Masonry Beam Design

Strength Design

Flexure and Shear

481

Code References

- 2003 International Building Code (IBC)
- 2006 International Building Code (IBC)
- 2002 ACI 530/ASCE5/TMS402 by the Masonry Standards Joint Committee (MSJC)
- 2005 ACI 530/ASCE5/TMS402 by the Masonry Standards Joint Committee (MSJC)

Note:

** indicates equations found within general text books that are not contained in IBC , ASCE, or MSJC codes or commentaries.

482

Code References for Beam Design

There are few changes between
the 2003 IBC/02 MSJC
and
the 2006 IBC /05 MSJC

Exceptions will be noted

Otherwise code references apply to either code

483

Beam Strength Design

Beam Design Using Factored Loads and Strength Reduction Factors

•2006 International Building Code (IBC)
(2006 IBC) Section 2108

•2005 ACI 530/ASCE5/TMS402 by the Masonry Standards Joint Committee (MSJC)
(2005 MSJC) Chapter 3

484

Beam Strength Design

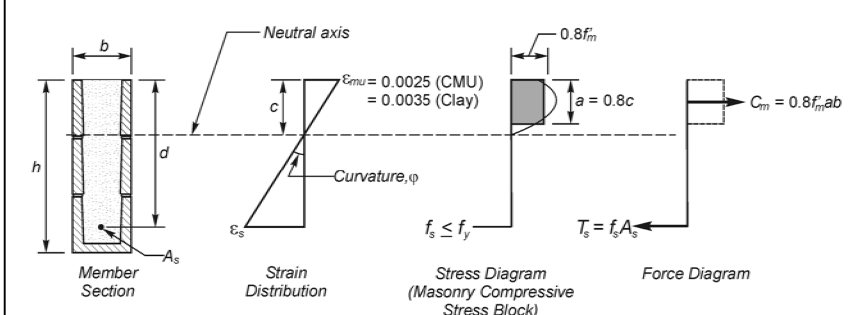
Masonry strength design is similar to concrete strength design using the Whitney Stress Block except that some assumptions are different

- Maximum Usable Strain
- Depth of Compression Block
- Average Stress on Compression Block

485

Beam Strength Design

2005 MSJC / 2006 IBC



486

Equilibrium of Beam Cross Section

$C_m = .8\ f'_m\ a\ b$

$T_s = A_s\ F_y$

$T_s = C_m$

$A_s\ F_y = .8\ f'_m\ a\ b$

a = depth of compression block

$$a = \frac{A_s f_y}{0.80\, f'_m\, b}$$

$$M_n = (T)(jd) = A_s f_y \left(d - \frac{a}{2}\right)$$

$$M_u = \phi M_n$$

487

Flexural Strength of Beam Cross Section

$$M_n = (T)(jd) = A_s f_y \left(d - \frac{a}{2}\right) \qquad a = \frac{A_s f_y}{0.80\, f'_m\, b}$$

$$M_u = \phi M_n$$

$$M_u = \phi A_s f_y \left(d - \frac{A_s f_y}{(2)(0.80) f'_m b}\right) \qquad A_s = \rho\ b\ d$$

$$M_u = \phi\ \rho\, b d f_y \left(d - \frac{(\rho\ b\ d) f_y}{(2)(0.80) f'_m b}\right)$$

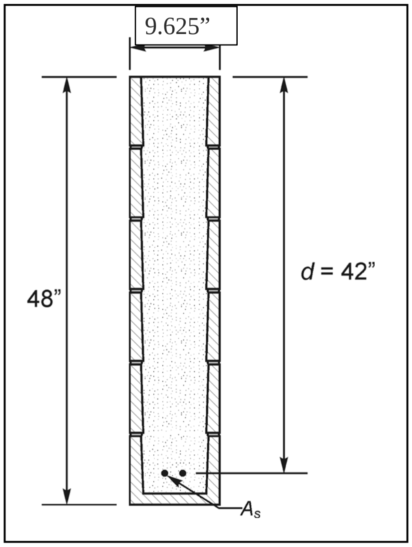
$$M_u = \phi\ \rho b d^2 f_y \left(1 - 0.63\left(\rho\ \frac{f_y}{f'_m}\right)\right) \qquad \text{Let}\ \ q = \rho\ \frac{f_y}{f_m'}$$

$$= \phi b\ d^2\ f'_m q (1 - 0.63\, q)$$

Solve for q to find required A_s

488

Beam Design Example



- Beam is solid grouted 10 inch CMU
- L_{clear} = 8’-0”
- M_u = 160 kip-ft; V_u = 80 k
- f'_m = 2500 psi
- Grade 60 steel
- Type S Mortar

