Appendix to Chapter 7

GEOTECHNICAL ULTIMATE STRENGTH DESIGN OF FOUNDATIONS AND FOUNDATION LOAD-DEFORMATION MODELING

A7.2 GENERAL DESIGN REQUIREMENTS

A7.2.2. Foundation load capacities. In current geotechnical engineering practice, foundation design is based on allowable stresses, with allowable foundation load capacities, Q_{as}, for dead plus live loads based on limiting static settlements and providing a large factor of safety against exceeding ultimate capacities. In current practice, allowable soil stresses for dead plus live loads are increased by one-third for load combinations that include wind or seismic forces. The one-third increase is overly conservative if the allowable stresses for dead plus live loads are far below ultimate soil capacity.

This appendix provides guidance for the direct use of ultimate foundation load capacity, Q_{us}, for load combinations including seismic effects. It is required that foundations be capable of resisting loads with acceptable deformations considering the short duration of seismic loading, the dynamic properties of the soil, and the ultimate load capacities, Q_{us}, of the foundations under vertical, lateral, and rocking loading.

A7.2.2.1. Determination of ultimate foundation load capacities. For competent soils that are not expected to degrade in strength during seismic loading (e.g., due to partial or total liquefaction of cohesionless soils or strength reduction of sensitive clays), use of static soil strengths is recommended for determining ultimate foundation load capacities, Q_{us}. Use of static strengths is somewhat conservative for such soils because rate-of-loading effects tend to increase soil strengths for transient loading. Such rate effects are neglected because they may not result in significant strength increase for some soil types and are difficult to confidently estimate without special dynamic testing programs. The assessment of the potential for soil liquefaction or other mechanisms for reducing soil strengths is critical, because these effects may reduce soil strengths greatly below static strengths in susceptible soils.

The best-estimated ultimate vertical load capacity of footings, Q_{us}, should be determined using accepted foundation engineering practice. In the absence of moment loading, the ultimate vertical load capacity of a rectangular footing of width B and length L may be written as

$$Q_{us} = q_c BL$$

where q_c = ultimate soil bearing pressure.

For rigid footings subject to moment and vertical load, contact stresses become concentrated at footing edges, particularly as footing uplift occurs. The ultimate moment capacity, M_{us}, of the footing as limited by the soil is dependent upon the ratio of the vertical load stress, q, to the ultimate soil bearing pressure q_c. Assuming that contact stresses are proportional to vertical displacements and remain elastic up to q_c, it can be shown that uplift will occur prior to plastic yielding of the soils 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 CA7.2.2-1. In general the ultimate moment capacity of a rectangular footing may be expressed as:

$$M_{us} = \frac{LP}{2}\left(1 - \frac{q}{q_c}\right)$$

where

P = Vertical Load

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

B = Footing width, and

L = Footing length in direction of rotation

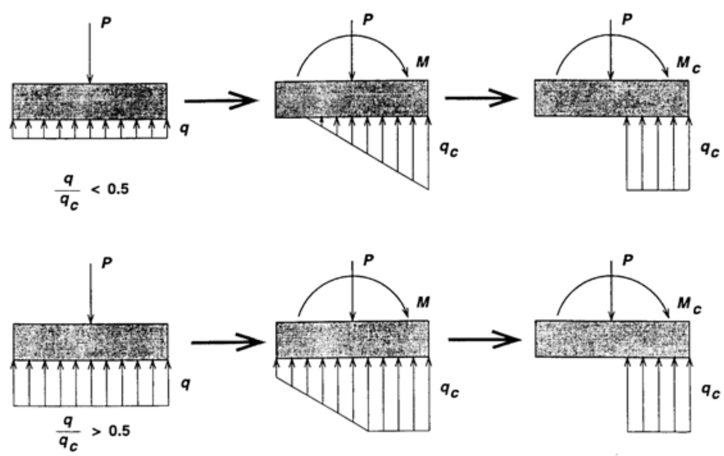


Figure CA7.2.2-1.

The ultimate lateral load capacity of a footing may be assumed equal to the sum of the best-estimated ultimate soil passive resistance against the vertical face of the footing plus the best-estimated ultimate soil friction force on the footing base. The determination of ultimate passive resistance should consider the potential contribution of friction on the face of the footing on the passive resistance.

For piles, the best-estimated ultimate vertical load capacity (for both axial compression and axial tensile loading) should be determined using accepted foundation engineering practice. When evaluating axial tensile load capacity, consideration should be given to the capability of pile cap and splice connections to take tensile loads.

The ultimate 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 overturning moments. However, full axial capacity of piles may be mobilized when computing ultimate moment capacity, in a manner analogous to that described for a footing in Figure CA7.2.2-1. The ultimate lateral capacity of a pile group may be assumed equal to the best-estimated ultimate passive resistance acting against the edge of the pile cap and the additional passive resistance provided by piles.

