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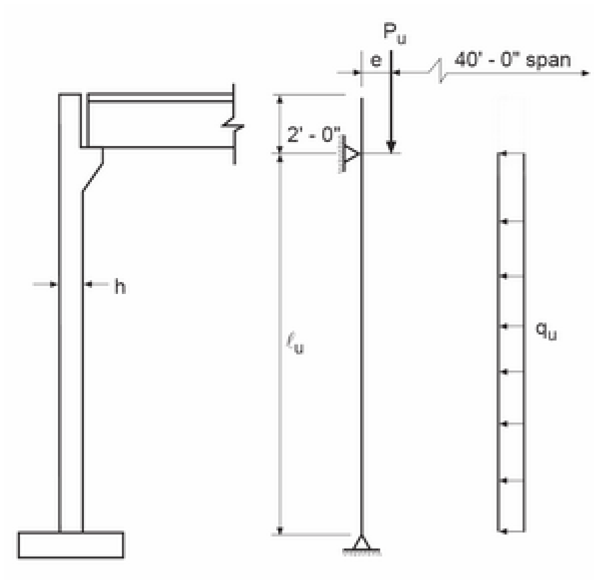
Slender Wall Design

Chapter 10 (ACI 318-05, 14.4)

Design of the wall shown is required. The wall is restrained at the top edge, and the roof load is supported through 4 in. tee stems spaced at 4 ft on center.



Design Data

Loads:

Roof dead load $q_D := 60psf$

Roof live load $q_L := 30psf$

Wind $q_W := 20psf$

Steel

$f_y := 60ksi$ $E_s := 29000ksi$

$\rho_{minh} := 0.0020$ $\rho_{minv} := 0.0012$

Concrete

$f'_c := 4ksi$ $E_c := 57000 \cdot \sqrt{\frac{f'_c}{psi}} \cdot psi$

$w_c := 150 \frac{lbf}{ft^3}$ $E_c = 3605\,ksi$ $\phi := 0.9$

$\beta_1 := if\left[f'_c \le 4000psi, 0.85, max\left[0.65, 0.85 - 0.05 \cdot \frac{(f'_c - 4000psi)}{1000psi}\right]\right]$ $\beta_1 = 0.85$

Geometry

Wall height $l_u := 16ft$

Parapet length $l_p := 2ft$

Roof load eccentiricity $e := 6.75in$

Roof span $l_r := 40ft$

Tee beam spacing $s := 4ft$

Stem width $w := 4in$

Effective length factor $k := 1.0$

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Calculations

1. Trial wall section

Trial wall thickness $h := 6.5\,in$ $\quad d := 0.5 \cdot h \quad d = 3.25\,in$

Minimum reinforcement area $A_{sminv} := \rho_{minv} \cdot h \quad A_{sminv} = 0.094\,\frac{in^2}{ft}$

Trial area of vertical reinforcement $A_s := 0.100\,\frac{in^2}{ft}$

2. Effective wall length for roof reaction

$l_{eff} := \min(w + 4 \cdot h, s) \quad l_{eff} = 2.5\,ft$

3. Roof loading per foot width of wall $b := 1\,ft$

Roof dead load $P_{D1} := q_D \frac{s}{l_{eff}} \cdot \frac{l_r}{2} \quad P_{D1} = 1920\,plf$

Wall dead load at midheight $P_{D2} := h \cdot \left(l_p + \frac{l_u}{2}\right) \cdot w_c \quad P_{D2} = 812.5\,plf$

Roof live load $P_L := q_L \frac{s}{l_{eff}} \cdot \frac{l_r}{2} \quad P_L = 960\,plf$

4. Factored load combinations
(1.2D + 0.5Lr + 1.6W)

Factored axial load at mid-height $P_u := b \cdot \left[1.2 \cdot \left(P_{D1} + P_{D2}\right) + 0.5 \cdot P_L\right]$

$P_u = 3.759\,kips$

Factored moment at mid-height $M_u := b \cdot \left(1.2 \cdot P_{D1} \cdot \frac{e}{2} + 0.5 \cdot P_L \cdot \frac{e}{2} + 1.6 \cdot \frac{q_W \cdot l_u^2}{8}\right)$

$M_u = 21.684\,in \cdot kips$

Sustained axial load at mid-heght $P_{us} := b \cdot \left[1.2 \cdot \left(P_{D1} + P_{D2}\right)\right]$

