4. Foundations and Geologic Site Hazards

4.1 Scope

This chapter sets forth general requirements for consideration of foundation load-deformation characteristics, seismic rehabilitation of foundations, and mitigation of geologic site hazards in the Systematic Rehabilitation of buildings.

Section 4.2 specifies data collection for site characterization and defines geologic site hazards. Section 4.3 outlines procedures for mitigation of geologic site hazards. Section 4.4 provides soil strength and stiffness parameters for consideration of foundation load-deformation characteristics. Section 4.5 specifies seismic earth pressures on building walls. Section 4.6 specifies requirements for seismic rehabilitation of foundations.

C4.1 Scope

This chapter provides geotechnical engineering provisions for building foundations and seismic-geologic site hazards. Acceptability of the behavior of the foundation system and foundation soils for a given performance level cannot be determined apart from the context of the behavior of the superstructure.

Geotechnical requirements for buildings that are suitable for Simplified Rehabilitation are included in Chapter 10. Structural engineering issues of foundation systems are discussed in the chapters on Steel and Cast Iron (Chapter 5), Concrete (Chapter 6), Masonry (Chapter 7), and Wood and Light Metal Framing (Chapter 8).

4.2 Site Characterization

Site characterization shall include collection of information on the building foundation as specified in Section 4.2.1, and on seismic geologic site hazards as specified in Section 4.2.2.

C4.2 Site Characterization

The guidance of the State Historic Preservation Officer should be obtained if historic or archeological resources are present at the site.

4.2.1 Foundation Information

Information on the foundation supporting the building to be rehabilitated, nearby foundation conditions, design foundation loads, and load-deformation characteristics of the foundation soils shall be obtained as specified in Sections 4.2.1.1 through 4.2.1.3.

4.2.1.1 Foundation Conditions

4.2.1.1.1 Structural Foundation Information

The following structural information shall be obtained for the foundation of the building to be rehabilitated:

1. Foundation type.
2. Foundation configuration, including dimensions, locations, depth of embedment of shallow foundations, pile tip elevations, and variations in cross-section along the length of pile or belled caissons.
3. Material composition and details of construction.

C4.2.1.1.1 Structural Foundation Information

Foundation types may consist of shallow isolated or continuous spread footings, mat foundations, deep foundations of driven piles, or cast-in-place concrete piers or drilled shafts of concrete.

Foundation material types include concrete, steel, and wood. Foundation installation methods include cast-in-place and open/closed-end driving.

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.

4.2.1.1.2 Subsurface Soil Conditions

The following information on subsurface soil conditions shall be obtained as required by the selected performance level:

1. For Collapse Prevention and Life Safety Performance Levels, the type, composition, consistency, relative density, and layering of soils shall be determined to a depth at which the stress imposed by the building is less than or equal to 10% of the building weight divided by the total foundation area. For buildings with friction piles, the depth so calculated shall be increased by two-thirds of the pile length. For end bearing piles, the depth of investigation shall be the pile length plus 10 feet.

2. The location of the water table and its seasonal fluctuations beneath the building shall be determined.

3. For enhanced rehabilitation objectives, the soil unit weight, γ; soil cohesion, c; soil friction angle, ϕ; soil compressibility characteristics, soil shear modulus, G; and Poisson's ratio, ν, for each type, shall be determined.

C4.2.1.1.2 Subsurface Soil Conditions

Specific foundation information developed for an adjacent or nearby building may be useful if subsurface soils and ground water conditions in the site region are known to be uniform. However, less confidence will result if subsurface data are developed from anywhere but the site of the building being rehabilitated. Adjacent sites where construction has been done recently may provide a guide for evaluation of subsurface conditions at the site being considered.

Design drawings may 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. 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.

4.2.1.2 Design Foundation Loads

Information on the design foundation loads shall be obtained including separate information on dead loads and live loads.

4.2.1.3 Load-Deformation Characteristics Under Seismic Loading

Load-deformation characteristics of foundations shall be obtained from geotechnical reports, or shall be determined in accordance with the requirements of Section 4.4.

C4.2.1.3 Load-Deformation Characteristics Under Seismic Loading

Traditional geotechnical engineering treats load-deformation characteristics for long-term dead loads plus frequently applied live loads only. In most cases, long-term settlement governs foundation design. Short-term (earthquake) load-deformation characteristics have not traditionally been used for design; consequently, such relationships are not generally found in the geotechnical reports for existing buildings.

4.2.2 Seismic Geologic Site Hazards

Seismic rehabilitation shall include an assessment of earthquake-induced hazards at the site due to fault rupture, liquefaction, differential compaction, landsliding, and an assessment of earthquake-induced flooding or inundation in accordance with Sections 4.2.2.1 through 4.2.2.5.

If the resulting ground movements cause unacceptable performance in the building for the selected performance level, then the hazards shall be mitigated in accordance with Section 4.3.

4.2.2.1 Fault Rupture

A geologic fault shall be defined as a plane or zone along which earth materials on opposite sides have moved differentially in response to tectonic forces.

Geologic site information shall be obtained to determine if an active geologic fault is present under the building foundation. If a fault is present, the following information shall be obtained:

1. The degree of activity based on the age of most recent movement.

2. The fault type; whether it is a strike-slip, normal-slip, reverse-slip, or thrust fault.

3. The sense of slip with respect to building geometry.

4. Magnitudes of vertical and/or horizontal displacements consistent with the selected earthquake hazard level.

5. The width and distribution of the fault-rupture zone.

C4.2.2.1 Fault Rupture

Buildings found to straddle active faults should be assessed to determine if any rehabilitation is warranted, possibly to reduce the collapse potential of the structure given the likely amount and direction of fault displacement.

4.2.2.2 Liquefaction

Liquefaction shall be defined as an earthquake-induced process in which saturated, loose, granular soils lose shear strength and liquefy as a result of an increase in pore-water pressure during earthquake shaking.

Subsurface soil and ground water information shall be obtained to determine if liquefiable materials are present under the building foundation. If liquefiable soils are present, the following information shall be obtained: soil type, soil density, depth to water table, ground surface slope, proximity of free-face conditions, and lateral and vertical differential displacements.

A site shall be regarded as free from liquefaction hazard if the site soils, or similar soils in the site vicinity, have not experienced historical liquefaction, and if any of the following criteria are met:

1. The geologic materials underlying the site are either bedrock or have a very low liquefaction susceptibility according to the relative susceptibility ratings based upon the type of deposit and geologic age of the deposit, as shown in Table 4-1.

2. The soils underlying the site are stiff clays or clayey silts.

3. The soils are not highly sensitive, based on local experience.

4. The soils are cohesionless with a minimum normalized Standard Penetration Test (SPT) resistance, $N_{1\ 60}$, value of 30 blows/foot as defined in *ASTM D1586-99*, for depths below the ground water table, or with clay content greater than 20%.

5. The ground water table is at least 35 feet below the deepest foundation depth, or 50 feet below the ground surface, whichever is shallower, including considerations for seasonal and historic ground-water level rises, and any slopes or free-face conditions in the site vicinity do not extend below the ground water elevation at the site.

If a liquefaction hazard is determined to exist at the site, then a more detailed evaluation of potential ground movements due to liquefaction shall be performed using approved procedures.

Table 4-1 Estimated Susceptibility to Liquefaction of Surficial Deposits During Strong Ground Shaking

Type of Deposit	General Distribution of Cohesionless Sediments in Deposits	Likelihood that Cohesionless Sediments, When Saturated, Would be Susceptible to Liquefaction (by Age of Deposit)			
		Modern < 500 yr.	Holocene < 11,000 yr.	Pleistocene < 2 million yr.	Pre-Pleistocene > 2 million yr.
(a) Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Flood plain	Locally variable	High	Moderate	Low	Very low
Alluvial fan, plain	Widespread	Moderate	Low	Low	Very low
Marine terrace	Widespread	—	Low	Very low	Very low
Delta, fan delta	Widespread	High	Moderate	Low	Very low
Lacustrine, playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dune	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	Low	Unknown	Unknown
Residual soils	Rare	Low	High	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low
(b) Coastal Zone Deposits					
Delta	Widespread	Very high	High	Low	Very low
Esturine	Locally variable	High	Moderate	Low	Very low
Beach, high energy	Widespread	Moderate	Low	Very low	Very low
Beach, low energy	Widespread	High	Moderate	Low	Very low
Lagoon	Locally variable	High	Moderate	Low	Very low
Foreshore	Locally variable	High	Moderate	Low	Very low
(c) Fill Materials					
Uncompacted fill	Variable	Very high	—	—	—
Compacted fill	Variable	Low	—	—	—

C4.2.2.2 Liquefaction

Soil liquefaction is a phenomenon in which a soil below the ground-water 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 following information may be necessary for evaluating the liquefaction potential of soils.

1. **Soil type:** Whether liquefiable soils—i.e., granular (sand, silty sand, nonplastic silt) soils—are present.

2. **Soil density:** Whether liquefiable soils are loose to medium dense.

3. **Depth to water table:** Whether liquefiable soils are saturated at any time during seasonal fluctuations of the water table.

4. **Ground surface slope and proximity of free-face conditions:** Whether liquefiable soils are at a gently sloping site or in the proximity of free-surface conditions.

5. **Lateral and vertical differential displacement:** Amount and direction at the building foundation should be calculated.

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). 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 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), 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. More guidance for evaluating liquefaction potential is contained in Section C4.2.2.2 of *FEMA 274*.

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.

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.

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 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). Relationships presented by Ishihara and Yoshimine (1992) are also available for assessing settlement.

Evaluating Increased Lateral Earth Pressures on Building Walls. Liquefaction of soils adjacent to building walls increases lateral earth pressures which can be approximated as a fluid pressure having a unit weight equal to the saturated unit weight of the soil plus the inertial forces on the soil equal to the hydrodynamic pressure.

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.

