vesic, 1975)

Shape of the Base	ζ_c	ζ_q	ζ_γ
Strip	1.00	1.00	1.00
Rectangle	$1 + \frac{B}{L}\frac{N_q}{N_c}$	$1 + \frac{B}{L}\tan\phi$	$1 - 0.4\frac{B}{L}$
Circle and Square	$1 + \frac{N_q}{N_c}$	$1 + \tan\phi$	0.60

For a rehabilitation project, normally some information on footing size and depth might be available; but rarely are the soil properties required for the above calculation readily available. The *Guidelines* allow the calculation of bearing capacity by a qualified geotechnical engineer or the use of conservative presumptive or prescriptive values.

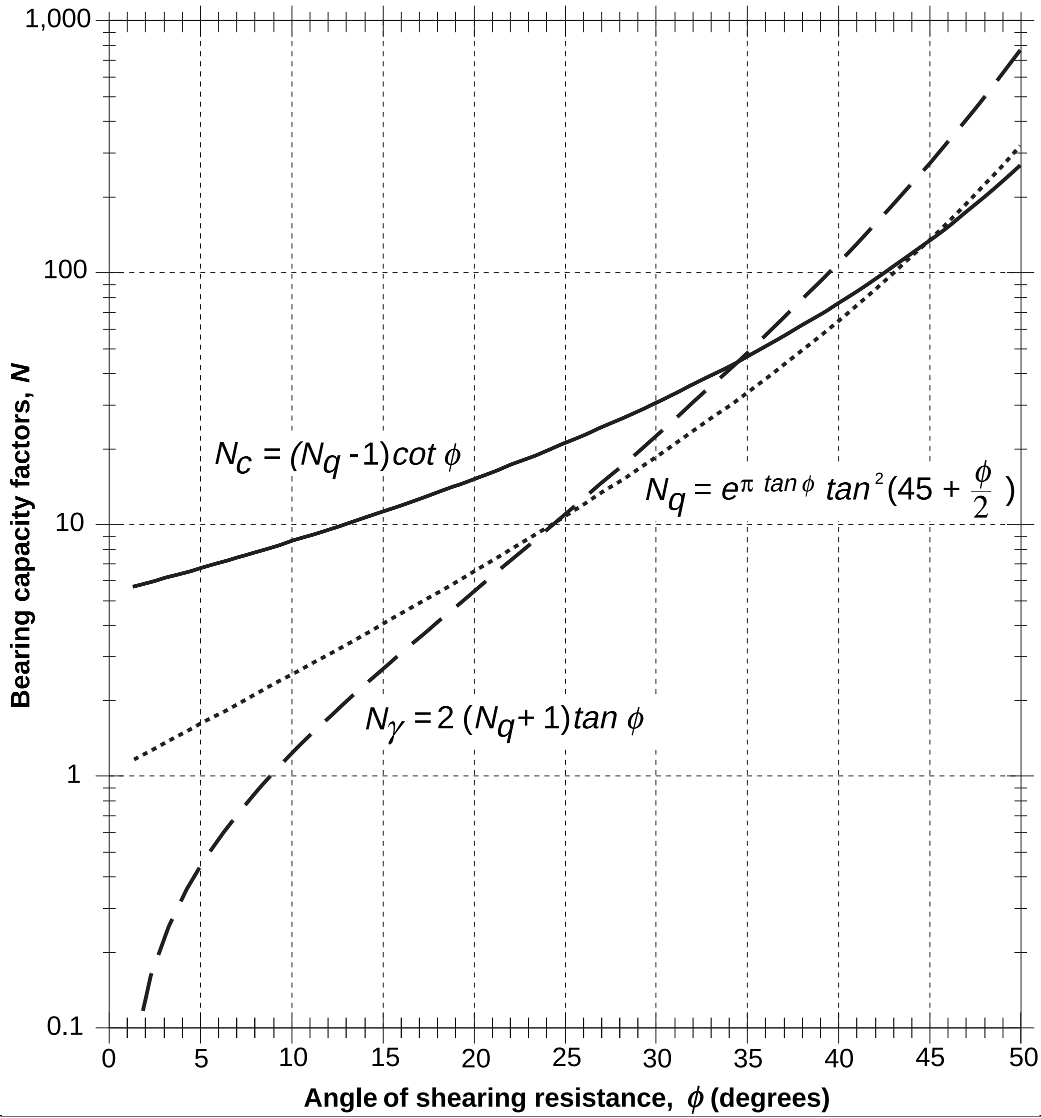

Figure C4-13 Bearing Capacity Factors (calculated from Vesic, 1975)

C4.4.1 Ultimate Bearing Capacities and Load Capacities

Presumptive and prescriptive procedures may be used to determine ultimate load capacities (Q_c) of structures that are located in areas of low seismicity and that are underlain by stable soil conditions (i.e., where a fault rupture, landsliding, and liquefaction are not anticipated). Presumptive ultimate bearing capacities for different foundation soils are provided in Table 4-2. Information developed for Table 4-2 was derived from the *Uniform Building Code* (UBC) and the allowable design values from the UBC were doubled to establish the ultimate bearing pressures for the *Guidelines*. This increase is based upon conventional geotechnical practice, which typically includes a factor of safety of two or more for spread footing foundations.

Alternatively, the ultimate load capacity may be assumed to be equal to 200% or 150% of the dead load, live load, and snow load (that were used for working stress design of the building) acting on a shallow or deep foundation, respectively. The increased uncertainty associated with deep foundations warrants the more conservative factor for these components. Performance of structures during past earthquakes has typically indicated that this empirical rule has provided adequate foundation performance without excessive occurrences of foundation failures, provided that the underlying soils remain stable (i.e., no fault rupture, liquefaction, or landslides).

Site-specific investigation by a qualified geotechnical engineer is the preferred method of determining foundation capacities, particularly for complex analyses.

C4.4.2 Load-Deformation Characteristics for Foundations

C4.4.2.1 Shallow Bearing Foundations

The lateral stiffness and capacity of footings arise from three components, as shown in Figure C4-14. The elastic stiffness solutions shown in Figure 4-2 arise from base contact only, whereas Figure 4-4 provides an elastic stiffness solution generated from passive resistance on the vertical face of the footing. The latter solution (after Wilson, 1988) was derived for bridge abutments, where the soil surface is level with the top of the wall. For buried footings, some judgment is needed in assessing an "equivalent" footing height. For practical purposes, where lateral loads approach the passive pressure, it may be reasonable to assume that the lateral displacement required to mobilize passive pressure is approximately 2% of an "equivalent" footing height (assuming the soil surrounding the footing is dense or stiff). Displacements of approximately 2% to 4% would be more appropriate for softer soils (Clough and Duncan, 1991).

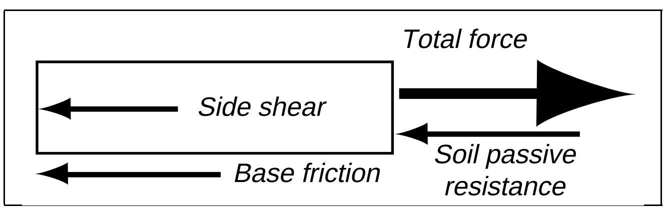

Figure C4-14 Footing Lateral Stiffness and Capacity Components

The determination of displacement as a function of load for a footing is complex (see Figure C4-15). Upon initial loading, the example footing may be relatively stiff as shearing strains are low, or alternatively, until a preconsolidation pressure due to previous overburden or drying (shrinkage) might be reached. At larger deformations, the material may soften progressively until a capacity plateau is reached. If the footing is unloaded, the rebound is usually not complete and permanent displacement occurs. For repeated cyclic loading the permanent displacement can accumulate. When reloaded, the footing can be substantially stiffer than for previous cycles. This information needs to be simplified and generalized for use in a structural analysis model. For this purpose the *Guidelines* promote a strength and stiffness envelope, shown here in Figure C4-15. The lower bound reflects the initial material properties during the first cycle of loading; the upper bound represents the effects of repeated loading. This allows the structural engineer to investigate the sensitivity of the analysis to the soils parameters. It may be that the stiff-strong assumption will give critical results for some structural elements while the flexible-weak will more adversely affect others.

