SEAOC Seismic Design Manual (1997 UBC Version)
Errata No. 2 for Volume I

Page 90, Example 29.

Replace the existing example in its entirety with the new example at the end of this errata.

Page 123, Example 40.

Add "and §1921.7" to right-hand side of the title.

Line 14, change the title of Part 1 to read:

> **Induced moment in ordinary column**.

Page 124, Example 40.

Change the title of Part 1 to read:

> **Induced moment in ordinary column.**

Add "and §1921.7" to right-hand side of the title for Part 1.

Part 1, delete the first paragraph and substitute the following:

> Both Sections 1633.2.4 and 1921.7 require that the elements not part of the designated lateral force-resisting system be designed and/or detailed to maintain support of the design dead and live loads ($1.2D + f_1L$) when subjected to the maximum inelastic response displacement Δ_M. Inelastic deformation of these supporting members and connections is permitted as long as appropriate detailing is provided to maintain the required support capacity at the Δ_M displacement. For the case of reinforced concrete frame elements §1921.7 prescribes the use of the element moment induced in the elastic model of the structure when subjected to Δ_M. This induced elastic moment (termed here as M_M) serves as an indicator of whether or not inelastic deformation occurs in a particular frame element. For a column having design strength ϕM_n under the interaction with factored axial loads ($1.2P_D + f_1P_L$):
>
> > if M_M does not exceed ϕM_n, then elastic behavior is indicated and the detailing requirements of §1921.7.2 apply;
> >
> > and if M_M exceeds ϕM_n, then inelastic behavior occurs and the special details of §1921.7.3 are required to maintain the vertical load support capacity. For this inelastic behavior case, the actual moment in the column would be held at the yield level represented by M_n.

> The ordinary columns located on the perimeter frames and the interior flat plate column system fall under these requirements and must be checked for the moments induced by the maximum inelastic response displacement. For this example, the ordinary column ϕM_n = 1200 k-in. at factored axial load ($1.2P_D + f_1P_L$) = 80 kips.

Change the second paragraph after Equation (30-17) to read:

> The moment induced in the ordinary column (considering PΔ effects) due to the maximum inelastic response displacement Δ_M on Line E must be determined.

Change the third paragraph after Equation (30-17) to read:

> For purposes of this example, a fixed-fixed condition is used for simplicity. In actual applications, column moment is usually determined from a frame analysis. The induced moment is:
>
> $$M_M = \frac{6E_c I_c \Delta_M}{h^2}$$
>
> The requirement to consider PΔ effects may be satisfied by compliance with the overall structural stability analysis specified in §1630.1.3. Refer to the SEAOC Blue Book Commentary §C105.1.3 for an acceptable procedure.

Change the last sentence of the fourth paragraph after Equation (30-17) to read:

> This requirement also applies to the elements that are not part of the lateral force-resisting system.

Page 125, Example 40.

Under Part 2, delete the first paragraph and substitute the following:

> The result that the induced moment M_M = 1875 k-in. exceeds ϕM_n = 1200 k-in. indicates that inelastic behavior will occur and the special detailing requirements of §1921.7.3 are necessary in order to maintain vertical load support capability at the Δ_M deformation. Although the ordinary column in this example is a corner column, it need not be checked for orthogonal displacements such as including 30 percent of the Δ_M on line A.

Under the Commentary, delete the first paragraph and substitute the following before the second paragraph:

> In actual applications, all frame members, including beams, not part of the lateral force-resisting system must comply with §1921.7. The interior flat slab and column system is also subject to this requirement. The flat plate slab must be checked for the flexure, direct shear, and punching shear due to gravity loads and the effects induced

by Δ_M. The Chapter 19 requirements for punching shear capacity and integrity steel should be considered as minimum requirements for the case where the induced M_M does not exceed ϕM_n, and the slab requirements of ACI 352 should be used to ensure proper detailing and performance when M_M exceeds the ϕM_n of the supporting column.

