

1 **PROPOSAL IT 3-01 (2009)**  
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5 **SCOPE: White paper to be included in Part 3 of the 2009 NEHRP**  
6 ***Recommended Provisions***  
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10 **PROPOSAL FOR CHANGE:**  
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12 **Add the following White Paper Titled *Appropriate Seismic Load***  
13 ***Combinations for base Plates, Anchorage and Foundations to Part 3 of***  
14 ***the 2009 Provisions:***

15  
16 *See attached Issue Team 3 White Paper*  
17

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

23  
24 **IT 3 VOTE:**  
25

26 IT 3 members were sent the white paper for review. The IT 3 chair received one editorial  
27 comment which was made. The white paper was re-sent to all IT 3 members and they  
28 voted unanimously to approve.  
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Issue Team #3 White Paper

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 (ASD) format towards a strength design, or LRFD, 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 or components, in which the maximum strength and deformation capacity of the combination is controlled by whichever component is the weakest in the series.

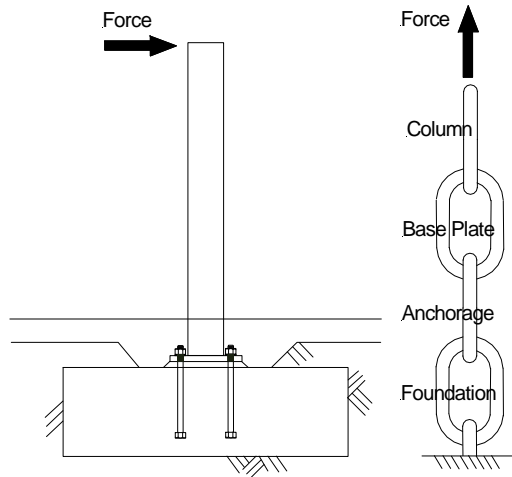
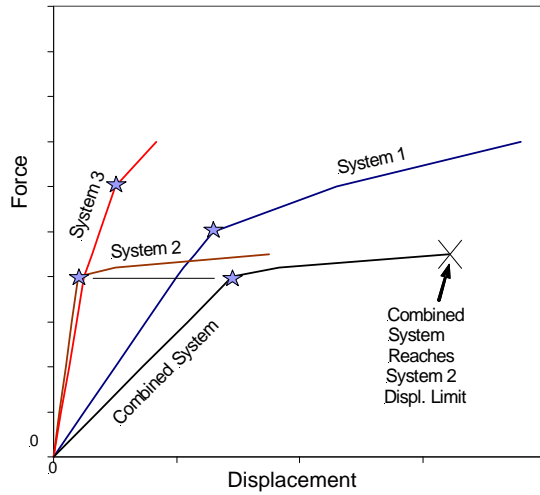


Figure 1: Structure Elements in Series  
(Column, Base Plate, Anchorage, Foundation, and Soil)

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

1 series, the combined strength and deformation capacity of the system would be determined by  
2 summation of the individual displacements of each element at any given force level, as shown in  
3 Figure 3. This type of combination is referred to as a force-dependent structural system.

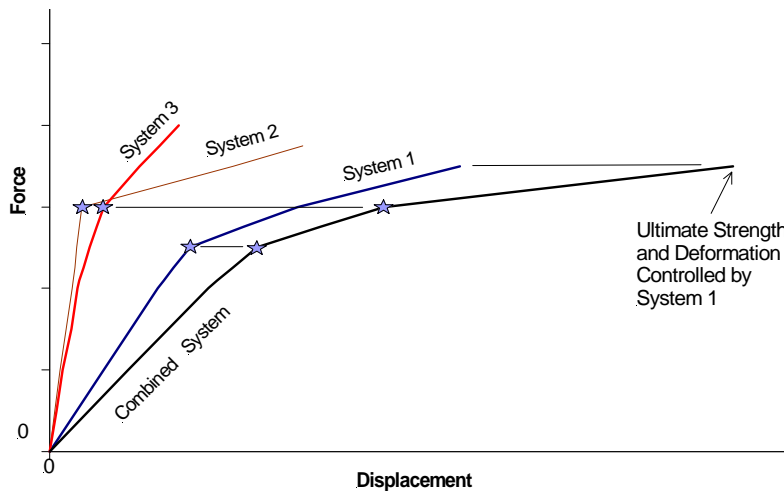


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Figure 2: Force vs. Displacement of Series-Connected Elements