The objective of the force-displacement relationships is to allow the structural engineer to incorporate the foundation characteristics into an analysis model. Consider the spread footing shown in Figure C4-16 with an applied vertical load (P), lateral load (H), and moment (M). The soil characteristics might be modeled as two translational springs and a rotational spring. More common, however, is the use of a Winkler spring model acting in conjunction with foundation structure to eliminate the rotational spring. The conversion to Winkler springs requires the consideration that rotational stiffness may differ substantially from vertical stiffness. Useful discussions of the concepts of rigid and flexible footing behavior are provided by Scott (1981) and Bowles (1982). Note that the values of Winkler or subgrade stiffness coefficients often tabulated in geotechnical textbooks reflect first loading values. Stiffness coefficients for unloading and reloading reflecting cyclic loading conditions can range from about two to five times stiffer, depending on the original density or stiffness of the soil.

A problem frequently encountered in seismic rehabilitation is the analysis of a shear wall or braced frame supported on spread footings. The relationship of the vertical load, overturning moment, and soil properties, and their effect on stiffness and energy dissipation was thoroughly studied by Bartlett (1976). Figure C4-17 illustrates the relationship between overturning moment and base rotation for a wall that is allowed to uplift and/or accommodate compression yielding in the supporting soil medium. This rocking behavior has several important effects on the seismic response of the structure. First of all, rocking results in a decrease in stiffness and lengthening of the fundamental period of the structure. This effect is amplitude-

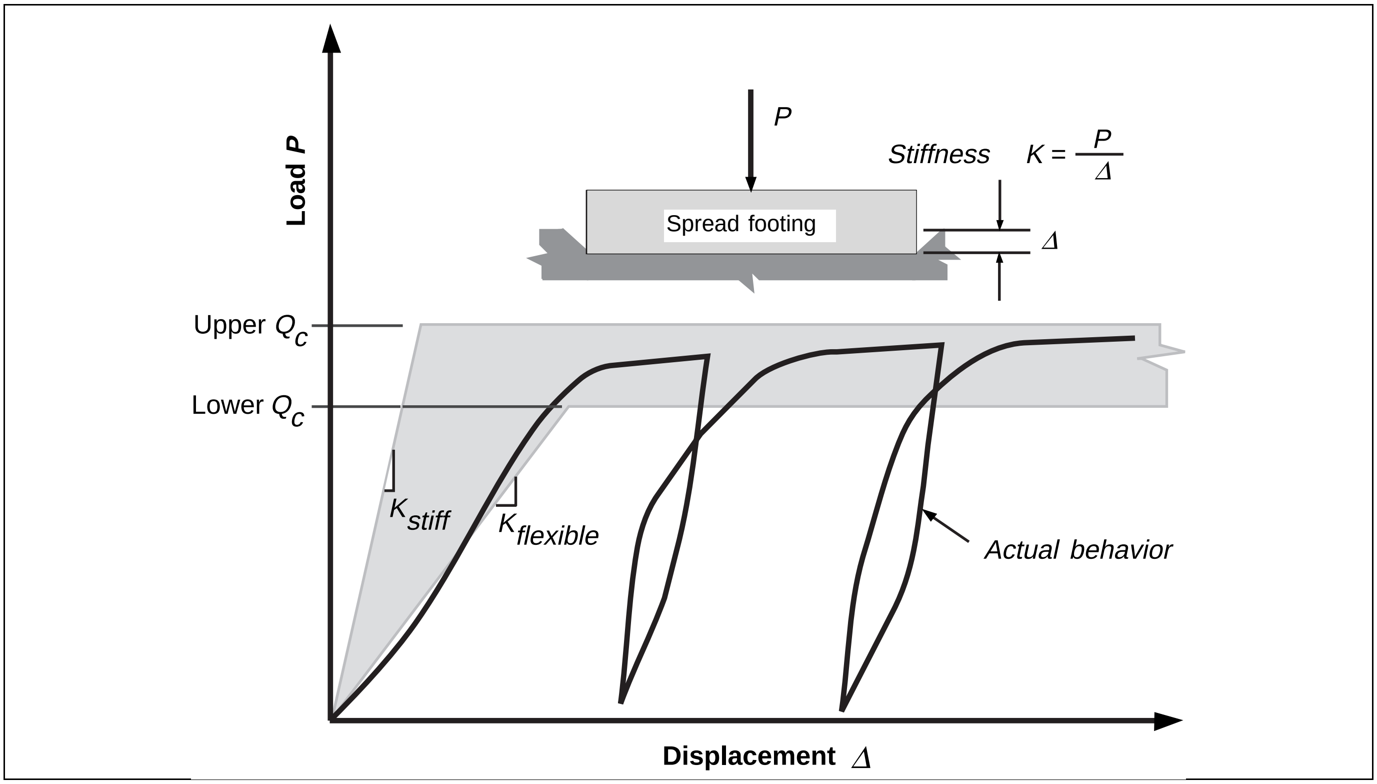

Figure C4-15 Load-Displacement Relationship for Spread Footing

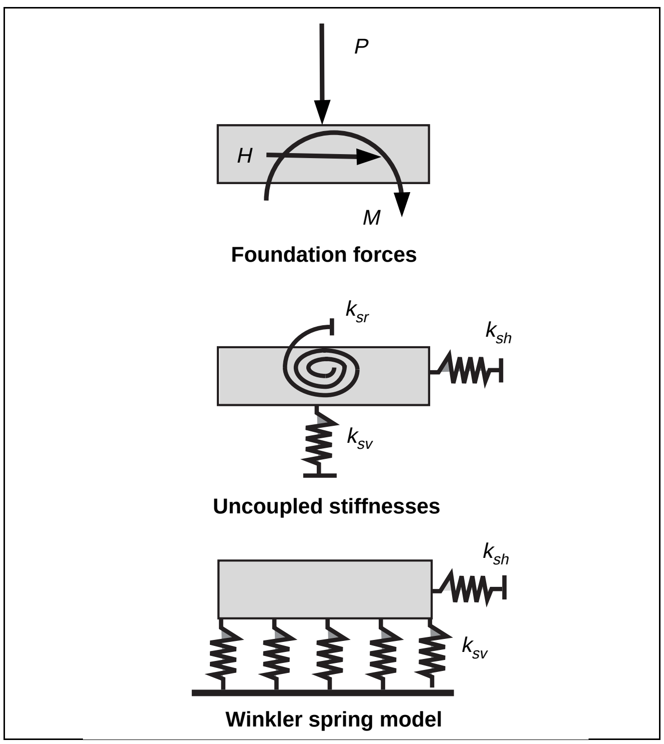

Figure C4-16 Analytical Models for Spread Footing

dependent and therefore highly nonlinear. The result is generally a reduction in the maximum seismic response. Depending on the ratio of initial bearing pressure to the ultimate capacity of the soil, significant amounts of energy may be dissipated by soil yielding. This behavior also can result in increased displacement response of the superstructure and permanent foundation displacements.

