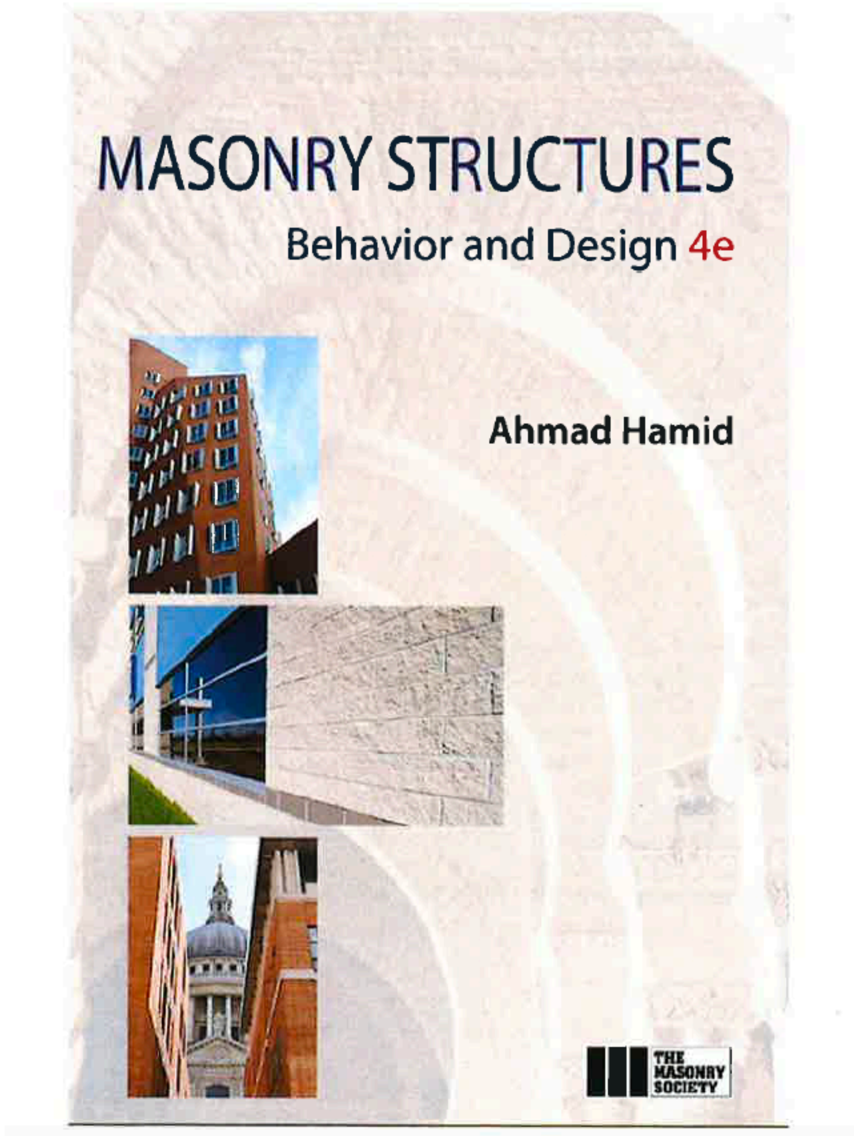
ERRATA

For



July 2021

CHAPTER 7

Page 278

$$\mu_m = (\phi_m\ f'_{tn} + P_f\ /\ A_e)\ /\ \phi_m\ f'_{tp} \le 1.00 \quad (7.17)$$

and f'_{tn}, f'_{tp} = flexural tensile strengths normal and parallel to the bed joints, respectively
P_f = factored axial load not to exceed self-weight of the panel plus A_e times 22 psi (0.15 MPa)
ϕ_m = strength reduction factor equals to 0.60 in CSA S304.1[7.4]
A_e = effective net cross sectional area.

