PROPOSAL IT 3-01 (2009)

SCOPE: White paper to be included in Part 3 of the 2009 *NEHRP Recommended Provisions*

PROPOSAL FOR CHANGE:

Add the following White Paper Titled *Appropriate Seismic Load Combinations for Base Plates, Anchorage and Foundations* to Part 3 of the 2009 *Provisions*:

See attached Issue Team 3 White Paper

REASON FOR PROPOSAL:

The purpose of this paper is to present findings from a study made to determine appropriate load conditions for base plates, anchorage (via anchor bolts, anchor rods, or other), and foundations (either shallow or deep).

Note: The attached white paper is all new material. It has not been underlined for easier reading.

Appropriate Seismic Load Combinations for Base Plates, Anchorage and Foundations

Introduction

The suitability of existing load combinations has been increasingly questioned as building code provisions have shifted from an allowable stress design (ASD) format towards an ultimate strength design (USD) format. Foundation design provisions have largely remained in ASD format because of a lack of consensus in how to convert the traditional foundation ASD approach into an ultimate strength format. Also, there has been disagreement regarding the appropriate requirements for base plates and anchorage, whereby building designers are inclined to specify use of the special seismic load combination for these elements, whereas designers of non-building structures tend to rely on inelastic behavior and, to some extent, uplift or sliding.

The purpose of this paper is to present findings from a study made to determine appropriate load conditions for base plates, anchorage (via anchor bolts, anchor rods, or other), and foundations (either shallow or deep).

Controlling Behavior of Structure Components in Series

The structural system that is created when a structure element is attached to a base plate, anchorage and foundation is a "series" combination of structure elements, as shown in Figure 1. In the simplest sense, a series combination can be conceptualized as a chain of components, in which the maximum strength and deformation capacity of the combination is controlled by whichever component is the weakest in the series.

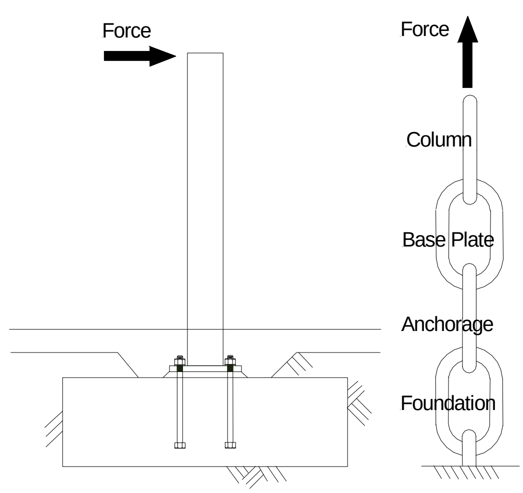

Figure 1: Structure Elements in Series
(Column, Base Plate, Anchorage, Foundation, and Soil)

In actuality, each component in a real structure has different strengths and deformation capacities. Figure 2 presents an example of the strengths and deformation capacities of three imaginary structure elements. System 1 is a flexible, ductile element; System 2 is a rigid, weaker but ductile element; and System 3 is a rigid, brittle but strong element. If these elements are connected into a series, the combined strength and deformation capacity of the system would be determined by summation of the individual displacements of each element at any given force level, as shown in

Figure 3. This type of combination is referred to as a force-dependent structural system.

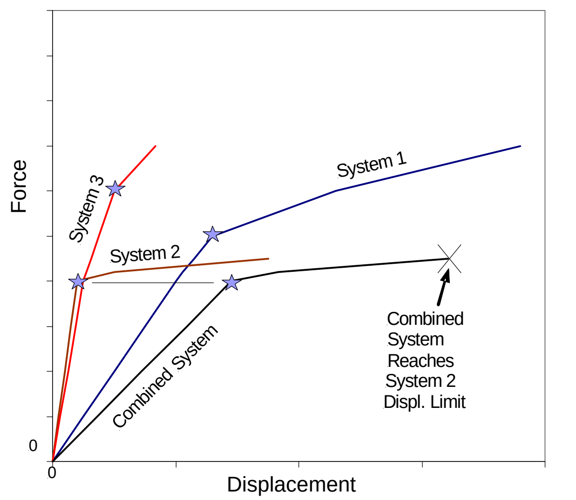

Figure 2: Force vs. Displacement of Series-Connected Elements

For the example shown in Figure 2, the combined strength and ductility capacity of the structural system is entirely controlled by System 2, because both the yield and ultimate strength of System 2 is less than the yield strength of either System 1 or System 3. For purposes of discussion, System 1 might be imagined as the behavior of a building structural element, System 2 might be the rocking behavior of a shallow foundation, and System 3 might be that of a low-ductility base plate and anchorage. The low ductility of System 3 is not a problem because this element always remains elastic, however the low strength of System 2 may be a problem because it prevents the relatively good ductility of System 1 from being utilized.

In order to transition the controlling behavior and mechanism from System 2 to that of System 1, the required strength of System 2 needs to be increased until the ultimate strength of System 1 is less than that of System 2, as shown in Figure 3. This demonstrates that careful scaling of load combination requirements for each component in any structural series is a necessary factor in controlling structure behavior.

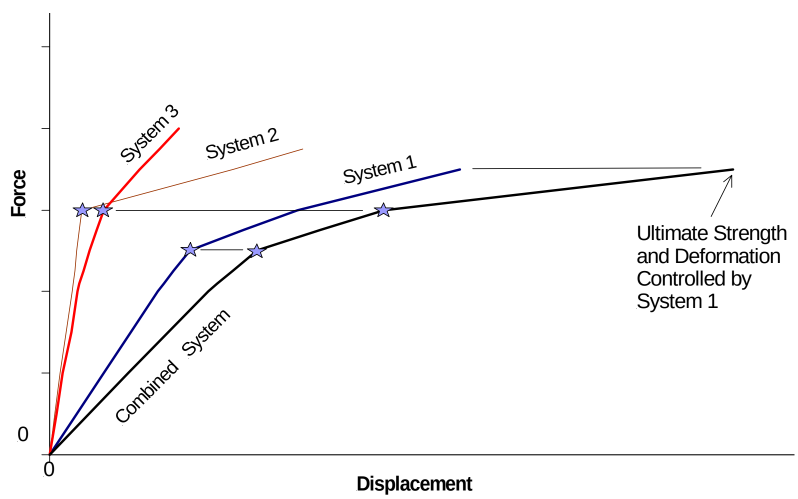

Figure 3: Using Load Factors to Increase Required Strength of System 2 Causes Behavior to be Controlled by System 1