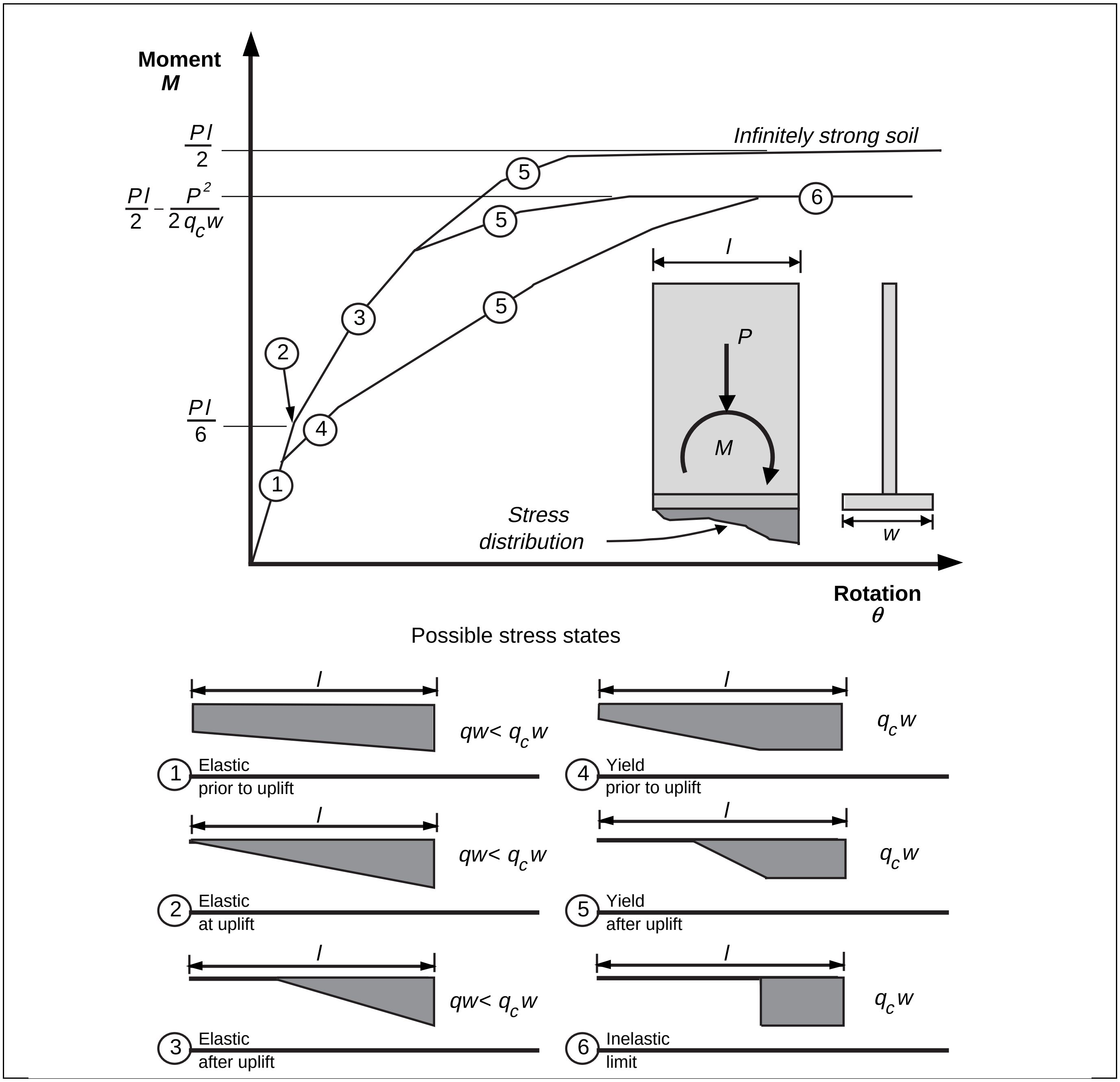

Figure C4-17 Rocking of Shear Wall on Strip Footing

A. Shear Wall and Frame Example

This example illustrates the effects of foundation flexibility on the results of analysis of an eight-story concrete shear wall and frame building, shown in Figure C4-18. The results of an LSP for this structure for both a fixed base and flexible base are summarized below:

Seismicity

Spectral response acceleration at short periods, $S_{XS} = 1.1$

Spectral response acceleration at one second, $S_{X1} = 0.75$

Soil properties

Soil unit weight, $\gamma = 110$ pcf

Shear wave velocity, $v_s = 1100$ ft/sec

Poisson's ratio, $\nu = 0.35$

Initial shear modulus, $G_o = \dfrac{\gamma v_s^2}{g} = 4097$ ksf

Effective shear modulus, $G = 0.35\ G_o = 1434$ ksf

(for $S_{XS}/2.5 = 0.40$ from Table 4-3)

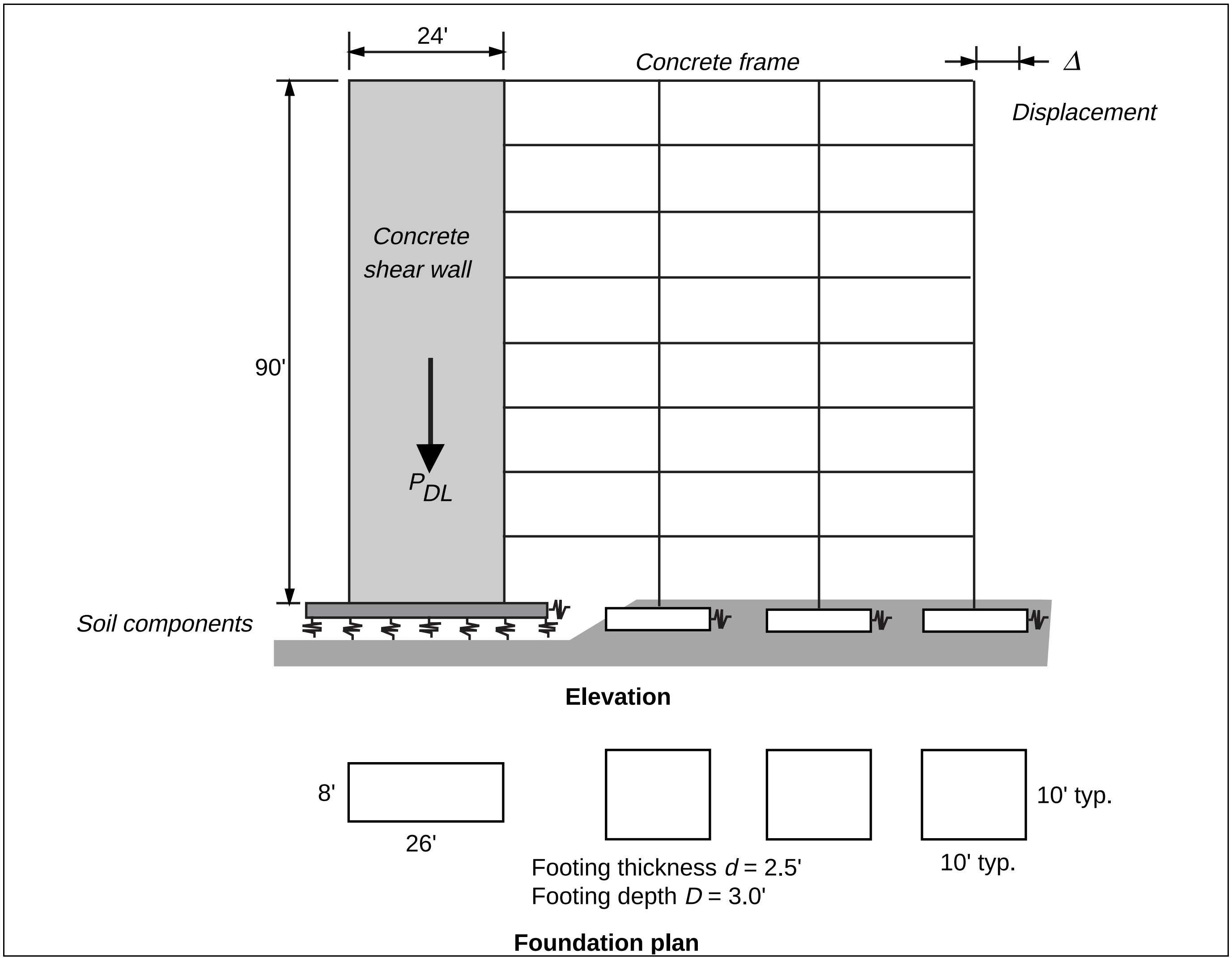

Figure C4-18 Shear Wall and Frame Example

Ultimate bearing capacity, q_c = 12 ksf

Upper bound q_c = 2(12)=24 ksf

Dead load bearing stress available to resist seismic overturning:

q = 5.85 ksf for P_{DL} = 1360 k and Q_G = 0.9 P_{DL}

Modification factors

$C_1 = C_2 = C_3 = 1.0$

Flexible Foundation Properties

Foundation stiffnesses, in accordance with Gazetas (1991), are:

Lateral stiffness, K_y = 4 footings x K_{yi} = 219,638 k/ft

Rotational stiffness, K_q = shear wall only = 13,155,000 ft-k/rad

Using the SSI procedures from BSSC (1995) (note that the equation for flexible base period presented there contains an error; the equation below is correct):

$$\text{Fixed base stiffness, } k' = 4\pi^2 \frac{W'}{gT^2} = 5229 \text{ k/ft}$$

Flexible base period:

$$T' = T\sqrt{1 + \frac{k'}{K_y}\left[1 + \frac{K_y(0.7h)^2}{K_q}\right]} = 0.93 \text{ sec}$$

	Fixed Base	**Flexible Base**
Period	0.58 sec	0.93 sec
Base shear	3246 k	2361 k
Overturning moment	194,769 k-ft.	142,368 k-ft.
Roof displacement	19.4 in.	25.9 in.

Checking the fixed base solution, in accordance with the *Guidelines*, Equation 4-11, at the base of the structure reveals that the base overturning moment from the seismic forces unacceptably exceeds twice the plastic capacity of the soil beneath the shear wall.

$$\begin{aligned} Q_C &= M_C = \frac{L}{2}Q_G\left(1 - \frac{q}{q_C}\right) \\ &= \frac{28}{2}(0.9(1360))\left(1 - \frac{5.85}{24}\right) \\ &= 12,959 \text{ k-ft} \end{aligned} \tag{C4-3}$$

For a fixed base condition, use force-controlled behavior to determine:

$$\begin{aligned} Q_{UF} &= \frac{Q_E}{C_1C_2C_3J} = \frac{194,769}{(1)(1)(1)(2)} \\ &= 97,384 > 12,959 \end{aligned} \tag{C4-4}$$

Although the flexible base overturning moment also greatly exceeds the plastic capacity of the soil, this condition is acceptable, provided that the performance of the structure is acceptable for the increased displacements associated with the rotating foundation beneath the shear wall. Of particular concern in this structure is the ability of the columns of the frame to undergo these displacements without losing vertical-load-carrying capacity. Note that the forces on the structure are reduced significantly by the flexible base assumption, in spite of the larger displacements.