7.12 PROBLEMS

7.1 A clay brick wall with a nominal thickness of 4 in. (10 cm) spans 12 ft (3.66 m) vertically. Data: Brick compression strength = 8000 psi (55 MPa) and type S mortar (compressive strength) = 2000 psi (13.8 MPa). Determine the maximum out-of-plane seismic load that can be carried using ~~either working stress design according to your local building code (alternatively use strength design).~~ using TMS 402 strength design

(a) Flexural action.

(b) Arching between rigid supports (factor of safety = 3.0).

(c) Gapped arching with a 1/32 in. (0.8 mm) gap at the top of the wall.

Comment on the results and the suitability of the methods. Seismic load is assumed to act as a uniformly distributed out-of-plane load.

CHAPTER 8

Page 336

and

$$I_{cr} = n\,[A_s + (P_u/f_y)(t_{sp}/2d)]\,(d-c)^2 + bc^3/3 \qquad (8.32)$$

$$c = (A_s f_y + P_u)/(0.64\,f'_m\,b) \qquad (8.33)$$

Note that th

Page 345

This factored moment value amounts to a moment magnification of 1.06 or, in another words, p-delta results in a 6 percent increase of primary moment.

→ Since M_u = 1,132 ft-lb/ft is less than ϕM_n = 0.9(1,671) = 1,504 ft-lb/ft No. 5 bars of 32 in. (.803 m) space is adequate

Checking serviceability, set the deflection at service load equal to the limit in Eq. 8.36. Thus

$$\delta_s = 0.007\,h = 0.007(16 \text{ x } 12) = 1.34 \text{ in. } (34 \text{ mm})$$

Then, from Eq. 8.25 and using D + 0.6W load combination for Allowable Stress Design [8.18]

$$M_s = \frac{wh^2}{8} + P_f\,\frac{4/12}{2} + (P_w + P_f)\delta_s$$

$$= \frac{0.6(32)(16)^2}{8} + \frac{300(2)}{12} + (8(44.8) + 300)\frac{1.34}{12} = 738 \text{ ft-lb/ft } (3.29 \text{ kN}\cdot\text{m/m})$$

Similarly, calculating deflection, from Eq. 8.27

$$\delta_s = \frac{5}{48}\,\frac{(6{,}397)(16(12))^2}{900(2{,}404)(148.7)} + \frac{5}{48}\,\frac{(738(12) - 6{,}397)(16(12))^2}{900(2{,}404)(9.6)} = 0.53 \text{ in. } (13 \text{ mm})$$

Since the calculated deflection is less than the limiting value, the calculation is valid and the deflection criteria have been satisfied.

→ It is to hoted that No. 5 bars at 40 in. (1.0 m) spacing is also adequate

8.6.3 Example 8.3: Analysis of Bearing Capacity Under Concentrated Load

CHAPTER 10

Page 410

- For running bond masonry not fully grouted;

$$V_n = 56A_{nv} + 0.45N_u \qquad (10.31)$$

- For masonry not laid in running bond, constructed of open end units and fully grouted;

$$V_n = 56A_{nv} + 0.45N_u \qquad (10.32)$$

- For running bond masonry fully grouted;

$$V_n = 90A_{nv} + 0.45N_u \qquad (10.33)$$

- For masonry not laid in running bond, constructed with other than open end units, and fully grouted;

$$V_n = 23A_{nv} \qquad (10.34)$$

Page 412

For values of $M_u/V_u d$ between the above limits, interpolation is permitted.

