4. Foundations and Geotechnical Hazards (Systematic Rehabilitation)

4.1 Scope

This chapter provides geotechnical engineering guidance regarding 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 (Chapter 5), Concrete (Chapter 6), Masonry (Chapter 7), and Wood (Chapter 8).

This chapter describes rehabilitation measures for foundations and geotechnical site hazards. Section 4.2 provides guidelines for establishing site soil characteristics and identifying geotechnical site hazards, including fault rupture, liquefaction, differential compaction, landslide and rock fall, and flooding. Techniques for mitigating these geotechnical site hazards are described in Section 4.3. Section 4.4 presents criteria for establishing soil strength capacity, stiffness, and soil-structure interaction (SSI) parameters for making foundation design evaluations. Retaining walls are discussed in Section 4.5. Section 4.6 contains guidelines for improving or strengthening foundations.

4.2 Site Characterization

The geotechnical requirements for buildings suitable for Simplified Rehabilitation are described in Chapter 10. For all other buildings, specific geotechnical site characterization consistent with the selected method of Systematic Rehabilitation is required. Site characterization consists of the compilation of information on site subsurface soil conditions, configuration and loading of existing building foundations, and seismic-geologic site hazards.

In the case of historic buildings, 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 Soil Information

Specific information describing the foundation conditions of the building to be rehabilitated is required. Useful information also can be gained from knowledge of the foundations of adjacent or nearby buildings. Foundation information may include subsurface soil and ground water data, configuration of the foundation system, design foundation loads, and load-deformation characteristics of the foundation soils.

4.2.1.1 Site Foundation Conditions

Subsurface soil conditions must be defined in sufficient detail to assess the ultimate capacity of the foundation and to determine if the site is susceptible to seismic-geologic hazards.

Information regarding the structural foundation type, dimensions, and material are required irrespective of the subsurface soil conditions. This information includes:

- Foundation type—spread footings, mat foundation, piles, drilled shafts.
- Foundation dimensions—plan dimensions and locations. For piles, tip elevations, vertical variations (tapered sections of piles or belled caissons).
- Material composition/construction. For piles, type (concrete/steel/wood), and installation method (cast-in-place, open/closed-end driving).

Subsurface conditions shall be determined for the selected Performance Level as follows.

A. Collapse Prevention and Life Safety Performance Levels

Determine type, composition, consistency, relative density, and layering of soils to a depth at which the stress imposed by the building is approximately 10% of the building weight divided by the total foundation area.

Determine the location of the water table and its seasonal fluctuations beneath the building.

B. Enhanced Rehabilitation Objectives and/or Deep Foundations

For each soil type, determine soil unit weight γ, soil shear strength c, soil friction angle ϕ, soil compressibility characteristics, soil shear modulus G, and Poisson's ratio ν.

4.2.1.2 Nearby Foundation 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 being rehabilitated. Adjacent sites where construction has been done recently may provide a guide for evaluation of subsurface conditions at the site being considered.

4.2.1.3 Design Foundation Loads

Information on the design foundation loads is required, as well as actual dead loads and realistic estimates of live loads.

4.2.1.4 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 soils and foundation reports for existing buildings. Load-deformation relationships are discussed in detail in Section 4.4.

4.2.2 Seismic Site Hazards

In addition to ground shaking, seismic hazards include surface fault rupture, liquefaction, differential compaction, landsliding, and flooding. The potential for ground displacement hazards at a site should be evaluated. The evaluation should include an assessment of the hazards in terms of ground movement. If consequences are unacceptable for the desired Performance Level, then the hazards should be mitigated as described in Section 4.3.

4.2.2.1 Fault Rupture

Geologic site conditions must be defined in sufficient detail to assess the potential for the trace of an active fault to be present in the building foundation soils. If the trace of a fault is known or suspected to be present, the following information may be required:

- The degree of activity—that is, the age of most recent movement (e.g., historic, Holocene, late Quaternary)—must be determined.
- The fault type must be identified, whether strike-slip, normal-slip, reverse-slip, or thrust fault.
- The sense of slip with respect to building geometry must be determined, particularly for normal-slip and reverse-slip faults.
- Magnitudes of vertical and/or horizontal displacements with recurrence intervals consistent with Rehabilitation Objectives must be determined.
- The width of the fault-rupture zone (concentrated in a narrow zone or distributed) must be identified.

4.2.2.2 Liquefaction

Subsurface soil and ground water conditions must be defined in sufficient detail to assess the potential for liquefiable materials to be present in the building foundation soils. If liquefiable soils are suspected to be present, the following information must be developed.

- **Soil type:** Liquefiable soils typically are granular (sand, silty sand, nonplastic silt).
- **Soil density:** Liquefiable soils are loose to medium dense.
- **Depth to water table:** Liquefiable soils must be saturated, but seasonal fluctuations of the water table must be estimated.
- **Ground surface slope and proximity of free-face conditions:** Lateral-spread landslides can occur on gently sloping sites, particularly if a free-face condition—such as a canal or stream channel—is present nearby.
- **Lateral and vertical differential displacement:** Amount and direction at the building foundation must be calculated.

The hazard of liquefaction should be evaluated initially to ascertain whether the site is clearly free of a hazardous condition or whether a more detailed evaluation is required. It can be assumed generally that a significant hazard due to liquefaction does not exist at a site 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:

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

Table 4-1 Estimated Susceptibility to Liquefaction of Surficial Deposits During Strong Ground Shaking (after Youd and Perkins, 1978)

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	?	?
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	—	—	—

- The soils underlying the site are stiff clays or clayey silts, unless the soils are highly sensitive, based on local experience; or, the soils are cohesionless (i.e., sand, silts, or gravels) with a minimum normalized Standard Penetration Test (SPT) resistance, $(N_1)_{60}$, value of 30 blows/foot for depths below the groundwater table, or with clay content greater than 20%. The parameter $(N_1)_{60}$ is defined as the SPT blow count normalized to an effective overburden pressure of 2 ksf. Clay has soil particles with nominal diameters $\leq$ 0.005 mm.

- The groundwater 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, by applying the above criteria, a possible liquefaction hazard at the site cannot be eliminated, then a more detailed evaluation is required. Guidance for detailed evaluations is presented in the *Commentary.*

4.2.2.3 Differential Compaction

Subsurface soil conditions must be defined in sufficient detail to assess the potential for differential compaction to occur in the building foundation soils.

Differential compaction or densification of soils may accompany strong ground shaking. The resulting differential settlements can be damaging to structures. Types of soil that are 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 groundwater table.

It can generally be assumed that a significant hazard due to differential compaction does not exist if the soil conditions meet both of the following criteria:

- The geologic materials underlying foundations and below the groundwater table do not pose a significant liquefaction hazard, based on the criteria in Section 4.2.2.2.

- The geologic materials underlying foundations and above the groundwater 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 $(N_1)_{60}$ of 20 blows/0.3 m (20 blows/foot).