6 For the example shown in Figure 2, the combined strength and ductility capacity of the structural  
7 system is entirely controlled by System 2, because both the yield and ultimate strength of System  
8 2 is less than the yield strength of either System 1 or System 3. For purposes of discussion,  
9 System 1 might be imagined as the behavior of a building structural element, System 2 might be  
10 the rocking behavior of a shallow foundation, and System 3 might be that of a low-ductility base  
11 plate and anchorage. The low ductility of System 3 is not a problem because this element always  
12 remains elastic, however the low strength of System 2 may be a problem because it prevents the  
13 relatively good ductility of System 1 from being utilized.

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



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

### 3 **Base Plates and Anchorage**

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

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

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

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

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

### 32 Summary and Recommendations

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

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<sup>1</sup> 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.

1 Anchor Rod/Bolt as Yield Mechanism

- 2 1. Design the base plate/anchorage to resist the actual (not specified) tensile strength of the  
3 anchor bolt
- 4 2. Design the foundation anchorage to resist the actual tensile strength of the anchor bolt
- 5 3. Use ductile steel for the anchor bolt. Use nuts capable of developing the anchor bolt.
- 6 4. In the case of cast-in and post-installed grouted anchors, consider de-bonding the anchor  
7 bolt from the concrete over a significant length (inelastic length) to permit development  
8 of meaningful displacements.
- 9 5. Use either continuously threaded rod to ensure uniform yielding over the inelastic length  
10 of the anchor bolt, or ensure that the rod material has sufficient tensile strength relative to  
11 its yield strength that the rod is fully yielded before tension fracture occurs. Upset threads  
12 are not considered necessary for anchors resisting seismic loads.
- 13 6. Consider use of nuts on both sides of base plate so that progressive elongation of the  
14 anchor bolt is reduced and cyclic reversals have a chance to cycle rod in compression  
15 (however, anchor bolts are not recommended for direct transfer of shear forces).
- 16 7. Provide adequate stretch length in the yielding section of anchor bolts to accommodate  
17 maximum expected inelastic displacements and rotations.

18 Anchorage/Base Plate as Yield Mechanism

- 19 1. Design the anchor bolt, particularly if non-ductile (such as an expansion bolt), to be  
20 stronger (elastic strength) than the yield strength of the anchorage assembly and with  
21 adequate displacement capacity to accommodate maximum joint movements.
- 22 2. Qualify post-installed anchor bolts by appropriate testing to confirm adequate strength  
23 and ductility characteristics under anticipated design conditions.
- 24 3. Although using an anchorage or base plate as the intended yield mechanism may be  
25 successful at protecting a non-ductile anchor bolt from failure, the total work performed  
26 in a small anchorage may not provide adequate hysteresis to reduce global structural  
27 seismic behavior.

28 Unyielding Anchorage/Anchor Bolt Assembly

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

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**Table 1**

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

<b>Nonstructural Components</b>	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 <sup>(1)</sup>
					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

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**3 Geotechnical vs. Structural Engineering Perspectives on Foundations**

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

14 Conventional design procedures cause geotechnical engineers to define soil strength values for  
 15 both seismic and long-term load conditions at an early point in the design process, when the size,  
 16 shape and ultimate loading on the foundations are at best only roughly known. Unless ultimate  
 17 foundation strengths can be re-evaluated by the geotechnical engineer at a design stage when the  
 18 sizes, shapes and loading of foundations are relatively definite, it is normal that the geotechnical  
 19 engineer maintain some degree of conservatism against potential geotechnical mechanisms.  
 20 However, the traditional practice of arbitrarily defining a one-third increase in permitted long-