4.2.2.3 Differential Compaction

Differential compaction shall be defined as an earthquake-induced process in which foundation soils compact and the foundation settles in a nonuniform manner across a site.

Subsurface soil information shall be obtained to determine if soils susceptible to differential compaction are present under the building foundation.

A site shall be regarded as free of a differential compaction hazard if the soil conditions meet both of the following criteria:

1. Geologic materials below the ground water table do not pose a liquefaction hazard, based on the criteria in Section 4.2.2.2.

2. Geologic deposits above the ground water table are either Pleistocene in geologic age (older than 11,000 years), stiff clays or clayey silts, or cohesionless sands, silts, and gravels with a minimum $(N_I)_{60}$ of 20 blows/0.3 m (20 blows/foot).

If a differential compaction hazard is determined to exist at the site, then a more detailed evaluation shall be performed using approved procedures.

C4.2.2.3 Differential Compaction

Differential compaction or densification of soils may accompany strong ground shaking. The resulting differential settlements can be damaging to structures. Types of soil susceptible to liquefaction (that is, relatively loose natural soils, or uncompacted or poorly compacted fill soils) are also susceptible to compaction. Compaction can occur in soils above and below the ground-water table.

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.

4.2.2.4 Landsliding

A landslide shall be defined as the down-slope mass movement of earth resulting from any cause. Subsurface soil information shall be obtained to determine if soils susceptible to a landslide that will cause differential movement of the building foundation are present at the site.

Slope stability shall be evaluated at sites with:

1. Existing slopes exceeding 18 degrees (three horizontal to one vertical).

2. Prior histories of instability (rotational or translational slides, or rock fall).

Use of pseudo-static analyses shall be permitted to determine slope stability if the soils are not susceptible to liquefaction based on Section 4.2.2.2 or otherwise expected to lose shear strength during deformation. If soils are susceptible to liquefaction based on Section 4.2.2.2 or otherwise expected to lose shear strength during deformation, dynamic analyses shall be performed to determine slope stability.

Pseudo-static analyses shall use a seismic coefficient equal to $S_{XS}/5$, to approximate one-half the peak ground acceleration at the site associated with the selected Rehabilitation Objective. Sites with a static factor of safety equal to or greater than 1.0 shall be judged to have adequate stability, and require no further stability analysis.

A sliding-block displacement analysis shall be performed for sites with a static factor of safety of less than 1.0. The displacement analysis shall determine the magnitude of ground movement and its effect upon the performance of the structure.

In addition to the effects of landslides that directly undermine the building foundation, the effects of rock fall or slide debris from adjacent slopes shall be evaluated using approved procedures.

C4.2.2.4 Landsliding

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.

4.2.2.5 Flooding or Inundation

For seismic rehabilitation of buildings for performance levels higher than Life Safety, site information shall be obtained to determine if the following sources of earthquake-induced flooding or inundation are present:

1. Dams located upstream, subject to damage by earthquake shaking or fault rupture.

2. Pipelines, aqueducts, and water storage tanks located upstream, subject to damage by fault rupture, earthquake-induced landslides, or strong shaking.

3. Coastal areas within tsunami zones or areas adjacent to bays or lakes, subject to seiche waves.

4. Low-lying areas with shallow ground water, subject to regional subsidence and surface ponding of water, resulting in inundation of the site.

Damage to buildings from earthquake-induced flooding or inundation shall be evaluated for its effect upon the performance of the structure.

In addition to the effects of earthquake-induced flooding or inundation, scour of building foundation soils from swiftly flowing water shall be evaluated using approved procedures.

4.3 Mitigation of Seismic-Geologic Site Hazards

Mitigation of seismic-geologic hazards identified in Section 4.2 shall be accomplished through modification of the structure, foundation, soil conditions, or other approved methods.

C4.3 Mitigation of Seismic-Geologic Site Hazards

Opportunities exist to improve seismic performance under the influence of some site hazards at reasonable cost; however, some site hazards may be so severe that they are economically impractical to include in risk-reduction measures. The discussions presented in this section are based on the concept that the extent of site hazards is discovered after the decision for seismic rehabilitation of a building has been made; however, the decision to rehabilitate a building and the selection of a Rehabilitation Objective may have been made with full knowledge that significant site hazards exist and must be mitigated as part of the rehabilitation.

Possible mitigation strategies for seismic geologic site hazards are presented in the following sections.

1. Fault Rupture. If the structural performance of a building evaluated for the calculated ground movement due to fault rupture during earthquake fails to comply with the requirements for the selected performance level, mitigation schemes should be employed that include one or more of the following measures to achieve acceptable performance: stiffening of the structure and/or its foundation; strengthening of the structure and/or its foundation; and modifications to the structure and/or its foundation to distribute the effects of differential vertical movement over a greater horizontal distance to reduce angular distortion.

Large movements caused by fault rupture generally cannot be mitigated economically. If the structural consequences of the estimated horizontal and vertical displacements are unacceptable for any performance level, either the structure, its foundation, or both, might be stiffened or strengthened to reach acceptable performance. Measures are highly dependent on specific structural characteristics and inadequacies. Grade beams and reinforced slabs are effective in increasing resistance to horizontal displacement. Horizontal forces are sometimes limited by sliding friction capacity of spread footings or mats. Vertical displacements are similar in nature to those caused by long-term differential settlement.

2. Liquefaction. If the structural performance of a building evaluated for the calculated ground movement due to liquefaction during an earthquake fails to comply with the requirements for the selected performance level, then one or more of the following mitigation measures should be implemented to achieve acceptable performance.

2.1 Modification of the Structure. The structure should be strengthened to improve resistance against the predicted liquefaction-induced ground deformation.

This solution may be feasible for small ground deformations.

2.2 Modification of the Foundation. The foundation system should be modified to reduce or eliminate the differential foundation displacements by underpinning existing shallow foundations to achieve bearing on deeper, nonliquefiable strata or by stiffening a shallow foundation system by a system of grade beams between isolated footings, or any other approved method.

2.3 Modification of the Soil Conditions. One or more of the following ground improvement techniques should be implemented to reduce or eliminate the liquefaction under existing buildings: soil grouting (either throughout the entire liquefiable strata beneath a building, or locally beneath foundation elements); installation of drains; or installation of permanent dewatering systems.

Other types of ground improvement widely used for new construction are less applicable to existing buildings because of the effects of the procedures on the building. Thus, removal and replacement of liquefiable soil or in-place densification of liquefiable soil by various techniques are not applicable beneath an existing building.

2.4 Mitigation of the Lateral Spreading. Large soil volumes should be stabilized and/or buttressing structures should be constructed.

If the potential for significant liquefaction-induced lateral spreading movements exists at a site, then the mitigation of the liquefaction hazard may be more difficult. This is because the potential for lateral spreading movements beneath a building may depend on the behavior of the soil mass at distances well beyond the building as well as immediately beneath it.

3. Differential Compaction. If the structural performance of a building evaluated for the calculated differential compaction during earthquake fails to comply with the requirements for the selected performance level, then one or more mitigation measures similar to those recommended for liquefaction should be implemented to achieve acceptable performance.

4. Landslide. If the structural performance of a building evaluated for the calculated ground movement due to landslide during earthquake fails to comply with the requirements for the selected performance level, then one or more of the following mitigation measures should be implemented to achieve acceptable performance:

1. Regrading.
2. Drainage.
3. Buttressing.

4. Structural Improvements:
    a. Gravity walls.
    b. Tieback/soil nail walls.
    c. Mechanically stabilized earth walls.
    d. Barriers for debris torrents or rock fall.
    e. Building strengthening to resist deformation.
    f. Grade beams.
    g. Shear walls.
5. Soil Modification/Replacement:
    a. Grouting.
    b. Densification.

5. Flooding or Inundation. If the structural performance of a building evaluated for the effects of earthquake-induced flooding and inundation fails to comply with the requirements for the selected performance level, then one or more of the following mitigating measures should be implemented to achieve acceptable performance:

1. Improvement of nearby dam, pipeline, or aqueduct facilities independent of the rehabilitated building.
2. Diversion of anticipated peak flood flows.
3. Installation of pavement around the building to reduce scour.
4. Construction of sea wall or breakwater for tsunami or seiche protection.

4.4 Foundation Strength and Stiffness

Foundation strength and stiffness shall be determined in accordance with this section.

C4.4 Foundation Strength and Stiffness

It is assumed that foundation soils are not susceptible to significant strength loss due to earthquake loading. In general, soils have considerable ductility unless they degrade significantly in stiffness and strength under cyclic loading. With this assumption, the provisions of this section provide an overview of the requirements and procedures for evaluating the ability of foundations to withstand the imposed seismic loads without excessive deformations.

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, foundation yield associated with mobilization at upper bound expected 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 upper bound expected soil capacities, it can be assumed that foundation deformations will be relatively small.

Parametric analyses to cover uncertainties in soil load-deformation characteristics are required. 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). In static analyses, a somewhat conservative interpretation of the results is recommended because cyclic loading effects cannot be incorporated directly.

4.4.1 Expected Capacities of Foundations

The expected capacity of foundation components shall be determined by presumptive, prescriptive, or site-specific methods as specified in Sections 4.4.1.1 through 4.4.1.3. Capacities shall be based on foundation information obtained as specified in Section 4.2.1.

C4.4.1 Expected Capacities of Foundations

Design values recommended by geotechnical engineers are generally consistent with lower bound values. It is important to obtain information on the actual factor of safety applied to arrive at design values so that ultimate soil capacities are understood and expected values can be properly derived.

4.4.1.1 Presumptive Capacities

4.4.1.1.1 Presumptive Capacities of Shallow Foundations

Calculation of presumptive expected capacities for spread footings and mats shall be permitted using the parameters specified in Table 4-2.

4.4.1.1.2 Presumptive Capacities of Deep Foundations

1. Capacities of Piles or Piers in Granular Soils

Calculation of presumptive expected capacities of piles or piers in granular soils shall be permitted using the procedure shown in Figure 4-1.