Base Plates and Anchorage

Base plates and anchorages are commonly used for steel structures, light-frame structures, large non-building structures such as tanks, vessels, signs and the like, equipment attachments, and for nonstructural component attachments. Design standards and ductility requirements vary considerably for these items; Table 1 summarizes some of the broad variety of criteria currently used to define the seismic strength requirements and permitted capacity values for various types of structural elements that typically use some form of anchor rods/bolts and base plates or anchorages.

Current design standards for steel buildings have, for high-seismic areas, specified use of the special load combination for base plates and anchor rods[1] for steel columns. While these provisions in principle also apply for low-seismic areas, alternative design procedures that avoid this requirement are more likely to be commonly used for many items. Where anchor rods may be needed to attach elements other than columns, increased strength requirements are not currently required.

Where anchor bolts are required for light-frame construction, current design standards generally do not require any different strength requirements than for the attached structure component.

Designers of some types of non-building structures have shown a preference for using foundation anchor bolts as a yield mechanism to provide structural ductility. For example, ASCE 7-05 Section 15.7.5 and API standards require that vertical vessel structures typically found in oil refineries, which do not have significant ductility, be intentionally designed to create a plastic mechanism of tensile yielding in the anchor bolts used to attach the vessel to its foundation. The anchor bolts are specified to use ductile material and installed in a manner to facilitate tensile yielding over a significant length of the bolt. The anchorage used to attach the anchor bolts to the vessel as well as the vessel itself is then designed to mobilize the full strength of the anchor bolts.

Nonstructural components such as fan motors, piping systems and building facades often have cast-in or post-installed anchors with limited or no ductility for support. In some instances, the anchorage or bracket used to attach the component to the anchor is the element most capable of providing some degree of ductility in the attachment. In many cases imposed displacements may be the controlling factor in the anchorage design.

<u>Summary and Recommendations</u>

There is too much variety in structure and attachment types to define any single target behavior about which load combinations might be developed. Considering the wide variety of structures and components that utilize base plates and anchorages, there exist valid justifications to define ductility requirements for the structural element, the base plate/anchorage, or the anchor bolt. Recommended future code development should instead target rational rules within the three basic arenas of yield mechanisms. For each situation, specific design and detailing rules are appropriate to include in conjunction with the intended yield mechanism.

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[1] While AISC has introduced the term "anchor rod" to describe a bolt that attaches steel to concrete, other standards groups currently still use the term "anchor bolt." This paper uses the term "anchor rod" when specifically referring to AISC standards, and "anchor bolt" where anchorage in general is concerned.

Anchor Rod/Bolt as Yield Mechanism

1. Design the base plate/anchorage to resist the actual (not specified) tensile strength of the anchor bolt.

2. Design the foundation anchorage to resist the actual tensile strength of the anchor bolt

3. Use ductile steel for the anchor bolt. Use nuts capable of developing the anchor bolt.

4. In the case of cast-in and post-installed grouted anchors, consider de-bonding the anchor bolt from the concrete over a significant length (inelastic length) to permit development of meaningful displacements.

5. Use either continuously threaded rod to ensure uniform yielding over the inelastic length of the anchor bolt, or ensure that the rod material has sufficient tensile strength relative to its yield strength that the rod is fully yielded before tension fracture occurs. Upset threads are not considered necessary for anchors resisting seismic loads.

6. Consider use of nuts on both sides of base plate so that progressive elongation of the anchor bolt is reduced and cyclic reversals have a chance to cycle rod in compression (however, anchor bolts are not recommended for direct transfer of shear forces).

7. Provide adequate stretch length in the yielding section of anchor bolts to accommodate maximum expected inelastic displacements and rotations.

Anchorage/Base Plate as Yield Mechanism

1. Design the anchor bolt, particularly if non-ductile (such as an expansion bolt), to be stronger (elastic strength) than the yield strength of the anchorage assembly and with adequate displacement capacity to accommodate maximum joint movements.

2. Qualify post-installed anchor bolts by appropriate testing to confirm adequate strength and ductility characteristics under anticipated design conditions.

3. Although using an anchorage or base plate as the intended yield mechanism may be successful at protecting a non-ductile anchor bolt from failure, the total work performed in a small anchorage may not provide adequate hysteresis to reduce global structural seismic response.

Unyielding Anchorage/Anchor Bolt Assembly

Design requirements for the non-ductile structural elements currently exist. Care must be taken that load-amplification provisions for the anchor bolt/rod and base plate which are expected to remain elastic do not overlap.

Table 1

	System Type	R-Max	Element	Required Seismic Load Effect	Design Criteria	Avg. Anchor or Attachment Strength Relative to Supported Item
Steel Buildings	High Seismic; SDC D-F (AISC Definition)	SPSW[6] R = 7	Attachments	*E*	AISC Seismic	Same
			Anchorage	*uncertain*	ACI D3.3 w/AISC mod.	Same
		Other System Types, $R_{max} = 8$	Base plates	E_m	AISC Seismic	Same
			Anchorage	E_m	ACI D3.3 w/AISC mod.	Same
	Low Seismic; SDC A-C (AISC Definition)	Systems with R>3	(Same as High-Seismic SDC D-F Requirements)			
		Systems with R ≤ 3.0	Base plates	*E*	AISC 360	Same or Weaker[2]
			Anchorage	*E*	ACI D3.3	Same or Weaker[2]
Light-Frame Buildings	Shear Wall	7.0	Uplift Devices	*E*/1.4	ICC-ES	Varies
			Uplift Anchorage	*E*	ACI D3.3, SDC C-F	Stronger[1]
				E	ACI D3.3, SDC A-B	Same
			Shear Anchorage	*E*	ACI D3.3, SDC C-F	Stronger[1]
				E	ACI D3.3, SDC A-B	Same
NonBuilding Structures	Having building-like structural systems	8.0	(Same as Steel Buildings, incl. High & Low Seismic categorization)			
	Other Types	3.5	Base Plates & Attachments	*E*	AISC 360 [5]	Same
			Anchorage	*E*	ACI D3.3, SDC C-F other industry stds may govern	Stronger[1]
				E	ACI D3.3, SDC A-B	Same