Determine required flexural reinforcement

489

Beam Design Example

Load Combinations

$1.2D + 1.6L$ IBC 1605.2.1

Flexural Strength Reduction Factor (No Axial Loads):

$P=0 < 0.05A_n f'_m$ MSJC: 3.3.4.2.1

$\phi = 0.90$ MSJC: 3.1.4.1

490

Beam Design Example

2005 MSJC:

$$M_u = \phi\, b\, d^2 f'_m\, q\,(1 - 0.63q)$$

$$(160)(12) = (0.9)(9.63)(42)^2 (2.5)\, q\,(1 - 0.63q)$$

$$0.0502 = q(1 - 0.63q)$$

$$0.63q^2 - q + 0.0502 = 0$$

Solve quadratic equation

$$q = \frac{-(-1) - \sqrt{1 - 4\cdot(0.63)\,(0.0502)}}{2\cdot(0.63)} \qquad q = 0.052$$

491

Beam Design Example

2005 MSJC:

$$q = 0.052 \qquad \rho = q\,\frac{f'_m}{f_y}$$

$$\rho = (0.052)\,\frac{2.5\text{ ksi}}{60\text{ksi}} \qquad \rho = 0.0022$$

$$A_s = \rho\, b\, d \qquad = 0.0022\,(9.625\text{ in})\,(42\text{ in})$$

$$A_s = 0.875\,\text{in}^2$$ Use 2-#6 A_s actual = 0.88 in^2

492

Beam Design Example

2005 MSJC:

Actual Nominal Flexural Capacity:

$M_n = b\, d^2\, f'_m\, q\, (1\text{-}0.63q)$ $q = [As/(bd)]\ (fy/f'_m)$

$q = [(0.88\ in^2)/(9.625")(42")]\ (60ksi/2.5ksi) = 0.0522$

$M_n = (9.625")\ (42in2)\ (2.5ksi)\ (0.522)[1\text{-}0.63(0.522)](1'/12"/')$

$M_n = 179$ k-ft (will need for upcoming check)

$\phi M_n = 0.9*179$ k-ft = 161 k-ft > $M_u = 160$ k-ft

Flexural Capacity OK
and efficient

493

Check Beam Cracking Moment

2005 MSJC / 2006 IBC

MSJC Section 3.3.4.2.2.2 Minimum Capacity: $1.3M_{cr} \leq M_n$

Table 3.1.8.2.1:
tensile stress parallel to bed joints, fully grouted, Type S mortar $f_r = 200$ psi

$1.3M_{cr} = 1.3S_x f_r$

$= 1.3*[(9.625\ in)*(48\ in)^2/6]*[(200\ psi)/[(12\ in/ft)*(1000\ lb/k)]]$

$1.3M_{cr} = 1.3(61.60$ ft-kip) = 80 ft-kip < $M_n = 179$ k-ft

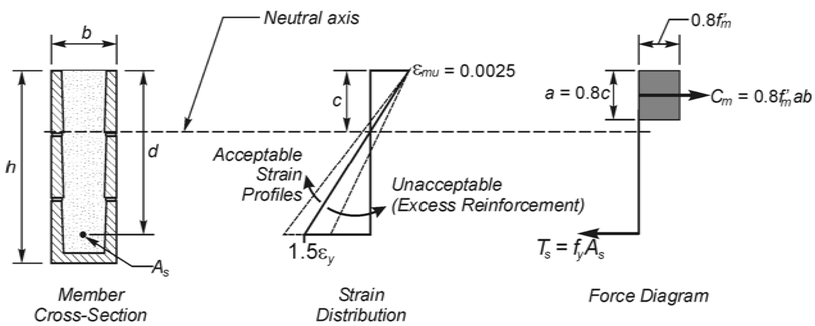
*Nominal Flexural Strength OK

494

Check Maximum Reinforcement

2005 MSJC / 2006 IBC

Section 3.3.3.5.1:



495

Check Maximum Reinforcement

2005 MSJC / 2006 IBC---changed from 2002 MSJC/2003 IBC

Section 3.3.3.5.1:

$$\rho_{\max} = \frac{A_{s,\max}}{bd} = 0.64\frac{f'_m}{f_y}\left(\frac{\varepsilon_{mu}}{\alpha\varepsilon_y + \varepsilon_{mu}}\right) \quad \text{(CC 3.3.3.5)}$$

For concrete masonry beams with no axial load:

$\alpha = 1.5$

$\varepsilon_{mu} = 0.0025$ for CMU (3.3.2.(c))

$$\rho_{\max} = 0.64\left(\frac{2.5}{60}\right)\left(\frac{0.0025}{1.5(60)/29000 + 0.0025}\right) = 0.012$$

$$\rho = \frac{A_s}{bd} = \frac{0.88}{(9.5)(42)} = 0.0022 < \rho_{\max} = 0.012$$

496

Beam Shear Strength

- Beam shear strength design is similar to concrete shear strength design.
- The total shear strength is based on combined masonry and steel contributions

$$V_n = V_m + V_s \qquad \text{MSJC Equation (3-18)}$$

There is no guidance for evaluating the shear force location at a distance of d/2 away from the support (similar to Allowable Stress Design). Therefore, we take V_u at the face of support.
(You may also reasonably chose to check at d/2 from support).

Beam Shear Strength

2005 MSJC / 2006 IBC

- $\phi = 0.80$ MSJC 3.1.4.3
- SHEAR: Section 3.3.4.1.2
- $V_m = [4 - 1.75\{M/(Vd_v)\}]\, A_n (f'_m)^{1/2} + 0.25P$
  where $M/(Vd_v) \leq 1.0$ Eqn (3-21)
  For beams with no axial loads, P = 0
- $V_s = 0.5(A_v/s)\, f_y\, d_v$ Eqn (3-22)
- both equations are empirical and are research derived

Beam Shear Strength

Total masonry shear strength is based on combined moment and shear components where M is the moment at the location that shear is checked.

$$V_m = [4 - 1.75\{M/(Vd_v)\}]\,A_n(f'_m)^{1/2}$$

If M & V due to a purely uniform load, at end of beam M=0 @ V_{max}, therefore

$$M/Vd_v=0$$

If a centered concentrated load, M=PL/4 and V=P/2 at the centerline. $M/(V\,d_v) = ½(L/d_v)$ so longer spans reduce shear capacity.

499

Beam Shear Strength

Example for the same beam problem:

- Span, L_{clear} = 8′-0″
- Axial Load, P = 0
- V_u = 80 kips
- f'_m = 2500 psi
- f_y = 60 ksi
- b = 9.625 in
- d_v= h = 48 in (MSJC 1.5)
- $A_n = b*d_v$ = 9.625″ * 48″ = 462 in^2

500

Beam Shear Strength

- The 2005 MSJC 3.1.3 provides an overall ductility requirement for shear only. Specifics follow.
- The intent is to provide a ductile failure mechanism in flexure prior to a shear failure.
- The shear capacity must correspond to a loading that produces a moment that is 25% higher than the actual flexural capacity of the member but less than 2.5 times the calculated V_u.

501

Beam Shear Strength

- MSJC ductile shear requirement will apply only to special reinforced shear walls, not beams, beginning in MSJC 2008 (1.17.3.2.6.1)
- Could defend that this corrected requirement should not apply to non tie-beam shear design (or any beam design) under the MSJC 2005 code.
- Both versions presented in the remainder of the example

502

Beam Shear Strength

- *MSJC 3.1.3: ΦV_n shall exceed the shear corresponding to 1.25 times the nominal flexural strength, $1.25M_n$, but need not exceed 2.5 times the required shear strength, $2.5V_u$. Call location of $1.25M_n$, the "ductile" location:*
  - *$M_{ductile} = 1.25M_n = 1.25(179\text{ k-ft}) = 224\text{ k-ft}$*
  - *$M = wL^2/8$ *uniform distributed loading*
    - *Solve for ductile w, $w = 8M/L^2$*
      $= 8(224\text{ ft-k})/(8\text{ ft})^2 = 28\text{ kips/ft}$
  - *Corresponding $V_{ductile}$ at $1.25\ M_n$*
    $= wL/2 = (28\text{ k/ft})(8\text{ ft})/2 = 112\text{ kips}$

503

Beam Shear Strength

$V_{ductile}$ at $1.25\ M_n$ loading $= 112$ kips
But need not exceed:

$V_{max} = 2.5Vu = 2.5(80\text{ k}) = 200\text{ k} > 112\text{ k}$
Use $V_{max} = 112$ k controls

$\Phi V_{n\ min} = 112\text{ k} > V_u = 80\text{ k}$

In future codes, this requirement will NOT apply to beams or other elements except special reinforced shear walls. The example will continue with both 80 kips and 112 kips for comparison. (The $V_{ductile}$ value will no longer be a beam shear parameter.)