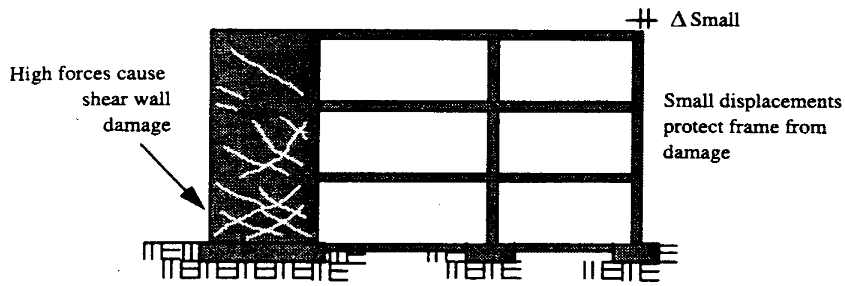
Resistance factors, ϕ, are provided to factor ultimate foundation load capacities, Q_{us}, to reduced capacities, ϕQ_{us}, used to check foundation acceptance criteria. The values of ϕ recommended in the *Provisions* are higher than those recommended in some codes and specifications for long-term static loading. The development of resistance factors for static loading has been based on detailed reliability studies and on calibrations to give designs and factors of safety comparable to those given by allowable stress design. As indicated in the first paragraph of this section, mobilized strengths for seismic loading conditions are expected to be somewhat higher than the static strengths specified for use in obtaining values of Q_{us}, especially for cohesive soils. In the absence of any detailed reliability studies for seismic loading conditions, Values of ϕ equal to 0.8 and 0.7 were selected for cohesive and cohesionless soils, respectively, where geotechnical site investigations, including laboratory or insitu tests, are conducted, and values of ϕ equal to 1.0 and 0.9 were selected where full-scale field testing of prototype foundations are conducted. These values are comparable to the values of 0.8 (for soil strengths determined based on a comprehensive site soil investigation including soil sampling and testing) and 0.9 (for soil strengths determined by site loading testing using plate bearing or near full scale foundation element testing) recommended by the SEAOC Seismology Committee Ad Hoc Foundation Committee (2001).

A7.2.2.2 Acceptance criteria. The factored load capacity, ϕQ_{us}, provides the basis for the acceptance criteria, particularly for the linear analysis procedures. The mobilization of ultimate capacity in the nonlinear analysis procedures does not necessarily mean unacceptable performance as structural deformations due to foundation displacements may be tolerable, as discussed by Martin and Lam (2000). For the nonlinear analysis procedures, it is also prudent to evaluate structural behavior utilizing parametric increases in foundation load capacities above Q_{us} by a factor of $1/\phi$, to check potential changes in structural ductility demands.

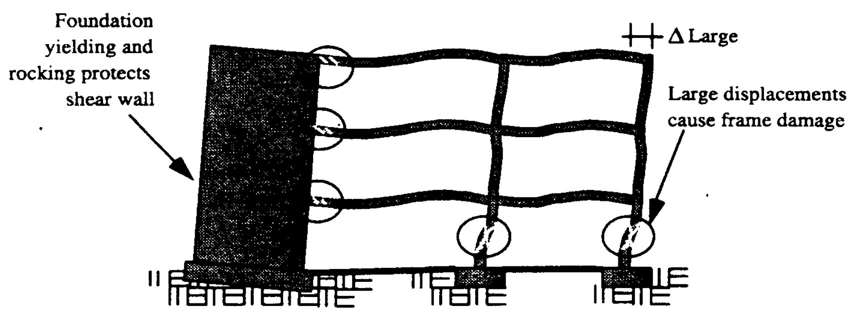
A7.2.3 Foundation load-deformation modeling. Analysis methods described in Sec. 5.3 (response spectrum procedure) and Sec. 5.4 (linear response history procedure), permit the use of realistic assumptions for foundation stiffness, as opposed to the assumption of a fixed base. In addition, the nonlinear response history procedure (Sec. 5.5) and the nonlinear static procedure (Appendix to Chapter 5) permit the use of realistic assumptions for the stiffness and load-carrying characteristics of the foundations. Guidance for flexible foundation (non-fixed base) modeling for the above analysis procedures are described herein.

Foundation load-deformation behavior characterized by stiffness and load capacity may significantly influence the seismic performance of a structure, with respect to both load demands and distribution among structural elements (ATC 1996, NEHRP 1997a, 1997b). This is illustrated schematically in Figure CA 7.2.3-1. While it is recognized that the load-deformation behavior of foundations is nonlinear, an equivalent elasto-plastic representation of load-deformation behavior is often assumed, as illustrated in Figure CA 7.2.3-2. To allow for variability and uncertainty in the selection of soil parameters and analysis methods used to determine stiffness and capacity, a range of parameters for foundation modeling should be used to permit sensitivity evaluations.