Table 1, Continued

Nonstructural Components	Supports and Attachments (for Ductwork or welded piping)	R_p = 10.0[4] max	Base Plates & Attachments	*E or E*/1.4	Generally from ICC-ES ESRs	Same
	Supports and Attachments (Other)	R_p = 6.0 max	Base Plates & Attachments	*E or E*/1.4	Generally from ICC-ES ESRs	Same
	Anchors (seismically-qualified or per ACI D3.3)	R_p = 6.0 max	Anchorage per ASCE 13.4	*E*	ICC-ES AC193, AC308	?
					ACI D3.3, SDC C-F	Stronger[1]
					ACI D3.3, SDC A-B	Same
	Other Anchors (nonductile)	R_p = 1.5	Anchorage per ASCE 13.4	$(1.5/R_p)\ E$	ICC-ES AC193, AC308	Stronger

Notes

1.) Presumed stronger because ACI D3.3 applies a 0.75 strength reduction factor to the anchor strength.
2.) Weaker where supported item strength is determined by drift or other considerations.
3.) ASD Strengths determined using ICC-ES reports are based on tests.
4.) Welded Piping R_p = 12 is effectively only R_p = 10 due to $F_{p\ min}$ requirement.
5.) API, AWWA requires anchorage to be designed for yield load of anchor
6.) SPSW = Steel Plate Shear Walls

Geotechnical vs. Structural Engineering Perspectives on Foundations

A geotechnical engineer tends to view the foundation as a rigid body that is imposing loads and deformations and concerned with stresses and deformations of the soil surrounding the foundations. A structural engineer tends to treat soil similar to a fluid pressure that produces stresses within the foundation. A geotechnical engineer tends to define the ultimate strength of a foundation at a point when either an unstable soil movement is imminent or a limiting value of displacement is reached. A structural engineer tends to define the ultimate strength of a foundation at a point when either an unstable mechanism within the structure is imminent (such as rocking) or a structural capacity is reached. The fallacy of the geotechnical engineer is the assumption that the soil will fail before the structure; the fallacy of the structural engineer is the assumption that soil behavior can be simplified to the extent of being a simple fluid or force.

Conventional design procedures cause geotechnical engineers to define soil strength values for both seismic and long-term load conditions at an early point in the design process, when the size, shape and ultimate loading on the foundations are at best only roughly known. Unless ultimate foundation strengths can be re-evaluated by the geotechnical engineer at a design stage when the sizes, shapes and loading of foundations are relatively definite, it is normal that the geotechnical engineer maintain some degree of conservatism against potential geotechnical mechanisms. However, the traditional practice of arbitrarily defining a one-third increase in permitted long-term soil pressures for seismic loading does not adequately reflect what is necessary to transition from an ASD to an ultimate strength design practice. While a one-third increase might remain suitable for checking stresses for a 100-year wind event, it is not suitable for determining adequacy for a limit-state seismic event. It therefore becomes necessary to separately define design limit values for limit-state and long-term load conditions. Table 1804.1 of the 2006 IBC (reproduced below) will require substantial revision as a part of any change to strength design procedures.

2006 IBC Table 1804.1
ALLOWABLE FOUNDATION AND BEARING PRESSURE

CLASS OF MATERIALS	Allowable Foundation Pressure (psf)	Lateral Bearing (psf/ft below natural grade)	Lateral Sliding	
			Coefficient of friction	Resistance (psf)
1. Crystalline bedrock	12,000	1,200	0.70	--
2. Sedimentary and foliated rock	4,000	400	0.35	--
3. Sandy gravel and/or gravel (GW and GP)	3,000	200	0.35	--
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM, and GC)	2,000	150	0.25	--
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500	100	--	130

(footnotes a. to c. omitted)

d. An increase of one-third is permitted when using the alternate load combinations in Section 1605.3.2 that include wind or earthquake loads.

Performance Statement for Soil Limit State Condition

In order to define soil and foundation strength values associated with limit state design, a definitive performance statement for structural and geotechnical conditions at the limit state needs to be developed.

<u>Settlement and Soil Movement.</u> When structural actions result in repeated cycles of loading at or near the limit-state soil pressure, some degree of progressive foundation settlement is expected to occur due to compaction and local shear movements of soil materials beneath the foundation as shown in Figure 4. The total and differential settlements resulting from repeated cycles of loading should be considered in the light of the performance-based design criteria. Large total settlement may not be detrimental if the differential settlements between adjacent foundations are within acceptable limits.

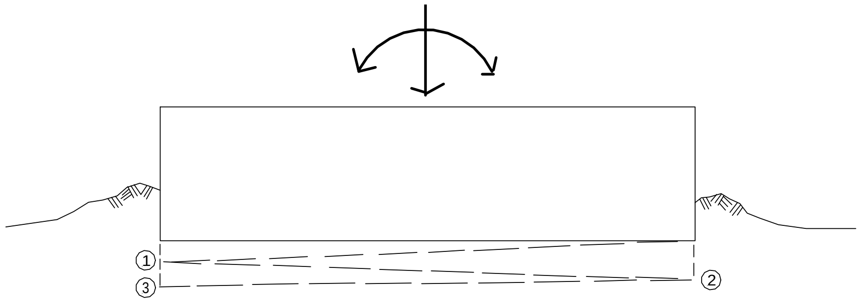


Figure 4: Progressive Settlement during repeated cycling

Rotational mechanisms of foundations due to soil shear failures as shown in Figure 5 should not be permitted. Maximum structure overturning moments should maintain a factor of safety against soil shear failure mechanisms of at least 2; otherwise foundations should be interconnected by grade beams so that the resulting soil loading will be primarily direct compression.