1 term soil pressures for seismic loading does not adequately reflect what is necessary to transition  
 2 from an ASD to an ultimate strength design practice. While a one-third increase might remain  
 3 suitable for checking stresses for a 100-year wind event, it is not suitable for determining  
 4 adequacy for a limit-state seismic event. It therefore becomes necessary to separately define  
 5 design limit values for limit-state and long-term load conditions. Table 1804.1 of the 2006 IBC  
 6 (reproduced below) will require substantial revision as a part of any change to strength design  
 7 procedures.

8 2006 IBC Table 1804.1  
 9 ALLOWABLE FOUNDATION AND BEARING PRESSURE

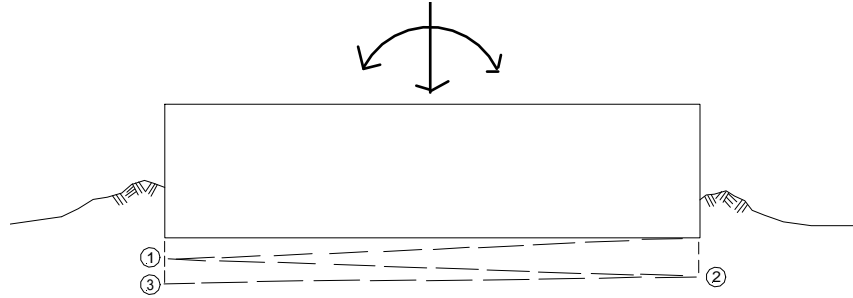
CLASS OF MATERIALS	ALLOWABLE FOUNDATIO N PRESSURE (psf)	LATERAL BEARING (psf/f below natural grade)	LATERAL SLIDING Coeffici ent of friction	Resistanc e (psf)
1. Crystalline bedrock	12,000	1,200	0.70	--
2. Sedimentary and foliated rock	4,000	400	0.35	--
3. Sandy gravel and/or gravel (GW and GP)	3,000	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

10 (footnotes a. to c. omitted)  
 11 d. An increase of one-third is permitted when using the alternate load combinations in Section  
 12 1605.3.2 that include wind or earthquake loads.

13 **Performance Statement for Soil Limit State Condition**

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

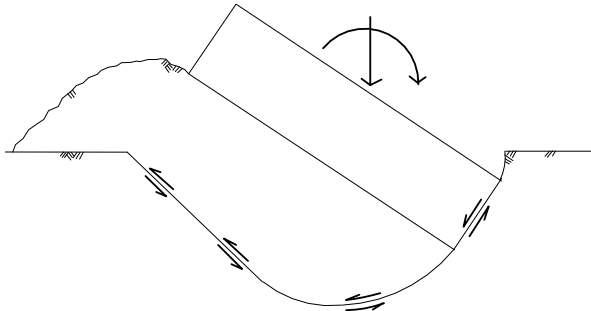
17 Settlement and Soil Movement. When structure actions result in repeated cycles of loading at or  
 18 near the limit-state soil pressure, some degree of progressive foundation settlement is expected to  
 19 occur due to compaction and local shear movements of soil materials beneath the foundation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



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Figure 4: Progressive Settlement during repeated cycling