Foundation stiffness and strength affect various structural components differently.



Stiff/Strong Foundation



Flexible/Weak Foundation

Stiff/strong is not always favorable; nor is flexible/weak always conservative.

Figure CA7.2.3-1.

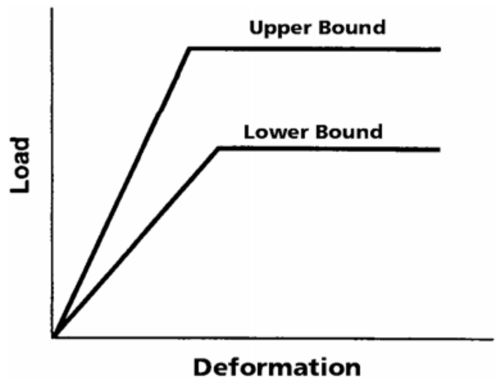


Figure CA7.2.3-2.

Consider the spread footing shown in Figure CA 7.2.3-3 with an applied vertical load (*P*), lateral load (*H*), and moment (*M*). The soil characteristics might be modeled as two translational springs and a rotational spring, each characterized by a linear elastic-stiffness and a plastic capacity. The use of a Winkler spring model acting in conjunction with the foundation to eliminate the rotational spring may also be used, as shown in Figure CA7.2.3-4. The Winkler model can capture more accurately progressive mobilization of plastic capacity during rocking behavior. Note the lateral action is normally uncoupled from the vertical and rotational action. Many foundation systems are relatively stiff and strong in the horizontal direction, due to passive resistance against the face of footings or basement walls, and friction beneath footings and floor slabs. Comparisons of horizontal stiffness of the foundation and the structure can provide guidance on the need to include horizontal foundation stiffness in demand or capacity analyses. In general, foundation rocking has the most influence on structural response. Slender shear wall structures founded on strip footings, in particular, are most sensitive to the effects of foundation rocking.

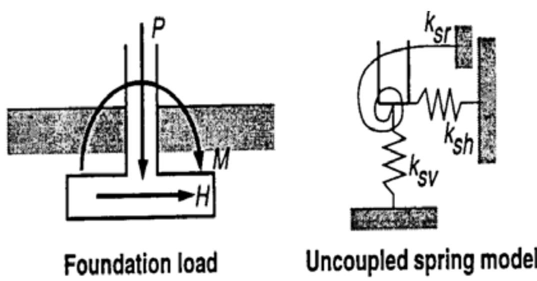


Figure CA7.2.3-3.

Assuming a shallow footing foundation may be represented by an embedded rigid plate in an elastic half-space, classical elastic solutions may be used to compute the uncoupled elastic stiffness parameters. Representative solutions are described in *Commentary* to Sec. 5.6. Solutions developed by Gazetas (1991) are also often used, as described in ATC (1996). Dynamic soil properties (i.e. properties consistent with seismic wave velocities and associated moduli of the soils as opposed to static soil moduli) should be used in dynamic soil solutions. The effects of nonlinearity on dynamic soil properties should be incorporated using the reduction factors in Sec. 5.6.2.1.1 or based on a site-specific study.

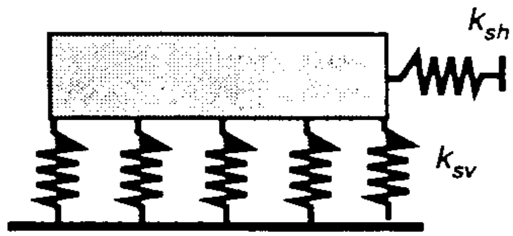


Figure CA 7.2.3-4.

In the case of pile groups, the uncoupled spring model shown in Figure CA 7.2.3-3 may also be used, where the footing represents the pile cap. In the case of the vertical and rotational 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 for footings. 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}, are generally in the range given by:

$$k_{sv} = \sum_{n=1}^{N} \frac{0.5AE}{L} \text{ to } \sum_{n=1}^{N} \frac{2AE}{L}$$

where

A = Cross-sectional area of a pile,

E = Modulus of elasticity of piles,

L = Length of piles , and

N = Number of piles in group.

Values of axial stiffness depend on complex nonlinear interaction of the pile and soil (NEHRP, 1997b). For simplicity, best estimate values of *AE/L* and 1.5 *AE/L* are recommended for piles where axial capacity is primarily controlled by end bearing and side friction, respectively.

The rocking spring stiffness values, k_{sr}, 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:

$$k_{sr} = \sum_{n=1}^{N} k_{vn} S_n^2$$

where

k_{vn} = Axial stiffness of the nth pile, and

S_n = Distance between nth pile and axis of rotation.

