Design of Beams

TMS20210210

February 10, 2021

Richard M. Bennett, Ph.D., P.E., FTMS
Professor
University of Tennessee

The Masonry Society
AIA Provider: 50119857

The Masonry Society is a registered Provider with the American Institute of Architects Continuing Education Systems. Credit earned on completion of this program will be reported to CES Records for AIA members. Certificates of completion for non-AIA members are available upon request.

This program is registered with AIA/CES for continuing professional education. As such, it does not include content that may be deemed or construed to be an approval or endorsement by the AIA of any material of construction or any method or manner of handling, using, distributing or dealing in any material or product.

Questions related to specific materials, methods, and services will be addressed at the conclusion of this presentation.

2

Course Description

This session will review the design of masonry beams and lintels, including an examination of whether arching action can be used to reduce the loads on these elements. Deflection calculations will be reviewed, along with detailing requirements to meet code minimum and maximum reinforcement percentages. Deep beam requirements will also be covered.

3

Learning Objectives

- Introduce the design masonry beams and lintels for bending moment and shear
- Describe deflection calculations for beams and lintels
- Review maximum and minimum reinforcing provisions
- Examine arching and discuss when it can be used

4

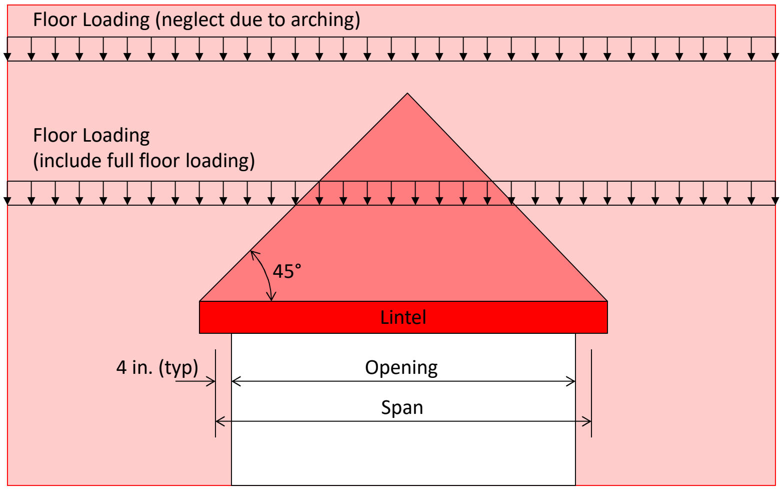
Determination of Loads, Shears, and Moments

- Arching
- Beam Depth
- Beam Span

Arching

(State College, PA)

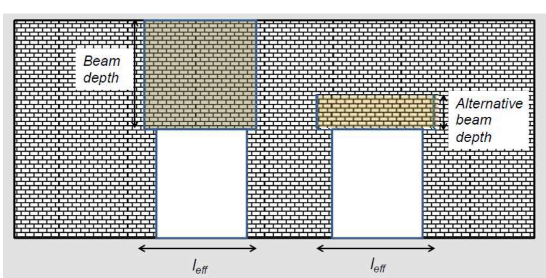
Arching



7

Beam Depth

Strength Design Guide 3.3.1



TMS 402 Figure CC-5.2-2

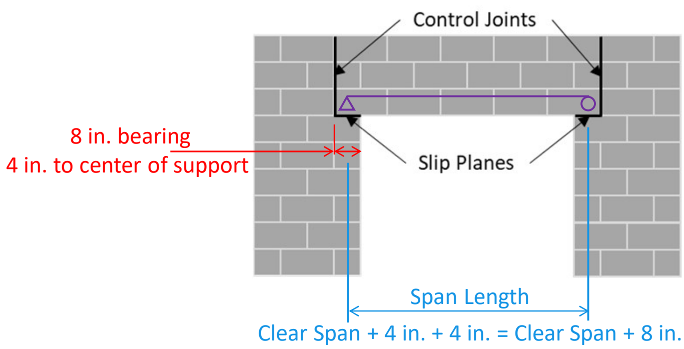
Strength Design Guide Figure 6.3.1-3

8

Beam Span: Beams not built integrally with supports

Strength Design Guide 6.3.1; TMS 402 5.2.1.1.1

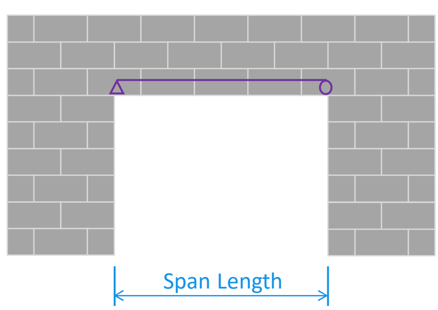
Span is minimum of:
- Clear span + depth of beam
- Distance between centers of supports



Beam Span: Beams built integrally with supports

Strength Design Guide 6.3.1; TMS 402 is silent

Reasonable approximation: Clear span
- without significant negative moment reinforcement
- Lee et al (1983) showed end restraint reduced deflection from 20-45% of simply support

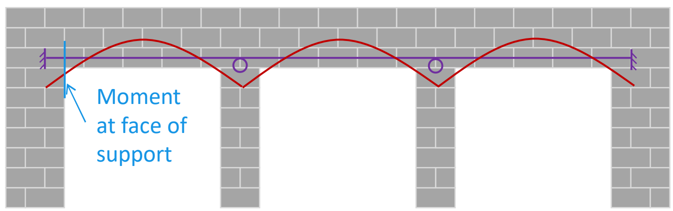


Beam Span: Continuous

Strength Design Guide 6.3.1; TMS 402 5.2.1.1.2

Requires negative moment reinforcement (otherwise built integrally with supports)