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



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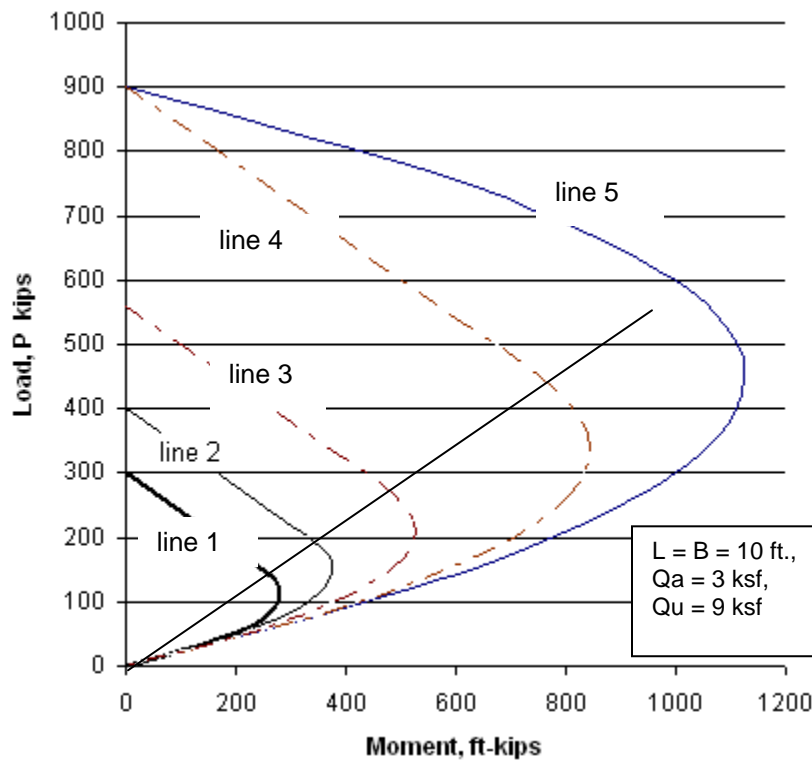
Figure 5: Foundation Rotational Mechanism within Soil

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



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

7 Traditionally, the structural design of shallow foundations assumes that soil pressure beneath the  
 8 foundations can be treated as a linearly-varying pressure across the length of the foundation,  
 9 forming a pressure diagram which, depending upon the degree of eccentricity,  $e = M/P$ , can be  
 10 described as either trapezoidal or triangular in shape. In 2003, FEMA 450-1 introduced a  
 11 foundation strength design approach that permits a Whitney stress-block approach to be used to  
 12 simulate an ultimate soil pressure condition to be used to design shallow foundations. Appendix  
 13 1 presents a summary of equations (1) and (2) that describe the ASD load limits of simple  
 14 rectangular-in-plan foundations. It also includes a similar equation (3) that describes the strength  
 15 limits for the strength design approach described in the Appendix to Chapter 7 of FEMA 450-1.  
 16 Using Equations 1 through 3, simple load vs. moment interaction curves can be developed for any  
 17 rectangular foundation shape, as shown in Figure 6.



18  
 19 Figure 6 – Example Interaction Curve for a Shallow Foundation

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

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

- 1 • Line 1 represents an interaction curve using ASD design assumptions, with an allowable soil  
2 pressure of 3 ksf.
- 3 • Line 2 represents the effect of a 33 percent allowable increase in soil pressure for temporary  
4 load conditions, to 4 ksf.
- 5 • Line 3 represents the effect of using IBC Section 1605.3.2 to design foundations (the  
6 reduction of  $E/1.4$  is represented as an increase in allowable soil pressure by a factor of 1.4).
- 7 • Line 4 represents the interaction curve at the ultimate soil pressure of 9 ksf, but using  
8 traditional triangular/trapezoidal soil pressure distribution (i.e., the ultimate soil pressure  
9 occurs only at the extreme edge of the foundation).
- 10 • Line 5 represents the interaction curve at the ultimate soil pressure of 9 ksf, and using a  
11 equal-pressure soil distribution.

12 The overstrength of the traditional ASD design approach can be expressed as the ratio between  
13 the presumed ultimate (line 5) and the design-level (line 3) interaction curves. The amount of  
14 overstrength that results using the ASD design approach is not constant, and varies significantly  
15 depending on how much vertical load is on the foundation. For more lightly loaded foundations  
16 (having  $P/P' < 0.5$ ), the amount of overstrength present varies significantly, to the extent that  
17 when a foundation is at  $P/P' = 0$  (such as when a foundation is loaded in direct uplift), the  
18 effective Factor of Safety present is 1.0 (i.e., no overstrength).