If a possible differential compaction hazard at the site cannot be eliminated by applying the above criteria, then a more detailed evaluation is required. Guidance for a detailed evaluation is presented in the *Commentary.*

4.2.2.4 Landsliding

Subsurface soil conditions must be defined in sufficient detail to assess the potential for a landslide to cause differential movement of the building foundation soils. Hillside stability shall be evaluated at sites with:

- Existing slopes exceeding approximately 18 degrees (three horizontal to one vertical)

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

Pseudo-static analyses shall be used to determine site stability, provided the soils are not liquefiable or otherwise expected to lose shear strength during deformation. Pseudo-static analyses shall use a seismic coefficient equal to one-half the peak ground acceleration (calculated as $S_{XS}/2.5$) at the site associated with the desired 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.

Sites with a static factor of safety of less than 1.0 will require a sliding-block displacement analysis (Newmark, 1965). The displacement analysis shall determine the magnitude of potential ground movement for use by the structural engineer in determining its effect upon the performance of the structure and the structure’s ability to meet the desired Performance Level. Where the structural performance cannot accommodate the computed ground displacements, appropriate mitigation schemes shall be employed as described in Section 4.3.4.

In addition to potential effects of landslides on foundation soils, the possible effects of rock fall or slide debris from adjacent slopes should be considered.

4.2.2.5 Flooding or Inundation

For Performance Levels exceeding Life Safety, site conditions should be defined in sufficient detail to assess the potential for earthquake-induced flooding or inundation to prevent the rehabilitated building from meeting the desired Performance Level. Sources of earthquake-induced flooding or inundation include:

- Dams located upstream damaged by earthquake shaking or fault rupture
- Pipelines, aqueducts, and water-storage tanks located upstream damaged by fault rupture, earthquake-induced landslides, or strong shaking
- Low-lying coastal areas within tsunami zones or areas adjacent to bays or lakes that may be subject to seiche waves
- Low-lying areas with shallow ground water where regional subsidence could cause surface ponding of water, resulting in inundation of the site

Potential damage to buildings from flooding or inundation must be evaluated on a site-specific basis. Consideration must be given to potential scour of building foundation soils from swiftly flowing water.

4.3 Mitigation of Seismic 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 below 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.

4.3.1 Fault Rupture

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. Mitigative techniques include modifications to the structure or its foundation to distribute the effects of differential vertical movement over a greater horizontal distance to reduce angular distortion.

4.3.2 Liquefaction

The effectiveness of mitigating liquefaction hazards must be evaluated by the structural engineer in the context of the global building system performance. If it has been determined that liquefaction is likely to occur and the consequences in terms of estimated horizontal and vertical displacements are unacceptable for the desired Performance Level, then three general types of mitigating measures can be considered alone or in combination.

Modify the structure: The structure can be strengthened to improve resistance against the predicted liquefaction-induced ground deformation. This solution may be feasible for small ground deformations.

Modify the foundation: The foundation system can be modified to reduce or eliminate the potential for large foundation displacements; for example, by underpinning existing shallow foundations to achieve bearing on deeper, nonliquefiable strata. Alternatively (or in concert with the use of deep foundations), a shallow foundation system can be made more rigid (for example, by a system of grade beams between isolated footings) in order to reduce the differential ground movements transmitted to the structure.

Modify the soil conditions: A number of types of ground improvement can be considered to reduce or eliminate the potential for liquefaction and its effects. Techniques that generally are potentially applicable to existing buildings include soil grouting, either throughout the entire liquefiable strata beneath a building, or locally beneath foundation elements (e.g., grouted soil columns); installation of drains (e.g., stone columns); and installation of permanent dewatering systems. Other types of ground improvement that are 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.

If potential for significant liquefaction-induced lateral spreading movements exists at a site, then the remediation 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. Thus, measures to prevent lateral spreading may, in some cases, require stabilizing large soil volumes and/or constructing buttressing structures that can reduce the potential for, or the amount of, lateral movements.

4.3.3 Differential Compaction

The effectiveness of mitigating differential compaction hazards must be evaluated by the structural engineer in the context of the global building system performance. For cases of predicted significant differential settlements of a building foundation, mitigation options are similar to those described above to mitigate liquefaction hazards. There are three options: designing for the ground movements, strengthening the foundation system, and improving the soil conditions.

4.3.4 Landslide

The effectiveness of mitigating landslide hazards must be evaluated by the structural engineer in the context of the global building system performance. A number of schemes are available for reducing potential impacts for earthquake-induced landslides, including:

- Regrading
- Drainage
- Buttressing
- Structural Improvements
  - Gravity walls
  - Tieback/soil nail walls
  - Mechanically stabilized earth walls
  - Barriers for debris torrents or rock fall
  - Building strengthening to resist deformation
    - Grade beams
    - Shear walls
- Soil Modification/Replacement
  - Grouting
  - Densification

The effectiveness of any of these schemes must be considered based upon the amount of ground movement that the building can tolerate and still meet the desired Performance Level.

4.3.5 Flooding or Inundation

The effectiveness of mitigating flooding or inundation hazards must be evaluated by the structural engineer in the context of the global building system performance. Potential damage caused by earthquake-induced flooding or inundation may be mitigated by a number of schemes, as follows:

- Improvement of nearby dam, pipeline, or aqueduct facilities independent of the rehabilitated building
- Diversion of anticipated peak flood flows
- Installation of pavement around the building to minimize scour
- Construction of sea wall or breakwater for tsunami or seiche protection

4.4 Foundation Strength and Stiffness

It is assumed in this section that the foundation soils are not susceptible to significant strength loss due to earthquake loading. With this assumption, the following paragraphs provide an overview of the requirements and procedures for evaluating the ability of foundations to withstand the imposed seismic loads without excessive deformations. If soils are susceptible to significant strength loss, due to either the direct effects of the earthquake shaking on the soil or the foundation loading on the soil induced by the earthquake, then either improvement of the soil foundation condition should be considered or special analyses should be carried out to demonstrate that the effects of soil strength loss do not result in excessive structural deformations.

Consideration of foundation behavior is only one part of seismic rehabilitation of buildings. Selection of the desired Rehabilitation Objective probably will be done without regard to specific details of the building, including the foundation. The structural engineer will choose the appropriate type of analysis procedures for the selected Performance Level (e.g., Systematic Rehabilitation, with Linear Static or Dynamic Procedures, or Nonlinear Static or Dynamic Procedures). As stated previously, foundation requirements for buildings that qualify for Simplified Rehabilitation are included in Chapter 10.

4.4.1 Ultimate Bearing Capacities and Load Capacities

The ultimate load capacity of foundation components may be determined by one of the three methods specified below. The choice of method depends on the completeness of available information on foundation properties (see Section 4.2.1.1) and the requirements of the selected Performance Level.

4.4.1.1 Presumptive Ultimate Capacities

Presumptive capacities are to be used when the amount of information on foundation soil properties is limited and relatively simple analysis procedures are used. Presumptive ultimate load parameters for spread footings and mats are presented in Table 4-2.

4.4.1.2 Prescriptive Ultimate Capacities

Prescriptive capacities may be used when either construction documents for the existing building or previous geotechnical reports provide information on foundation soils design parameters.

