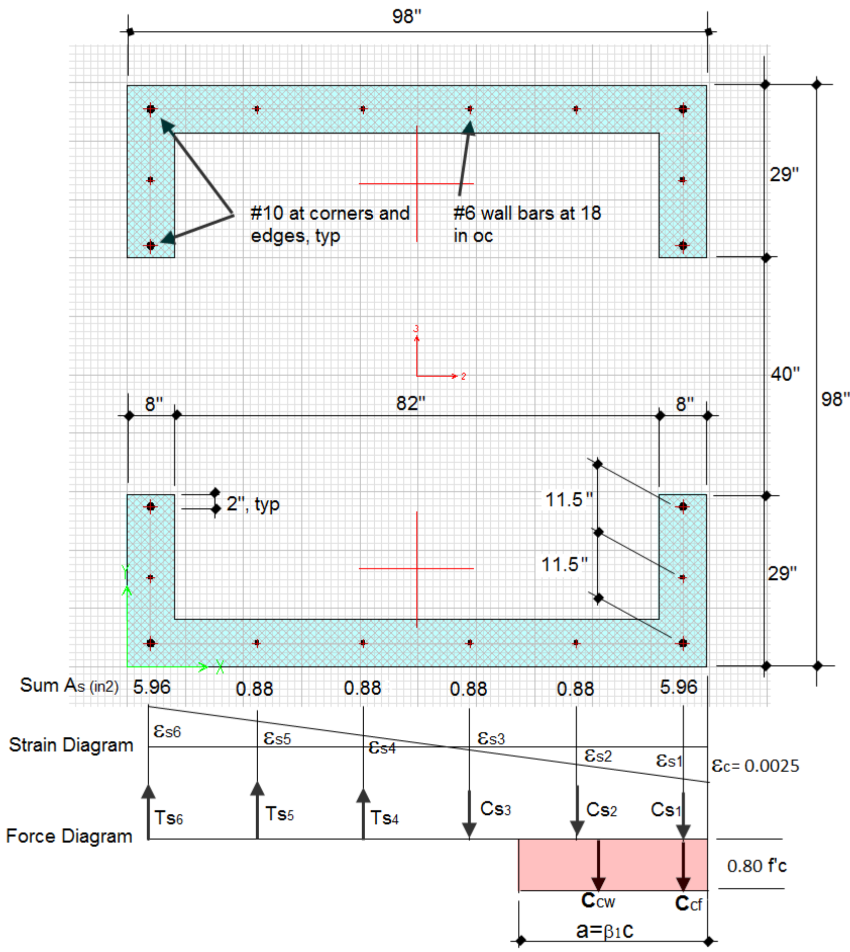
EXAMPLE ACI 530-11 Masonry Wall-002

P-M INTERACTION CHECK FOR WALL

EXAMPLE DESCRIPTION

The Demand/Capacity ratio for a given axial loading and moment is tested in this example. The wall is reinforced as shown below. The concrete core wall is loaded with a factored axial load P_u = 1496 k and moments M_{u3} = 7387 k-ft. The design capacity ratio is checked by hand calculations and the results are compared with ETABS program results.

GEOMETRY, PROPERTIES AND LOADING



Material Properties		Section Properties	Design Properties
E =	3600 k/in^2	tb = 8 in	f'_c = 4 k/in^2
v =	0.2	h = 98 in	f_y = 60 k/in^2
G =	1500 k/in^2	As1= As6 = 2-#10,2#6 (5.96 in^2)	
		As2, As3, As4 and As5 = 2-#6 (0.88 in^2)	

TECHNICAL FEATURES OF ETABS TESTED

- Concrete wall flexural Demand/Capacity ratio

RESULTS COMPARISON

Independent results are hand calculated and compared with ETABS design check.

Output Parameter	ETABS	Independent	Percent Difference
Column Demand/Capacity Ratio	0.998	1.00	-0.20%

COMPUTER FILE: ACI 530-11 MASONRY WALL-002

CONCLUSION

The ETABS results show an acceptable comparison with the independent results.

HAND CALCULATION

Wall Strength under compression and bending

1) A value of e = 59.24 inches was determined using $e = M_u / P_u$ where M_u and P_u were taken from the ETABS test model interaction diagram. The values of M_u and P_u were large enough to produce a flexural D/C ratio very close to or equal to one. The depth to the neutral axis, c, was determined by iteration using an excel spreadsheet so that equations 1 and 2 below were equal.