#### 19 2003 FEMA 450-1 Ultimate Strength Approach

20 While the foundation strength design approach defined in the Chapter 7 appendix defines  
21 procedures that can be used to determine an ultimate strength design such as shown in Line 5 if  
22 Figure 6, it is silent regarding which strength load combinations to use for design. The available  
23 alternatives to use are either the seismic load combinations defined in ASCE 7-05 Section  
24 12.4.2.3 or the Special Load Combinations defined in Section 12.4.3.2.

- 25 • The basic strength load combinations are not generally appropriate to use in conjunction with  
26 ultimate foundation strength values. Using load combinations incorporating 1.0 E together  
27 with the ultimate foundation strength means that the design procedure permits no  
28 overstrength to be present at all in the design, i.e., that foundation failure will always be the  
29 dominant controlling mechanism in any structure. It also means that the expected ductility  
30 capacity of the resulting foundation mechanism must equal or exceed the value of R used in  
31 the design (whereas for the building structure the expected ductility demand is  $R_d = R / R_0$ ).
- 32 • If the special load combination is used in conjunction with ultimate foundation strength  
33 values, then foundation rocking or sliding mechanisms are unlikely to be a controlling or  
34 participating mechanism in the structure response. While this might be an acceptable or  
35 desired characteristic for structures using high-R systems or for essential facilities, it is  
36 probably an undesirable characteristic for ordinary-use structures using moderate- or low-R  
37 systems.

## 1 **Strength of Deep Foundations**

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

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

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

## 29 **Overstrength of Deep Foundations**

30 Deep and shallow foundations are significantly different in that deep foundations can have tensile  
31 strength, overturning strength with low gravity loads, and element over-strength properties  
32 similar to superstructure elements. Deep foundations might therefore be capable of internally  
33 developing overstrength values in the range of tabulated  $\Omega_o$  values, provided that adequate  
34 ductility is present in the piles. There is therefore no clear need for specifying a special or  
35 increased load combination in order to offset a lack of overstrength in the foundation system, as  
36 there is for lightly-loaded shallow foundation systems. However, earthquake damage in deep  
37 foundations is difficult to detect, is probably frequently overlooked in post-earthquake damage  
38 investigations, and even if detected is very costly to repair. It would therefore be justified to  
39 require increased strength for deep foundation systems of structures having occupancies and sites  
40 resulting in higher seismic design categories, since foundations for those structures might be  
41 expected to experience more than one damaging earthquake during the foundation service life,  
42 and the potential loss-of-use for repairs would be detrimental to critical occupancy types.

1 **Recommendations for Foundations**

2 The foundation design including soil pressures for either shallow or deep foundation systems may  
 3 use USD load combinations in which the value of E used would be replaced as shown from the  
 4 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 $R \geq 5$	2.0 E	1.5 E
R 3 to <5	1.5 E	1.0 E
$R < 3$	1.0 E	1.0 E

5

**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
$R \leq 3$	1.0 E	1.0 E

6

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

11 Reason:

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

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

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

4 ASD Equivalents

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

11

12 **References**

- 13 1. “Background information for some of the proposed earthquake design provisions for the 2005  
 14 edition of the National Building Code of Canada,” Ronald H. DeVall, NRC Research Press, 4  
 15 April 2003.
- 16 2. National Building Code of Canada, 2005, Vol. 1 and Commentary.
- 17 3. International Building Code, 2006 edition, ICC.
- 18 4. ACI 318-05, “Building Code Requirements for Structural Concrete and Commentary”
- 19 5. AISC 341-05, “Seismic Provisions for Structural Steel Buildings”