2. Capacities of Piles or Piers in Cohesive Soils

Calculation of presumptive expected capacities of piles or piers in cohesive soils shall be permitted using the procedure shown in Figure 4-2.

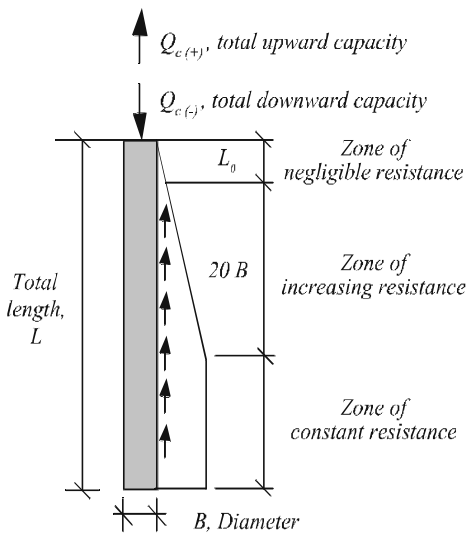
C4.4.1.1.2 Presumptive Capacities of Deep Foundations

The calculation procedures for presumptive expected capacities of piles or piers specified in this section are adapted from *ATC-40* and NAVFAC (1986a and 1986b).

Table 4-2 Parameters for Calculating Presumptive Expected Foundation Load Capacities of Spread Footings and Mats

Class of Materials[2]	Vertical Foundation Pressure[3] Lbs/Sq. Ft. (q_c)	Lateral Bearing Pressure Lbs/Sq. Ft./Ft of Depth Below Natural Grade[4]	Lateral Sliding[1]	
			Coefficient[5]	Resistance[6] Lbs/Sq. Ft
Massive Crystalline Bedrock	8000	2400	0.80	—
Sedimentary and Foliated Rock	4000	800	0.70	—
Sandy Gravel and/or Gravel (GW and GP)	4000	400	0.70	—
Sand, Silty Sand, Clayey Sand, Silty Gravel, and Clayey Gravel (SW, SP, SM, SC, GM, and GC)	3000	300	0.50	—
Clay, Sandy Clay, Silty Clay, and Clayey Silt (CL, ML, MH, and CH)	2000[7]	200	—	260

1. Lateral bearing and lateral sliding resistance shall be permitted to be combined.
2. For soil classifications OL, OH, and PT (i.e., organic clays and peat), a foundation investigation shall be required.
3. All values of expected bearing capacities are for footings having a minimum width of 12 inches and a minimum depth of 12 inches into natural grade. Except where Footnote 7 applies, an increase of 20% is allowed for each additional foot of width or depth to a maximum value of three times the designated value.
4. Shall be permitted to be increased by the amount of the designated value for each additional foot of depth to a maximum of 15 times the designated value.
5. Coefficient applied to the dead load.
6. Lateral sliding resistance value to be multiplied by the contact area. In no case shall the lateral sliding resistance exceed one-half of the dead load.
7. No increase for width shall be permitted.



Soil Profile

Thickness L_i	Wt. Density γ_i	Angle of Shearing Resistance ϕ_i
L_0	γ_0	ϕ_0
L_1	γ_1	ϕ_1
L_2	γ_2	ϕ_2
etc.	etc.	etc.
etc.	etc.	etc.
L_t	γ_t	ϕ_t

Downward Capacity

$$Q_c\,(-) = P_t N_q A_t + \sum_{i=1}^{t-1} F_{di} P_i \tan\delta_i a_s L_i$$

Upward Capacity

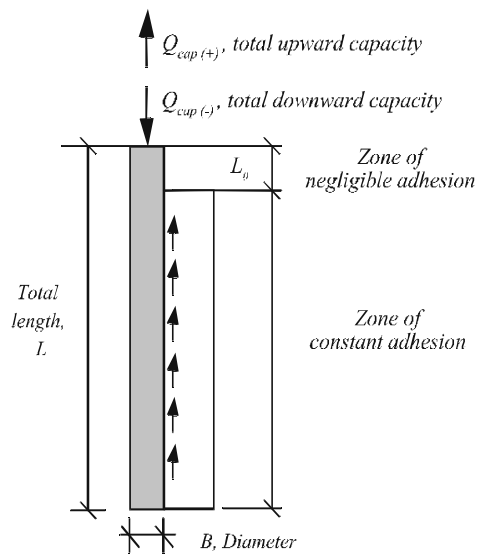
$$Q_c\,(+) = \sum_{i=1}^{t-1} F_{ui} P_i \tan\delta_i a_s L_i$$

$$P_i = \sum_{j=0}^{i} L_j \gamma_j \le P_c$$

where:

B = Pile or pier diameter
L_0 = 3B for use with high values of F_{di} or F_{ui}
= 5B for use with low values of F_{di} or F_{ui}
P_i = Effective vertical stress at depth i
P_c = Effective vertical stress P_i at depth $L_0 + 20B$
P_t = Effective vertical stress P_i at tip or at $i=t$
N_q = Bearing capacity factor (see Table 4-3)
A_t = Bearing area at tip
F_{di} = Effective horizontal stress factor for downward load (see Table 4-4)
F_{ui} = Effective horizontal stress factor for upward load (see Table 4-4)
δ_i = Friction angle between pile/pier at soil depth i (see Table 4-5)
a_s = Surface area of pile/pier per unit length

Figure 4-1 Presumptive Expected Capacities of Piles or Piers in Granular Soils



Soil Profile

Thickness L_i	Wt. Density γ_i	Adhesion C_{ai}
L_0	γ_0	C_0
L_1	γ_1	C_1
L_2	γ_2	C_2
etc.	etc.	etc.
etc.	etc.	etc.
L_t	γ_t	C_t

Downward Capacity

$$Q_c\,(-) = c_t N_c A_t + \sum_{i=1}^{t} c_{ai} a_s L_i$$

Upward Capacity

$$Q_c\,(+) = \sum_{i=1}^{t} c_{ai} a_s L_i$$

where:

B = Pile or pier diameter
L_0 = 4B
c_t = Cohesion strength of soil at tip (see Table 4-6)
N_c = Bearing capacity factor = 9.0 for L > 4B
= 9L/4B for $L \leq L_0$
A_t = Bearing area at tip
c_{ai} = Adhesion strength of soil (see Table 4-6) at depth i
a_s = Surface area of pile/pier per unit length

Figure 4-2 Presumptive Expected Capacities of Piles or Piers in Cohesive Soils

Table 4-3 Typical Pile and Pier Capacity Parameters: Bearing Capacity Factors, N_q

	Angle of Shearing Resistance for Soil, ϕ (degrees)												
Placement	**26**	**28**	**30**	**31**	**32**	**33**	**34**	**35**	**36**	**37**	**38**	**39**	**40+**
Driven pile	10	15	21	24	29	35	42	50	62	77	86	120	145
Drilled pier	5	8	10	12	14	17	21	25	30	38	43	60	72

Table 4-4 Typical Pile and Pier Capacity Parameters: Effective Horizontal Stress Factors, F_{di} and F_{ui}

	Downward F_{di}		Upward F_{ui}	
Pile or Pier Type	**low**	**high**	**low**	**high**
Driven H-pile	0.5	1.0	0.3	0.5
Driven straight prismatic pile	1.0	1.5	0.6	1.0
Driven tapered pile	1.5	2.0	1.0	1.3
Driven jetted pile	0.4	0.9	0.3	0.6
Drilled pier	0.7	0.7	0.4	0.4

Table 4-5 Typical Pile and Pier Capacity Parameters: Friction Angle, δ (degrees)

Pile or Pier Material	δ
Steel	20
Concrete	0.75 ϕ
Timber	0.75 ϕ

4.4.1.2 Prescriptive Expected Capacities

Prescriptive expected capacities shall be used when construction documents or previous geotechnical reports for the existing building are available and provide information on foundation soil design parameters. Calculation of prescriptive expected capacities by the following methods shall be permitted.

1. The prescriptive expected bearing capacity, q_c, for a spread footing shall be calculated using Equation (4-1).

$$q_c = 3q_{allow} \tag{4-1}$$

where:

q_{allow} = Allowable bearing pressure specified in available documents for the gravity load design of shallow foundations (dead plus live loads)

2. For deep foundations, the prescriptive expected vertical capacity, Q_c, of individual piles or piers shall be calculated using Equation (4-2).

$$Q_c = 3Q_{allow} \tag{4-2}$$

where:

Q_{allow} = Allowable vertical capacity specified in available documents for the gravity load design of deep foundations (dead plus live loads)

3. Alternatively, the prescriptive expected capacity, q_c or Q_c, of any foundation, shallow or deep, shall be calculated using Equation (4-3).

$$q_c \ \ or \ \ Q_c = 1.5Q_G \tag{4-3}$$

where Q_G = gravity load action as specified in Section 3.2.8, expressed in terms of pressure or load.

Table 4-6 Typical Pile and Pier Capacity Parameters: Cohesion, c_t, and Adhesion, c_a (psf)

Pile Material	Consistency of Soil (approximately STP blow count)	Cohesion, c_t		Adhesion, c_a	
		low	high	low	high
Timber and Concrete	Very soft (<2)	0	250	0	250
	Soft (2–4)	250	500	250	480
	Medium stiff (4–8)	500	1000	480	750
	Stiff (8–15)	1000	2000	750	950
	Very stiff (>15)	2000	4000	950	1300
Steel	Very soft (<2)	0	250	0	250
	Soft (2–4)	250	500	250	460
	Medium stiff (4–8)	500	1000	460	700
	Stiff (8–15)	1000	2000	700	720
	Very stiff (>15)	2000	4000	720	750

4.4.1.3 Site-Specific Capacities

For buildings where the methods specified in Sections 4.4.1.1 and 4.4.1.2 do not apply, a subsurface geotechnical investigation shall be conducted to determine expected ultimate foundation capacities based on the specific characteristics of the building site.