The ultimate prescriptive bearing pressure for a spread footing may be assumed to be twice the allowable dead plus live load bearing pressure specified for design.

$$q_c = 2q_{allow.D+L} \tag{4-1}$$

Table 4-2 Presumptive Ultimate Foundation Pressures

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 may 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 ultimate foundation pressure are for footings having a minimum width of 12 inches and a minimum depth of 12 inches into natural grade. Except where Footnote 7 below applies, increase of 20% allowed for each additional foot of width or depth to a maximum value of three times the designated value.
4. May 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 the dead load.
7. No increase for width is allowed.

For deep foundations, the ultimate prescriptive vertical capacity of individual piles or piers may be assumed to be 50% greater than the allowable dead plus live loads specified for design.

$$Q_c = 1.5 Q_{allow.D+L} \tag{4-2}$$

As an alternative, the prescriptive ultimate capacity of any footing component may be assumed to be 50% greater than the total working load acting on the component, based on analyses using the original design requirements.

$$Q_c = 1.5 Q_{max.} \tag{4-3}$$

where $Q_{max.} = Q_D + Q_L + Q_S$

4.4.1.3 Site-Specific Capacities

A detailed analysis may be conducted by a qualified geotechnical engineer to determine ultimate foundation capacities based on the specific characteristics of the building site.

4.4.2 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.

Foundation systems for buildings can in some cases be complex, but for the purpose of simplicity, three foundation types are considered in these *Guidelines*:

- shallow bearing foundations
- pile foundations
- drilled shafts

While it is recognized that the load-deformation behavior of foundations is nonlinear, because of the difficulties in determining soil properties and static foundation loads for existing buildings, together with the likely variability of soils supporting foundations, an equivalent elasto-plastic representation of load-deformation behavior is recommended. In addition, to allow for such variability or uncertainty, an upper and lower bound approach to defining stiffness and capacity is recommended (as shown in Figure 4-1a) to permit evaluation of structural response sensitivity. The selection of uncertainty represented by the upper and lower bounds should be determined jointly by the geotechnical and structural engineers.

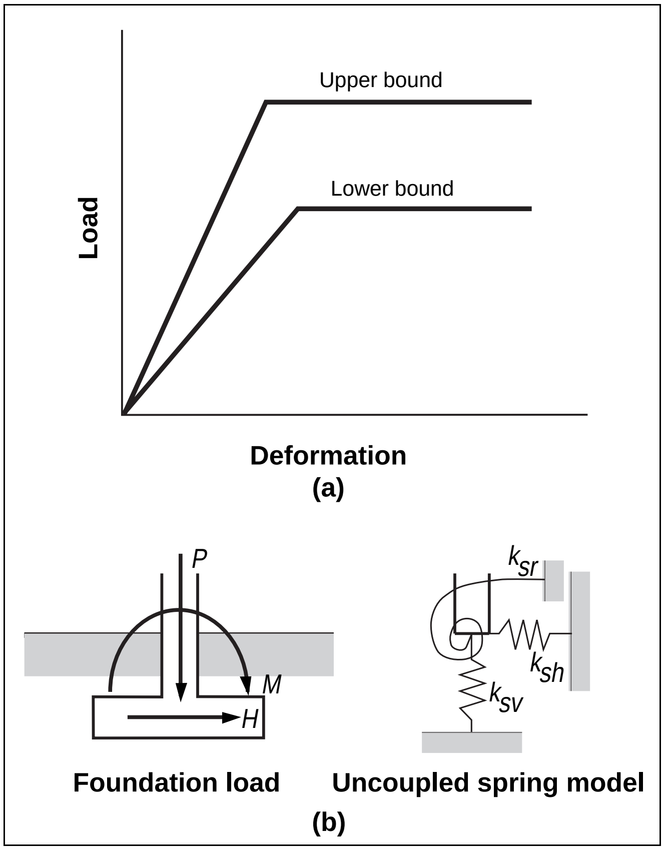


Figure 4-1 (a) Idealized Elasto-Plastic Load-Deformation Behavior for Soils (b) Uncoupled Spring Model for Rigid Footings

4.4.2.1 Shallow Bearing Foundations

A. Stiffness Parameters

The shear modulus, G, for a soil is related to the modulus of elasticity, E, and Poisson's ratio, ν, by the relationship

$$G = \frac{E}{2(1+\nu)} \tag{4-4}$$

Poisson's ratio may be assumed as 0.35 for unsaturated soils and 0.50 for saturated soils.

The initial shear modulus, G_o, is related to the shear wave velocity at low strains, v_s, and the mass density of the soil, ρ, by the relationship

$$G_o = \rho v_s^2 \tag{4-5}$$

(In the fonts currently in use in the *Guidelines*, the italicized v is similar to the Greek ν.) Converting mass density to unit weight, γ, gives an alternative expression

$$G_o = \frac{\gamma v_s^2}{g} \tag{4-6}$$

where g is acceleration due to gravity.

The initial shear modulus also has been related to normalized and corrected blow count, $(N_1)_{60}$, and effective vertical stress, σ_o', as follows (from Seed et al., 1986):

$$G_o \cong 20{,}000\,(N_1)_{60}^{1/3}\sqrt{\sigma_o'} \tag{4-7}$$

where:

$(N_1)_{60}$	=	Blow count normalized for 1.0 ton per square foot confining pressure and 60% energy efficiency of hammer
σ_o'	=	Effective vertical stress in psf

and

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 water level

It should be noted that the G_o in Equation 4-7 is expressed in pounds per square foot, as is σ_o'.

Most soils are intrinsically nonlinear and the shear wave modulus decreases with increasing shear strain. The large-strain shear wave velocity, v_s', and the effective shear modulus, G, can be estimated based on the Effective Peak Acceleration coefficient for the earthquake under consideration, in accordance with Table 4-3.

Table 4-3 Effective Shear Modulus and Shear Wave Velocity

	Effective Peak Acceleration, $S_{XS}/2.5$	
	0.10	**0.70**
Ratio of effective to initial shear modulus (G/G_o)	0.50	0.20
Ratio of effective to initial shear wave velocity (v_s'/v_s)	0.71	0.45

Notes:

1. Site-specific values may be substituted if documented in a detailed geotechnical site investigation.
2. Linear interpolation may be used for intermediate values.

To reflect the upper and lower bound concept illustrated in Figure 4-1a in the absence of a detailed geotechnical site study, the upper bound stiffness of rectangular footings should be based on twice the effective shear modulus, G, determined in accordance with the above procedure. The lower bound stiffness should be based on one-half the effective shear modulus. Thus the range of stiffness should incorporate a factor of four from lower to upper bound.

Most shallow bearing footings are stiff relative to the soil upon which they rest. For simplified analyses, an uncoupled spring model, as shown in Figure 4-1b, may be sufficient. The three equivalent spring constants may be determined using conventional theoretical solutions for rigid plates resting on a semi-infinite elastic medium. 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.

Figure 4-2 presents stiffness solutions for rectangular plates in terms of an equivalent circular radius.

Stiffnesses are adjusted for shape and depth using factors similar to those in Figure 4-3. Other formulations incorporating a wider range of variables may be found in Gazetas (1991). For the case of horizontal translation, the solution represents mobilization of base traction (friction) only. If the sides of the footing are in close contact with adjacent in situ foundation soil or well-compacted fill, significant additional stiffness may be assumed from passive pressure. A solution for passive pressure stiffness is presented in Figure 4-4.