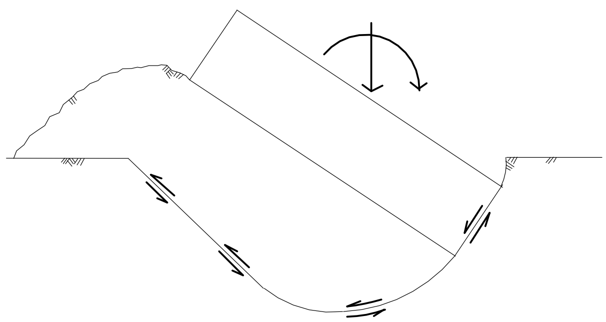


Figure 5: Foundation Rotational Mechanism within Soil

Lateral sliding of buildings and other structures may be resisted by both friction and passive soil pressure. Lateral displacement or sliding of foundations during the design event may be permissible, however, structural stability must be maintained.

Proposed new IBC Table (to follow existing Table 1804.1)
LIMIT STATE FOUNDATION AND BEARING PRESSURE

CLASS OF MATERIALS	Ultimate Foundation Pressure (psf)	Lateral Bearing (psf/ft below natural grade)	Lateral Sliding	
			Coefficient of friction	Resistance (psf)
6. Crystalline bedrock	24,000	2,500	0.70	--
7. Sedimentary and foliated rock	10,000	1,000	0.35	--
8. Sandy gravel and/or gravel (GW and GP)	8,000	600	0.35	--
9. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM, and GC)	6,000	500	0.25	--
10. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	4,500	300	--	400

Strength and Overstrength of Shallow Foundations

Current Code Approach

Past and current code provisions for both shallow and deep foundations have been based on Allowable Strength design methodologies. FEMA 450-1 includes an appendix that has proposed a new strength design methodology. An evaluation of the foundation design provisions therefore must address both methodologies.

Selected 2006 IBC sections relative to seismic load combination requirements for shallow foundations follow:

- 1605.2.1 – permits use of Strength load combinations in conjunction with the maximum 25% reduction in overturning moment permitted in ASCE 7 section 12.13.4.

- 1605.3.1 – permits use of ASD load combinations [$D + H + F + 0.7E$] and [$0.6D + 0.7E + H$].

- 1605.3.2 – permits use of alternative ASD load combinations [$D + L + S + \mathrm{E}/1.4$] and [$0.9D + E/1.4$] without the overturning reduction permitted by ASCE 7 section 12.13.4.

- Table 1804.2, Footnote "d." permits a one-third increase in allowable soil pressures when using the alternate load combinations that include seismic loads.

In current practice, the load combinations defined in Section 1605.3.2, in combination with the one-third increase permitted in Table 1804.2 are commonly used.

An unusual additional load combination provision is found in ACI-318 section 15.2.2. "*Base area of footing or number and arrangement of piles shall be determined from unfactored forces and moments transmitted by footing to soil or piles and permissible soil pressure of permissible pile capacity determined through principles of soil mechanics.*" Although ACI-318 section 21.10 (seismic foundation requirements) does not over-ride this section, this is in conflict with IBC section 1605, which would govern over the ACI provision.

Traditionally, the structural design of shallow foundations assumes that soil pressure beneath the foundations can be treated as a linearly-varying pressure across the length of the foundation, forming a pressure diagram which, depending upon the degree of eccentricity, e = M/P, can be described as either trapezoidal or triangular in shape. In 2003, FEMA 450-1 introduced a foundation strength design approach that permits a Whitney stress-block approach to be used to simulate an ultimate soil pressure condition to be used to design shallow foundations. Appendix 1 presents a summary of equations (1) and (2) that describe the ASD load limits of simple rectangular-in-plan foundations. It also includes a similar equation (3) that describes the strength limits for the strength design approach described in the Appendix to Chapter 7 of FEMA 450-1. Using Equations 1 through 3, simple load vs. moment interaction curves can be developed for any rectangular foundation shape, as shown in Figure 6.

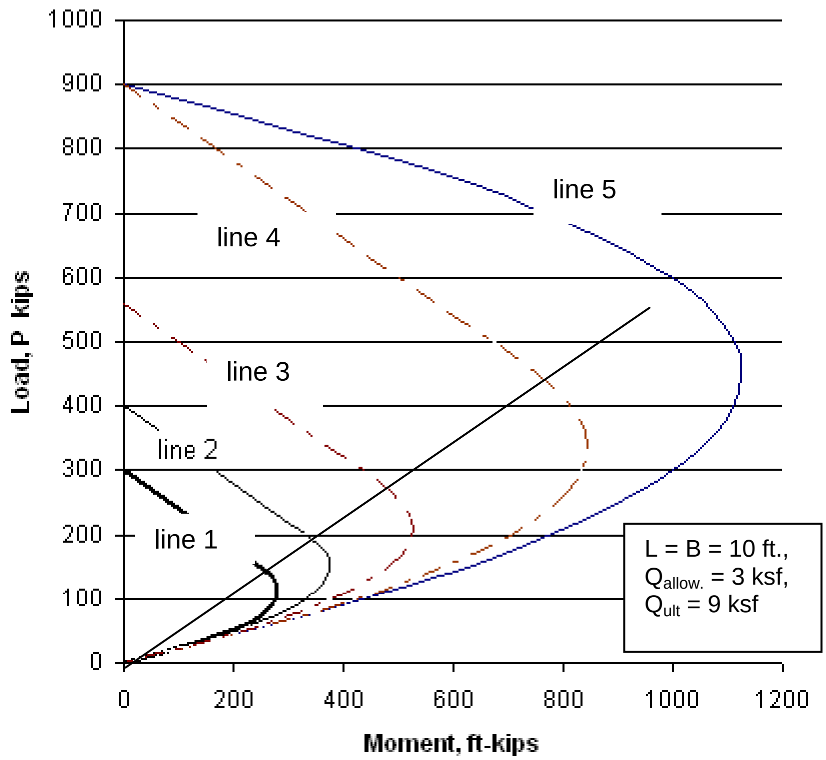


Figure 6 – Example Interaction Curve for a Shallow Foundation

Figure 6 presents an example interaction curve for a 10-foot square foundation, with an allowable long-term soil pressure of Q_{allow} = 3 ksf, and assumed ultimate soil strength of Q_{ult} = 9 ksf. In the figure:

- The radial line occurs at e = L/6, the transition from trapezoidal to triangular soil pressure distribution.