4.4.2 Load-Deformation Characteristics for Foundations

If building foundations are explicitly modeled in the mathematical model of the building, the load-deformation characteristics shall be calculated in accordance with Section 4.4.2.1 for shallow bearing foundations, Section 4.4.2.2 for pile foundations, and Section 4.4.2.3 for drilled shafts.

For explicit modeling of other types of foundations, load deformation characteristics shall be calculated by an approved method.

Nonlinear behavior of foundations shall be represented by an equivalent elasto-plastic load-deformation relationship unless another approved relationship is available.

Building analyses, including load-deformation characteristics of foundations, shall be bounded by upper and lower bound assumptions of stiffness and capacity as illustrated in Figure 4-3a and defined in this section. In lieu of an explicit evaluation of uncertainties of the foundation characteristics, it shall be permitted to take the respective upper and lower bound stiffness and strength values as twice-and-one-half the values given in this section.

C4.4.2 Load-Deformation Characteristics for Foundations

Load-deformation characteristics are required where the effects of foundations are to be taken into account in Linear Static or Dynamic Procedures (LSP or LDP), Nonlinear Static (pushover) Procedures (NSP), or Nonlinear Dynamic (time-history) Procedures (NDP). Foundation load-deformation parameters characterized by both stiffness and capacity can have a significant effect on both structural response and load distribution among structural elements.

While it is recognized that the load-deformation behavior of foundations is nonlinear, an equivalent elasto-plastic representation of load-deformation behavior is recommended because of the difficulties in determining soil properties and the likely variability of soils supporting foundations. In addition, to allow for such variability or uncertainty, an upper and lower bound approach to defining stiffness and capacity is required to evaluate the sensitivity of the structural response to these parameters.

The sources of this uncertainty include variations due to rate of loading, assumed elasto-plastic soil behavior, level of strain, cyclic loading, and variability of soil properties. These sources of variability produce results that are generally within a factor of two above or below the expected value. It is conceivable that certain conditions will fall outside the bounds prescribed in this standard. However, it is not the objective to guarantee that the answer is always within the applied factor. Instead, the intent is that 1) solution sensitivity be identified, and 2) that the bounds, considered reasonably, capture the expected behavior. Current practice (both conventional and within the nuclear industry) has suggested that variation by a factor of two is generally appropriate. Geotechnical engineers often use a safety factor of two to establish lower bound values for use in design. Consistent with the approach taken in *ASCE 4*, if additional testing is performed, the range could be narrowed to that defined by multiplying and dividing by $(1+C_v)$, where the coefficient of variation, C_v, is defined as the standard deviation divided by the mean. In no case should C_v be taken to be less than 0.5.

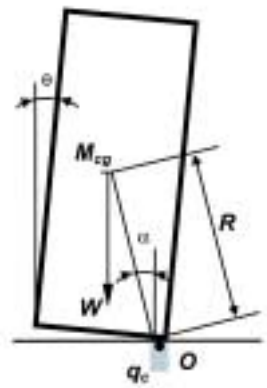
It is important that geotechnical engineers report the average results obtained and the actual factor of safety applied to arrive at design values. The design values recommended by geotechnical engineers are generally consistent with the lower bound. If such reduced values were used by the structural engineer as expected values, the application of the prescribed upper and lower bound variations would not achieve the intended aim.

1. Consideration of Foundation Rocking: Buildings may rock on their foundations in an acceptable manner provided the structural components can accommodate the resulting displacements and deformations. Consideration of rocking can be used to limit the force input to a building; however, rocking should not be considered simultaneously with the effects of soil flexibility.

The design professional is directed to *FEMA 274* and the work of Yim and Chopra (1985), Housner (1963), Makris and Roussos (1998), and Priestly and Evison (1978) for additional information on rocking behavior.

A possible procedure for considering rocking is outlined in Figure C4-1. The procedure involves the following steps:

- Calculation of the mass, weight, and center of gravity for the rocking system (or subsystem);
- Calculation of the soil contact area, center of contact, and rocking system dimension, *R;*
- Determination of whether rocking will initiate;
- Calculation of the effective viscious damping of the rocking system (and the corresponding design displacement spectrum);
- Calculation (graphically or iteratively) of the period and amplitude of rocking (the solution will not converge if overturning will occur—that is, when $\theta > \alpha$.



Mass, weight, and center of gravity

Note that, in general, the mass and weight will not be consistent with each other. The mass, *M*, is the total seismic mass tributary to the wall. The weight, *W*, is the vertical gravity load reaction. For the purposes of these calculations, the vertical location of the center of gravity is taken at the vertical center of the seismic mass and the horizontal location of the center of gravity is taken at the horizontal center of the applied gravity loads.

Soil contact area and center of contact

The soil contact area is taken as W/q_c. The wall rocks about point O located at the center of the contact area.

Wall rocking potential

Determine whether the wall will rock by comparing the overturning moment to the restoring moment. For this calculation, S_a is based on the fundamental, elastic (no-rocking) period of the wall. The wall will rock if $S_a > (W/Mg)\tan\alpha$. If rocking is not indicated, discontinue these calculations.

Rocking calculations

Calculate I_O, the mass moment of inertia of the rocking system about point O.

Calculate the effective viscous damping, β, of the rocking system as follows:

$$\beta = 0.4(1-\sqrt{r}) \text{ where } r = \left[1 - \frac{MR^2}{I_O}(1-\cos(2\alpha))\right]^2$$

Construct the design response spectrum at this level of effective damping using the procedure defined in Section 1.6.1.5. By iteration or graphical methods, solve for the period and displacement that simultaneously satisfy the design response spectrum and the following rocking period equation:

$$T = \frac{4}{\sqrt{\frac{WR}{I_O}}}\cosh^{-1}\left(\frac{1}{1-\frac{\theta}{\alpha}}\right) \text{ where } \theta = \frac{\delta_{rocking}}{R\cos\alpha}$$

Also recall that $S_d = S_a g \frac{T^2}{4\pi^2}$

At the desired solution, $\delta_{rocking} = S_d$

Figure C4-1 Outline Procedure for Consideration of Rocking Behavior

4.4.2.1 Shallow Bearing Foundations

4.4.2.1.1 Stiffness Parameters

The initial shear modulus, G_o, shall be calculated in accordance with Equation (4-4) or (4-5) where v_s is the shear wave velocity at low strains, γ is the weight density of the soil, and g is the acceleration due to gravity.

$$G_o = \frac{\gamma v_s^2}{g} \tag{4-4}$$

$$G_o \cong 20{,}000\ (N_1)_{60}^{1/3} \sqrt{\sigma_o'} \tag{4-5}$$

where:

$(N_1)_{60}$ = Standard Penetration Test blow count normalized for an effective stress of 1.0 ton per square foot confining pressure and corrected to an equivalent hammer energy efficiency of 60%

σ_o' = Effective vertical stress in psf

σ_o' = $\gamma_t d - \gamma_w (d - d_w)$

γ_t = Total unit weight of soil

γ_w = Unit weight of water

d = Depth to sample

d_w = Depth to ground-water level

G_o in Equation (4-5) is expressed in pounds per square foot, as is σ_o'.

The effective shear modulus, G, shall be calculated in accordance with Table 4-7.

Table 4-7 Effective Shear Modulus Ratio (G/G_0)

	Effective Peak Acceleration, $S_{XS}/2.5$			
Site Class	$S_{XS}/2.5$ =0	$S_{XS}/2.5$ =0.1	$S_{XS}/2.5$ =0.4	$S_{XS}/2.5$ =0.8
A	1.00	1.00	1.00	1.00
B	1.00	1.00	0.95	0.90
C	1.00	0.95	0.75	0.60
D	1.00	0.90	0.50	0.10
E	1.00	0.60	0.05	*
F	*	*	*	*

Note: Use straight-line interpolation for intermediate values of $S_{XS}/2.5$.

* Site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Based on relative stiffnesses of the foundation structure and the supporting soil, the foundation stiffness shall be calculated using one of the following three methods.

C4.4.2.1.1 Stiffness Parameters

Table 4-7 is consistent with the site classification Tables 1-4 and 1-5 in that the layout and level of complexity is identical, and the indication of problem soils that require site-specific investigation (Site Class F) is consistent. The following observations on the relationship between shear modulus reduction and peak ground acceleration can be made:

1. As the peak ground acceleration approaches zero, the modulus reduction factor approaches unity,

2. Modulus reduction effects are significantly more pronounced for softer soils, and

3. The modulus reduction factors given in both *FEMA 273* and the *FEMA 302 NEHRP Recommended Provisions* overestimate the modulus reduction effects for Site Classes A, B, and C.

The shears and moments in foundation elements are conservative when such elements are considered rigid. However, soil pressures may be significantly underestimated when foundation flexibility is ignored. The flexibility and nonlinear response of soil and of foundation structures should be considered when the results would change.

For beams on elastic supports (for instance, strip footings and grade beams) with a point load at midspan, the beam may be considered rigid when:

$$\frac{EI}{L^4} > \frac{2}{3} k_{sv} B \qquad \text{(C4-1)}$$

The above equation is generally consistent with traditional beam-on-elastic foundation limits (NAVFAC, 1986b; Bowles, 1988). The resulting soil bearing pressures are within 3% of the results, including foundation flexibility.

For rectangular plates (with plan dimensions L and B, and thickness t, and mechanical properties E_f and v_f) on elastic supports (for instance, mat foundations or isolated footings) subjected to a point load in the center, the foundation may be considered rigid when:

$$4k_{sv} \sum_{m=1}^{5} \sum_{n=1}^{5} \frac{\sin^2\left(\frac{m \cdot \pi}{2}\right) \sin^2\left(\frac{n \cdot \pi}{2}\right)}{\left[\pi^4 D_f \left(\frac{m^2}{L^2} + \frac{n^2}{B^2}\right)^2\right] + k_{sv}} < 0.03 \qquad \text{(C4-2)}$$

where:

$$D_f = \frac{E_f t^3}{12(1 - \nu_f)^2} \qquad \text{(C4-3)}$$

The above equation is based on Timoshenko's solutions for plates on elastic foundations (Timoshenko, 1959). The general solution has been simplified by restriction to a center load. Only the first five values of m and n (in the infinite series) are required to achieve reasonable accuracy.