Nonlinear Procedure Results. This example has also been analyzed using the NSP, including the effects of foundation uplift and soil yielding on the inelastic response (Hamburger, 1994). The nonlinear model of the structure included springs representing the stiffness and strength of the soil beneath the shear wall (Figure C4-19). These springs were preloaded with the effect of vertical loads from the structure, but uplift was allowed if the preload was overcome by rotation.

Rocking and compressional soil yielding initiate early in the response of the structure; in fact, it was found that over two-thirds of the deformation demand was absorbed in the foundation soils materials. As a consequence, the inelastic demand on the shear walls was very small, within acceptable limits for the Life Safety Performance Level for the structure as a whole. The stiffness and strength of the soil were varied by factors of 67% and 150% in an effort to test the sensitivity of the analysis results to these parameters. The behavior was not significantly affected, leading to the conclusion that the response is most sensitive to nonlinear rocking itself rather than exact soil properties.

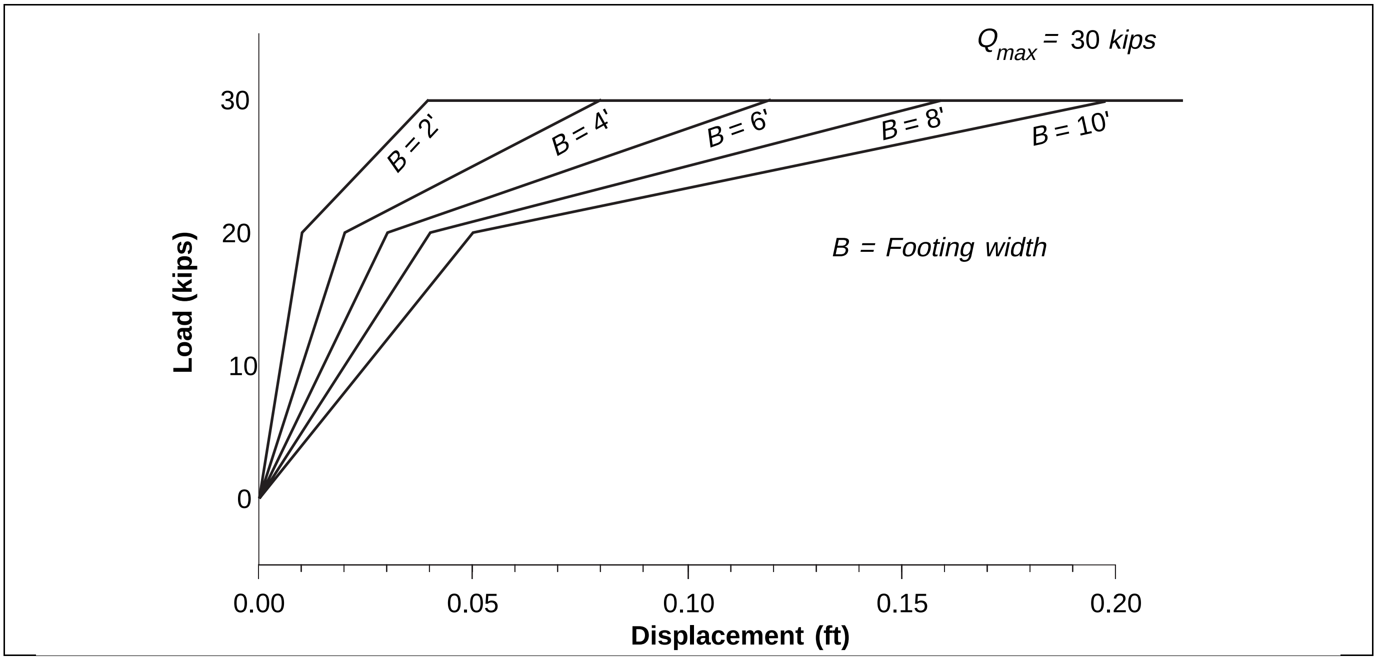

Figure C4-19 Foundation Stiffness and Strength Properties

These nonlinear analysis results have different implications for response than does the linear procedure. The foundation rocking effectively protects the shear walls from large inelastic demand. Modification to the walls and their foundations is not necessary. However, the resulting large lateral movement of the structure could cause undesirable shear failure in some of the columns of the concrete frame. This leads to the conclusion that the columns should be retrofitted to provide greater shear strength, by jacketing or other techniques to provide increased confinement. In contrast, the linear procedure might indicate that a relatively expensive retrofit of the walls and footings is warranted. Perhaps more significantly, the linear procedure with the rigid base assumption might fail to identify the potential problem with the columns.

B. Short Stout Walls on Flexible Grade Beam Example

Figure C4-20 depicts a structural model of one exterior wall of a two-story masonry building (Taner, 1994). The rehabilitation design includes the addition of reinforced concrete shear walls against the unreinforced masonry. A reinforced concrete grade beam couples the three shear wall panels at their base; the tops of the panels are linked together by a bond beam at the roof. The ultimate moment capacity, M_c, of the shear wall panels controls the lateral strength of the structure. Assuming a fixed base for the shear wall panels, displacement at the roof was tolerable at the strength limit state. The designer was concerned, however, that foundation rocking and flexibility might magnify this displacement.

The nonlinear model predicts the incremental displacement, Δ, at the roof due to the interaction of the flexible grade beam with a flexible supporting soil. The model allows unrestrained uplift of the grade beam and footing once the dead load is overcome. The spring constant, k_{sv}, for compressibility of the soil was varied in an effort to assess the sensitivity of the results to this parameter.

The results indicate that significant uplift occurs for any soil stiffness. The distribution and maximum magnitude of foundation contact pressure is highly dependent on the relative stiffness of the soil and the grade beam. The extremely flexible soil virtually allows a rigid body rotation of the structure and a very large incremental roof displacement. The more flexible soils also result in larger moments, M_{max}, in the grade beam. Fortunately, the actual soil is relatively stiff and the incremental displacement is small.