$P_{us} = 3.279\,kips$

$\beta_d := \frac{P_{us}}{P_u} \quad \beta_d = 0.872$

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5. Check Wall slenderness

$$\frac{k \cdot l_u}{0.3 \cdot h} = 98.462$$

$$\text{concat}\left(s1, \text{if}\left(\frac{k \cdot l_u}{0.3 \cdot h} \le 100, s2, s3\right)\right) = \text{"Approximate methods of Chapter 10 can be used"}$$

$$\text{if}\left(\frac{k \cdot l_u}{0.3 \cdot h} > 40, s4, s5\right) = \text{"Slenderness can't be neglected"}$$

6. Calculate magnified moments for non-sway case

For members with transverse loads between supports

$$C_m := 1.0$$

Gross moment of intertia

$$I_g := \frac{b \cdot h^3}{12} \qquad I_g = 274.625\ \text{in}^4$$

Reinforcement ratio

$$\rho := \frac{A_s}{h} \qquad \rho = 1.282 \times 10^{-3}$$

Calculations of EI in accordance with PCA Notes (Chapter 21, Eq 1) based on approach for single layer of reinforcment suggested by J.G.MacGregor in "Design and Safety of Reinforced Concrete Compression Members" presented at International Association for Bridge and Structural Engineering Symposium, Quebec, 1974.

$$\beta := 0.9 + 0.5\left(\beta_d\right)^2 - 12\rho \qquad \beta = 1.265$$

$$EI := \frac{E_c \cdot I_g}{\beta} \cdot \min\left(0.4, \max\left(0.1, 0.5 - \frac{e}{h}\right)\right)$$

$$EI = 78258.062\ \text{in}^2 \cdot \text{kips}$$

Critical buckling loads

$$P_c := \frac{\pi^2 \cdot EI}{\left(k \cdot l_u\right)^2} \qquad P_c = 20.952\ \text{kips}$$

Magnification factors

$$\delta_{ns} := \max\left[\frac{C_m}{1 - \left(\frac{P_u}{0.75 \cdot P_c}\right)}, 1.0\right] \qquad \delta_{ns} = 1.314$$

Magnified moments

$$M_c := \delta_{ns} \cdot M_u \qquad M_c = 28.502\ \text{in} \cdot \text{kips}$$

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7. Check design strength vs. required strength

Depth of equivalent concrete stress block

$$a := \frac{\frac{P_u}{\phi} + A_s \cdot b \cdot f_y}{0.85 \cdot f'_c \cdot b} \qquad a = 0.249 \text{ in}$$

Neutral axis depth

$$c := \frac{a}{\beta_1} \qquad c = 0.293 \text{ in}$$

Net tensile steel strain

$$\varepsilon_t := \frac{0.003}{c} \cdot (d - c) \qquad \varepsilon_t = 0.03$$

Nominal moment capacity

$$M_n := f_y \cdot \left(b \cdot A_s + \frac{P_u}{\phi \cdot f_y} \right) \cdot \left(d - \frac{a}{2} \right)$$

$$M_n = 31.805 \text{ in} \cdot \text{kips}$$

Factored capacity

$$\phi \cdot M_n = 28.624 \text{ in} \cdot \text{kips}$$

Capacity check

$$\frac{M_c}{\phi \cdot M_n} = 0.996$$

$$\text{if}\left(\frac{M_c}{\phi \cdot M_n} \le 1, \text{"OK"}, \text{"NG"} \right) = \text{"OK"}$$

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Comparison of cracking coefficients

$b = 12\,\text{in}$ $\quad P_u = 3.759\,\text{kips}$

$c = 0.293\,\text{in}$ $\quad f_y = 60\,\text{ksi}$

$d = 3.25\,\text{in}$ $\quad E_c = 3605\,\text{ksi}$

$I_g = 274.625\,\text{in}^4$ $\quad A_s = 0.1\,\dfrac{\text{in}^2}{\text{ft}}$

$$n := \frac{E_s}{E_c}$$

Based on cracked section with actual area of steel, A_s

$$I_{cr} := \frac{b\cdot(c)^3}{3} + n\cdot A_s\cdot b\cdot(d-c)^2 \qquad I_{cr} = 7.133\,\text{in}^4 \qquad \frac{I_{cr}}{I_g} = 0.026$$

Based on Eq. 14-7, ACI 318-05, cracked section with effective area of steel, A_{se}

$$A_{se} := A_s + \frac{P_u}{b\cdot f_y} \qquad A_{se} = 0.163\,\frac{\text{in}^2}{\text{ft}}$$

$$I_{cr} := \frac{b\cdot(c)^3}{3} + n\cdot A_{se}\cdot b\cdot(d-c)^2 \qquad I_{cr} = 11.538\,\text{in}^4 \qquad \frac{I_{cr}}{I_g} = 0.042$$