For more complex analyses, a finite element representation of linear or nonlinear foundation behavior may be accomplished using Winkler component models. Distributed vertical stiffness properties may be calculated by dividing the total vertical stiffness by the area. Similarly, the uniformly distributed rotational stiffness can be calculated by dividing the total rotational stiffness of the footing by the moment of inertia of the footing in the direction of loading. In general, however, the uniformly distributed vertical and rotational stiffnesses are not equal. The two may be effectively decoupled for a Winkler model using a procedure similar to that illustrated in Figure 4-5. The ends of the rectangular footing are represented by end zones of relatively high stiffness over a length of approximately one-sixth of the footing width. 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.

In some instances, the stiffness of the structural components of the footing may be relatively flexible compared to the soil material; for example, a slender grade beam resting on stiff soil. Classical solutions for beams on elastic supports can provide guidance on when such effects are important. For example, a grade beam supporting point loads spaced at a distance of L might be considered flexible if:

$$\frac{EI}{L^4} < 10k_{sv}B \qquad (4\text{-}8)$$

where, for the grade beam,

E = Effective modulus of elasticity

I = Moment of inertia

B = Width

For most flexible foundation systems, the unit subgrade spring coefficient, k_{sv}, may be taken as

$$k_{sv} = \frac{1.3G}{B(1-\nu)} \qquad (4\text{-}9)$$

B. Capacity Parameters

The specific capacity of shallow bearing foundations should be determined using fully plastic concepts and the generalized capacities of Section 4.4.1. Upper and lower bounds of capacities, as illustrated in Figure 4-1a, should be determined by multiplying the best estimate values by 2.0 and 0.5, respectively.

In the absence of moment loading, the vertical load capacity of a rectangular footing of width B and length L is

$$Q_c = q_c BL \qquad (4\text{-}10)$$

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 vertical stress capacity, q_c. Assuming that contact stresses are proportional to vertical displacement and remain elastic up to the vertical stress 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 4-6. In general the moment capacity of a rectangular footing may be expressed as:

$$M_c = \frac{LP}{2}\left(1 - \frac{q}{q_c}\right) \qquad (4\text{-}11)$$

where

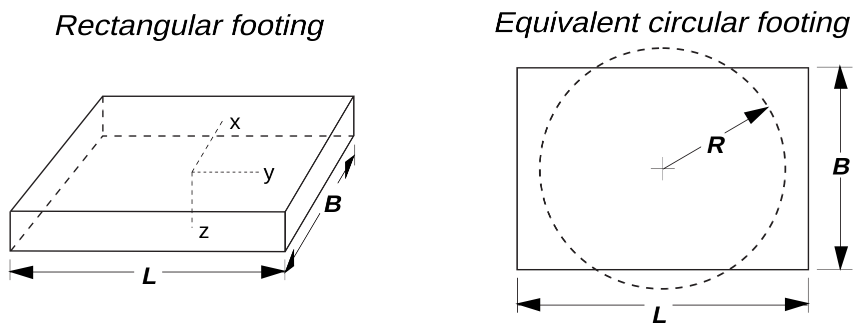
P = Vertical load

$q = \dfrac{P}{BL}$

B = Footing width

L = Footing length in direction of bending

Radii of circular footings equivalent to rectangular footings



	Degree of freedom			
	Translation	***Rocking***		***Torsion***
		About x-axis	*About y-axis*	*About z-axis*
Equivalent radius, R	$\left(\frac{BL}{\pi}\right)^{1/2}$	$\left(\frac{BL^3}{3\pi}\right)^{1/4}$	$\left(\frac{B^3L}{3\pi}\right)^{1/4}$	$\left[\frac{BL(B^2+L^2)}{6\pi}\right]^{1/4}$

Spring constants for embedded rectangular footings

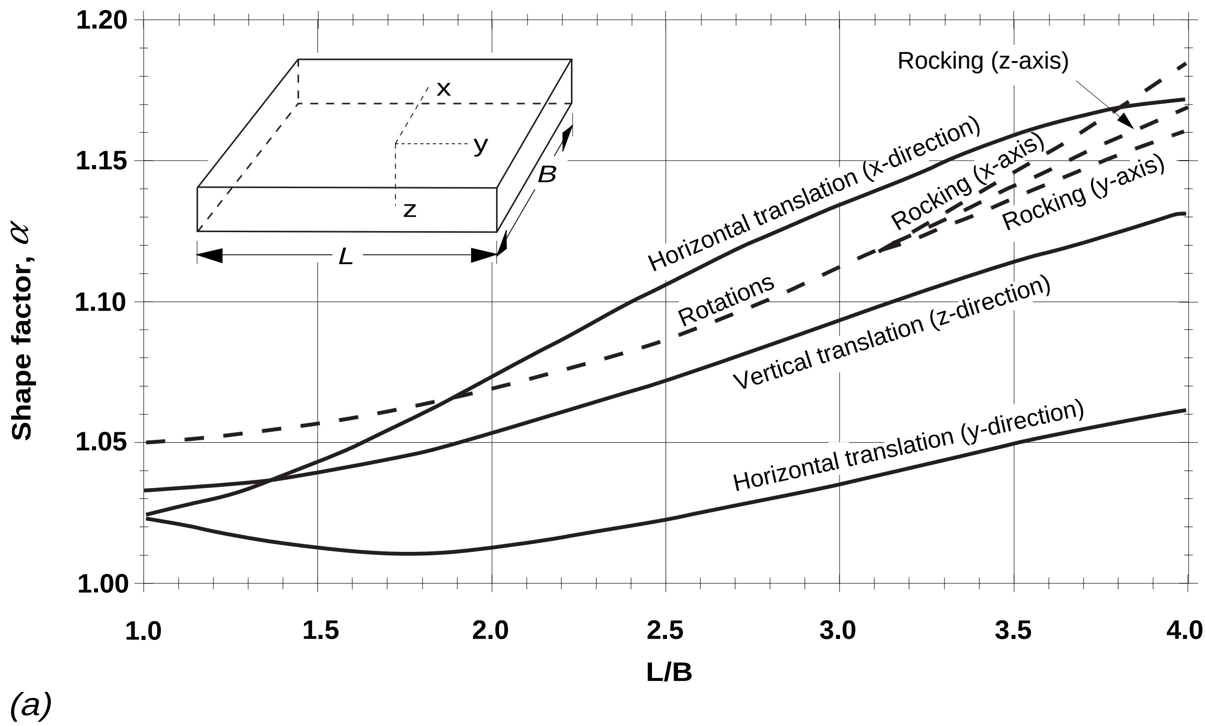
Spring constants for shallow rectangular footings are obtained by modifying the solution for a circular footing, bonded to the surface of an elastic half-space, i.e., $k = \alpha\beta k_o$
where
k_o = Stiffness coefficient for the equivalent circular footing
α = Foundation shape correction factor (Figure 4-3a)
β = Embedment factor (Figure 4-3b)

To use the equation, the radius of an equivalent circular footing is first calculated according to the degree of freedom being considered. The figure above summarizes the appropriate radii. k_o is calculated using the table below:

Displacement degree of freedom	k_o
Vertical translation	$\frac{4GR}{1-\nu}$
Horizontal translation	$\frac{8GR}{2-\nu}$
Torsional rotation	$\frac{16GR^3}{3}$
Rocking rotation	$\frac{8GR^3}{3(1-\nu)}$

Note:
G and ν are the shear modulus and Poisson's ratio for the elastic half-space.
G is related to Young's modulus, E, as follows:
$E = 2(1+\nu)G$
R = Equivalent radius

Figure 4-2 ***Elastic Solutions for Rigid Footing Spring Constants (based on Gazetas, 1991 and Lam et al., 1991)***



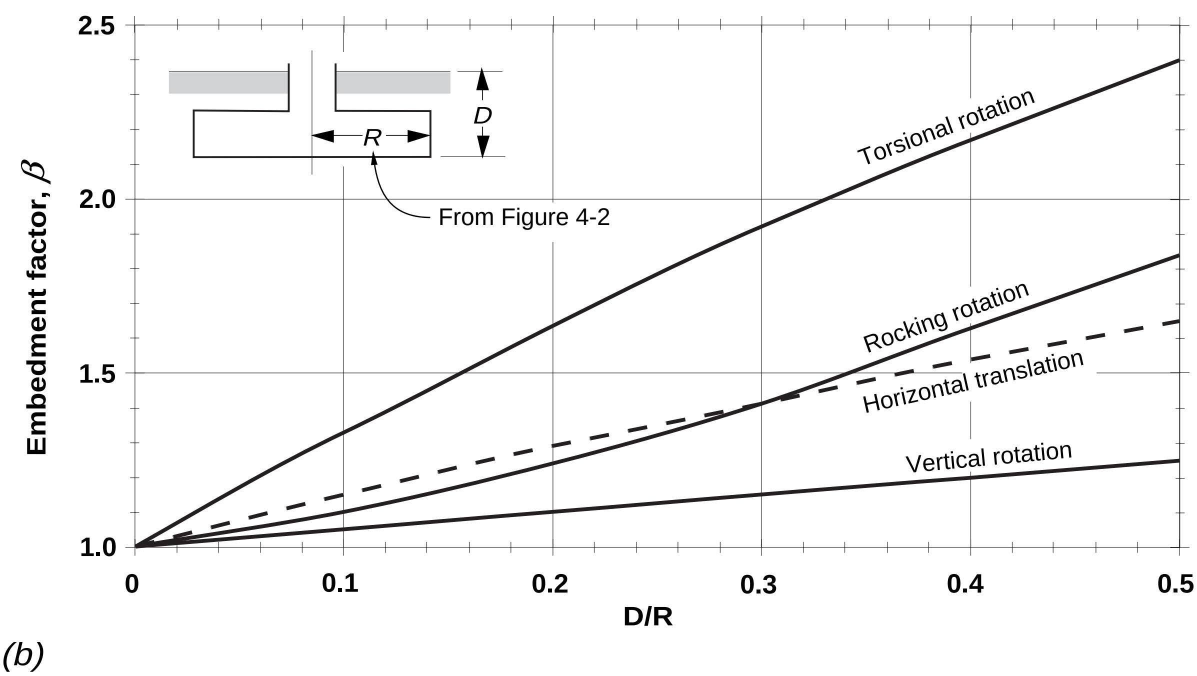


Figure 4-3 ***(a) Foundation Shape Correction Factors (b) Embedment Correction Factors***

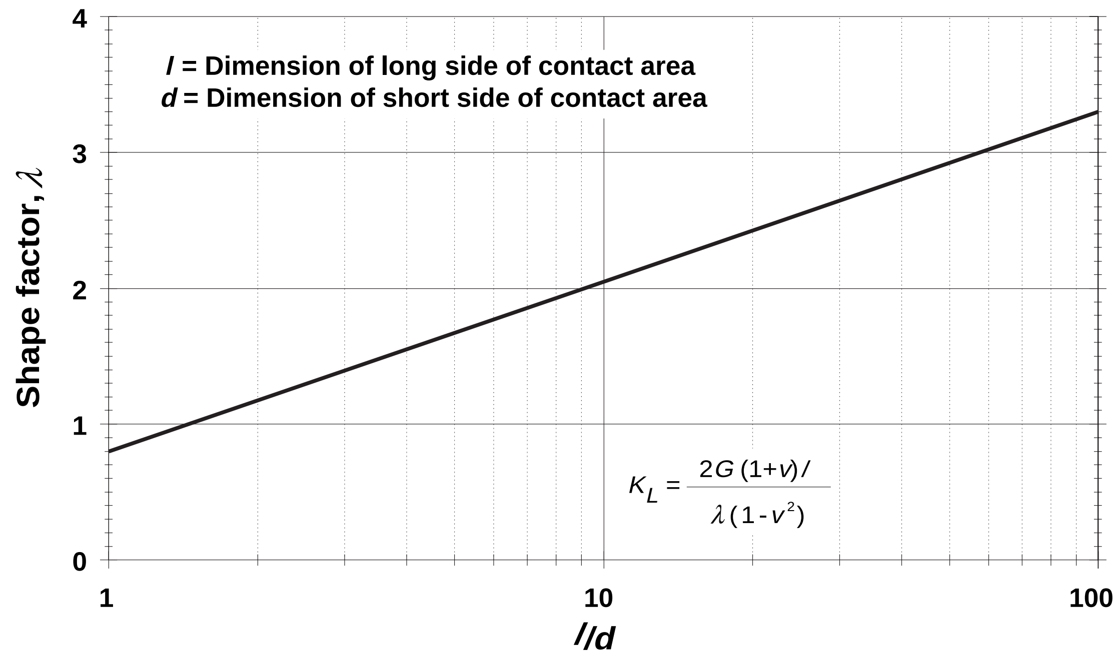


Figure 4-4 Lateral Foundation-to-Soil Stiffness for Passive Pressure (after Wilson, 1988)

The lateral capacity of a footing should assumed to be attained when the displacement, considering both base traction and passive pressure stiffnesses, reaches 2% of the thickness of the footing. Upper and lower bounds of twice and one-half of this value, respectively, also apply.

4.4.2.2 Pile Foundations

Pile foundations, in the context of this subsection, refer to those foundation systems that are composed of a pile cap and associated driven or cast-in-place piles, which together form a pile group. A single pile group may support a load-bearing column, or a linear sequence of pile groups may support a shear wall.

Generally, individual piles in a group could be expected to be less than two feet in diameter. The stiffness characteristics of single large-diameter piles or drilled shafts are described in Section 4.4.2.3.

A. Stiffness Parameters

For the purpose of simplified analyses, the uncoupled spring model as shown in Figure 4-1b may be used where the footing in the figure represents the pile cap. In the case of the vertical and rocking springs, it can be assumed that the contribution of the pile cap is relatively small compared to the contribution of the piles. In general, mobilization of passive pressures by either the pile caps or basement walls will control lateral spring stiffness. Hence, estimates of lateral spring stiffness can be computed using elastic solutions as described in Section 4.4.2.1A. In instances where piles may contribute significantly to lateral stiffness (i.e., very soft soils, battered piles), solutions using beam-column pile models are recommended.