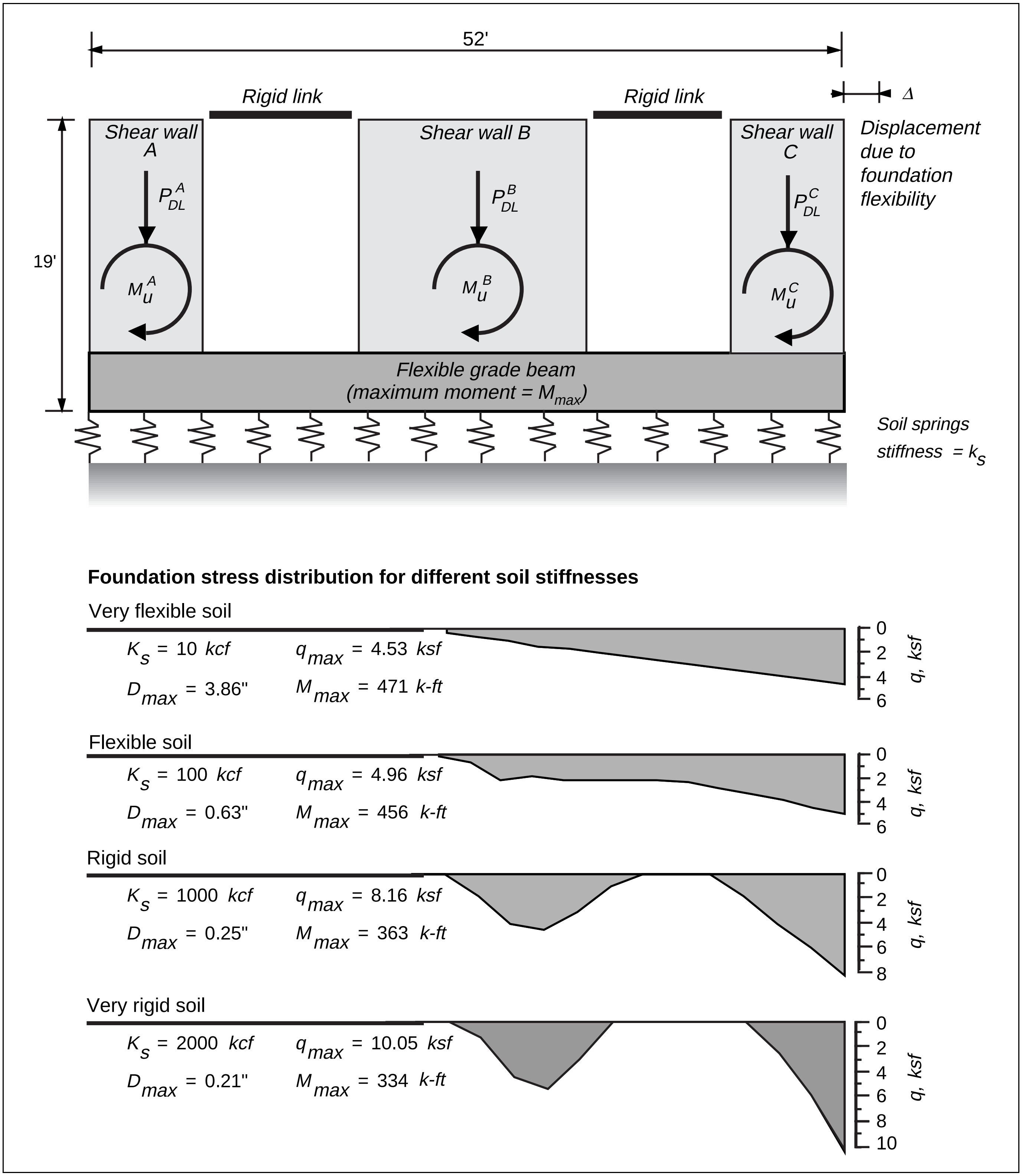

Figure C4-20 Structural Model, One Exterior Wall of Two-Story Masonry Building

C4.4.2.2 Pile Foundations

Axial Loading. Earthquake-induced axial loading of pile groups may be of significant design importance in the analysis of the seismic rocking response of rigid shear walls for buildings when subjected to lateral loading. Analyses also show that the rotational stiffness of a pile group is generally dominated by the axial stiffness of individual piles. The rotational or rocking behavior of a pile group may have a significant influence on the seismic response of a structure and could significantly influence column moments.

Although elastic solutions exist for the pile head stiffness for piles embedded in linear elastic media (Poulos and Davis, 1980; Pender, 1993), the complexities of the nonlinear load transfer mechanisms to the pile shaft and tip make the selection of an equivalent linear elastic modulus for the soil very difficult. The use of the nonlinear Winkler spring approach provides an alternate procedure that has been widely adopted in practice.

The various components of the axial pile load transfer problem are illustrated in Figure C4-21. The overall pile behavior depends on the axial pile stiffness (*AE*) and the load transfer characteristics (*t*-*z* curves) along the side of the pile and at the pile tip (tip *q*-*z* curve). The fundamental problem in an analysis of piles under axial loading relates to the uncertainties of the load transfer characteristics at the side and at the pile tip, which in turn influence the pile head load-deflection behavior. Factors that need to be considered in developing the load transfer characteristics include:

- The side-friction capacity along the length of the pile
- The ultimate resistance at the pile tip
- The form of the load transfer-deflection curves associated with each of the above forms of soil resistance

The ultimate capacity of a pile depends on numerous factors, including:

- The soil conditions and pile type
- The geologic history of the site
- The pile installation methods

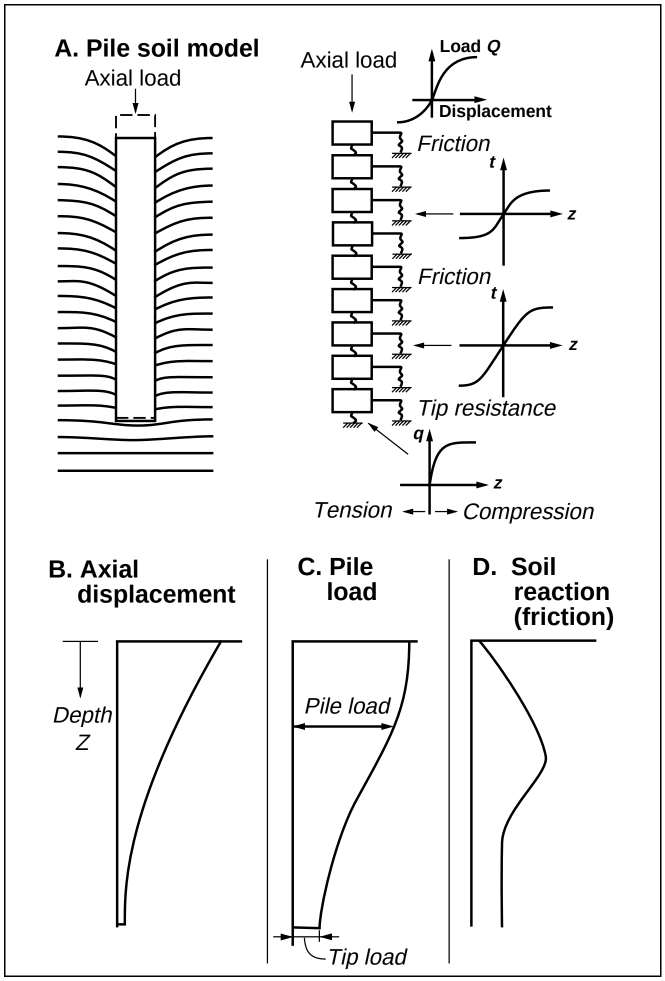

Figure C4-21 Schematic Representation of Axial Pile Loading (Matlock and Lam, 1980)

Numerous methods have been proposed to predict the axial capacity of piles, and can lead to widely varying capacity estimates, as documented in Finno (1989). Incorporation of site-specific pile load test data has been perceived to be the most reliable method for pile capacity determination.

In addition to the ultimate side friction and end-bearing capacity, some assumptions need to be made to develop the load transfer-displacement relationships (for both side friction and end bearing) to evaluate the overall pile behavior. The form of the load transfer-displacement relationship is complex, and there is no uniform agreement on the subject.

A computer approach provides the most convenient means of solving axial pile behavior. Many of the well-established computer programs, such as BMCOL 76 and PILSET (Olsen, 1985), allow for prescription of the *t-z* curves at various depths along the length of the pile (e.g., at the boundaries of each soil layer) and will automatically perform interpolations to develop support curves at all the pile stations. The *t-z* curves for side friction usually are assumed to be symmetrical, and the *q-z* curve at the pile tip usually is assumed to be nonsymmetric.

Uncertainty in axial soil-pile interaction analysis relates largely to uncertainties in soil parameters, including the ultimate pile capacity (skin-friction and end-bearing) and load-displacement relationships. Computers can be used for rigorous nonlinear solutions. However, an approximate nonlinear graphical solution method has been presented by Lam and Martin (1984, 1986). The procedure is shown schematically in Figure C4-22 (for a 70-foot-long, 1-foot-diameter pipe pile embedded in sand, $\phi = 30^{o}$) and involves the following steps:

1. **Soil Load-Displacement Relationships.** Side-friction and end-bearing load-displacement curves are constructed for a given pile capacity scenario (accumulated skin-friction and ultimate tip