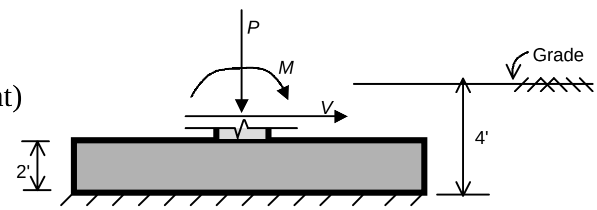
For steel frame structures the columns not part of the lateral force-resisting system should have compact sections.

Example 29
At Foundation §1630.8.3

Foundation reports for new building construction typically provide soil bearing pressures on an allowable stress design basis while seismic forces in the 1997 UBC, and in most concrete design under ACI 318, are on a strength design basis. This requires that the designer make a transition from the ASD procedure used to size the footing to the USD procedures used to design the footing. The purpose of this example is to illustrate footing design under this situation.

In this example, a spread footing supports a reinforced concrete column. The soil classification at the site is sand (SW). The following information is given:

Zone 4, $C_a = 0.4$, $I = 1.0$, $f_1 = 0.5$, and
$\rho = 1.0$ for structural system
$P_D = 80$ k $M_D = 15$ k - ft
(P_D includes the footing and imposed soil weight)
$P_L = 30$ k $M_L = 6$ k - ft
$P_E = \pm 40$ k $V_E = 30$ k $M_E = \pm 210$ k - ft
(these are the E_h loads due to base shear V)
Snow load $S = 0$
Wind load $W < \frac{E_h}{1.4}$



To properly apply load combinations when the interaction of axial load P and flexural load M is involved, it is necessary to define a sign convention:

Axial load P is positive (+) when acting downward on the top of the footing.

Flexural load M is positive (+) when acting clockwise on the top of the footing.

Note that when the applied seismic forces on the structure are acting in the left-to-right direction, the resulting seismic actions P_E and M_E are both positive (+). For the right-to-left force direction, both P_E and M_E are negative (-). The loads given above follow this sign convention.

Find the following:

1. **Determine the design criteria and allowable bearing pressure.**
2. **Determine footing size.**
3. **Check resistance to sliding.**

4. **Determine soil pressure reactions for strength design of the footing section.**

Calculations and Discussion — Code Reference

1. **Determine the design criteria and allowable bearing pressure.** **§1630.8.3**

Requirements for footing design are specified in §1915. The external forces on the footing are required to be transferred to the soil without exceeding permissible soil pressures (specified on an allowable stress design basis). In the 1997 UBC, however, the seismic force reactions on the footing are based on strength design. This apparent conflict is resolved by §1629.1, which states that allowable stress design may be used for sizing the foundation using the load combinations given in §1612.3. It should be noted that the intent of §1915 (see §1915.2.2) is the same as that of §1629.1. However, the last "italicized" sentence of §1915.2.2 is expressed in the terms of the 1994 UBC and is in error for the 1997 UBC. To be consistent with §1629.1, this sentence should be corrected to read, "*External forces and moments are those resulting from the load combinations of §1612.3.*"

In this example, it is elected to use the alternate basic load combinations of §1612.3.2.

$$D + L + S \quad (12\text{-}12)$$

$$D + L + \frac{E}{1.4} \quad (12\text{-}13)$$

$$0.9D \pm \frac{E}{1.4} \quad (12\text{-}16\text{-}1)$$

Because foundation investigation reports for buildings typically specify bearing pressures on an allowable stress design basis, criteria for determining footing size are also on this basis.

The earthquake loads to be resisted are specified in §1630.1.1 by Equation 30-1.

$$E = \rho E_h + E_v \quad (30\text{-}1)$$

Since $E_v = 0$ for allowable stress design, Equation 30-1 reduces to §1630.1.1

$$E = \rho E_h = (1.0)E_h$$

Table 18-1-A of §1805 gives the allowable foundation pressure, lateral bearing pressure, and the lateral sliding friction coefficient. These are default values to be used in lieu of site-specific recommendations given in a foundation report for the building. They will be used in this example.