- Line 1 represents an interaction curve using ASD design assumptions, with an allowable soil pressure of 3 ksf.

- Line 2 represents the effect of a 33 percent allowable increase in soil pressure for temporary load conditions, to 4 ksf.

- Line 3 represents the effect of using IBC Section 1605.3.2 to design foundations (the reduction of E/1.4 is represented as an increase in allowable soil pressure by a factor of 1.4).

- Line 4 represents the interaction curve at the ultimate soil pressure of 9 ksf, but using traditional triangular/trapezoidal soil pressure distribution (i.e., the ultimate soil pressure occurs only at the extreme edge of the foundation).

- Line 5 represents the interaction curve at the ultimate soil pressure of 9 ksf, and using a equal-pressure soil distribution.

The overstrength of the traditional ASD design approach can be expressed as the ratio between the presumed ultimate (line 5) and the design-level (line 3) interaction curves. The amount of overstrength that results using the ASD design approach is not constant, and varies significantly depending on how much vertical load is on the foundation. Let us define P as the actual vertical load on a foundation and *P'* and the theoretical maximum permitted vertical load capacity of a concentrically loaded foundation (equal to the maximum permitted soil pressure times the total footing area). For more lightly loaded foundations (having $P/P' < 0.5$), the amount of overstrength present varies significantly, to the extent that when a foundation is at $P/P' = 0$ (such as when a foundation is loaded in direct uplift), the effective Factor of Safety present is 1.0 (i.e., no overstrength).

<u>2003 FEMA 450-1 Ultimate Strength Approach</u>

While the foundation strength design approach defined in the Chapter 7 appendix defines procedures that can be used to determine an ultimate strength design such as shown in Line 5 in Figure 6, it is silent regarding which strength load combinations to use for design. The available alternatives to use are either the seismic load combinations defined in ASCE 7-05 Section 12.4.2.3 or the Special Load Combinations defined in Section 12.4.3.2.

- The basic strength load combinations are not generally appropriate to use in conjunction with ultimate foundation strength values. Using load combinations incorporating 1.0 E together with the ultimate foundation strength means that the design procedure permits no overstrength to be present at all in the design, i.e., that foundation failure will always be the dominant controlling mechanism in any structure. It also means that the expected ductility capacity of the resulting foundation mechanism must equal or exceed the value of R used in the design (whereas for the building structure the expected ductility demand is in some manner proportional to R/Ω_O).

- If the special load combination is used in conjunction with ultimate foundation strength values, then foundation rocking or sliding mechanisms are unlikely to be a controlling or participating mechanism in the structure response. While this might be an acceptable or desired characteristic for structures using high-R systems or for essential facilities, it is probably an undesirable characteristic for ordinary-use structures using moderate- or low-R systems. Because modest levels of foundation nonlinearity is generally considered to be acceptable for ordinary structures using moderate or lower R systems, the use of special load combinations would prevent such action and would result in an increase in their expected construction cost.

Strength of Deep Foundations

Although the ultimate strength of a deep foundation cannot be simplified in the same manner as a shallow foundation, simplified methods can be used to predict ultimate strength values that have slight resemblance to reality. Geotechnical engineers can determine allowable ultimate and long-term load capacities of assumed pile groups, translate that into individual-pile ultimate and long-term load values for the structural engineer, and then the structural engineer can translate those back into predicted ultimate and long-term pile group capacities that may or may not resemble the values originally determined by the geotechnical engineer.

Appendix 2 presents two examples of how a structural engineer might estimate the ultimate strength of a pile group based on individual pile capacities. Both of these approaches are vast over-simplifications of the actual interaction and response that occurs between the structure and soil of a deep foundation, but they are both simple enough for practicing engineers to adopt as design practice. The first approach is a modification of a current common design practice for multi-pile foundations that assumes the ultimate strength point is reached when the outermost pile reaches a defined ultimate strength. The second is a plastic-analysis approach that assumes that all piles in a pile group are eventually able to reach their defined ultimate strengths. The plastic analysis approach likely over-estimates the strength that a multi-pile group is capable of developing, however, for either approach the phi-factor of 0.7 will provide significant compensation. Also, since for either approach the pile cap structure would need to have sufficient strength to accommodate the full expected strength of the foundation capacity that is used, many engineers would probably prefer the more conventional linear-strain approach in order to reduce the required strength of pile caps.

More accurate methods to predict the ultimate strength of deep foundations include field testing of individual piles, reduced-scale testing of pile groups, and prediction of strength and deformation states of both foundation and soil through complex models of the combined foundation and surrounding soil. Analysis of soil seismic behavior in this manner should include the strain-dependent strength of the soil materials due to both foundation loading and ongoing seismic deformations.

Overstrength of Deep Foundations

Deep and shallow foundations are significantly different in that deep foundations can have tensile strength, overturning strength with low gravity loads, and element over-strength properties similar to superstructure elements. Deep foundations might therefore be capable of internally developing overstrength values in the range of tabulated Ω_O values, provided that adequate ductility is present in the piles. There is therefore no clear need for specifying a special or increased load combination in order to offset a lack of overstrength in the foundation system, as there is for lightly-loaded shallow foundation systems. However, earthquake damage in deep foundations is difficult to detect, is probably frequently overlooked in post-earthquake damage investigations, and even if detected is very costly to repair. It might therefore justify an increase of deep foundation strength for structures having occupancies and sites resulting in higher seismic design categories, since foundations for those structures might be expected to experience more than one damaging earthquake during the foundation service life, because the potential loss-of-use and cost for repairs is less acceptable.