20

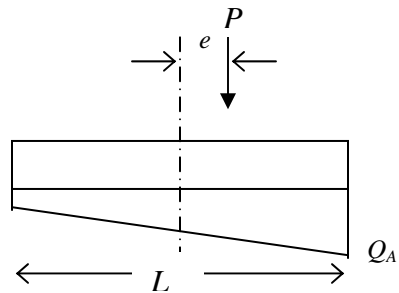
21 **Appendix 1**

22 **Derivation of Shallow Foundation Equations**

23 **Traditional ASD Design – Full Contact**

24 Given:

- 25  $P$  = Vertical Load  
 26  $M$  = Overturning Moment  
 27  $L$  = Length of Rectangular Footing  
 28  $B$  = Width of Rectangular Footing  
 29  $e = M / P$  = Eccentricity of Loading  
 30  $Q_A$  = Maximum ASD Allowable Soil Pressure



31 From the standard ending stress equation:

32 
$$\sigma = \frac{P}{A} \pm \frac{M}{S}$$
, the maximum soil pressure,  $Q_A$  will be:

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

2 Rearranging;

3 
$$e = \frac{L}{6} \left( \frac{Q_A}{P} BL - 1 \right)$$

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

5 
$$\frac{e}{L} = \frac{1}{6} \left( \frac{P}{P'} - 1 \right) \quad \text{(Equation 1)}$$

6

7 **Traditional ASD Design – Partial Contact**

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

9 
$$Q_A = \frac{2P}{3B \left( \frac{L}{2} - e \right)}$$

10 Rearranging;

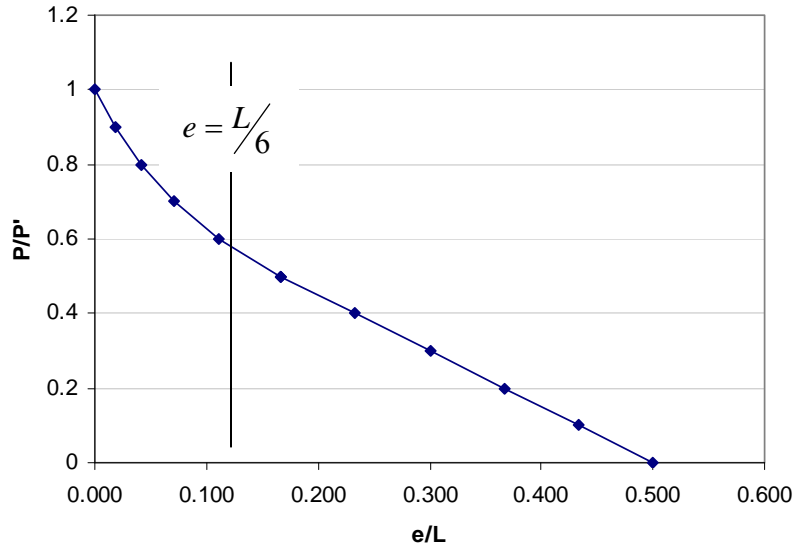
11 
$$e = \frac{L}{2} - \frac{2}{3} \frac{P}{Q_A B}$$

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

13 
$$\frac{e}{L} = \frac{1}{2} \left[ 1 - \frac{4}{3} \left( \frac{P}{P'} \right) \right] \quad \text{(Equation 2)}$$

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

15 The following is a graph of Equations 1 and 2:



1

2 **NEHRP Ultimate Strength Design Method**

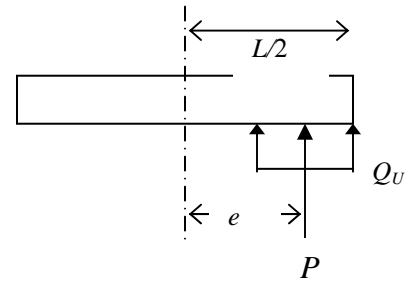
3 A simplified ultimate strength design approach, based on the Whitney Stress Block Method, follows:

4 Define (in addition to terms used above):

5  $Q_U =$  Ultimate Soil Pressure

6 For an assumed rectangular soil pressure distribution;

7 
$$Q_U = \frac{P}{2B(L/2 - e)}$$

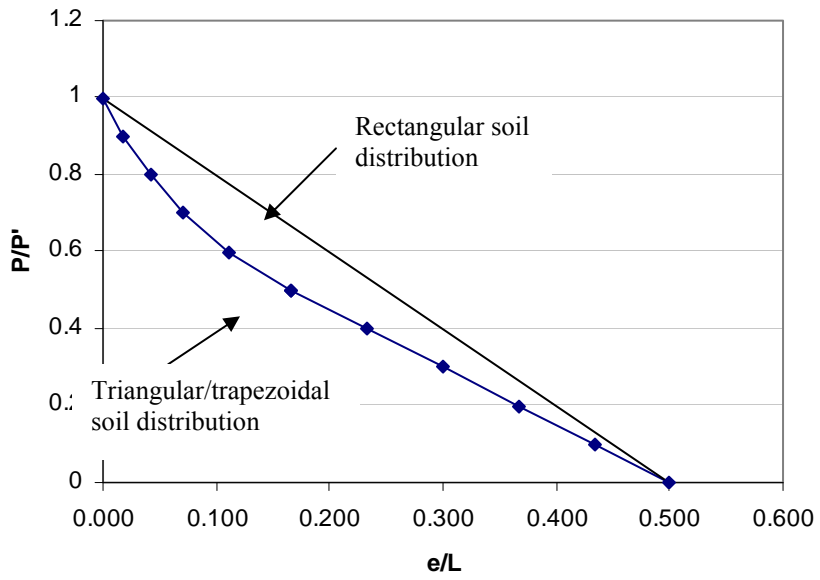


8 Substituting  $B = \frac{P'}{LQ_A}$ ,

9 
$$\frac{e}{L} = \frac{1}{2} \left[ 1 - \left( \frac{Q_A}{Q_U} \right) \left( \frac{P}{P'} \right) \right]$$
 (Equation 3)

10 Graphing this equation against the ASD equations;

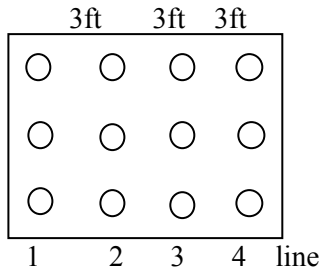
### Comparison of Equations for Ultimate vs. ASD Soil Pressure Distribution



1

**Appendix 2**  
**ASD and LRFD Interaction Diagrams for Deep Foundations**

Linear Strain Assumption



Assumed ASD Allowable Pile Capacities

$$P = 100 \text{ kips}$$

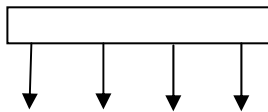
$$T = -50 \text{ kips}$$

$$\text{Ultimate/ASD} = 1.7 \text{ tension}$$

$$\text{Ultimate/ASD} = 2.5 \text{ compression}$$

(does not include  $\phi = 0.7$ )

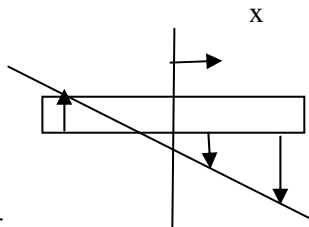
Point 1 - Pure compression



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:  
 $X_{na} = -1.50 \text{ ft}$

USD:  
 $X_{na} = -2.22 \text{ ft}$

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

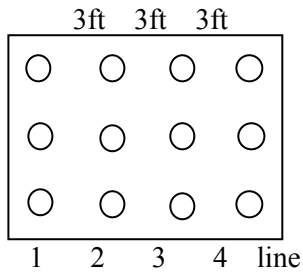
USD Capacity	
990	kips
5025.0	ft kips

Point 3 - Pure tension



$P = 4 * 3 * -50$	<u>ASD Capacity</u>	<u>USD Capacity</u>
= -600 kips		-1020 kips
M = 0.0 ft kips		0.0 ft kips