4.4.2.1.2 Method 1

For shallow bearing footings that are rigid with respect to the supporting soil, an uncoupled spring model, as shown in Figure 4-3(b), shall represent the foundation stiffness.

The equivalent spring constants shall be calculated as specified in Figure 4-4.

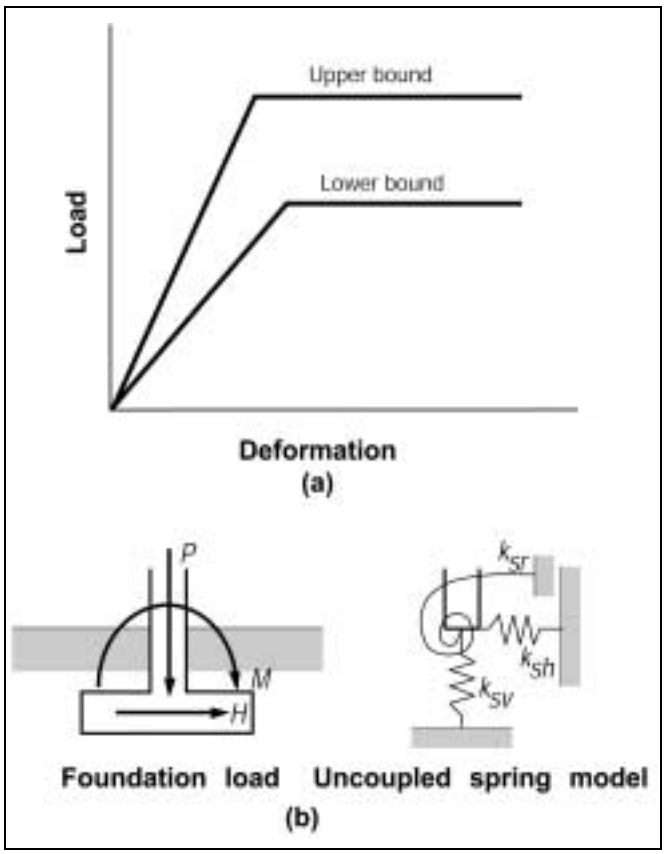
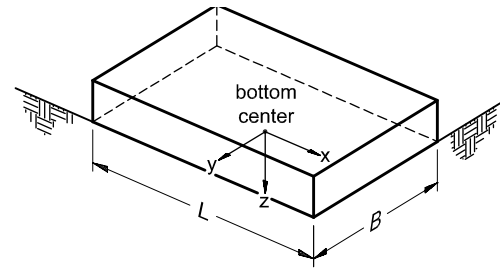


Figure 4-3 ***(a) Idealized Elasto-Plastic Load-Deformation Behavior for Soils (b) Uncoupled Spring Model for Rigid Footings***

Degree of Freedom	Stiffness of Foundation at Surface	Note
Translation along x-axis	$K_{x,\,sur} = \frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 1.2\right]$	
Translation along y-axis	$K_{y,\,sur} = \frac{GB}{2-\nu}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 0.4\frac{L}{B} + 0.8\right]$	
Translation along z-axis	$K_{z,\,sur} = \frac{GB}{1-\nu}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right]$	
Rocking about x-axis	$K_{xx,\,sur} = \frac{GB^3}{1-\nu}\left[0.4\left(\frac{L}{B}\right) + 0.1\right]$	
Rocking about y-axis	$K_{yy,\,sur} = \frac{GB^3}{1-\nu}\left[0.47\left(\frac{L}{B}\right)^{2.4} + 0.034\right]$	Orient axes such that $L \geq B$
Torsion about z-axis	$K_{zz,\,sur} = GB^3\left[0.53\left(\frac{L}{B}\right)^{2.45} + 0.51\right]$	

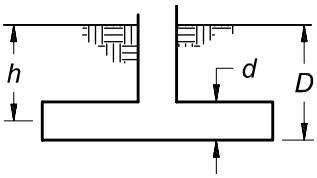
Degree of Freedom	Correction Factor for Embedment	Note
Translation along x-axis	$\beta_x = \left(1 + 0.21\sqrt{\frac{D}{B}}\right)\cdot\left[1 + 1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]$	
Translation along y-axis	$\beta_y = \beta_x$	
Translation along z-axis	$\beta_z = \left[1 + \frac{1}{21}\frac{D}{B}\left(2 + 2.6\frac{B}{L}\right)\right]\cdot\left[1 + 0.32\left(\frac{d(B+L)}{BL}\right)^{2/3}\right]$	d = height of effective sidewall contact (may be less than total foundation height)
Rocking about x-axis	$\beta_{xx} = 1 + 2.5\frac{d}{B}\left[1 + \frac{2d}{B}\left(\frac{d}{D}\right)^{-0.2}\sqrt{\frac{B}{L}}\right]$	h = depth to centroid of effective sidewall contact
Rocking about y-axis	$\beta_{yy} = 1 + 1.4\left(\frac{d}{L}\right)^{0.6}\left[1.5 + 3.7\left(\frac{d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.6}\right]$	For each degree of freedom, calculate
Torsion about z-axis	$\beta_{zz} = 1 + 2.6\left(1 + \frac{B}{L}\right)\left(\frac{d}{B}\right)^{0.9}$	$K_{emb} = \beta K_{sur}$

Figure 4-4 Elastic Solutions for Rigid Footing Spring Constraints

C4.4.2.1.2 Method 1

Researchers have developed spring stiffness solutions that are applicable to any solid basemat shape on the surface of, or partially or fully embedded in, a homogeneous halfspace (Gazetas). Rectangular foundations are most common in buildings. Therefore, the general spring stiffness solutions were adapted to the general rectangular foundation problem, which includes rectangular strip footings.

Using Figure 4-4, a two step calculation process is required. First, the stiffness terms are calculated for a foundation at the surface. Then, an embedment correction factor is calculated for each stiffness term. The stiffness of the embedded foundation is the product of these two terms. Figure C4-2 illustrates the effects of foundation aspect ratio and embedment.

According to Gazetas, the height of effective sidewall contact, *d*, should be taken as the average height of the sidewall that is in good contact with the surrounding soil. It should, in general, be smaller than the nominal height of contact to account for such phenomena as slippage and separation that may occur near the ground surface. Note that *d* will not necessarily attain a single value for all modes of oscillation. When *d* is taken larger than zero, the resulting stiffness includes sidewall friction and passive pressure contributions

Although frequency-dependent solutions are available, results are reasonably insensitive to loading frequencies within the range of parameters of interest for buildings subjected to earthquakes. It is sufficient to use static stiffnesses as representative of repeated loading conditions. Other formulations incorporating a wider range of variables may be found in Gazetas and Lam, et al.