Axial pile group stiffness spring values, k_{sv}, may be assumed to be in an upper and lower bound range, respectively, given by:

$$k_{sv} = \sum_{n=1}^{N} \frac{0.5\ A\ E}{L} \quad \text{to} \quad \sum_{n=1}^{N} \frac{2\ A\ E}{L} \tag{4-12}$$

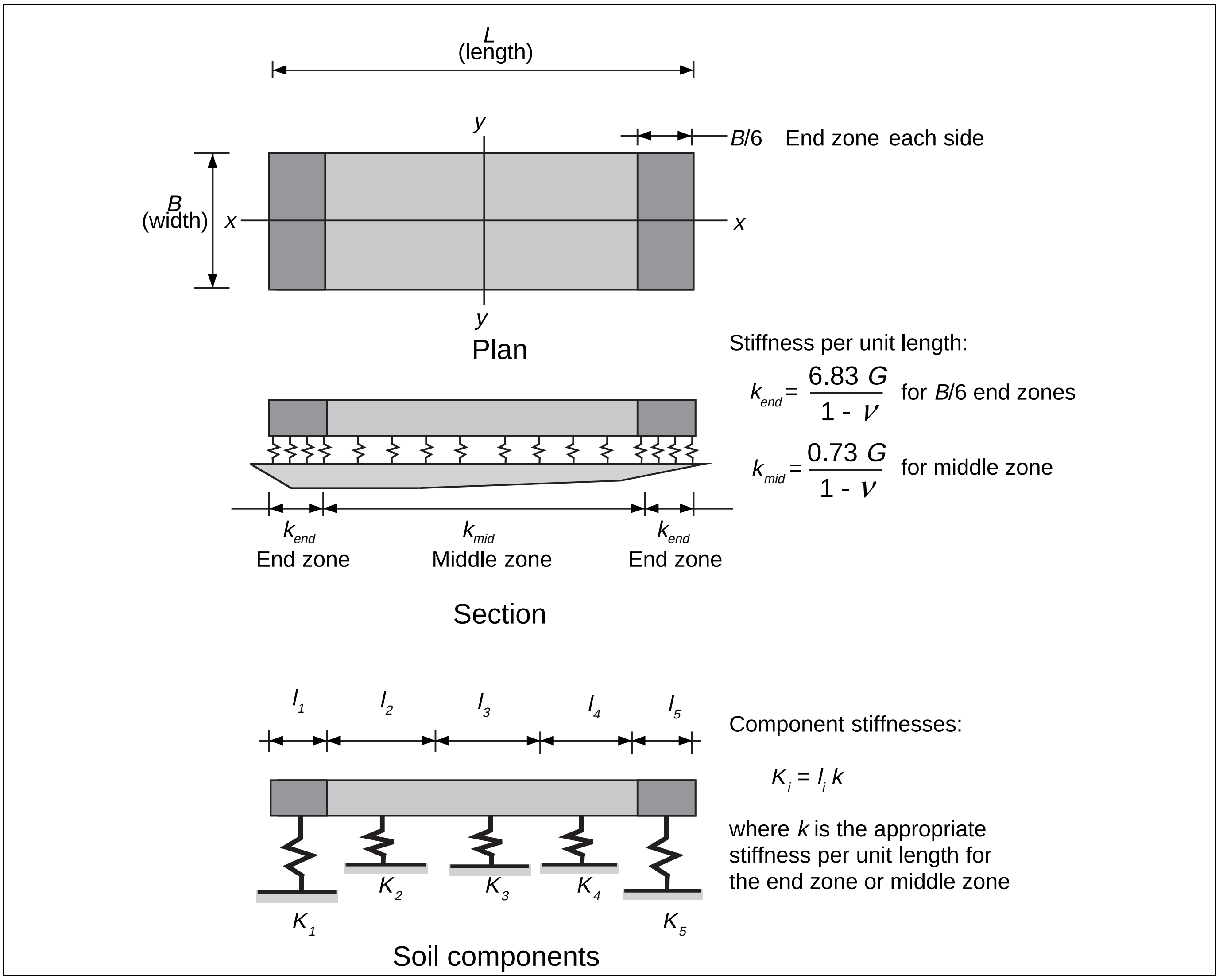


Figure 4-5 ***Vertical Stiffness Modeling for Shallow Bearing Footings***

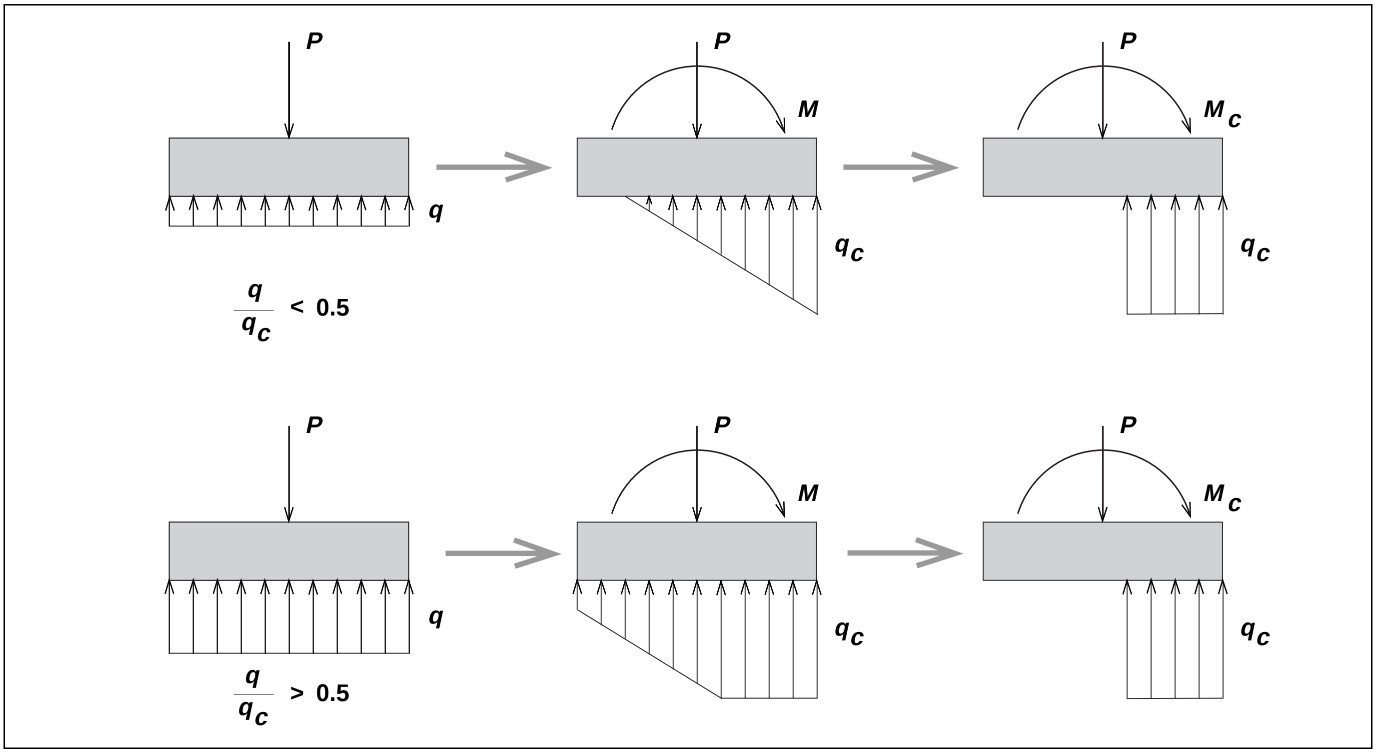


Figure 4-6 Idealized Concentration of Stress at Edge of Rigid Footings Subjected to Overturning Moment

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 may be computed by assuming each axial pile spring acts as a discrete Winkler spring. The rotational spring constant (moment per unit rotation) is then given by:

$$k_{sr} = \sum_{n=1}^{N} k_{vn} S_n^2 \tag{4-13}$$

where

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

Whereas 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 expressed for axial pile stiffness.