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	USD Capacity
1	3	-255	-4.5	1147.5	
2	3	750	-1.5	-1125.0	
3	3	750	1.5	1125.0	
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	USD Capacity
1	3	-255	-4.5	1147.5	
2	3	-255	-1.5	382.5	
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	USD Capacity
1	3	-255	-4.5	1147.5	
2	3	-255	-1.5	382.5	
3	3	-255	1.5	-382.5	-15 kips
4	3	750	4.5	3375.0	4522.5 ft kips
Sum =		-15		4522.5	

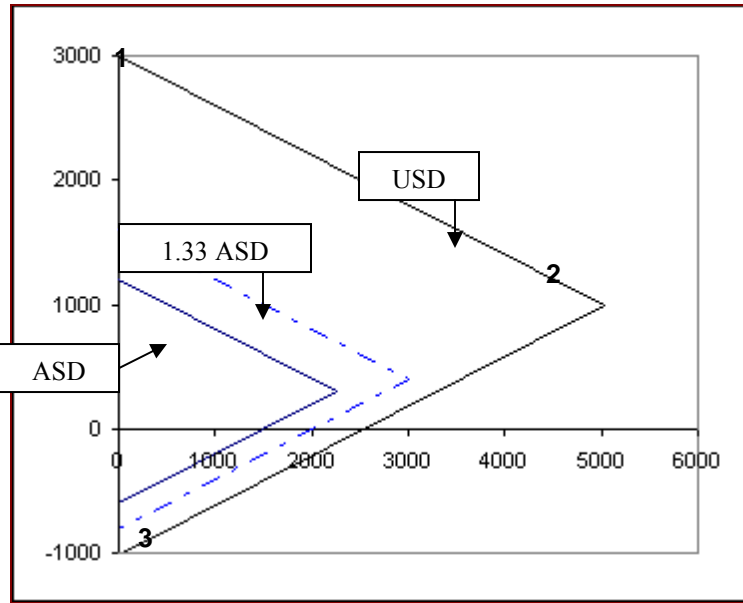
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Point 5

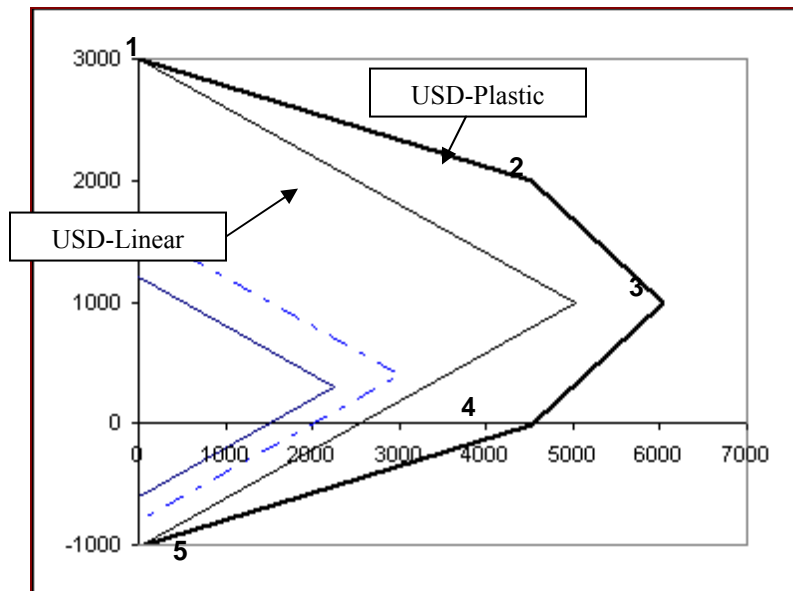
Line	# piles	#piles x Pile force	x	Px	USD Capacity	
1	3	-255	-4.5	1147.5	-1020	kips
2	3	-255	-1.5	382.5		
3	3	-255	1.5	-382.5	0.0	ft kips
4	3	-255	4.5	-1147.5		
Sum =		-1020		0.0		

2



3  
4

Linear Strain Assumption



5  
6  
7

Fully Plastic Assumption,  
Superimposed on Linear Strain Assumption