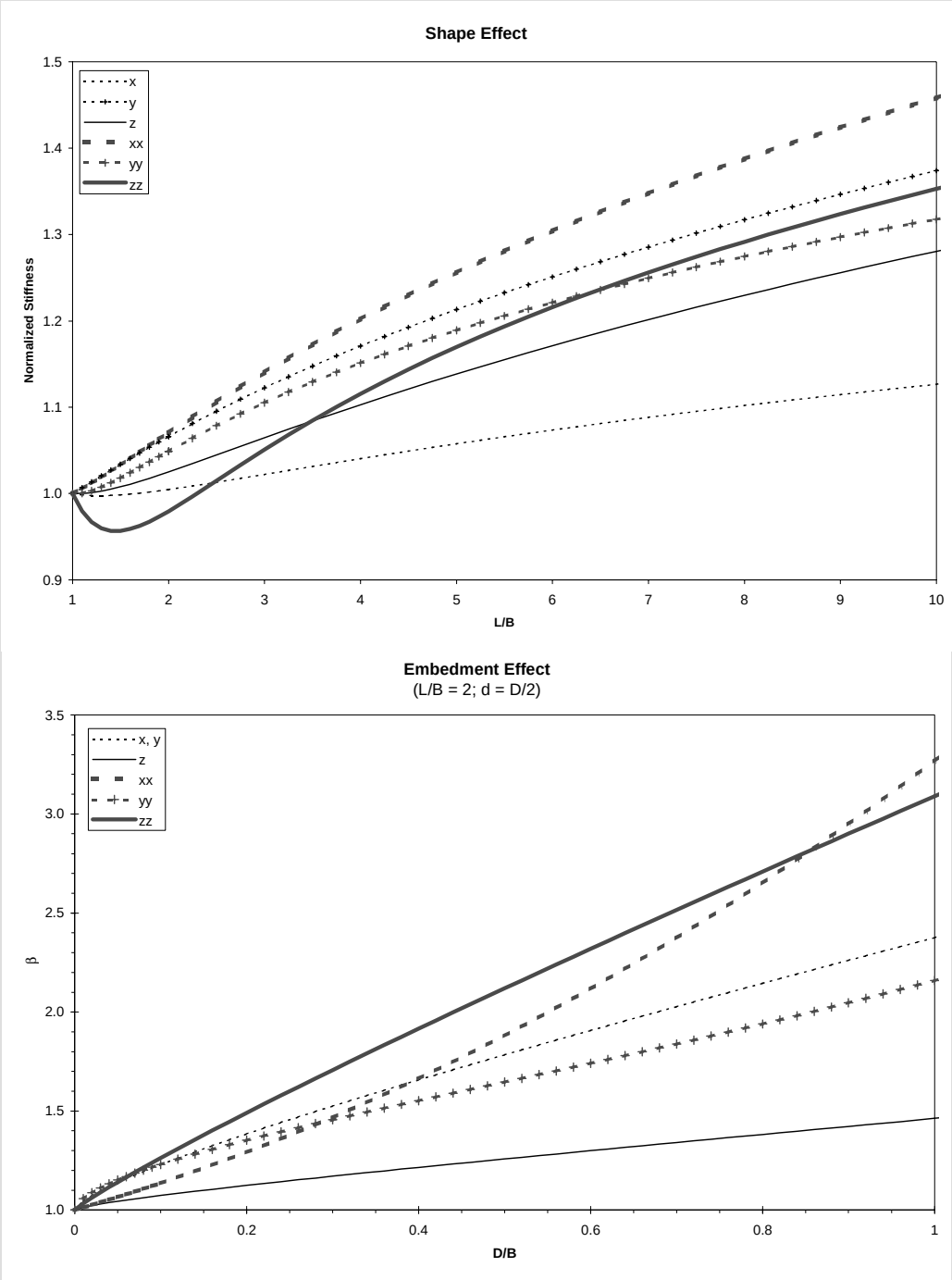


Figure C4-2 ***(a) Foundation Shape Effect***
(b) Foundation Embedment Effect

4.4.2.1.3 Method 2

For shallow bearing foundations that are not rigid with respect to the supporting soils, a finite element representation of linear or nonlinear foundation behavior using Winkler models shall be used. Distributed vertical stiffness properties shall be calculated by dividing the total vertical stiffness by the area. Uniformly distributed rotational stiffness properties shall be calculated by dividing the total rotational stiffness of the footing by the moment of inertia of the footing in the direction of loading. Vertical and rotational stiffnesses shall be decoupled for a Winkler model. It shall be permitted to use the procedure illustrated in Figure 4-5 to decouple these stiffnesses.

C4.4.2.1.3 Method 2

The stiffness per unit length in these end zones is based on the vertical stiffness of a B x B/6 isolated footing. The stiffness per unit length in the middle zone is equivalent to that of an infinitely long strip footing.

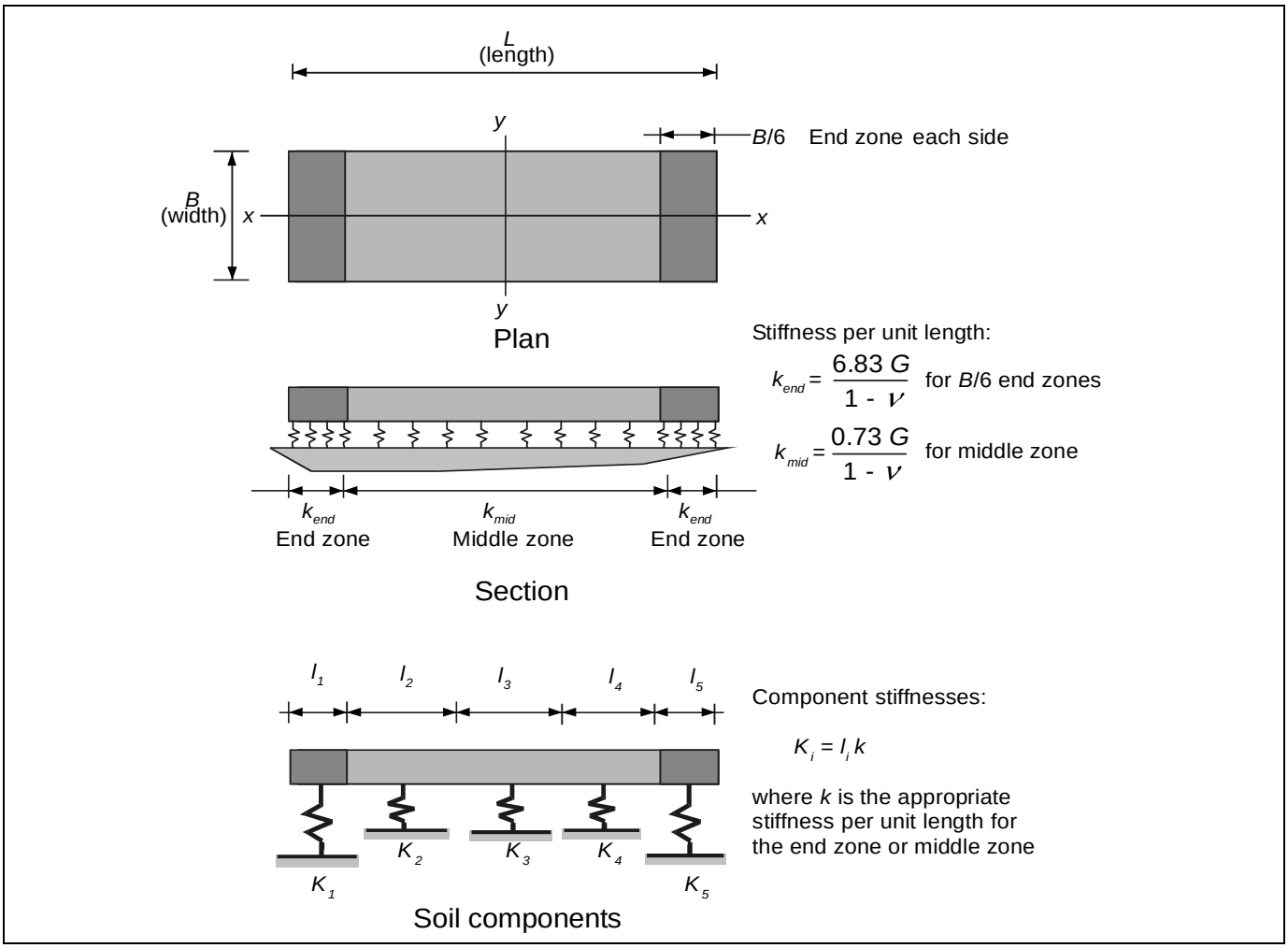


Figure 4-5 Vertical Stiffness Modeling for Shallow Bearing Footings

4.4.2.1.4 Method 3

For shallow bearing foundations that are flexible relative to the supporting soil, based on approved theoretical solutions for beams or plates on elastic supports, the foundation stiffness shall be permitted to be calculated by a decoupled Winkler model using a unit subgrade spring coefficient. For flexible foundation systems, the unit subgrade spring coefficient, k_{sv}, shall be calculated by Equation (4-6).

$$k_{sv} = \frac{1.3G}{B(1-\nu)} \tag{4-6}$$

where:

G = Shear modulus
B = Width of footing
ν = Poisson's ratio

4.4.2.1.5 Capacity Parameters

The vertical expected capacity of shallow bearing foundations shall be determined using the procedures of Section 4.4.1.

In the absence of moment loading, the expected vertical load capacity, Q_c, of a rectangular footing shall be calculated by Equation (4-7).

$$Q_c = q_c BL \tag{4-7}$$

where:

q_c = Expected bearing capacity determined in Section 4.4.1
B = Width of footing
L = Length of footing

The moment capacity of a rectangular footing shall be calculated by Equation (4-8):

$$M_c = \frac{LP}{2}\left(1 - \frac{q}{q_c}\right) \tag{4-8}$$

where:

P = Vertical load on footing
q = $\frac{P}{BL}$ = vertical bearing pressure
B = Width of footing (parallel to the axis of bending)
L = Length of footing in the direction of bending
q_c = Expected bearing capacity determined in Section 4.4.1

The lateral capacity of shallow foundations shall be calculated using established principles of soil mechanics and shall include the contributions of traction at the bottom and passive pressure resistance on the leading face. Mobilization of passive pressure shall be calculated using Figure 4-6.

C4.4.2.1.5 Capacity Parameters

For rigid footings subject to moment and vertical load, contact stresses become concentrated at footing edges, particularly as uplift occurs. The ultimate moment capacity, M_c, is dependent upon the ratio of the vertical load stress, q, to the expected bearing capacity, q_c. Assuming that contact stresses are proportional to vertical displacement and remain elastic up to the expected bearing capacity, q_c, it can be shown that uplift will occur prior to plastic yielding of the soil when q/q_c is less than 0.5. If q/q_c is greater than 0.5, then the soil at the toe will yield prior to uplift. This is illustrated in Figure C4-3.

For footings subjected to lateral loads, the base traction strength is given by $V = C + N\mu$; where C is the effective cohesion force (effective cohesion stress, c, times footing base area), N is the normal (compressive) force and μ is the coefficient of friction. If included, side traction is calculated in a similar manner. The coefficient of friction is often specified by the geotechnical consultant. In the absence of such a recommendation, μ may be based on the minimum of the effective internal friction angle of the soil and the friction coefficient between soil and foundation from published foundation references. The ultimate passive pressure strength is often specified by the geotechnical consultant in the form of passive pressure coefficients or equivalent fluid pressures. The passive pressure problem has been extensively investigated for more than 200 years. As a result, countless solutions and recommendations exist. The method used should, at a minimum, include the contributions of internal friction and cohesion, as appropriate.

As shown in Figure 4-6, the force-displacement response associated with passive pressure resistance is highly nonlinear. However, for shallow foundations, passive pressure resistance generally accounts for much less than half of the total strength. Therefore, it is adequate to characterize the nonlinear response of shallow foundations as elastic-perfectly plastic using the initial, effective stiffness and the total expected strength. The actual behavior is expected to fall within the upper and lower bounds prescribed in this standard.

4.4.2.2 Pile Foundations

A pile foundation shall be defined as a deep foundation system composed of one or more driven or cast-in-place piles and a pile cap cast-in-place over the piles, which together form a pile group supporting one or more load-bearing columns, or a linear sequence of pile groups supporting a shear wall.

The requirements of this section shall apply to piles less than or equal to 24 inches in diameter. The stiffness characteristics of single large-diameter piles or drilled shafts larger than 24 inches in diameter shall comply with the requirements of Section 4.4.2.3.

