Calculations and Discussion	Code Reference

$$F_v = \sqrt{f'_m} < 50 \text{ psi}$$

$$F_v = \sqrt{2{,}500} = 50 \text{ psi}$$ Eq. (7-4)

$$F_v = \sqrt{1{,}500} = 38.7 \text{ psi}$$

For wind, $F_v = 1.33(50 \text{ psi}) = 66.5 \text{ psi}$ 5.3.2

For this case, $f_v = 5.8 \text{ psi} < 66.5 \text{ psi}$, ∴ OK

<u>For Compression in the Concrete Block Wythe:</u>

From MDG Example 11.1-6: (with the same wind pressure assumed in both directions)

$d = 8.63$ in.

$jd = 7.94$ in.

$$j = \frac{7.94 \text{ in.}}{8.63 \text{ in.}} = 0.92$$ 7.5.2.1

$$f_v = \frac{V}{bjd}$$ * $f_v = \frac{V}{bd}$ current code

$$f_v = \frac{300 \text{ lb}}{(12 \text{ in.})(0.92)(8.63 \text{ in.})}$$

$$f_v = 3.2 \text{ psi}$$

The maximum $F_v = \sqrt{1{,}500} = 38.7 \text{ psi}$ 7.5.2.2

For wind, $F_v = 1.33(38.7 \text{ psi}) = 51.5 \text{ psi}$ 5.3.2

This design is okay for shear.

Calculations and Discussion	Code Reference

See Note in MDG Example 11.1-6 regarding delamination.

Check for potential delamination

Maximum allowable collar joint shear = 10 psi 5.8.1.2

Increase 1/3 for wind = 13.3 psi 5.3.2

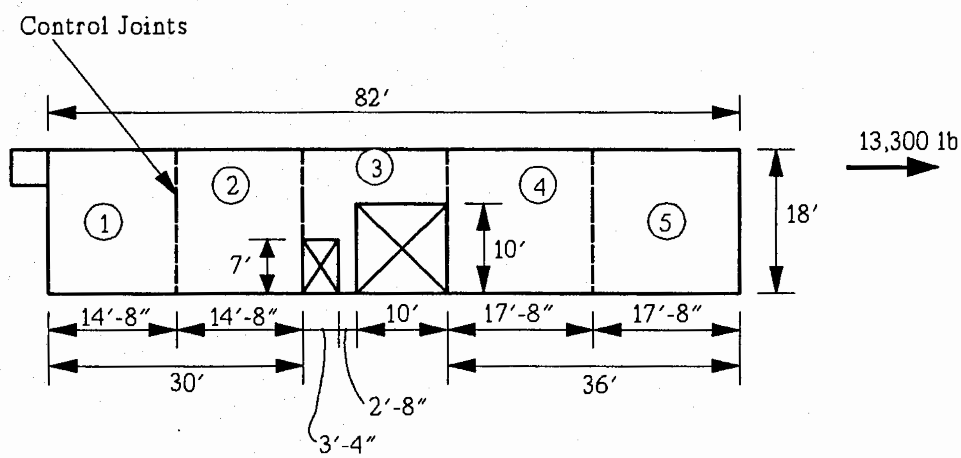
Delamination computations are generally based on the resistance provided by the contact area. However, since this example is based on a cracked section the transverse shear computed above will be conservatively used for this in plane shear check.