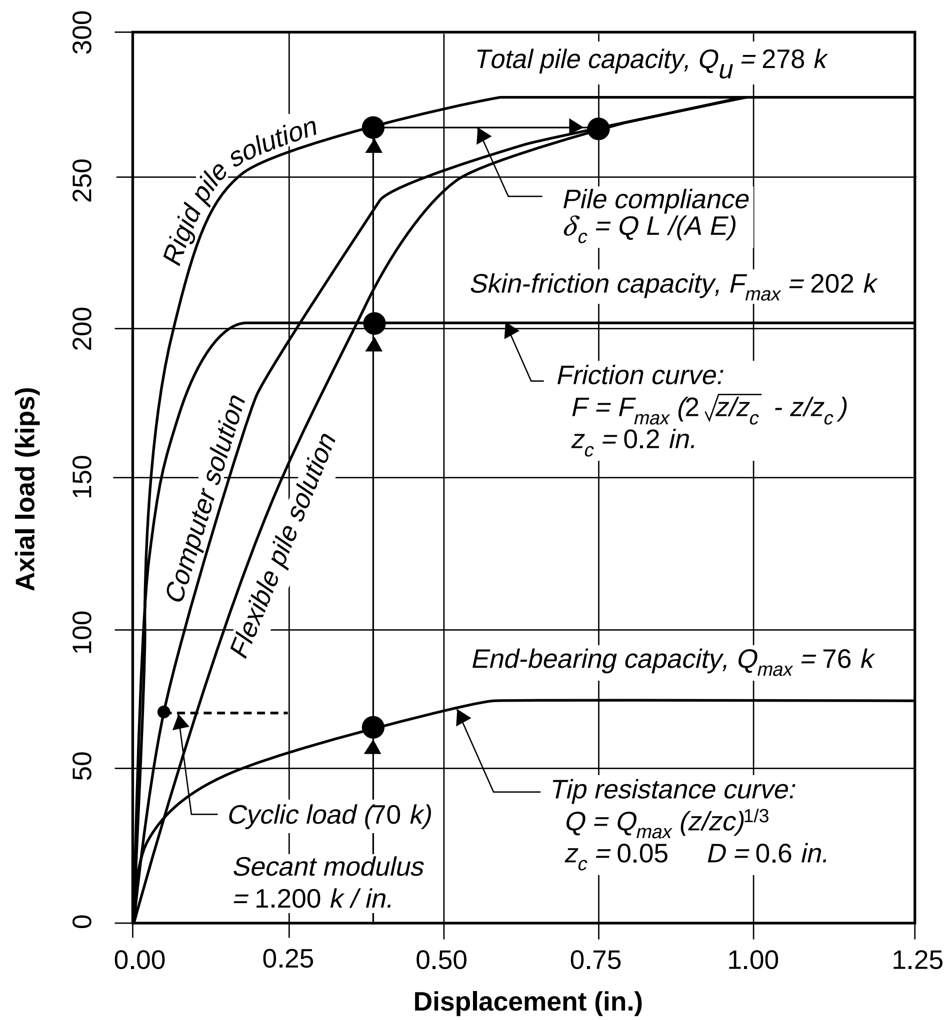

Figure C4-22 Graphical Solution for Axial Pile Stiffness (Lam et al., 1991)

resistance). In the example shown, skin friction is assumed mobilized at a displacement of 0.2 inches, and end bearing at a displacement of 0.5 times the pile diameter.

2. **Rigid Pile Solution.** Using the above load-displacement curves, the rigid pile solution can be developed by summation of the side-friction and end-bearing resistance values at each displacement along the load-displacement curves.

3. **Flexible Pile Solution.** From the rigid pile solution, the flexible pile solution can be developed by adding an additional component of displacement at each load level Q to reflect the pile compliance. For the most flexible pile scenario, corresponding to a uniform thrust distribution along the pile shaft, the pile compliance is given by:

$$\delta_c = \frac{QL}{AE} \qquad \text{(C4-5)}$$

where:

L = Pile length
A = Cross-sectional area
E = Young's modulus of the pile

4. **Intermediate Pile Stiffness Solution.** The "correct" solution, as indicated by the computer solution, is bounded by the rigid pile and flexible pile solutions. In most cases, a good approximation can be developed by averaging the load-displacement curves for the rigid and flexible pile solutions. The above graphical method can be used to solve for the load-displacement curve for any combination of pile/soil situations (end-bearing and friction piles) as well as any pile type or pile material.

As described by Gohl (1993), as an even simpler approximation, pile head stiffness values under normal loading (not exceeding the capacity) may be expressed as some multiple α of AE/L, with the constant α depending on the proportions of shaft and end bearing resistance mobilized. For example, a value of $\alpha = 1.0$ would be appropriate for an end bearing pile on rock with negligible shaft friction. Values of α closer to 2.0 would be reasonable for friction piles with negligible end tip resistance. The range of α from 0.5 to 2.0 in the *Guidelines* encompasses the uncertainties involved with existing foundations, albeit more complex analyses could be used if reliable data are available.

Under earthquake conditions, some magnitude of cyclic axial load will be superimposed on a static bias load (e.g., the static dead weight). Figure C4-23 illustrates the various factors that come into the picture due to a static bias loading. As shown, in a normal design range, where the maximum load level (from superimposing the cyclic load on the static bias) does not exceed the pile capacity (for both the peak compressive or tensile load), the static dead weight can be neglected in solving for the secant stiffness of the pile. The magnitude of cyclic loading, along with the backbone load-displacement curve, can be used to develop the secant stiffness of the pile at the various load levels. However, the load-displacement behavior of the pile will be more complex when the pile capacity (compressive or tensile) is exceeded. Permanent displacement of the pile will occur when the capacity is exceeded.

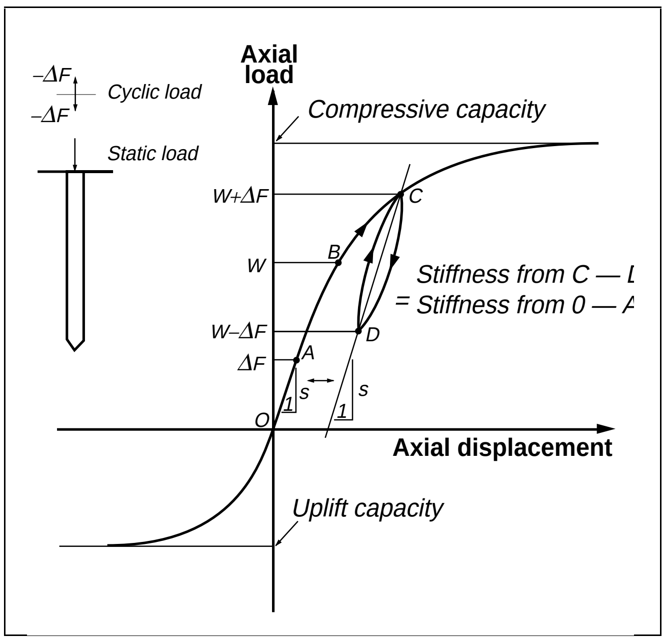

Figure C4-23 ***Load-Displacement Characteristics under Axial Loading (Lam and Martin, 1986)***

Moment-Rotation Capacity. The moment-rotational characteristics and the capacity of a pile footing depend on the following factors:

- The configuration (number of piles and spatial dimension) of the pile footing
- The capacity of each pile for both compression and uplift loading

To illustrate the above concern, Lam (1994) presents an example problem involving a typical pile footing as shown in Figure C4-24. The analyses presented assume a rigid pile cap for the footing, and are quasi-static analyses. The load-displacement curves for each individual pile in the pile group are shown in Figure C4-25. The pile is modeled as an elastic beam-column, and nonlinear axial soil springs are distributed along the pile to represent the soil resistance in both compression and uplift. It can be seen from the figure that the ultimate soil capacities of the pile for