504

Beam Shear Strength

M=0 @ Max V and the shear strength of the masonry is then,

$V_m = [4 - 1.75\{M/(Vd_v)\}] A_n(f'_m)^{1/2}$

$V_m = [4 - 0] (462in^2)(2500psi)^{1/2} + 0 = 92.4$ kips

$\Phi V_m = (0.8)92.4k = 73.9 \text{ kips} < V_u = 112\text{ k}, 80\text{ k}$

*MSJC 3.3.4.2.3 *Transverse reinforcement is Required (Stirrups)*

505

Beam Shear Strength

- Check $A_{v\ min}$:
  - MSJC 3.3.4.2.3(c):
    - (a) and (b) provide details for stirrups
  - $A_{vmin} = 0.0007bd_v = 0.0007(9.625in)48in = 0.32\ in^2$
  - Use #5 bar -- Within 5% of A_{vmin}=0.32 in^2 and #6 bars are not practical for beam stirrups.
  - Could use #4, 2-leg for A_v=0.40 in^2 but difficult to position in a 10 inch wide beam

506

Beam Shear Strength

- Calculate stirrup requirements
  - $V_u \le \Phi(V_m+V_s)$
    112 (80K) kips ≤ 0.8(92.4k+V_s)
    Solve for V_s, $V_{u\ ductile}$ =112k, V_s = 47.6 kips
    V_u = 80k, V_s = 7.60 kips
  - *MSJC 3.3.4.1.2.3: Eqn (3-22)*
    - $V_s = 0.5(A_v/s)\, f_y\, d_v$

507

Beam Shear Strength

Vu =112 k, Vs $_{ductile}$ = 47.6 kips
Vu =80 k, Vs = 7.60 kips

$$s = \frac{0.5A_V f_y d_V}{V_s} = \frac{0.5(0.31\ \text{in}^2)(60\ \text{ksi})(48\ \text{in})}{(47.6\ \text{kips})}$$

= 9.38 in ⟶ #5 at 8 in (block module)
actual Vs at 8"oc = 55.8 k

$$s = \frac{0.5A_V f_y d_V}{V_s} = \frac{0.5(0.31\ \text{in}^2)(60\ \text{ksi})(48\ \text{in})}{(7.60\ \text{kips})}$$

= 58.74 in ⟶ #5 at d/2 = <u>24 inch</u> or 48 inch max
actual Vs at 24"oc = 18.60 k

508

Beam Shear Strength

where maximum spacing, S_{max} , is from:

- MSJC 3.3.4.2.3(e):
- S_{max} = h/2 = (48 in)/2 = 24”
- S_{max} = 48 in

509

Beam Shear Strength

- Check actual Max V_n to preclude brittle failures:
  - MSJC 3.3.4.1.2 (a-c):
  - $(M_u/V_ud_v \leq 0.25)$: Eqn (3-19)

    $V_n \leq 6A_n[f'_m]^{1/2}$
  - $(M_u/V_ud_v \leq 1.0)$: Eqn (3-20)

    $V_n \leq 4A_n[f'_m]^{1/2}$
  - V_{nmax} is permitted to be interpolated between the two values.

510

Beam Shear Strength

M_u=0 @ Max V_u and thus $M_u/V_u d_v$ = 0:

- *Eqn 3-19:* $V_{n\,max} = 6A_n[f'_m]^{1/2}$
  $V_{n\,max} = 6(462in^2)(2500psi)^{1/2}$ = <u>139 kips MAX</u>

- *Eqn 3-18* $V_n = V_m + V_s$ $_{or}$ *Vs ductile*
  V_n *= 92.4k + 55.8 k = 148.2 kips so =139k max capacity*

  ΦV_n *= 0.8(139 k) = 111 k ~* $V_{u\,ductile}$ *= 112 k,*
  But within 5%, Shear Strength OK for ductile requirement

- Eqn 3-18 $V_n = V_m + V_s$ $_{or}$ Vs ductile
  V_n = 92.4k + 18.6 k = 111.0 kips capacity<139k

  ΦV_n = 0.8(111 k) = 88.8 k > V_u = 80 k, OK

511

Beam Shear Strength

MSJC 3.3.4.2.3(d):

The first transverse bar shall be located within $d_v/4$ of the beam end.

$d_v/4$ = 48″/4 = 12″ > s = 8″ for block module

*Start first stirrup in first cell at 4 inches from the face of the opening to be centered in the first cell.

512

Beam Shear Strength

The spacing of the stirrups can be increased for the locations farther away from the support as actual shear decreases - similar to allowable stress design or concrete strength design.

Although shear reinforcing is not required by code for some locations in the beam where $V_u \leq \Phi V_m$, in higher seismic areas, it is good practice to use min $d_v/2$ (or at least d_v) spaced shear reinforcing throughout the beam length for ductility purposes.

513