$f_v = 5.8$ psi < 13.3 psi $\therefore$ OK

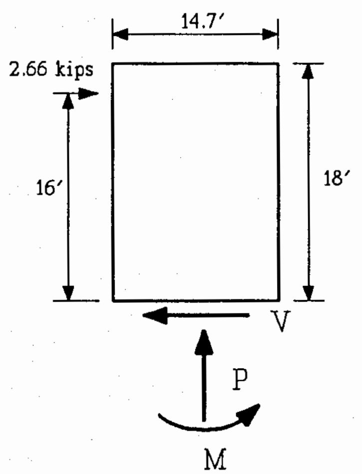
$f_v = 3.2$ psi < 13.3 psi $\therefore$ OK

Example 13.2-1 **TMS Shopping Center - Unreinforced Shear Wall Design**

Design one of the piers in the perforated shear wall elements of the TMS Shopping Center, located on Grid Line 3. between Grid Lines A and C. Assume Wall Construction Option A (unreinforced concrete masonry). The analysis shown in MDG Example 9.3-12 indicates that pier 1 is subjected to a shear load from the diaphragm of 2.66 kips, applied at the mean roof height of 16 ft.



Calculations and Discussion | Code Reference



The loading of pier 1 is shown below. It is assumed that there is no gravity roof load applied to this wall pier.

Assume that 12 in., hollow, face shell bedded CMU's are used with a weight of 46.5 psf. See MDG Appendix A for unit and wall properties tables.

$$P = (46.5 \text{ psf})(18 \text{ ft})\frac{(14.7 \text{ ft})}{(1{,}000 \text{ lb/kip})} = 12.3 \text{ kips}$$

$$M = (2.66 \text{ kips})(16 \text{ ft}) = 42.6 \text{ ft-kips}$$

Check Normal Stresses:

Assuming face shell bedding (t = 1.5 in. from MDG Appendix A)

$$\therefore\ A_n = 2 \times (14.7 \text{ ft} \times 12 \text{ in./ft}) \times 1.5 \text{ in.} = 529 \text{ in.}^2$$

$$S = \frac{bd^2}{6} = \frac{2 \times (1.5 \text{ in.}) \times (14.7 \text{ ft} \times 12 \text{ in./ft})^2}{6} = 15{,}600 \text{ in.}^3$$

$$\textit{Max. tensile stress} = -\frac{P}{A_n} + \frac{M}{S}$$

$$= -\frac{(12.3 \times 10^3) \text{ lb}}{529 \text{ in.}^2} + \frac{(42.6 \times 10^3 \times 12) \text{ in.-lb}}{15{,}600 \text{ in.}^3}$$

$$= -23.3 \text{ psi} + 32.9 \text{ psi} = 9.6 \text{ psi}$$

Tension stresses are not allowed for unreinforced wall elements subjected to in-plane forces since values in Code Table 6.3.1.1 apply only to out-of-plane loading. 6.3.1.1

Try 12 in. solid grouted CMU's (Assume 100 pcf)

$$P = \frac{(11.63 \text{ in.})}{(12 \text{ in./ft})}(100 \text{ pcf})(18 \text{ ft})\frac{(14.7 \text{ ft})}{(1{,}000 \text{ lb/kip})} = 25.6 \text{ kips}$$

$$A_n = (11.63 \text{ in.})(14.7 \text{ ft} \times 12 \text{ in./ft}) = 2{,}050 \text{ in.}^2$$

$$S = \frac{(11.63 \text{ in.}) \times (14.7 \text{ ft} \times 12 \text{ in./ft})^2}{6} = 60{,}300 \text{ in.}^3$$

Calculations and Discussion | **Code Reference**

$$Max.\ tensile\ stress = -\frac{25{,}600\ \text{lb}}{2{,}050\ \text{in.}^2} + \frac{(42.6 \times 10^3 \times 12)\ \text{in.-lb}}{60{,}300\ \text{in.}^3}$$

$$= -12.5\ \text{psi} + 8.49\ \text{psi} = -4.0\ \text{psi}$$

∴ No net tensile stresses.