For the contribution of masonry,

$$V_m = \left[4.0 - 1.75\frac{M_u}{V_u d}\right]A_{nv}\sqrt{f'_m} + 0.25P_u \qquad (10.45)$$

Page 419

Even when the reinforcement limits associated with required strain gradient are not satisfied, other conditions may be met which will avoid the need for special boundary elements. For instance, if $P_u < 0.01\,A_g\,f'_m$ for symmetric wall sections or $P_u < 0.05\,A_g\,f'_m$ for unsymmetrical sections and either

$$M_u / V_u d_v \leq 1.0 \qquad (10.56)$$

Page 425

and the strength level moment strength is

$$M_u \text{ at base} = 1.0(780) + 1.0(120)\frac{120}{12} = 1980 \text{ ft kips } (2{,}684 \text{ kN m})$$

$$f_m = -\frac{P_u}{A_n} \pm \frac{M_u y}{I_n} = -\frac{360(1000)}{1350} \pm \frac{1{,}980\,(12000)120}{6{,}480{,}000} = -267 \pm 440$$

173 psi (1.19 MPa) tension or 707 psi (4.88 MPa) compression

The resulting stress distribution is shown in Figure 10.42(b). Because tensile stress exceeds TMS 402 code limit of ϕf_{tn} =0.6 (163) = 98 psi (0.67 MPa), the section should be reinforced (see Section 10.6.2).

Page 426

Solution B. Assume that the intersecting walls are connected. For an effective flange width of $6t$ on either side of the web:

$$b_{eff} = 5.625 + 2\,(6) \times (5.625) = 73.1 \text{ in. } (1857 \text{ mm})$$

Section Properties: Adding the flanges

$$A_n = 1350 + 2(73.1)\,(5.625) = 2172 \text{ in.}^2 \quad (1.35 \times 10^6 \text{ mm}^2)$$

$$I_n = I_{web} + I_{flanges} = \frac{5.625(240)^3}{12} + 2\left[\frac{73.1(5.625)^3}{12} \quad 73.1\,(5.625)(122.8)^2\right]$$

$$= 18{,}883{,}452 \text{ in.}^4 \qquad (7.86 \times 10^{12} \text{ mm})$$

Stresses Due to Axial Compression and Bending. For the distance from the centroid to the extreme fiber of y = 125.62 in. (3,191 mm),

$$f_m = -\frac{P}{A} \pm \frac{My}{I} = \frac{-360(1{,}000)}{2172} \pm \frac{1{,}980(12{,}000)\,(125.62)}{18{,}883{,}452} = -166 \pm 158$$

8 psi (0.06 MPa) compression or 324 psi (2.24 MPa) compression

Page 428

$$P_u/\phi = C - T = 0.8 f'_m\, b(0.8c) - A_{s_1} f_y - A_{s_2} f_y - A_{s_3}\left(\frac{d_3 - c}{c}\right) 0.0025\, E_s$$

$$108{,}000/\cancel{8.9} = 0.8(3{,}000)(5.625)(0.8c) - 0.44(60{,}000) - 0.44(60{,}000)$$