B. Capacity Parameters

Best-estimate vertical load capacity of piles (for both axial compression and axial tensile loading) should be determined using accepted foundation engineering practice, using best estimates of soil properties. Consideration should be given to the capability of pile cap and splice connections to take tensile loads when evaluating axial tensile load capacity. Upper and lower bound axial load capacities should be determined by multiplying best-estimate values by factors of 2.0 and 0.5, respectively.

The upper and lower bound moment capacity of a pile group should be determined assuming a rigid pile cap, leading to an initial triangular distribution of axial pile loading from applied seismic moments. However, full axial capacity of piles may be mobilized when computing ultimate moment capacity, leading to a rectangular distribution of resisting moment in a manner analogous to that described for a footing in Figure 4-6.

The lateral capacity of a pile group is largely dependent on that of the cap as it is restrained by passive resistance of the adjacent soil material. The capacity may be assumed to be reached when the displacement reaches 2% of the depth of the cap in a manner similar to that for a shallow bearing foundation.

4.4.2.3 Drilled Shafts

In general, drilled shaft foundations or piers may be treated similarly to pile foundations. 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. In these instances, the interaction of the soil and shaft may be represented using Winkler type models (Pender, 1993; Reese et al., 1994).

4.4.3 Foundation Acceptability Criteria

This section contains acceptability criteria for the geotechnical components of building foundations. Structural components of foundations shall meet the appropriate requirements of Chapters 5 through 8.

Geotechnical components include the soil parts of shallow spread footings and mats, and friction- and end-bearing piles and piers. These criteria, summarized in Table 4-4, apply to all actions including vertical loads, moments, and lateral forces applied to the soil.

4.4.3.1 Simplified Rehabilitation

The geotechnical components of buildings qualified for and subject to Simplified Rehabilitation may be considered acceptable if they comply with the requirements of Chapter 10.

4.4.3.2 Linear Procedures

The acceptability of geotechnical components subject to linear procedures depends upon the basic modeling assumptions utilized in the analysis, as follows.

Fixed Base Assumption. If the base of the structure has been assumed to be completely rigid, actions on geotechnical components shall be as on force-controlled components governed by Equation 3-15 and component capacities may be assumed as upper-bound values. A fixed base assumption is not recommended for the Immediate Occupancy Performance Level for buildings sensitive to base rotations or other types of foundation movement.

Table 4-4 Soil Foundation Acceptability Summary

Analysis Procedure	Foundation Assumption	Performance Level: Collapse Prevention and Life Safety	Performance Level: Immediate Occupancy
Simplified Rehabilitation		See Chapter 10	Not applicable.
Linear Static or Dynamic	Fixed	Actions on geotechnical components shall be assumed as on force-controlled components governed by Equation 3-15 and component capacities may be assumed as upper bound values.	Not recommended for buildings sensitive to base rotation or other foundation movements.
	Flexible	$m = \infty$ for use in Equation 3-18	$m = 2.0$ for use in Equation 3-18
Nonlinear Static or Dynamic	Fixed	Base reactions limited to upper bound ultimate capacity.	Not recommended for buildings sensitive to base rotation or other foundation movements.
	Flexible	Geotechnical component displacements need not be limited, provided that structure can accommodate the displacements.	Estimate and accommodate possible permanent soil movements.

Flexible Base Assumption. If the base of the structure is modeled using linear geotechnical components, then the value of m, for use in Equation 3-18, for Life Safety and Collapse Prevention Performance Levels may be assumed as infinite, provided the resulting displacements may be accommodated within the acceptability criteria for the rest of the structure. For the

Immediate Occupancy Performance Levels, *m* values for geotechnical components shall be limited to 2.0.

4.4.3.3 Nonlinear Procedures

The acceptability of geotechnical components subject to nonlinear procedures depends upon the basic modeling assumptions utilized in the analysis, as follows.

Fixed Base Assumption. If the base of the structure has been assumed to be completely rigid, then the base reactions for all geotechnical components shall not exceed their upper-bound capacity to meet Collapse Prevention and Life Safety Performance Levels. A rigid base assumption is not recommended for the Immediate Occupancy Performance Level for buildings sensitive to base rotations or other types of foundation movement.

Flexible Base Assumption. If the base of the structure is modeled using flexible nonlinear geotechnical components, then the resulting component displacements need not be limited to meet Life Safety and Collapse Prevention Performance Levels, provided the resulting displacements may be accommodated within the acceptability criteria for the rest of the structure. For the Immediate Occupancy Performance Level, an estimate of the permanent nonrecoverable displacement of the geotechnical components shall be made based upon the maximum total displacement, foundation and soil type, soil layer thicknesses, and other pertinent factors. The acceptability of these displacements shall be based upon their effects on the continuing function and safety of the building.

4.5 Retaining Walls

Past earthquakes have not caused extensive damage to building walls below grade. In some cases, however, it may be advisable to verify the adequacy of retaining walls to resist increased pressure due to seismic loading. These situations might be for 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 seismic earth pressure acting on a building wall retaining nonsaturated, level soil above the ground-water table may be approximated as:

$$\Delta p = 0.4 k_h \gamma_t H_{rw} \tag{4-14}$$

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 = The total unit weight of soil

H_{rw} = The height of the retaining wall

The seismic earth pressure given above should be added to the unfactored static earth pressure to obtain the total earth pressure on the wall. The expression in Equation 4-14 is a conservative approximation of the Mononabe-Okabe formulation. The pressure on walls during earthquakes is a complex action. 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.

4.6 Soil Foundation Rehabilitation

This section provides guidelines for modification to foundations to improve anticipated seismic performance. Specifically, the scope of this section includes suggested approaches to foundation modification and behavioral characteristics of foundation elements from a geotechnical perspective. These must be used in conjunction with appropriate structural material provisions from other chapters. Additionally, the acceptability of a modified structure is determined in accordance with Chapter 2 of the *Guidelines.*

4.6.1 Soil Material Improvements

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 the *Commentary* 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.

Options that can be considered to increase the passive resistance of soils adjacent to foundations or grade beams include removal and replacement of soils with stronger, well-compacted soils or with treated (e.g., cement-stabilized) soils; in-place mixing of soils with strengthening materials (e.g., cement); grouting, including permeation grouting and jet grouting; and in-place densification by impact or vibratory compaction (if the soil layers to be compacted are not too thick and vibration effects on the structure are tolerable).