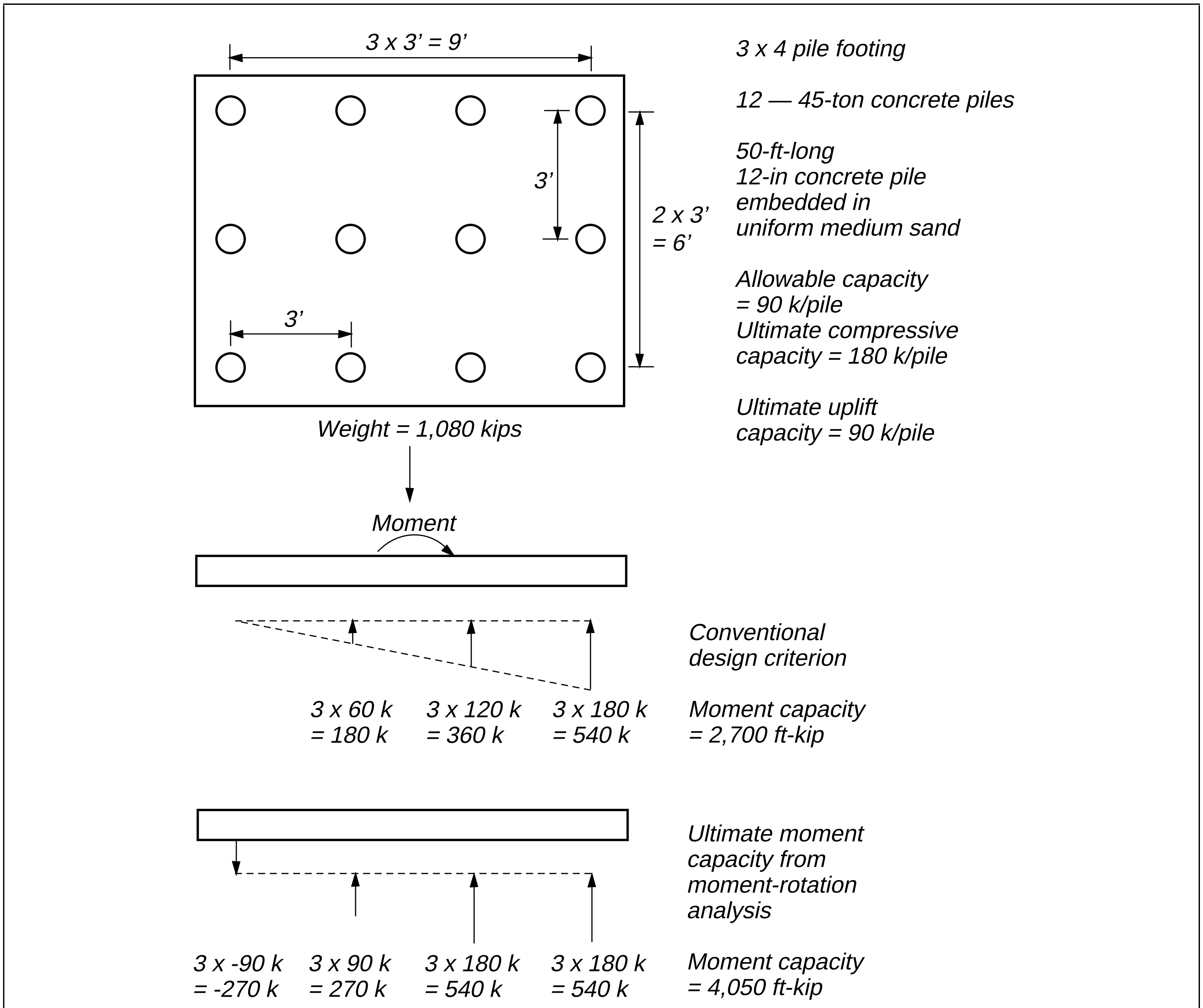

Figure C4-24 Pile Footing Configuration for Moment-Rotation Study (Lam, 1994)

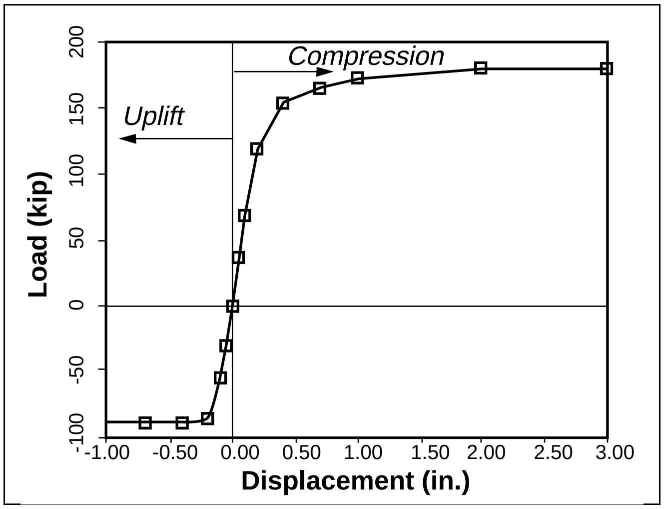

Figure C4-25 Axial Load-Displacement Curve for Single Pile (Lam, 1994)

compression and tension are 180 and 90 kips per pile, respectively, if the connection details and the pile member are adequate to enforce the failure to take place in the soil. The pile has been assumed to be a 50-foot-long, 12-inch concrete pile driven into uniform medium sand, which has a design load capacity of 45 tons per pile. The adopted ultimate capacity values (i.e., 180 kips compression and 90 kips uplift) are the default values commonly assumed by the California Department of Transportation in seismic retrofit projects for the 45-ton class pile. In the example, it is assumed that the footing has been designed for a static factor of safety of 2, or the piles are loaded to half of the ultimate compression capacity prior to the earthquake loading condition.

Figure C4-24 presents various capacity criteria for the pile footing. Under conventional practice, the moment capacity of the pile footing would be 2,700 ft-kip. This capacity arises from assuming a linear distribution in pile reaction across the pile footing. The moment capacity of 2,700 ft-kip is limited by the ultimate compressive capacity value of the most heavily loaded pile (180 kip per pile) while maintaining vertical equilibrium of the overall pile group (i.e., static load of 1,080 kips). The lowest part of Figure C4-24 presents the moment capacity that can be achieved from a nonlinear moment-rotation analysis of the pile footing, in which the moment load increases above the conventional capacity. Nonlinear load-displacement characteristics of the pile are simulated to allow additional load be distributed to the other less loaded

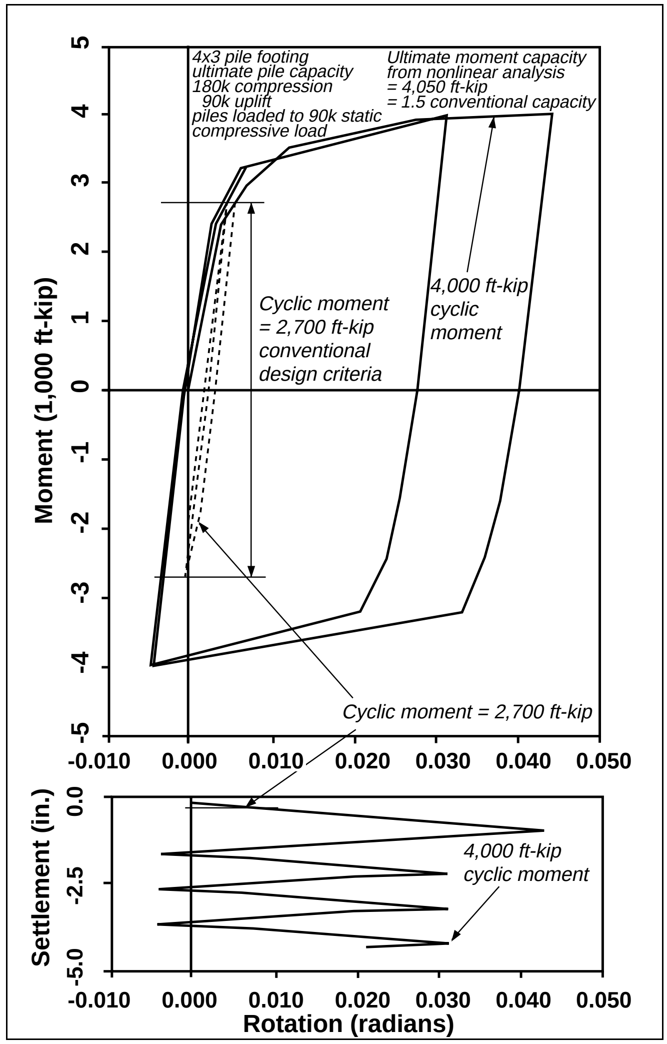

Figure C4-26 Cyclic Moment-Rotation and Settlement-Rotation Solutions (Lam, 1994)

piles in the pile group. As shown, a maximum ultimate capacity of 4,050 ft-kip (1.5 times the conventional capacity) can potentially be achieved by virtue of such nonlinear analysis.

Figure C4-26 presents the cyclic moment-rotation solutions associated with the footing example problem discussed above. The dotted line in the moment-rotation plot defines the monotonic loading path of the moment-rotation relation. Solutions for two uniform cyclic moment loads are presented: a lower cyclic moment level of 2,700 ft-kip corresponding to the conventional design capacity, and a higher cyclic moment load of 4,000 ft-kip. As shown in the figure, at the lower cyclic moment of 2,700 ft-kip, the moment-rotation characteristic is quite linear, and both the moment-rotation characteristics and settlement will equilibrate to the final value very quickly within a few cycles of loading. However, at the higher cyclic moment load of 4,000 ft-kip, progressive settlement of the footing can occur, and within about four cycles of loading, the footing can settle almost five inches. The moment-rotation relationship also indicates that some level of permanent rotation of the footing will likely occur even if the load is symmetric between positive and negative cyclic moments. The potential for the permanent rotation is associated with the change in the state of stress in the soil—from a virgin (unstressed) condition to the equilibrated state—after cyclic loading, unloading, and reloading. A similar analysis, using a static factor of safety of 3 (instead of 2) corresponding to a dead load of 720 kips, resulted in a ultimate moment capacity of 1.3 times the conventional capacity, and a reduced settlement of about 0.25 inches under loading cycles at the increased ultimate capacity level.