0.9

$$-0.44\left(\frac{28 - c}{c}\right)0.0025(29 \times 10^6)$$

$$c = 17.71 \text{ in. } (500 \text{ mm})$$

Page 429

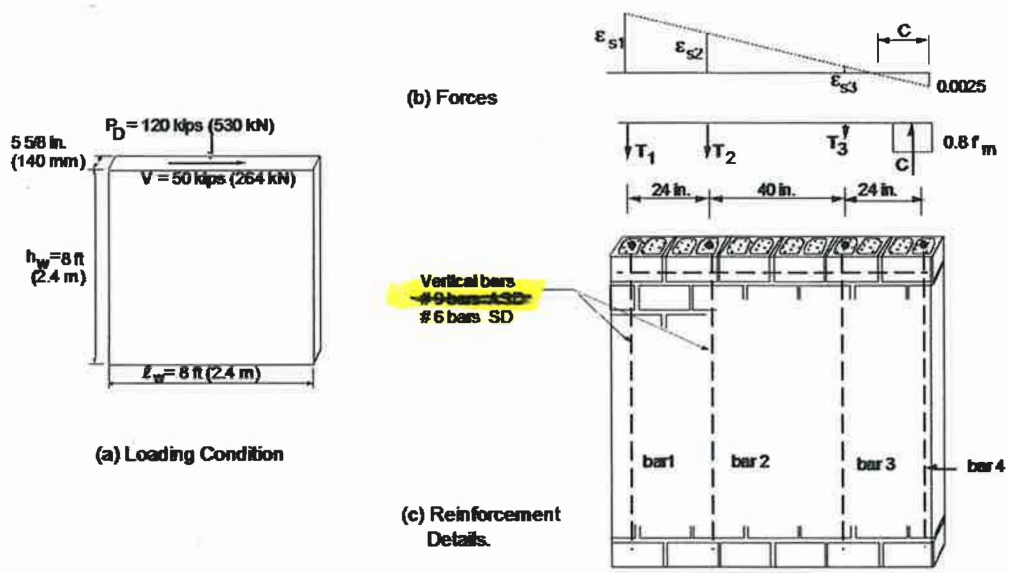


Figure 10.46 Strength design of masonry shear wall for Example 10.4.

Shear. Using the 1.0 load factor, $V_u = 1.0(80) = 80$ kips (356 kN) and to satisfy $V_u \le \phi V_n$ where V_n is defined by Eq. 10.42 and, from Eq. 10.44 for $M_u/V_u d_v \ge 1$, must not exceed

$$V_n = 4\, A_{nv} \sqrt{f'm}\; \gamma_g = 4(5.525)\ (96)\ \sqrt{3{,}000}\ \ (1.0)/1{,}000 = 118 \text{ Kips } \ (525 \text{ kN})$$

Where γ_g is taken equal to 1.0 for fully grouted construction. This indicates that the 6 in. (152 mm) block wall can be designed to resist the 80 kip (356 kN) factored shear force. Then, using Eq. 10.45 to calculate the masonry contribution to shear strength,

$$V_{nm} = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_{nv}\sqrt{f'_m} + 0.25 P_u$$

taking $M_u/V_u d_v$ as the maximum value of 1.0

$$V_{nm} = \left[4 - 1.75(1)\ (5.625\right](96)\sqrt{3{,}000}\,/1000 \quad 0.25(108)$$

$$= 93.5 \text{ kips } \ (416 \text{ kN})$$

Page 429- Continue

Then, since V_u is greater than $\phi V_{nm} = 0.80(93.5) = 74.8$ kips (333 kN), shear reinforcement is required. Taking $\gamma_g = 1.0$ for fully grouted masonry and using design Eq. 10.43, the required V_{ns} is

$$V_{ns} = V_u / \phi - V_{nm} = 80/0.80 - 93.5 = 6.5 \text{ kips} \ \ (28.9 \text{ kips})$$

and assuming joint reinforcement every second courses (that is, spacing $s = 16$ in. (406 mm))

$$A_v = \frac{2V_{ns}s}{f_y d} = 2(6,500)(16)/(70,000)(92) \quad 0.032 \text{ in.}^2 \ \ (20.8 \text{ mm}^2)$$

Page 431

Since at least 0.2 in.^{2} (129 mm^2) of vertical reinforcement is required around openings and at ends of walls (Section 10.6.3), use a No. 4 bar ($A_s = 0.20$ in.^{2}) in the end cell of both ends of the pier. Using Eq. 10.53 for the minimum axial load and assuming that the tension bar yields,

$$P_u / \phi = C - T = 0.8 f'_m b(0.8c) - A_s f_y$$

$$55,860/0.9 = 0.8(2,500)7.625(0.8c) - 0.20(60,000)$$

$$c = 6.02 \text{ in.} \ \ (153 \text{ mm})$$

Page 432

This shows that the strain in the tension steel reinforcing bars is $((d - c)/c)\,0.0025 = ((48 - 6.02)/6.02)0.0025 = 0.0174$ which is well above the yield strain of $60,000/29\times10^6 = 0.00207$. Therefore, we can continue and calculate the corresponding moment capacity using Eq. 10.38

$$M_n = C\left(\frac{l_w}{2} - \frac{0.8}{2}c\right) + T\left(d - \frac{l_w}{2}\right) = 0.8(2,500)(7.625)(0.8)(6.02)(24 - 0.4(6.02))$$

$$+ 0.20(60,000)(44 - 48/2) = 1.83\times10^6 \text{ in.-lb.} = 152.3 \text{ ft.-kips} \ \ (198 \text{ kN.m})$$