4.4.2.2.1 Stiffness Parameters

The uncoupled spring model shown in Figure 4-3(b) shall be used to represent the stiffness of a pile foundation where the footing in the figure represents the pile cap. In calculating the vertical and rocking springs, the contribution of the soil immediately beneath the pile cap shall be neglected. The total lateral stiffness of a pile group shall include the contributions of the piles (with an appropriate modification for group effects) and the passive resistance of the pile cap. The lateral stiffness of piles shall be based on classical methods or on analytical solutions using approved beam-column pile models. The lateral stiffness contribution of the pile cap shall be calculated using the passive pressure mobilization curve in Figure 4-6.

Pile group axial spring stiffness values, k_{sv}, shall be calculated using Equation (4-9).

$$k_{sv} = \sum_{n=1}^{N} \frac{A\,E}{L} \qquad (4\text{-}9)$$

where:

A = Cross-sectional area of a pile
E = Modulus of elasticity of piles
L = Length of piles
N = Number of piles in group

The rocking spring stiffness values about each horizontal pile cap axis shall be computed by modeling each pile axial spring as a discrete Winkler spring. The rotational spring constant, k_{sr} , (moment per unit rotation) shall be calculated using Equation (4-10).

$$k_{sr} = \sum_{n=1}^{N} k_{vn} S_n^2 \qquad (4\text{-}10)$$

where:

k_{vn} = Axial stiffness of the nth pile
S_n = Distance between nth pile and axis of rotation.

C4.4.2.2.1 Stiffness Parameters

As the passive pressure resistance may be a significant part of the total strength, and deep foundations often require larger lateral displacements than shallow foundations to mobilize the expected strength, it may not be appropriate to base the force-displacement response on the initial, effective stiffness alone. Instead, the contribution of passive pressure should be based on the passive pressure mobilization curve provided in Figure 4-6.

Although the effects of group action and the influence of pile batter are not directly accounted for in the form of the above equations, it can be reasonably assumed that the latter effects are accounted for in the range of uncertainties that must be considered in accordance with Section 4.4.1.

4.4.2.2.2 Capacity Parameters

The axial load capacity of piles in compression and tension shall be determined using the procedures in Section 4.4.1. The axial capacity in tension shall not exceed the tensile load capacity of the pile cap and splice connections.

The moment capacity of a pile group shall be determined assuming a rigid pile cap. Lower-bound moment capacity shall be based on triangular distribution of axial pile loading and lower-bound axial capacity of the piles. Upper-bound moment capacity shall be based on a rectangular distribution of axial pile load using full, upper-bound axial capacity of the piles.

The lateral capacity of a pile group shall include the contributions of the piles (with an appropriate modification for group effects) and the passive resistance of the pile cap. The lateral capacity of the piles shall be calculated using the same method used to calculate the stiffness. The lateral capacity of the pile cap, due to passive pressure, shall be calculated using established principles of soil mechanics. Passive pressure mobilization shall be calculated using Figure 4-6.

C4.4.2.2.2 Capacity Parameters

The lateral capacity of a pile cap should be calculated in the same way that the capacity of a shallow foundation is computed, except that the contribution of base traction should be neglected. Section C4.4.2.1.5 provides a more detailed description of the calculation procedure.

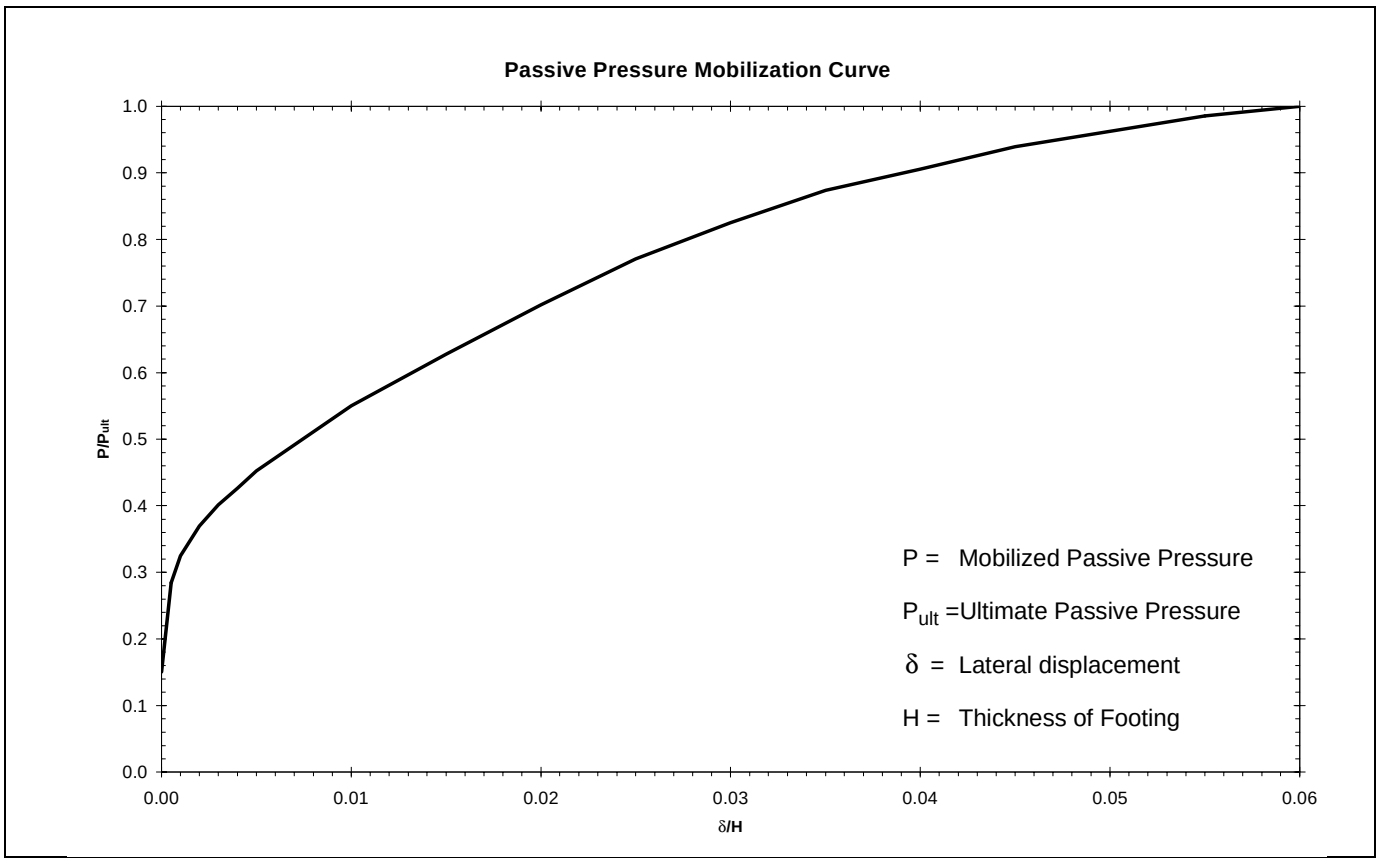


Figure 4-6 Passive Pressure Mobilization Curve

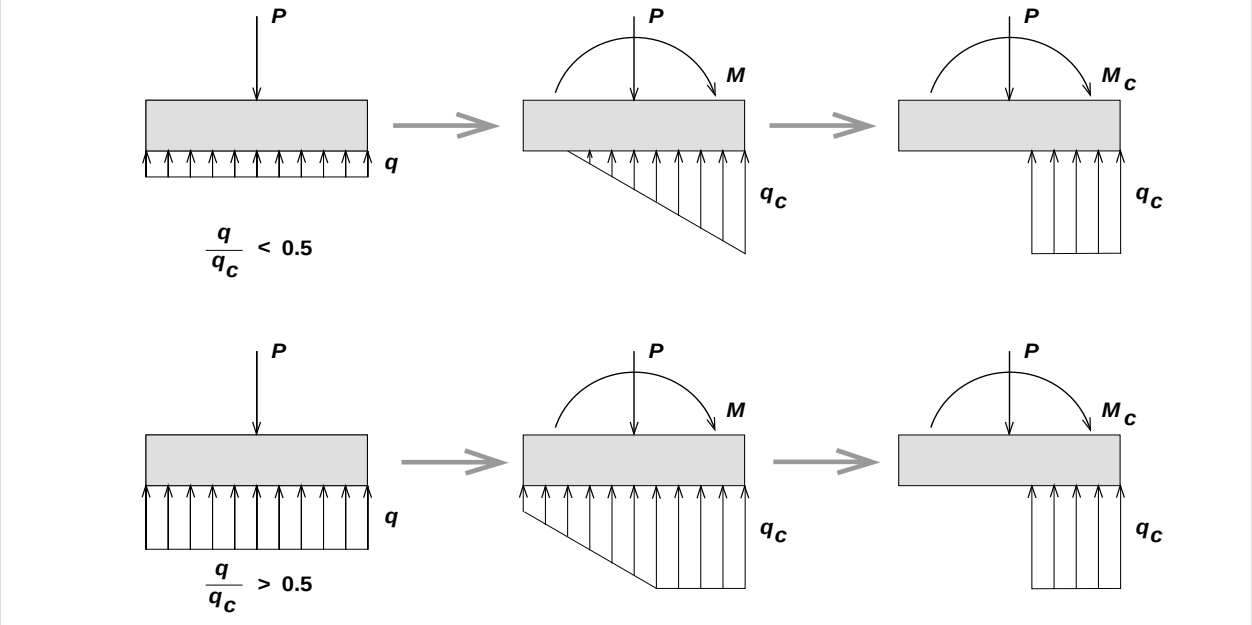


Figure C4-3 Idealized Concentration of Stress at Edge of Rigid Footings Subjected to Overturning Moment

4.4.2.3 Drilled Shafts

The stiffness and capacity of drilled shaft foundations and piers of diameter less than or equal to 24 inches shall be calculated using the requirements for pile foundations specified in 4.4.2.2. For drilled shaft foundations and piers of diameter greater than 24 inches, the capacity shall be calculated based on the interaction of the soil and shaft where the soil shall be represented using Winkler type models specified in Section 4.4.2.2.