For the sand (SW) class of material and footing depth of 4 feet, the allowable gross foundation pressure p_a is

$$p_a = 1.50 + (4\text{ ft} - 1\text{ ft})(0.2)(1.50) = \underline{\underline{2.40\text{ ksf}}}$$ Table 18-1-A and Footnote 2

A one-third increase in p_a is permitted for the load combinations that include earthquake load.

2. **Determine footing size.**

The trial design axial load and moment will be determined for load combination of Equation (12-13) and then checked for the other combinations. Earthquakes loads are in both directions, but the positive values are used in this calculation to create the largest bearing pressures.

$$P_a = D + L + \frac{E}{1.4} = P_D + P_L + \frac{P_E}{1.4} = 80 + 30 + \frac{40}{1.4} = 138.6\text{ kips or } 81.4\text{ kips} \quad (12\text{-}13)$$

$$M_a = D + L + \frac{E}{1.4} = M_D + M_L + \frac{M_E}{1.4} = 15 + 6 + \frac{210}{1.4} \quad (12\text{-}13)$$
$$= 171.0\text{ k-ft or } -129.0\text{ k-ft}$$

Select trial footing size.

Try 9 ft x 9 ft footing size, B = L = 9 ft. Footing area A and section modulus S are computed as

$$A = BL = 81\text{ ft}^2, \quad S = \frac{BL^2}{6} = \frac{9^3}{6} = 121.5\text{ ft}^3$$

Calculated soil pressures P due to axial load and moment using the largest values of P_a and M_a are

$$p = \frac{P_a}{A} \pm \frac{M_a}{S} = \frac{138.6}{81} \pm \frac{171.0}{121.5} = 1.71 \pm 1.41 = 3.12\text{ ksf or } 0.30\text{ ksf}$$

This is the gross soil pressure since P_D includes the footing and imposed soil weight.

Check bearing pressure against gross allowable with the one-third increase for seismic loads,

$$3.12\text{ ksf} < 1.33p_a = 1.33(2.40) = 3.20\text{ ksf}, \ o.k.$$

Check for the load combination of Equation (12-16-1).

$$P_a = 0.9D \pm \frac{E}{1.4} = 0.9P_D \pm \frac{P_E}{1.4} = 0.9\,(80) \pm \frac{40}{1.4} = 100.6 \text{ kips or } 43.4 \text{ kips} \qquad (12\text{-}16\text{-}1)$$

$$M_a = 0.9D \pm \frac{E}{1.4} = 0.9M_D \pm \frac{M_E}{1.4} = 0.9\,(15) \pm \frac{210}{1.4} \qquad (12\text{-}16\text{-}1)$$
$$= 163.5 \text{ k-ft or } -136.5 \text{ k-ft}$$

Note that according to the stated sign convention, the positive (+) values for P_E and M_E correspond to an applied seismic force loading from left to right, and minus (-) values are due to the right to left load direction. A positive (+) P_E cannot occur at the same time as minus (-) M_E since each sign corresponds to the particular direction of the applied seismic force loading.

Eccentricity $e = \dfrac{M_a}{P_a} = \dfrac{163.5 \text{ k-ft}}{100.6} = 1.63 \text{ ft, or } \dfrac{-136.5 \text{ k-ft}}{43.4} = -3.15 \text{ ft}$

Check for partial uplift. This occurs when the magnitude of e exceeds $\dfrac{L}{6}$, where $L =$ width of footing and $\dfrac{L}{6}$ is the limit of the kern area

$$\frac{L}{6} = \frac{9}{6} = 1.5 \text{ ft}$$

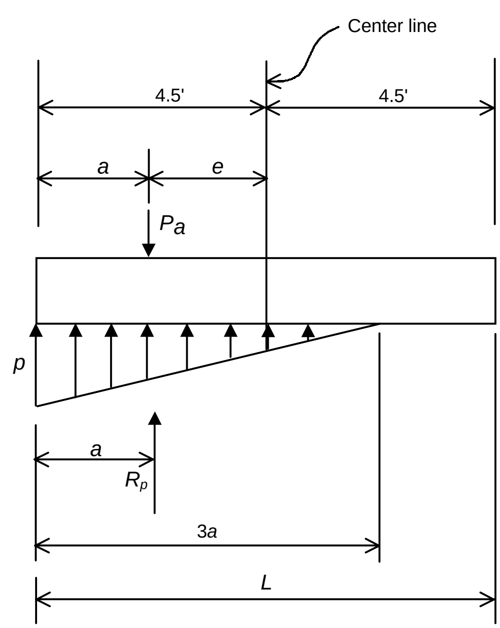
Since the magnitude of $e = 1.63 \text{ or } -3.15 > 1.5,$ there is partial uplift, and a triangular pressure distribution is assumed to occur.