Recommendations for Foundations

The foundation design including soil pressures for either shallow or deep foundation systems may use USD load combinations in which the value of E used in ASCE 7-05 Section 2.3 load combinations would be replaced for foundation elements as shown from the following tables:

Buildings and Building-like Non-building Structures

R value from ASCE 7-05, Table 12.2-1,12.14-1 or 15.4-1	Fixed Base Analysis	If Foundation deformations per ASCE-41 are included
for R >=5	2.0 E	1.5 E
R 3 to <5	1.5 E	1.0 E
R<3	1.0 E	1.0 E

Nonbuilding Structures not similar to Buildings

R value from ASCE 7-05, Table 15.4-2	Fixed Base Analysis	If Foundation deformations per ASCE-41 are included
R > 3	1.5 E	1.0 E
R<=3	1.0 E	1.0 E

It is likely that this scaling would apply to the full value of $E = E_h + E_v$ used in design, with no other reduction permitted, although it is recognized that the full effects of design including redundancy factors, importance factors and the vertical seismic component has not been studied in depth and might warrant some further improvements in the future.

Reason:

The load factor scaling factors selected were chosen in conjunction with the foundation-soil strength values (including phi-factors) that have been presented in this paper. Inherently, the load factor of 2.0E is intended to result in a structure in which inelastic response is preferred in the portions of structure that are above the foundation and base plate, while the load factor of 1.0 E was selected with an intent that some inelastic response might be preferred in the foundation of light structures. The Load factor of 1.5 E was selected as a value in which inelastic response might occur either in the foundation, in the supported structure, or in both elements.

It is recognized that simple rules often yield imperfect results, and that some structural systems might be identified that defy the logic of this reasoning. For instance, foundations beneath shear panels of light-frame buildings would be required to be designed using 2.0 E, suggesting that it is preferred that the foundations beneath these elements remain relatively elastic, whereas many engineers might argue that a load factor of 1.5 E might be more appropriate. However, until a more rational means of determining R values is determined this relatively simple table was judged to be generally effective in resulting in the preferred inelastic behavior distribution.

No distinction was made between the load factor recommendations for deep and shallow foundations because, in the final consideration, the load factors recommended appear likely to result in at least as much successful behavior for deep as opposed to shallow foundations.

<u>ASD Equivalents</u>

An underlying intent of this white paper is to address load combinations needed for a strength design approach for foundations; however it must be recognized that ASD load combinations may continue to be recognized in building codes. Common ASD methods currently used for seismic design can be approximately matched with ultimate foundation strengths discussed herein by dividing calculated ultimate foundation strengths by a factor of safety of 3.0, and using the earthquake forces recommended above reduced by a factor of 1.4.

References:

1. "Background information for some of the proposed earthquake design provisions for the 2005 edition of the National Building Code of Canada," Ronald H. DeVall, NRC Research Press, 4 April 2003.

2. National Building Code of Canada, 2005, Vol. 1 and Commentary.

3. International Building Code, 2006 edition, ICC.

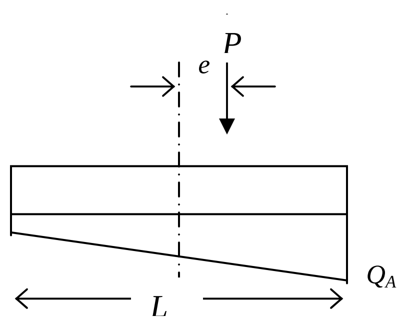
4. ACI 318-05, "Building Code Requirements for Structural Concrete and Commentary"

5. AISC 341-05, "Seismic Provisions for Structural Steel Buildings"

6. ASCE/SEI 41-06, "Seismic Rehabilitation of Existing Buildings"

Appendix 1
Derivation of Shallow Foundation Equations

Traditional ASD Design – Full Contact



Given:

P = Vertical Load
M = Overturning Moment
L = Length of Rectangular Footing
B = Width of Rectangular Footing
e = M / P = Eccentricity of Loading
Q_A = Maximum ASD Allowable Soil Pressure

From the standard ending stress equation:

$\sigma = \dfrac{P}{A} \pm \dfrac{M}{S}$, the maximum soil pressure, Q_A will be:

$$Q_A = \frac{P}{BL} + \frac{6M}{BL^2} = \frac{P}{BL}\left(1 + \frac{6e}{L}\right)$$

Rearranging;

$$e = \frac{L}{6}\left(\frac{Q_A}{P}BL - 1\right)$$

Introduce the term: $P' = Q_A BL$ so that we can substitute $BL = \frac{P'}{Q_A}$ resulting in;

$$\frac{e}{L} = \frac{1}{6}\left(\frac{P}{P'} - 1\right) \qquad \text{(Equation 1)}$$

Traditional ASD Design – Partial Contact

For $e \geq \frac{L}{6}$ the soil pressure is assumed as a triangular distribution.

$$Q_A = \frac{2P}{3B\left(L/2 - e\right)}$$

Rearranging;

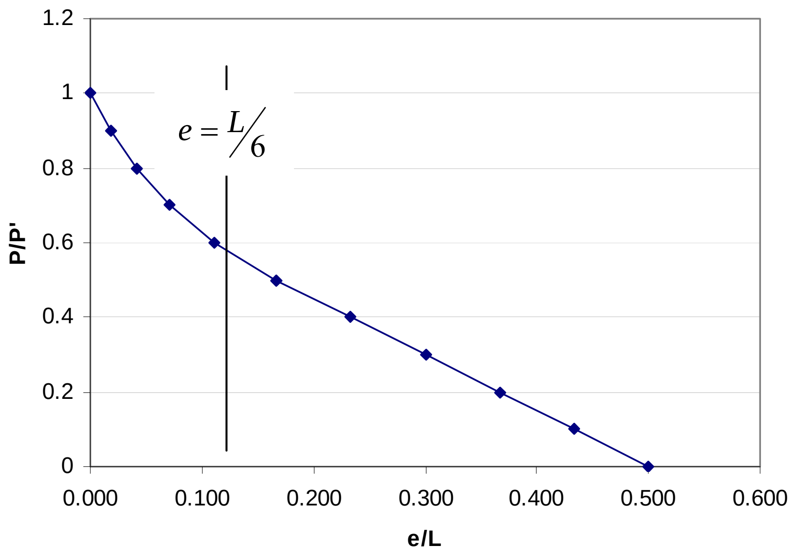
$$e = \frac{L}{2} - \frac{2}{3}\frac{P}{Q_A B}$$