C4.4.2.3 Drilled Shafts

When the diameter of the shaft becomes large (> 24 inches), the bending and the lateral stiffness and strength of the shaft itself may contribute to the overall capacity. This is obviously necessary for the case of individual shafts supporting isolated columns.

4.4.3 Foundation Acceptability Criteria

The foundation soil shall comply with the acceptance criteria specified in this section. The structural components of foundations shall meet the appropriate requirements of Chapters 5 through 8. The foundation soil shall be evaluated to support all actions, including vertical loads, moments, and lateral forces applied to the soil by the foundation.

4.4.3.1 Simplified Rehabilitation

The foundation soil of buildings for which the Simplified Rehabilitation Method is selected in accordance with Section 2.3.1 shall comply with the requirements of Chapter 10.

4.4.3.2 Linear Procedures

The acceptance criteria for foundation soil analyzed by linear procedures shall be based on the modeling assumptions for the base of the structure specified in Section 4.4.3.2.1 or 4.4.3.2.2.

4.4.3.2.1 Fixed Base Assumption

If the base of the structure is assumed to be completely rigid, the geotechnical components shall be classified as deformation-controlled. Component actions shall be determined by Equation (3-18). Acceptance criteria shall be based on Equation (3-20), *m*-factors for geotechnical components shall not exceed 3, and the use of upper-bound component capacities shall be permitted. A fixed base assumption shall not be used for buildings being rehabilitated to the Immediate Occupancy Performance Level that are sensitive to base rotations or other types of foundation movement.

If the alternative overturning method described in Section 3.2.10.1 is used, the geotechnical components shall be classified as force-controlled. Component actions shall not exceed the calculated capacities and upper-bound component capacities shall not be used.

4.4.3.2.2 Flexible Base Assumption

If the base of the structure is assumed to be flexible and modeled using linear geotechnical components, then the geotechnical components shall be classified as deformation-controlled. Component actions shall be determined by Equation (3-18). Soil strength need not be evaluated. Acceptability of soil displacements shall be based on the ability of the structure to accommodate these displacements within the acceptance criteria for the selected Rehabilitation Objective.

4.4.3.3 Nonlinear Procedures

The acceptance criteria for geotechnical components analyzed by nonlinear procedures shall be based on the modeling assumptions for the base of the structure specified in Section 4.4.3.3.1 or 4.4.3.3.2.

4.4.3.3.1 Fixed Base Assumption

If the base of the structure is assumed to be completely rigid, then the base reactions for all geotechnical components shall be classified as force-controlled, as determined by Equation (3-19), and shall not exceed upper-bound component capacities. A fixed base assumption shall not be used for buildings being rehabilitated for the Immediate Occupancy Performance Level that are sensitive to base rotations or other types of foundation movement.

4.4.3.3.2 Flexible Base Assumption

If the base of the structure is assumed to be flexible and modeled using flexible nonlinear geotechnical components, then the geotechnical components shall be classified as deformation-controlled and the displacements at the base of the structure shall not exceed the acceptance criteria of this section. For the Life Safety and Collapse Prevention Structural Performance Levels, acceptability of soil displacements shall be based on the ability of the structure to accommodate these displacements within the acceptance criteria for the selected Rehabilitation Objective. For the Immediate Occupancy Structural Performance Level, the permanent, nonrecoverable displacement of the geotechnical components shall be calculated by an approved method based on the maximum total displacement, foundation and soil type, thickness of soil layers, and other pertinent factors. The acceptability of these displacements shall be based upon the ability of the structure to accommodate them within the acceptance criteria for the Immediate Occupancy Structural Performance Level.

4.5 Seismic Earth Pressure

Building walls retaining soil shall be evaluated to resist additional earth pressure due to seismic forces. Unless otherwise determined from a site-specific geotechnical investigation, the seismic earth pressure acting on a building wall retaining nonsaturated, level soil above the ground water table shall be calculated using Equation (4-11):

$$\Delta p = 0.4 k_h \gamma_t H_{rw} \tag{4-11}$$

where:

Δp	=	Additional earth pressure due to seismic shaking, which is assumed to be a uniform pressure
k_h	=	Horizontal seismic coefficient in the soil, which may be assumed equal to $S_{XS}/2.5$
γ_t	=	Total unit weight of soil
H_{rw}	=	Height of the retaining wall
S_{XS}	=	Spectral response acceleration parameter as specified in Section 1.6

The seismic earth pressure shall be added to the unfactored static earth pressure to obtain the total earth pressure on the wall. The wall shall be evaluated as a force-controlled component using acceptance criteria based on the type of wall construction and approved methods.

C4.5 Seismic Earth Pressure

Past earthquakes have not caused extensive damage to building walls below grade. In some cases, however, it is advisable to verify the adequacy of retaining walls to resist increased pressure due to seismic loading. These situations include walls of poor construction quality, unreinforced or lightly reinforced walls, walls of archaic materials, unusually tall or thin walls, damaged walls, or other conditions implying a sensitivity to increased loads.

The expression in Equation (4-11) is a simplified approximation of the Mononobe-Okabe formulation. The actual magnitude and distribution of pressure on walls during earthquakes is very complex. If walls do not have the apparent capacity to resist the pressures estimated from the above approximate procedures, detailed investigation by a qualified geotechnical engineer is recommended.

Seismic earth pressures calculated in accordance with this section are intended for use in checking acceptability of local wall components and should not be used to increase total base shear on the building.

4.6 Foundation Rehabilitation

Foundation rehabilitation schemes shall be evaluated in conjunction with any rehabilitation of the superstructure and according to the general principles and requirements of this standard to assure that the complete rehabilitation achieves the selected building performance level for the selected earthquake hazard level. When new rehabilitation elements are used in conjunction with existing elements, the effects of differential foundation stiffness on the modified structure shall be demonstrated to meet the acceptance criteria. If existing loads are not redistributed to all the elements of rehabilitated foundation by shoring and/or jacking, the effects of differential strengths and stiffnesses among individual foundation elements shall be included in the analysis of the rehabilitated foundation. The effects of rehabilitation on stiffness, strength, and deformability shall be taken into account in an analytical model of the rehabilitated structure. The compatibility of new and existing components and/or elements shall be checked at displacements consistent with the performance level chosen.

C4.6 Foundation Rehabilitation

Guidance for modification of foundations to improve seismic performance is provided below.

1. Soil Material Improvements. Improvement in existing soil materials may be effective in the rehabilitation of foundations by achieving one or more of the following results: (a) improvement in vertical bearing capacity of footing foundations, (b) increase in the lateral frictional resistance at the base of footings, (c) and increase in the passive resistance of the soils adjacent to foundations or grade beams.

Soil improvement options to increase the vertical bearing capacity of footing foundations are limited. Soil removal and replacement and soil vibratory densification usually are not feasible because they would induce settlements beneath the footings or be expensive to implement without causing settlement. Grouting may be considered to increase bearing capacity. Different grouting techniques are discussed in *FEMA 274* Section C4.3.2. Compaction grouting can achieve densification and strengthening of a variety of soil types and/or extend foundation loads to deeper, stronger soils. The technique requires careful control to avoid causing uplift of foundation elements or adjacent floor slabs during the grouting process. Permeation grouting with chemical grouts can achieve substantial strengthening of sandy soils, but the more fine-grained or silty the sand, the less effective the technique becomes. Jet grouting could also be considered. These same techniques also may be considered to increase the lateral frictional resistance at the base of footings.

Soil improvement by the following methods may be effective in increasing the passive resistance of soils adjacent to foundations or grade beams; removal and replacement of existing soils with stronger, well-compacted soils or with treated (e.g., cement-stabilized) soils; in-place mixing of existing soils with strengthening materials (e.g., cement); grouting, including permeation grouting and jet grouting; and in-place densification by impact or vibratory compaction. In-place densification by impact or vibratory compaction should be used only if the soil layers to be compacted are not too thick and vibration effects on the structure are tolerable.

2. Shallow Foundation Rehabilitation. The following measures may be effective in the rehabilitation of shallow foundations:

2.1 New isolated or spread footings may be added to existing structures to support new structural elements such as shear walls or frames.

2.2 Existing isolated or spread footings may be enlarged to increase bearing or uplift capacity. Consideration of existing contact pressures on the strength and stiffness of the modified footing may be required unless uniform distribution is achieved by shoring and/or jacking.

2.3 Existing isolated or spread footings may be underpinned to increase bearing or uplift capacity. Underpinning improves bearing capacity by lowering the contact horizon of the footing. Consideration of the effects of jacking and load transfer may be required.

2.4 Uplift capacity may be improved by increasing the resisting soil mass above the footing.

2.5 Mitigation of differential lateral displacement of different portions of a building foundation may be carried out by provision of interconnection with grade beams, reinforced grade slab or ties.

3. Deep Foundation Rehabilitation. The following measures may be effective in the rehabilitation of deep foundation consisting of driven piles made of steel, concrete, or wood, or cast-in-place concrete piers, or drilled shafts of concrete.

3.1 Shallow foundation of spread footings or mats may be provided to support new shear walls or frames or other new elements of the lateral force-resisting system, provided the effects of differential foundation stiffness on the modified structure are analyzed and meet the acceptance criteria.

3.2 New wood piles may be provided for an existing wood pile foundation. A positive connection should be provided to transfer the uplift forces from the pile cap or foundation above to the new wood piles. Existing wood piles should be inspected for deterioration caused by decay, insect infestation, or other signs of distress prior to undertaking evaluation of existing wood pile foundation.

3.3 Driven piles made of steel, concrete, or wood, or cast-in-place concrete piers or drilled shafts of concrete, may be provided to support new structural elements such as shear walls or frames.

3.4 Driven piles made of steel, concrete, or wood, or cast-in-place concrete piers or drilled shafts of concrete, may be provided to supplement the vertical and lateral capacities of existing pile and pier foundation groups.