Since the compressive stresses are so low the unity equation:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$ 6.3.1

is OK by inspection.

$$Shear\ Stress = \frac{VQ}{Ib_w}$$ 6.5.1

for rectangular sections $f_v = \frac{3}{2}\frac{V}{A_n}$

$$\therefore \quad f_v = \frac{3}{2}\left(\frac{2.66 \times 10^3\ \text{lb}}{2{,}050\ \text{in.}^2}\right) = 1.94\ \text{psi}$$

The allowable shear stress (F_v) is the least of: 6.5.2

a) $1.5\sqrt{f'_m}$

b) 120 psi

c) $v + 0.45\frac{N_v}{A_n}$

Assume f_m = 1,000 psi from prism testing of 1,300 psi units and Type N mortar.

Use f'_m = 1,000 psi

Calculations and Discussion | **Code Reference**

a) $F_v = 1.5\ (\sqrt{1{,}000\ \text{psi}}) = 47.4\ \text{psi}$

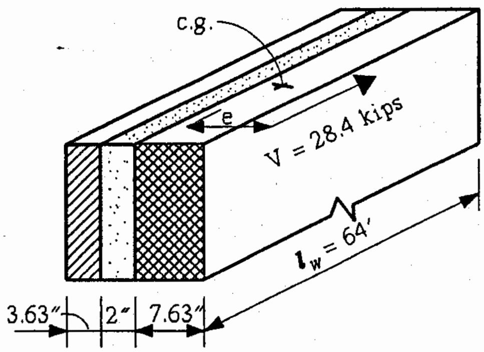
c) $F_v = 60\ \text{psi}$ (solid grouted units)

$\therefore$ $F_v = 60\ \text{psi} + 0.45(12.5\ \text{psi}) = 65.6\ \text{psi}$

$\therefore$ $F_v = 47.4$ psi governs and is much greater than $f_v = 1.95$ psi $\therefore$ OK

Use grouted 12 in. CMU's with a minimum compressive strength of 1,300 psi and Type N Mortar.

Design the East Wall of the DPC Gymnasium on Grid Line 2 for Seismic Zone 2. The East Wall is subject to a seismic in-plane shear load of 28,400 lb from MDG Example 9.2-2. Use Wall Construction Option B, unreinforced composite wall.



$f_g = 5{,}100$ psi

$E_g = 2.55 \times 10^6$ psi

$E_{Block} = 2.08 \times 10^6$ psi

$E_{Brick} = 2.40 \times 10^6$ psi

$(f'_m)_{block} = 1{,}500$ psi

$(f'_m)_{brick} = 2{,}400$ psi

The concrete block wythe is ungrouted.

Calculations and Discussion | **Code Reference**

$$\text{Average Wall Height} = \frac{24.67 \text{ ft} + 30 \text{ ft}}{2} = 27.3 \text{ ft}$$

Wall Weight = Concrete Masonry + Clay Masonry + Grout

Wall Weight **= 40 psf + 36.25 psf + 23.33 psf = 99.6 psf**

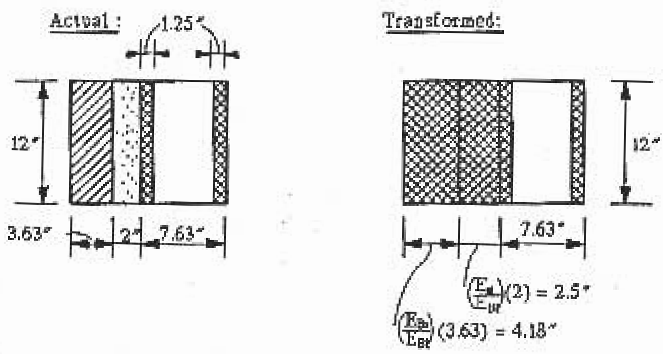
Check to see if wall is subject to in-plane flexural tension:

Overturning Moment = 28.4 kips (27.3 ft) = 776 ft-kips = 9,320 in.-kips

For nonloadbearing wall, P is due only to the dead weight

P = (99.6 psf)(24.67 ft) = 2,460 plf (using conservative minimum height)