Page 436

From Eq. 10.45

$$Vnm = \left[4.0 - 1.75\frac{M_u}{V_u d_v}\right] A_{nv}\sqrt{f'_m} + 0.25P = [4 - 1.75(1)]40(7.625)\sqrt{2,500} + 0.25(106,727)$$

$$= 34,313 + 26,682 = 60,995 \text{ lb. } (271 \text{ kN})$$

Therefore, no shear reinforcement is required since $\phi V_n = 0.8(60,995) = 48,796$ lb. (217 kN) > $V_u = 25,252$ lb. (112.6 kN).

However, for seismic design the shear capacity ϕV_n should not be less than 1.25 times the shear corresponding to the moment capacity, M_n. The shear corresponding to $M_n = 219,360$ ft.-lb. is 25,252(219,360/126,260) = 43,872 lb. (195 kN). Therefore, the nominal shear strength is adequate. At the other extreme for Pier 1,

$$\phi V_n = \phi V_{nm} = 0.8(34,313 + 0.25(16,701)) = 30,790 \text{ lb.} \quad (137 \text{ kN})$$

Page 437

which indicates no coupling between shear walls. Using Eq. 10.41 but without the ϕ factor and assuming that bar 5 is yielding in compression and the other bars are yielding in tension.

$$P_u = C - T + C_s$$

$$260,000 = 0.8(3,000)10(0.8c) - 4(0.79)(60,000) + 1(0.79)(60,000)$$

$$c = 20.95 \text{ in. } (532 \text{ mm})$$

From observation of Figure 10.49,

$$\varepsilon_t = 0.0035\left(\frac{36 - 20.95}{20.95}\right) = 0.0025$$

$$\varepsilon_5 = 0.0035\left(\frac{20.95 - 4}{20.95}\right) = 0.0028$$

Compared to steel yield strain of 0.002069, this indicates that assumptions regarding stresses in the reinforcement are correct. From a ductility point of view, strain in the extreme tension steel is

$$\varepsilon_t = \left(\frac{132 - 20.95}{20.95}\right)0.0035 = 0.019$$

Page 438

At ultimate conditions, the strain condition shown in Figure 10.49(d) is reached. Assuming that Bars 1 and 4 yield in tension and Bar 5 yields in compression, equilibrium of axial load and internal forces gives

$$280,000 = C_m + C_s - T$$

$$280,000 = 0.80(3,000)(10)(0.8c) + 0.79(60,000) - 4(0.79)60,000$$

$$c = 21.99 \text{ in.} \quad (559 \text{ mm})$$

Page 445

(a) Determine the required thickness if it is to be constructed with solid clay bricks: (f'_m = 6000 psi (41.4 MPa)). 8 in. (200 mm)

(b) Determine the required amount of vertical and horizontal reinforcement if it is constructed with ~~6 in. (150 mm)~~ hollow clay masonry. Use TMS 402 strength design method. Assume 50% of the axial load is dead load and the other 50% is live load.

(c) Show the reinforcement details for the steel calculated in part (b).

(d) Determine wall displacement ductility using the charts in Figure ~~10.33~~. 10.37(a)

[*Note*: For parts (b) and (d), use f'_m = 3,000 psi (20.7 MPa) and f_y = 60 ksi (414 MPa) with fully grouted construction.]

10.6 For the shear wall of Problem 10.5, if the axial load is doubled, how will this increase affect wall reinforcement requirements and ductility?