For the footing free-body:

$$P_a = R_p = \frac{p}{2}(3a)B$$

$R_p =$ Pressure resultant

Note that R_p must be co-linear with P_a such that the length of the triangular pressure distribution is equal to $3a$.



For the load combination $0.9D - \frac{E}{1.4}$, the load combinations with $P_a = 43.4$ kips and $M_a = -136.5$ k-ft or with $P_a = 100.6$ kips and $M_a = 163.5$ k-ft, must be checked. This is shown below. (12-10)

For $e = -3.15$ ft, $a = \frac{L}{2} - e = 4.5 - 3.15 = 1.35$ ft

For $e = 1.63$ ft, $a = \frac{L}{2} - e = 4.5 - 1.63 = 2.87$ ft

$$P_a = \frac{p}{2}(3a)\,L$$

For $e = -3.15$ ft, the bearing pressure is

$$p = \frac{2}{3}P_a\left(\frac{1}{aL}\right) = \frac{2}{3}(43.4)\left[\frac{1}{(1.35)(9.0)}\right] = 2.38\text{ ksf} < 1.33p_a = 3.20\text{ ksf} \quad o.k.$$

For $e = 1.63$ ft, the bearing pressure is

$$p = \frac{2}{3}P_a\left(\frac{1}{aL}\right) = \frac{2}{3}(100.6)\left[\frac{1}{(2.87)(9.0)}\right] = 2.60\text{ ksf} < 1.33p_a = 3.20\text{ ksf} \quad o.k.$$

If p had been greater than $1.33p_a$, the footing size would have to be increased.

Finally, check the gravity load combination (12-12) for $p < p_a = 2.40$ ksf .

$$P_a = D + L = P_D + P_L = 80 + 30 = 110\text{ kips} \qquad (12\text{-}12)$$

$$M_a = D + L = M_D + M_L = 15 + 6 = 21\text{ k-ft} \qquad (12\text{-}12)$$

$$p = \frac{P_a}{A} \pm \frac{M_a}{S} = \frac{110}{81} \pm \frac{21}{121.5} = 1.53\text{ ksf or } 1.19\text{ ksf} < 2.40\text{ ksf}, \quad o.k.$$

All applicable load combinations are satisfied, therefore a 9ft x 9ft footing is adequate.

3. **Check resistance to sliding.**

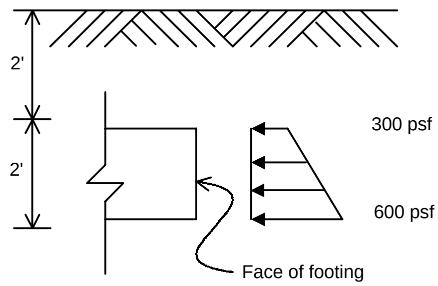
Unless specified in the foundation report for the building, the friction coefficient and lateral bearing pressure for resistance to sliding can be determined from Table 18-1-A. These values are:

Friction coefficient $\mu = 0.25$ Table 18-1-A

Lateral bearing resistance $p_L = 150\,\text{psf/foot} \times$ depth in feet below grade Table 18-1-A

Assume the footing is 2 feet thick with its base 4 feet below grade. Average resistance on the 2 feet deep by 9 feet wide footing face is $\frac{300+600}{2} = 450\,\text{psf}$.