The contribution of the wall flanges will be neglected in this problem. Transform a 12 in. wall length to equivalent concrete block: 5.13.1.2



$$\text{Axial Stress, } f_a = \frac{P}{A} = \frac{2{,}460 \text{ lb}}{(4.18 \text{ in.} + 2.5 \text{ in.} + 2.5 \text{ in.})(12 \text{ in.})} = 22.3 \text{ psi}$$

Bending Stress due to overturning in-plane moments:

$$f_b = \frac{M}{S}$$

$$f_b = \frac{(9{,}320 \text{ in.-kips})(6)(1{,}000 \text{ lb/kip})}{(4.18 \text{ in.} + 2.5 \text{ in.} + 2.5 \text{ in.})(64 \text{ ft})^2(144 \text{ in./ft})}$$

$$f_b = 10.3 \text{ psi}$$

Walls subjected to flexural tension must be reinforced and designed for shear according to Code 7.5.2.

$$\frac{P}{A} - \frac{M}{S} = 22.3 \text{ psi} - 10.3 \text{ psi} = 12.0 \text{ psi compression}$$

∴ Wall is not subject to flexural tension; thus, check the wall design as an unreinforced shear wall, Code Chapter 6.

Calculations and Discussion | **Code Reference**

Consider shear stresses:

Note that a combined shear due to direct in-plane shear plus shear due to twisting exists as illustrated by the eccentricity distance, *e*, at the beginning of the problem. Therefore, a torsion stress, τ, exists and can be computed from:

$$\tau = \frac{Tc}{J}$$

where *c* is the distance from the center of gravity to location of torsion stress, *T* is the torsional moment, and *J* is the polar moment of inertia of the cross section. This torsion stress τ can be computed and added to the direct shear stress. However, this torsional shear stress is usually small. For this problem, if the direct shear stress is close to the allowable, then the torsional shear stress would need to be computed, otherwise it can be neglected.

Calculate direct shear stress: 6.5

$$f_v = \frac{VQ}{Ib} = \frac{3V}{2A} = \frac{3V}{2Lb}$$

$$f_v = \frac{3(28{,}400\ \text{lb})}{2(64\ \text{ft})(12\ \text{in./ft})(4.18\ \text{in.} + 2.5\ \text{in.} + 2.5\ \text{in.})}$$

$$f_v = 6.0\ \text{psi} \quad \text{in the CMU}$$ Eq. (6.7)

$$f_v = 6\ \text{psi}\left(\frac{2.4 \times 10^6\ \text{psi}}{2.08 \times 10^6\ \text{psi}}\right) = 6.9\ \text{psi} \quad \text{in brick}$$

$$f_v = 6\ \text{psi}\left(\frac{2.55 \times 10^6\ \text{psi}}{2.08 \times 10^6\ \text{psi}}\right) = 7.4\ \text{psi in grout}$$

Calculations and Discussion | **Code Reference**

Checking the CMU the allowable shear stress, F_v, is the least of 6.5.2

(a) $F_v = 1.5\sqrt{f'_m} = 1.5\sqrt{1{,}500\text{ psi}} = 58\text{ psi}$

(Note: could split shear by the proportional amount carried by each wythe and use allowable for each. Since stresses are small, the lower f'_m will conservatively be used.)

(b) $F_v = 120$ psi

(c) $F_v = v + 0.45\left(\frac{N_v}{A_n}\right)$

Use running bond and not solidly grouted, so $v = 37$ psi

thus,

$$F_v = v + 0.45\left(\frac{N_v}{A_n}\right) = 37\text{ psi} + 0.45\left(\frac{2{,}460\text{ lb}}{(4.18\text{ in.} + 2.5\text{ in.} + 2.5\text{ in.})(12\text{ in.})}\right)$$

$$F_v = 47.0\text{ psi}$$

(d) does not apply since units laid in running bond

The allowable shear stress, $F_v = 1.33(47.0\text{ psi}) = 62.6$ psi 5.3.2

$f_v < F_v$ i.e. 6.0 psi < 62.6 psi ∴ OK

It is obvious that the shear in the brick and grout are OK.