4.6.2 Spread Footings and Mats

New isolated or spread footings may be added to existing structures to support new structural elements such as shear walls or frames. In these instances, capacities and stiffness may be determined in accordance with the procedures of Section 4.4.

Existing isolated or spread footings may be enlarged to increase bearing or uplift capacity. Generally, capacities and stiffness may be determined in accordance with the procedures of Section 4.4; however, consideration of existing contact pressures on the strength and stiffness of the modified footing may be required, unless a uniform distribution is achieved by shoring and/or jacking.

Existing isolated or spread footings may be underpinned to increase bearing or uplift capacity. This technique improves bearing capacity by lowering the contact horizon of the footing. Uplift capacity is improved by increasing the resisting soil mass above the footing. Generally, capacities and stiffness may be determined in accordance with the procedures of Section 4.4. Considerations of the effects of jacking and load transfer may be required.

Where potential for differential lateral displacement of building foundations exists, provision of interconnection with grade beams or a well-reinforced grade slab can provide good mitigation of these effects. Ties provided to withstand differential lateral displacement should have a strength based on rational analysis, with the advice of a geotechnical engineer when appropriate.

4.6.3 Piers and Piles

Piles and pile caps shall have the capacity to resist additional axial and shear loads caused by overturning forces. Wood piles cannot resist uplift unless a positive connection is provided for the loads. Piles must be reviewed for deterioration caused by decay, insect infestation, or other signs of distress.

Driven piles made of steel, concrete, or wood, or cast-in-place concrete piers may be used to support new structural elements such as shear walls or frames. Capacities and stiffnesses may be determined in accordance with the procedures of Section 4.4. When used in conjunction with existing spread footing foundations, the effects of differential foundation stiffness should be considered in the analysis of the modified structure.

Driven piles made of steel, concrete, or wood, or cast-in-place concrete piers may be used to supplement the vertical and lateral capacities of existing pile and pier foundation groups and of existing isolated and continuous spread footings. Capacities and stiffnesses may be determined in accordance with the procedures of Section 4.4. If existing loads are not redistributed by shoring and/or jacking, the potential for differential strengths and stiffnesses among individual piles or piers should be included.

4.7 Definitions

Allowable bearing capacity: Foundation load or stress commonly used in working-stress design (often controlled by long-term settlement rather than soil strength).

Deep foundation: Piles or piers.

Differential compaction: An earthquake-induced process in which loose or soft soils become more compact and settle in a nonuniform manner across a site.

Fault: Plane or zone along which earth materials on opposite sides have moved differentially in response to tectonic forces.

Footing: A structural component transferring the weight of a building to the foundation soils and resisting lateral loads.

Foundation soils: Soils supporting the foundation system and resisting vertical and lateral loads.

Foundation springs: Method of modeling to incorporate load-deformation characteristics of foundation soils.

Foundation system: Structural components (footings, piles).

Landslide: A down-slope mass movement of earth resulting from any cause.

Liquefaction: An earthquake-induced process in which saturated, loose, granular soils lose a substantial amount of shear strength as a result of increase in pore-water pressure during earthquake shaking.

Pier: Similar to pile; usually constructed of concrete and cast in place.

Pile: A deep structural component transferring the weight of a building to the foundation soils and resisting vertical and lateral loads; constructed of concrete, steel, or wood; usually driven into soft or loose soils.

Prescriptive ultimate bearing capacity: Assumption of ultimate bearing capacity based on properties prescribed in Section 4.4.1.2.

Presumptive ultimate bearing capacity: Assumption of ultimate bearing capacity based on allowable loads from original design.

Retaining wall: A free-standing wall that has soil on one side.

Shallow foundation: Isolated or continuous spread footings or mats.

SPT N-Values: Using a standard penetration test (ASTM Test D1586), the number of blows of a 140-pound hammer falling 30 inches required to drive a standard 2-inch-diameter sampler a distance of 12 inches.

Ultimate bearing capacity: Maximum possible foundation load or stress (strength); increase in deformation or strain results in no increase in load or stress.

4.8 Symbols

A	Footing area; also cross-section area of pile
B	Width of footing
D	Depth of footing bearing surface
E	Young’s modulus of elasticity
G	Shear modulus
G_o	Initial or maximum shear modulus
H	Horizontal load on footing
H_{rw}	Height of retaining wall
I	Moment of inertia
K_L	Passive pressure stiffness
L	Length of footing in plan dimension
L	Length of pile in vertical dimension
M	Moment on footing
M_c	Ultimate moment capacity of footing
N	Number of piles in a pile group
$(N_1)_{60}$	Standard Penetration Test blow count normalized for an effective stress of 1 ton per square foot and corrected to an equivalent hammer energy efficiency of 60%
P	Vertical load on footing
Q_D	Dead (static) load
Q_E	Earthquake load
Q_L	Live (frequently applied) load
$Q_{allow.D+L}$	Allowable working dead plus live load for a pile as specified in original design documents
Q_c	Ultimate bearing capacity
Q_S	Snow load
R	Radius of equivalent circular footing
S_{XS}	Spectral response acceleration at short periods for any hazard level or damping, g
S_n	Distance between nth pile and axis of rotation of a pile group
S_S	Spectral response acceleration at short periods, obtained from response acceleration maps, g
c	Cohesive strength of soil, expressed in force/unit area (pounds/ft^2 or Pa)

d	Short side of footing lateral contact area
d	Depth to sample
d_w	Depth of ground-water level
g	Acceleration of gravity (386.1 in/sec.^{2}, or 9,800 mm/sec.^{2} for SI units)
k_h	Horizontal seismic coefficient in soil acting on retaining wall
k_o	Stiffness coefficient for equivalent circular footing
k_{sh}	Winkler spring coefficient in horizontal direction, expressed as force/unit displacement/unit area
k_{sr}	Winkler spring coefficient in overturning (rotation), expressed as force/unit displacement/unit area
k_{sv}	Winkler spring coefficient in vertical direction, expressed as force/unit displacement/unit area
k_{vn}	Axial stiffness of nth pile in a pile group
l	Long side of footing lateral contact area
m	A modification factor used in the acceptance criteria of deformation-controlled components or elements, indicating the available ductility of a component action, and a multiplier of ultimate foundation capacity for checking imposed foundation loads in Linear Static or Dynamic Procedures
q	Vertical bearing pressure
$q_{allow.D+L}$	Allowable working dead plus live load pressure for a spread footing as specified in original design documents
q_c	Ultimate bearing capacity
v_s	Shear wave velocity at low strain
v_s'	Shear wave velocity at high strain
Δp	Additional earth pressure on retaining wall due to seismic shaking
α	Foundation shape correction factor
β	Embedment factor
γ	Unit weight, weight/unit volume (pounds/ft^3 or N/m^3)
γ_t	Total unit weight of soil
γ_w	Unit weight of water
δ_c	Pile compliance
λ	Shape factor for lateral stiffness
ν	Poisson's ratio
ρ	Soil mass density
σ_o'	Effective vertical stress
ϕ	Angle of internal friction, degrees

4.9 References

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