Substituting $Q_A B = \frac{P'}{L}$,

$$\frac{e}{L} = \frac{1}{2}\left[1 - \frac{4}{3}\left(\frac{P}{P'}\right)\right] \qquad \text{(Equation 2)}$$

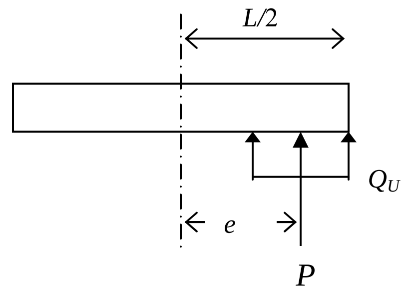
At the transition point between Equations 1 and 2, $e = \frac{L}{6}$ and $\frac{P}{P'} = \frac{1}{2}$.

The following is a graph of Equations 1 and 2:



Simplified Ultimate Strength Design Method

A simplified ultimate strength design approach, based on the Whitney Stress Block Method, follows:



Define (in addition to terms used above):

$Q_U =$ Ultimate Soil Pressure

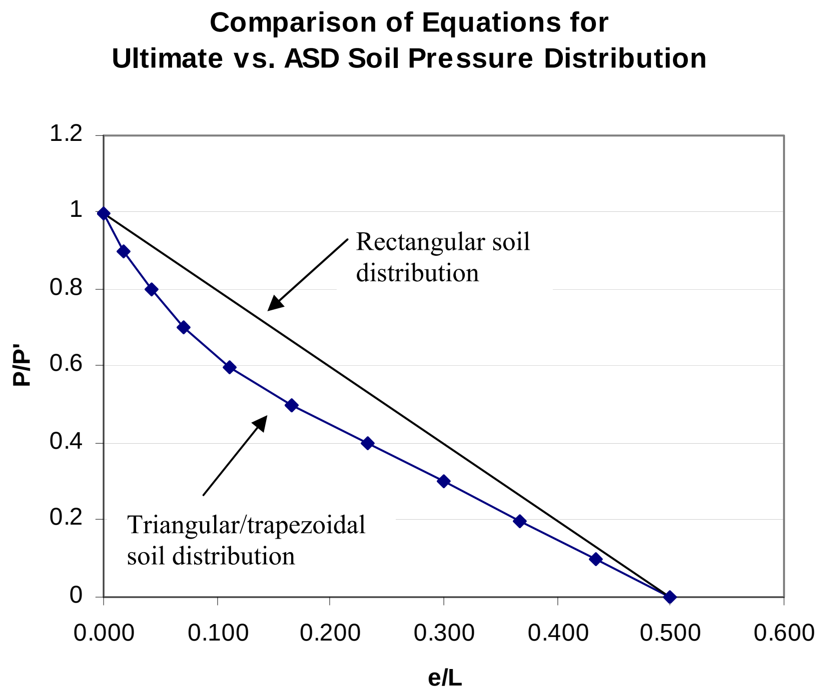
For an assumed rectangular soil pressure distribution;

$$Q_U = \frac{P}{2B\left(L/2 - e\right)}$$

Substituting $B = \dfrac{P'}{LQ_A}$,

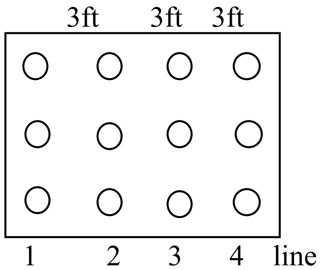
$$\frac{e}{L} = \frac{1}{2}\left[1 - \left(\frac{Q_A}{Q_U}\right)\left(\frac{P}{P'}\right)\right] \qquad \text{(Equation 3)}$$

Graphing this equation against the ASD equations;


Comparison of Equations for
Ultimate vs. ASD Soil Pressure Distribution
1.2
1
0.8
0.6
0.4
0.2
0
P/P'
0.000
0.100
0.200
0.300
0.400
0.500
0.600
e/L
Rectangular soil
distribution
Triangular/trapezoidal
soil distribution

Appendix 2
ASD and LRFD Interaction Diagrams for Deep Foundations

Linear Strain Assumption

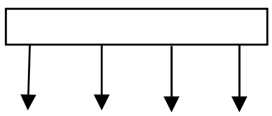


Assumed ASD Allowable Pile Capacities

P =	100	kips
T =	-50	kips
Ultimate/ASD =	1.7	tension
Ultimate/ASD =	2.5	compression

(does not include phi = 0.7)

Point 1 - Pure compression

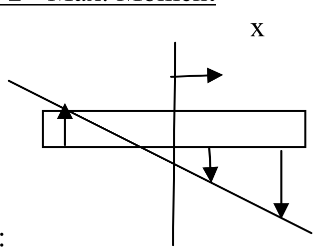


Line	# piles	#piles x Pile force	x	Px
1	3	300	-4.5	-1350.0
2	3	300	-1.5	-450.0
3	3	300	1.5	450.0
4	3	300	4.5	1350.0
Sum =		1200		0.0

ASD Capacity
1200 kips
0.0 ft kips

USD Capacity
3000 kips
0.0 ft kips

Point 2 - Max. Moment



ASD:
X na = -1.50 ft

USD:
X na = -2.22 ft

ASD:

Line	# piles	#piles x Pile force	x	Px
1	3	-150	-4.5	675.0
2	3	0	-1.5	0.0
3	3	150	1.5	225.0
4	3	300	4.5	1350.0
Sum =		300		2250.0