Considering the inherent conservatism in pile capacity determinations (especially for compressive loading), most existing pile footings probably have an inherent static factor of safety for dead load of over 3. Hence, it can be speculated that the potential for significant settlement or rotation of a pile footing would not be too high, except for poor soil sites where cyclic degradation of soil strengths can be significant. Typically, the most likely cause of foundation failure would be some form of permanent rotation of the pile group if the size of the footing and the number of piles are inadequate. Therefore, it is important to have a better appreciation of the magnitude of foundation rotation that is tolerable by the pile-supported structure, particularly for retrofit seismic design—where unnecessary conservation can be expensive.

A state-of-the-practice commentary on the seismic design of pile foundations, including a discussion of design uncertainties and structural design issues, has been presented by Martin and Lam (1995). A useful computer program, suitable for determining lateral, moment, and axial stiffness parameters for a vertical pile group, has been documented by Reese and others (1994). For battered pile systems, the computer program PILECAP has been developed for assembling a pile cap stiffness matrix, and is documented by Lam and Martin (1986).

C4.4.2.3 Drilled Shafts

No commentary is provided for this section.

C4.4.3 Foundation Acceptability Criteria

Geotechnical parts and actions of foundations are those whose behavior is characterized by the properties of the soil materials supporting the building. Bearing pressures beneath spread footings or friction forces on a pile are examples of geotechnical actions. These are differentiated from structural actions—such as the bending of a concrete footing, or the compression capacity of a steel pile—covered in other chapters. As with other elements and components, the acceptability of geotechnical parts depends on the performance goal for the building. Additionally, however, the basic procedure for rehabilitation, and the specific assumptions used in the analysis of the building, limit the use of the results with respect to foundation parts.

C4.4.3.1 Simplified Rehabilitation

Chapter 10 presents Simplified Rehabilitation appropriate for use on some buildings. These procedures include some investigation of foundation conditions and, in some cases, requirements for basic modifications.

C4.4.3.2 Linear Procedures

If the foundation is assumed to be fixed in the analysis, geotechnical component displacements are, by definition, zero. Thus, for these actions, acceptability can only be assessed by considering the geotechnical components to be force-controlled. This reduces the seismic force contribution to a more realistic level. Since geotechnical components are actually “ductile” in contrast to most other force-controlled components, acceptable force levels for these fixed-base actions may be based on upper-bound capacities. If these capacities are exceeded, the implication is that actual geotechnical component displacements may be large enough to increase displacement demands significantly in other parts of the structure. The practical consequence is to require the designer to model the elastic properties of the foundation.

If the analysis includes elastic modeling of the foundation, then for shallow and deep foundations, no limit of uplift or compression displacement is necessary for Collapse Prevention or Life Safety Performance Levels. In essence, m = infinity for these cases. This is reasonable, since soil bearing capacity does not degrade for short-term cyclic loads and the consequences of foundation movements are reflected in an approximate manner by the response of the structure in the model. This is true even though fictitious “tension” is allowed to develop between a footing and the soil. This is considered to be analogous to tension yielding in bending of a structural element where the estimate of inelastic displacements assumes that the beam remains elastic. Even if the seismic overturning moment is equal to the maximum resisting moment due to gravity, this situation changes quickly with seismic load reversal. Experience with past earthquakes does not indicate that gross overturning is a problem for buildings. If the calculated displacements do not result in adverse behavior in the structure, there is no need to limit foundation displacements.

However, the situation for the Immediate Occupancy Performance Level is different, since foundation displacements may result in damage that impedes the use of the facility. For this reason, fixed-base conditions should not be assumed for structures sensitive to base movement.

C4.4.3.3 Nonlinear Procedures

The assumption that the base of the structure is rigid in nonlinear procedures is acceptable, provided that the resulting forces do not exceed upper-bound component capacities. The rationale for this limitation is similar to that for linear procedures.

If the foundation is modeled with appropriate nonlinear force-displacement relationships, the acceptability of geotechnical components for Collapse Prevention or Life Safety Performance is analogous to that for linear procedures. For Immediate Occupancy, the amount of the total structural displacement due to foundation movement must be calculated. Some percentage of this foundation-related movement is assumed to be permanent, and the effects of this must be included in considering whether the building can remain functional. Permanent foundation movement is controlled by foundation soil type and thickness, and foundation system characteristics (footing dimensions and geometry).

C4.5 Retaining Walls

The equation in the *Guidelines* for the seismic increment of earth pressure acting on a building retaining wall is a rounded-off form of the equation developed by Seed and Whitman (1970). (In their equation, the fraction 3/8 rather than the rounded-off decimal 0.4 is used. In view of the uncertainty in these pressures, the rounding off is justified.) This equation was developed as an approximation of a seismic earth pressure formulation presented by Seed and Whitman ("Mononobe-Okabe method," 1970) for yielding (free-standing) retaining walls. Because building walls retaining soil (e.g., basement walls) are relatively nonyielding due to the restraint provided by the interior floors, the applicability of these equations to building walls is a matter of some debate. Alternative elastic solutions for seismic wall pressures have been proposed. The most widely used elastic solution is that of Wood (1973), which provides seismic pressures of the order of twice those given by the Seed and Whitman expression. The argument for the lower values of the Seed and Whitman expression is that a limited number of dynamic finite element analyses and one case history (Chang et al., 1990) have found that the calculated and observed seismic earth pressures were of the same order of magnitude as those given by the Mononobe-Okabe formulations and lower than those of the Wood elastic solutions. In a state-of-the-art paper, Whitman (1991) concluded that the Mononobe-Okabe equation should suffice for nonyielding walls, except for the case where a structure, founded on rock, has walls retaining soil. Other publications that discuss seismic lateral earth pressures include Martin (1993), Soydemir (1991), and the *ASCE Standard 4* (ASCE, 1986; under revision).

If building retaining walls are required to be utilized as part of the foundation system to resist seismically-induced structure inertia forces, then higher pressures may be required to be developed on the walls. The maximum pressures that can be mobilized by the soil are passive earth pressures. Because of uncertainty regarding the direction or significance of soil inertia forces affecting the passive pressure capacity, it is suggested that passive pressures be obtained using conventional static earth pressure formulations.

C4.6 Soil Foundation Rehabilitation

Foundation enhancements may be required because of inadequate capacity of existing foundations to resist overturning effects (inadequate footing bearing capacities) or inadequate shear resistance of the foundations. Additionally, foundation enhancements may be required to support structural improvements, such as new shear walls or strengthening of existing shear walls. In either event, the foundation enhancements may be accomplished by a combination of one or several of the following schemes:

- Soil improvement

- Footing improvement (new footing/enlargement of existing footing)
- Foundation underpinning

C4.6.1 Soil Material Improvements

Foundation soil improvements may be undertaken to address global concerns, such as the development of liquefaction, or to improve bearing capacity of the underlying foundation soils. Compaction grouting or chemical grouting are likely choices in either scenario. The level of foundation improvement with either technique may require field testing to verify that the density of soil has improved to the desired level and the extent of grout permeation is consistent with design objectives. Because of the difficulty of working beneath the existing structures to accomplish this goal of soil improvement, a test program may be needed to first verify the procedure and then establish realistic criteria for the level of soil improvement. This may need to be done well in advance of design to indicate the feasibility and economics of these solutions.

C4.6.2 Spread Footings and Mats

Footing improvements could include both constructing new footings to support new shear walls or columns for the structural retrofitting, and enlarging existing footings to support improvements to existing shear walls or additional loads anticipated through the existing shear walls. In either event, planners of the new construction will need to evaluate the relative impact of the new addition (new footing or enlarged footing) upon the existing structure to determine whether the new construction will induce settlements that may affect the integrity of the existing structure.

Footing underpinning is another solution that may be used to resist overturning effects. This solution may typically involve construction of micropiles around the perimeter of an existing footing, then the casting of a grade beam/pile cap integrally with the existing footing. Micropiles may range in size from three inches to as much as eight inches in diameter. Load capacities of the micropiles will vary depending upon subsurface soil conditions; however, load capacities on the order of 50 to 100 tons are not uncommon. This type of foundation strengthening may be used to resist both compression and tension loads, provided that the micropiles are adequately designed and installed in an appropriate bearing stratum. However, the evaluation of this foundation enhancement must consider that the stiffness of the micropiles is much greater than that of the spread footing foundation; the micropiles will deflect less—and thereby attract more—foundation loads than did the original spread footing foundation. This difference in stiffness must be considered in the structural analysis.

C4.6.3 Piers and Piles

No commentary is provided for this section.

C4.7 Definitions

No commentary is provided for this section.

C4.8 Symbols

No commentary is provided for this section.

C4.9 References

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