$$p_L = 450\,\text{psf} = 0.45\,\text{ksf}$$



Load combination of Equation (12-16-1) will be used because it has the lowest value of vertical load ($0.9D = 0.9P_D$). The vertical and lateral loads to be used in the sliding resistance calculations are:

$$P = 0.9P_D = 0.9\,(80) = 72\,\text{kips}$$

Lateral load = $\frac{V_E}{1.4} = \frac{30}{1.4} = 21.4\,\text{kips}$

The resistance due to friction is

$$P(\mu) = 72(0.25) = 18.0\,\text{kips}$$

The resistance from lateral bearing is

$$p_L\ \text{(face area)} = 0.45\,(2' \times 9') = 8.1\,\text{kips}$$

The total resistance is then the sum of the resistance due to friction and the resistance due to lateral bearing pressure. Table 18-1-A, Note 4

Total resistance = $18.0 + 8.1 = 26.1 > 21.4\,\text{kips}$, *o.k.*

∴ No sliding occurs

4. **Determine soil pressure reactions for strength design of footing section.** **§1915.2.1**

To obtain the moment and shear actions prescribed in §1915.4 and §1915.5 for the strength design of the reinforced concrete footing section, §1915.2.1 is interpreted as follows. The induced reactions necessary to compute the design moments and shears may be obtained by applying an appropriate factor to the allowable stress design soil pressures found in Part 2 for the determination of the footing area. The appropriate factor is taken equal to 1.5 for all of the allowable stress load combinations. This value (which has been approved by the SEAOC Seismology Committee) provides a reasonably conservative envelope for the strength design load combinations for the common case where live load L is less than dead load D. (Note: *Refer to the*

Commentary at the end of this example for discussion and justification of the following procedure.)

The applicable allowable soil pressures to be considered are due to the following allowable stress load combinations:

$$D + L + S = D + L \qquad (12\text{-}12)$$

$$D + L + E/1.4 = D + L + E_h/1.4 \qquad (12\text{-}13)$$

$$0.9D \pm E/1.4 = 0.9D \pm E_h/1.4 \qquad (12\text{-}16\text{-}1)$$

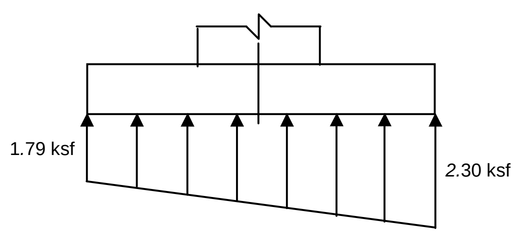
The corresponding soil pressures have been calculated in Part 2 of this example.

For both simplicity and conservatism, in this example gross value of dead load P_D, which includes the footing and imposed soil weight, will be used.

a. Factored soil pressure due to load combination $D + L$.

Using the assigned load factor of 1.5, the resulting strength design reaction soil pressures are:

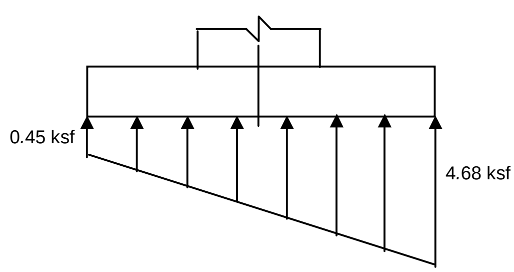
$$1.5(1.53 \text{ ksf or } 1.19 \text{ ksf}) = 2.30 \text{ ksf or } 1.79 \text{ ksf}$$



b. Factored soil pressure due to load combination $D + L + E_h/1.4$.

The resulting strength design reaction soil pressures are:

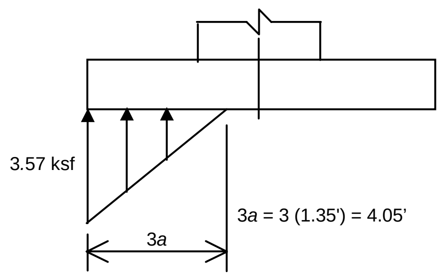
$$1.5(3.12 \text{ ksf or } 0.30 \text{ ksf}) = 4.68 \text{ ksf or } 0.45 \text{ ksf}$$



c. Factored soil pressure due to load combination $0.9D \pm E_h/1.4$.