2) From the equation of equilibrium:

$$P_{n1} = C_c + C_s - T$$

where

$$C_c = \beta_1 f'_m ab = 0.8 \bullet 2.5 \bullet 12a = 24.0a$$

$$C_s = A'_1\left(f_{s1} - 0.8 f'_m\right) + A'_2\left(f_{s2} - 0.8 f'_m\right) + A'_3\left(f_{s3} - 0.8 f'_m\right)$$

$$T = A_{s4} f_{s4} + A_{s5} f_{s5} + A_{s6} f_{s6}$$

$$\begin{aligned} P_{n1} = {} & 24a + A'_1\left(f_{s1} - 0.8 f'_m\right) + A'_2\left(f_{s2} - 0.8 f'_m\right) + \\ & A'_3\left(f_{s3} - 0.8 f'_m\right) - A_{s4} f_{s4} - A_{s5} f_{s5} - A_{s6} f_{s6} \end{aligned} \qquad \text{(Eqn. 1)}$$

3) Taking moments about A_{s6}:

$$P_{n2} = \frac{1}{e'}\left[\begin{aligned} & C_{cf}\left(d - d'\right) + C_{cw}\left(d - \frac{a - t_f}{2}\right) + C_{s1}\left(d - d'\right) + C_{s2}\left(4s\right) + \\ & C_{s3}\left(3s\right) - T_{s4}\left(2s\right) - T_{s5}\left(s\right) \end{aligned}\right] \qquad \text{(Eqn. 2)}$$

where $C_{s1} = A'_1\left(f_{s1} - 0.8 f'_m\right)$; $C_{sn} = A'_n\left(f_{sn} - 0.8 f'_m\right)$; $T_{sn} = f_{sn} A_{sn}$; and the bar strains are determined below. The plastic centroid is at the center of the section and d'' = 45 inch

$$e' = e + d'' = 59.24 + 45 = 104.24 \text{ inch.}$$

4) Iterating on a value of c until equations 1 and 2 are equal c is found to be c = 41.15 inches.

$$a = 0.8 \bullet c = 0.8 \bullet 41.15 = 32.92 \text{ inches}$$

5) Assuming the extreme fiber strain equals 0.0025 and c = 41.15 inches, the steel stresses and strains can be calculated. When the bar strain exceeds the yield strain, then $f_s = f_y$:

$$\varepsilon_{s1} = \left(\frac{c-d'}{c}\right)0.0025 = 0.00226; \; f_s = \varepsilon_s E \le F_y \; ; \; f_{s1} = 60.00 \text{ ksi}$$

$$\varepsilon_{s2} = \left(\frac{c-s-d'}{c}\right)0.0025 = 0.00116 \qquad f_{s2} = 33.74 \text{ ksi}$$

$$\varepsilon_{s3} = \left(\frac{c-2s-d'}{c}\right)0.0025 = 0.00007 \qquad f_{s3} = 2.03 \text{ ksi}$$

$$\varepsilon_{s4} = \left(\frac{d-c-2s}{d-c}\right)\varepsilon_{s6} = 0.00102 \qquad f_{s4} = 29.7 \text{ ksi}$$

$$\varepsilon_{s5} = \left(\frac{d-c-s}{d-c}\right)\varepsilon_{s6} = 0.00212 \qquad f_{s5} = 60.00 \text{ ksi}$$

$$\varepsilon_{s6} = \left(\frac{d-c}{c}\right)0.0025 = 0.00321 \qquad f_{s6} = 60.00 \text{ ksi}$$

Substituting the above values of the compression block depth, a, and the rebar stresses into equations Eqn. 1 and Eqn. 2 give

$$P_{n1} = 1662 \text{ k}$$
$$P_{n2} = 1662 \text{ k}$$

$$M_n = P_n e = 1662(41.15)/12 = 8208 \text{ k-ft}$$

6) Calculate the capacity,

$$\phi P_n = 0.9(1622) = 1496 \text{ kips}$$
$$\phi M_n = 0.9(8208) = 7387 \text{ k-ft.}$$