The effects of group action and the influence of pile batter are not accounted for in the above equations. These effects should be evaluated if judged significant.

Design Examples. In order to study and illustrate the effects of the change from allowable stress to ultimate strength design of foundations a series of examples were generated. The examples compared the size of foundations resulting from ultimate strength designs (USD) according the new procedures with those that would be obtained from conventional allowable stress designs (ASD).

The examples were based upon a single six-story reinforced concrete building with shear walls and gravity frame (see Figure 7.2.3-5). One set of examples was for a shallow spread footing design beneath a shear wall. The other set applied to deep CIDH piers placed beneath the same wall. For each set of examples, individual designs reflected a range of soil strengths and ASD factors of safety. The vertical loads were not changed, but two levels of seismic overturning demand were imposed.

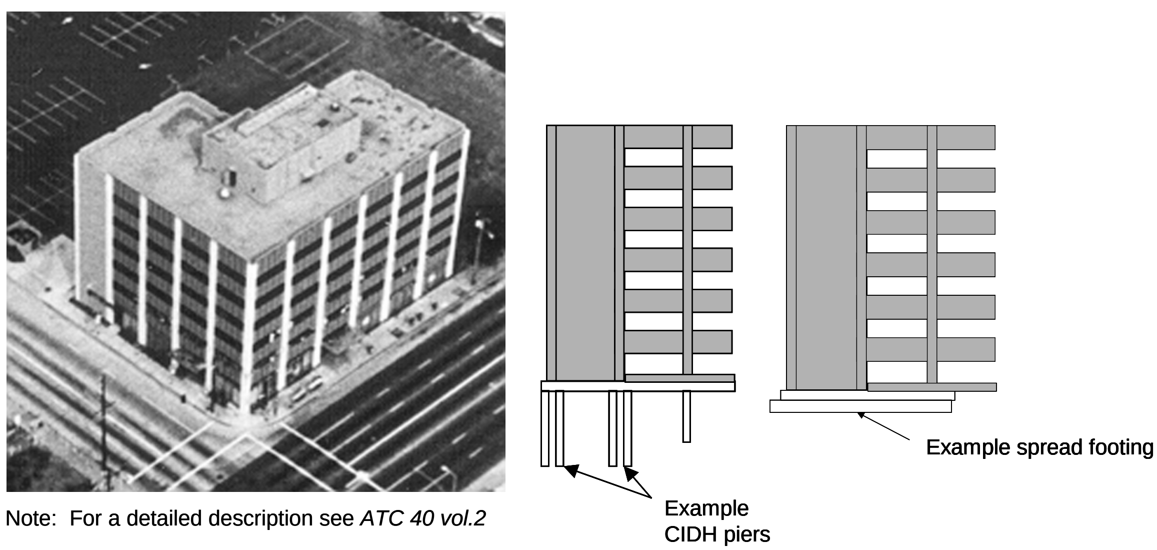


Figure 7.2.3-5 Example building.

While is not possible to generalize the results of these examples to apply universally, they are representative of the effects of the change to USD for a realistic case study. For the spread footing foundation the area of the footing for USD compared to that for ASD is controlled by the factor of safety applied to the soil strength for vertical loads. This reduction ranged from 0% to 20 percent for a low FOS (2) up to 25 percent to 40 percent for a high FOS (4). This is not surprising; where ASD uses a high factor of safety and is thus most conservative, USD results in a smaller footing size. However, the footing size cannot be smaller than that required for allowable stresses for static design under vertical dead plus live loads. For the pier example, the required length for USD was actually about 50 percent greater than for ASD for a low FOS (1.5) and up to 40 percent less for a high FOS (4).

REFERENCES

ATC, 1996, ATC 40, *Seismic Evaluation and Retrofit of Concrete Buildings*, 2 Volumes, The Applied Technology Council, Redwood City, CA.

Gazetas, G. (1991), “Foundation Vibrations,” *Foundation Engineering Handbook*, 2nd Edition, Vas Nostrand Reinhold, Edited by Hsai-Yang Fang.

Martin, G.R. and Lam, I.P., 2000, “Earthquake Resistant Design of Foundations: Retrofit of Existing Foundations,” *Proceedings*, GeoEng 2000, International Conference on Geological and Geotechnical Engineering, Melbourne, Australia, November.

NEHRP, 1997a, and 1997b, *National Earthquake Hazard Reduction Program Guidelines and Commentary for the Seismic Rehabilitation of Buildings*, developed for the Building Seismic Safety Council for the Federal Emergency Management Agency, (Publications FEMA 273 and 274).

SEAOC Seismology Committee, Ad Hoc Foundations Committee, 2001, “USD/LRFD/Limit State Approach to Foundation Design”, *Proceedings* of the 70th Annual SEAOC Convention, San Diego, California.