Noting that $0.9D + E_h/1.4$ is governed by $D + L + E_h/1.4$, then only $0.9D - E_h/1.4$ needs to be considered. The resulting strength design soil pressure reactions for the triangular distribution are:

$$1.5\,(2.38\text{ ksf or } 0) = 3.57\text{ ksf or } 0$$



The factored soil pressures due to the $D + L + E_h/1.4$ load combination governs. Note that the resulting moment and shear actions must be multiplied by 1.1 per Exception 1 of §1612.2.1.

Note also that the factored value of p need not be less than $1.33p_a = 3.20$ ksf, since it is used as a load for concrete section design rather than for determining footing size.

The "1.5 factor" method shown above can be used for the design of individual spread footings without further consideration of the actions on the column. For footings with two or more columns, however, the method may result in unstable solutions. This is because the soil bearing pressure has a 1.5 load factor, while dead, live, and earthquake loads factors are 0.9 or 1.2 for dead, f_1 or 1.6 for live, and 1.0 for earthquake. Thus, static equilibrium very likely will not be achieved. In this situation, the designer may need to determine the contribution of each load case to the "factored" soil pressure.

Commentary

The following additional discussion is provided for the particular procedure employed in Part 4 of this example that involves the use of a 1.5 factor times the ASD soil pressures. Section1915.2.1 of the code is not particularly clear and may be interpreted two ways:

1. Use the reactions (i.e., the footing section moments and shears) due to the soil pressures resulting from the strength design load combinations of §1612.2. If this interpretation were to be used, the load combination Equation (12-6): $0.9D - 0.5C_a I D - \rho E_h$ can result in an eccentricity $e = M_u / P_u$ that is beyond the edge of the footing having the area determined by §1915.2.2.
   For example, using the loads in this example:

   $$P_u = 0.9P_D - 0.5C_a P_D - P_E = 0.7P_D - P_E = (0.7)(80) - 40 = 16\,\text{k}$$

   $$M_u = 0.9M_D - 0.5C_a M_D - M_E = 0.7M_D - M_E = (0.7)(15) - 210 = 209.5\,\text{k - ft}$$

   $$e = M_u / P_u = (209.5)/(16) = 13.1\,\text{ft} > 5\,\text{ft}$$

   Thus, the reaction is beyond the edge of the 9'x9' footing.

   For this condition of large eccentricity, the corresponding soil pressure reactions cannot be evaluated because there is full uplift. Either piles would have to be provided for flexural tensile resistance at the soil interface or a footing area greater than that required by the allowable stress design loads of §1629.1. This would violate the intent of §1629.1.

2. Use the reactions due to a factor of 1.5 times the soil pressures from the gravity load combinations and the seismic load combinations of §1612.3. The 1.5 factor provides a reasonably conservative envelope for the strength design load combinations for the common case where dead load D > live load L, and is based on the traditional 1.4 conversion to the strength design seismic load combinations with a small increase to 1.5 in order to represent the effects of the vertical component E_v. The results of 1.5 times the governing ASD load combinations are compared below with the corresponding strength design load combinations:

   $$1.5D > 1.4D \qquad (12\text{-}1)$$

   $$1.5(D + L) > 1.2D + 1.6L \text{ for } D > L \qquad (12\text{-}2)$$

   $$1.5(D + L + \rho E_h/1.4) > 1.2D + 0.5L + (0.5C_a I D + \rho E_h) \qquad (12\text{-}5)$$

   but with

$$1.5(0.9D - \rho E_h / 1.4) < 0.9D - (0.5 C_a I D + \rho E_h) \quad (12\text{-}6)$$

such that combination Equation (12-6) does not govern the footing area design.

In summary, the use of Interpretation 1 can result in either a set of strength factored loads that cannot be resisted by the design footing area, such as eccentricity of vertical load beyond the edge of the footing or the footing area or support system must be changed. Consequently, Interpretation 2 is preferred, since it does not change the soil-foundation interface design provided by §1629.1 and §1915.2.