3. Interface stresses due to differential volume changes:

Note that clay brick expansion coupled with concrete shrinkage may induce an interface shear stress that should be checked. These differential displacements are not part of this problem and are discuss elsewhere in this MDG (See Chapter 10).

Calculations and Discussion | **Code Reference**

4. Interface shear stresses for multiwythe walls:

The ties across the interface between wythes must be capable of taking the interface shear stress, if this stress is deemed to be beyond the usual small amount. Code 5.8.1.2 provides for an allowable of 10 psi. In this case the proportional amount of shear carried across the interfaces does not need to be computed since the f_v = 4.0 psi < 10 psi ⇒ already OK

5. Ties across the interface:

Code 5.8.1.1 requires wall ties across the grouted collar joint. Code 5.8.1.5 requires at least one #9 gage wall ties per 2.67 ft^2 of wall with a horizontal spacing ≤ 36 in. and vertical spacing ≤ 24 in.

Place a #9 gage wall tie (styles other than "Z" wall ties can be selected from manufacturer's catalogs) at 16 in. on center vertically and 24 in. on center horizontally. Z wall ties are not acceptable for this wall as per Code 5.8.1.5.

6. Check the unity equation for the compression side of the in-plane flexure:

This check will be illustrated (even though the stresses are small and could be ignored.)

From Code 6.3.1:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$$ Eq. (6-1)

f_a = 22.3 psi (see above)

Calculations and Discussion | **Code Reference**

$$F_a = \frac{1}{4}(f'_m)\left[1 - \left(\frac{h}{140r}\right)^2\right],\ if\ \frac{h}{r} < 99$$ Eq. (6-3)

$$F_a = \frac{1}{4}(f'_m)\left(\frac{70r}{h}\right)^2,\ if\ \frac{h}{r} > 99$$ Eq. (6-4)

Based on $\frac{h}{r}$:

$r = 0.287t$

$r = 0.287(4.18\text{ in.} + 2.5\text{ in.} + 2.5\text{ in.}) = \mathbf{2.63\text{ in.}}$

Use peak height as conservative slenderness:

$$\frac{h}{r} = \frac{(30\text{ ft})(12\text{ in./ft})}{2.63\text{ in.}} = 136.9 > 99$$

using $f'_m = 1{,}500$ psi

Thus,

$$F_a = \frac{1}{4}(1{,}500\text{ psi})\left(\frac{70(2.63\text{ in.})}{30\text{ ft}(12\text{ in./ft})}\right)^2$$ 6.3.1

$F_a = 1.33(98\text{ psi}) = 130$ psi 5.3.2

$$F_b = 1.33\left(\frac{1}{3}\right)f'_m$$ Eq. (6-5)

(with the 1.33 factor from Code 5.3.2 since in-plane bending is due to seismic load)

$$F_b = 1.33\left(\frac{1}{3}\right)(1{,}500\text{ psi})$$

$F_b = 665$ psi

Calculations and Discussion	Code Reference

Unity Eq. :

$$\frac{22.3 \text{ psi}}{130 \text{ psi}} + \frac{10.3 \text{ psi}}{665 \text{ psi}} = 0.187 < 1.0$$ Eq. (6-1)

thus, as stated previously, this check was not expected to be a problem, but is included for illustrative purposes.

Since the shear wall is in Seismic Zone 2 both vertical and horizontal steel must be provided. Provide vertical reinforcement of 0.2 in.^{2} (#4 reinforcing bar) at the two wall ends. Provide horizontal reinforcement of 0.2 in.^{2} (#4 reinforcing bar) at top and bottom of wall and intermediate locations with maximum vertical spacing of 10 ft. Place both vertical and horizontal reinforcement in grouted collar joint. A.3.8