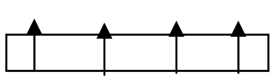
ASD Capacity
300 kips
2250.0 ft kips

USD:

Line	# piles	#piles x Pile force	x	Px
1	3	-255	-4.5	1147.5
2	3	80	-1.5	-120.0
3	3	415	1.5	622.5
4	3	750	4.5	3375.0
Sum =		990		5025.0

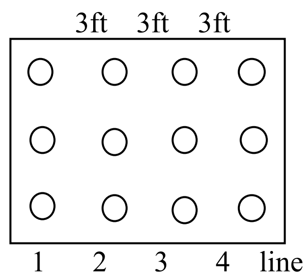
USD Capacity
990 kips
5025.0 ft kips

Point 3 - Pure tension



	ASD Capacity		USD Capacity	
P = 4 * 3 * -50				
=	-600	kips	-1020	kips
M =	0.0	ft kips	0.0	ft kkips

Fully Plastic Assumption



ASD Allowable Pile Capacities

P =	100	kips
T =	-50	kips
Ultimate/ASD =	1.7	tension
Ultimate/ASD =	2.5	compression

(does not include phi = 0.7)

Point 1

Line	# piles	#piles x Pile force	x	Px	USD Capacity	
1	3	750	-4.5	-3375.0	3000	kips
2	3	750	-1.5	-1125.0	0.0	ft kips
3	3	750	1.5	1125.0		
4	3	750	4.5	3375.0		
Sum =		3000		0.0		

Point 2

Line	# piles	#piles x Pile force	x	Px		
1	3	-255	-4.5	1147.5		
2	3	750	-1.5	-1125.0		
3	3	750	1.5	1125.0	USD Capacity	
4	3	750	4.5	3375.0	1995	kips
Sum =		1995		4522.5	4522.5	ft kips

Point 3

Line	# piles	#piles x Pile force	x	Px		
1	3	-255	-4.5	1147.5		
2	3	-255	-1.5	382.5	USD Capacity	
3	3	750	1.5	1125.0	990	kips
4	3	750	4.5	3375.0	6030.0	ft kips
Sum =		990		6030.0		

Point 4

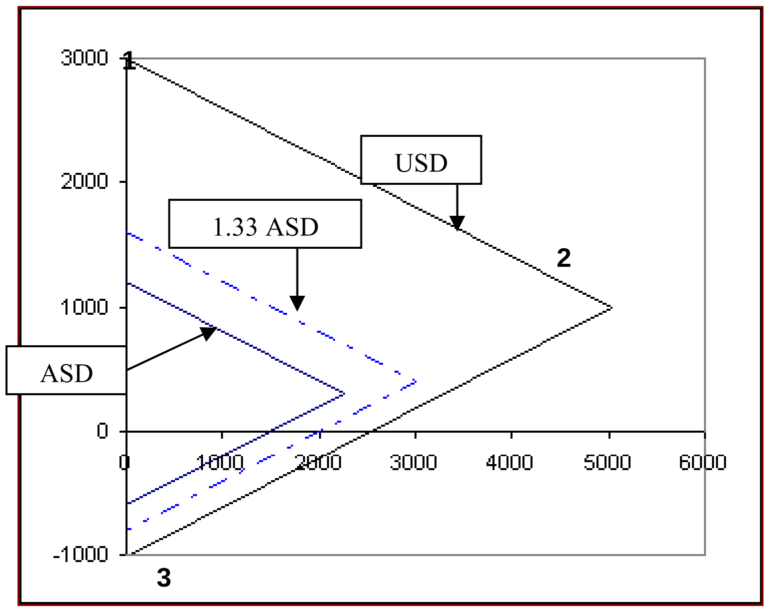
Line	# piles	#piles x Pile force	x	Px	
1	3	-255	-4.5	1147.5	
2	3	-255	-1.5	382.5	USD Capacity

3	3	-255	1.5	-382.5	-15	kips
4	3	750	4.5	3375.0	4522.5	ft kips
Sum =		-15		4522.5		

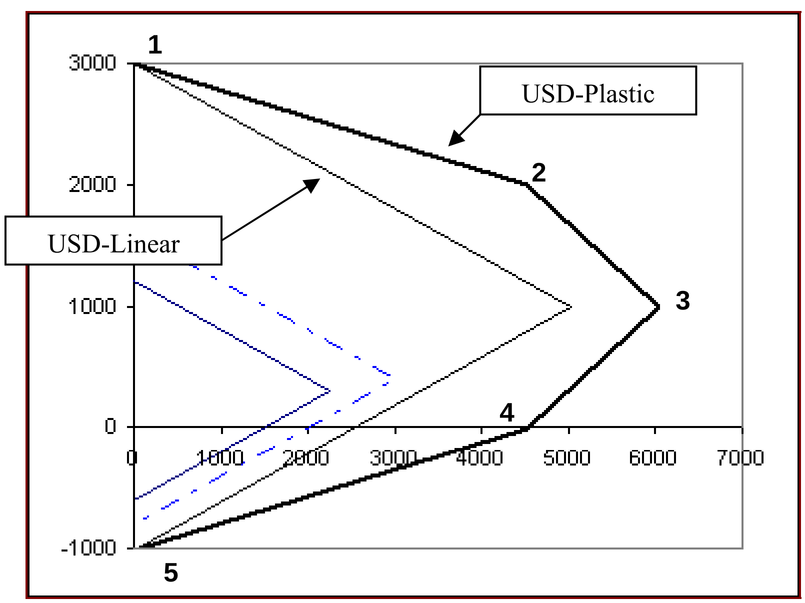
Point 5

Line	# piles	#piles x Pile force	x	Px
1	3	-255	-4.5	1147.5
2	3	-255	-1.5	382.5
3	3	-255	1.5	-382.5
4	3	-255	4.5	-1147.5
Sum =		-1020		0.0

USD Capacity	
-1020	kips
0.0	ft kips



Linear Strain Assumption



Fully Plastic Assumption,
Superimposed on Linear Strain Assumption