- Span length distance between centers of support for determining moments
- Reasonable approximation: Design for moment at face of supports



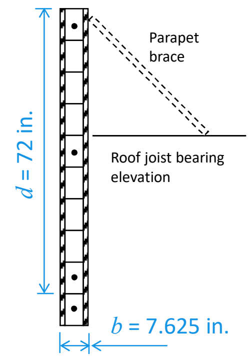
Beam Requirements

- Chapter 5
- Chapter 9

General Beam Design

Strength Design Guide 6.3.1; TMS 402 5.2.1

- Lateral support (TMS 402 5.2.1.2)
  - Minimum of:
    - $32b$
    - $120b^2/d$
- Bearing length (TMS 402 5.2.1.3)
  - Minimum of 4 in. in direction of span



$$32b = 32(7.625in.) = 244in.$$

$$120b^2/d = 120(7.625in.)^2/72in. = 96.9\ in.$$

Brace at every roof joist, which is 5 ft spacing

13

Beams

Strength Design Guide 6.3.1; TMS 402 9.3.4.2

- $P_u \leq 0.05A_n f'_m$ (TMS 402 9.3.4.2.1)
- ~~The variation in longitudinal reinforcing bars in a beam shall not be greater than one bar size. Not more than two bar sizes shall be used in a beam.~~ (TMS 402 9.3.4.2.1.1)
- Beams are required to be fully grouted (TMS 402 9.3.4.2.4)

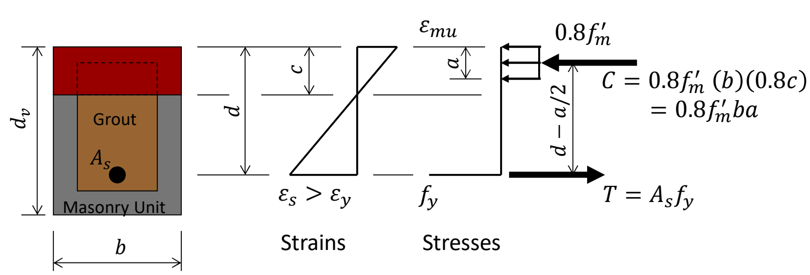
14

Flexural Design

- Design Moment Strength
- Flexural Design
- Minimum/Maximum Reinforcement
- Beam Construction

Nominal Moment Capacity

Strength Design Guide 6.2.1; TMS 402 9.3.2 (Design Assumptions)



ε_{mu} = 0.0025 CMU

ε_{mu} = 0.0035 SCU (Clay)

$$a = \frac{A_s f_y}{0.8 f'_m b}$$

$$M_n = A_s f_y \left(d - \frac{1}{2} \frac{A_s f_y}{0.8 f'_m b} \right)$$

ϕ = 0.9 for flexure

(TMS 402 9.1.4.4)

Flexural Design

Strength Design Guide 6.2.1.1

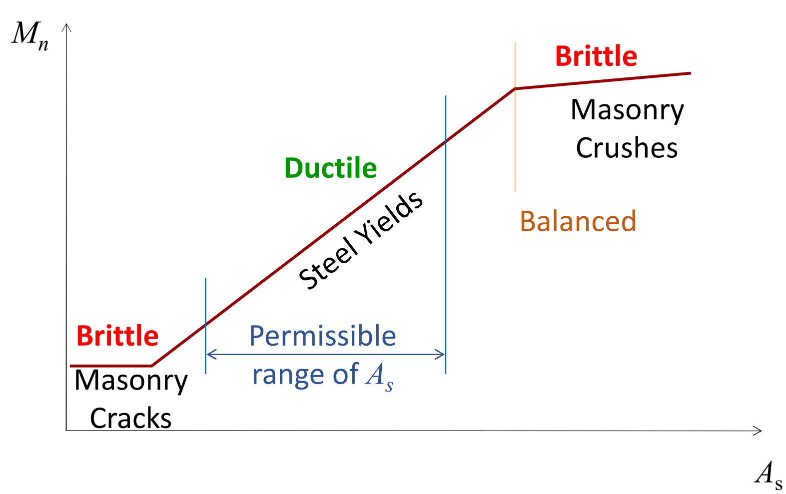
1. Determine material properties $(f_y,\ f'_m)$
2. Choose beam dimensions
   A. Thickness: 8 in., 12 in.
   B. Depth: if possible, choose so no shear reinforcement is required
3. Determine a, depth of compressive stress block

$$a = d - \sqrt{d^2 - \frac{2M_n}{0.8f'_m b}} = d - \sqrt{d^2 - \frac{2M_u}{0.8\phi f'_m b}}$$

4. Solve for $A_{s,reqd}$ $\qquad A_{s,reqd} = \frac{0.8f'_m ba}{f_y}$

17

Reinforcement Limitations



18

Minimum Reinforcement

Strength Design Guide 6.3.1.3; TMS 402 Section 9.3.4.2.2.2, 9.3.4.2.2.3

- $M_n \geq 1.3M_{cr}$, or $M_u \geq 1.17M_{cr}$
  - $M_{cr} = f_r \frac{bh^2}{6} = f_r \frac{bd_v^2}{6}$
- or $A_s \geq (4/3)A_{s,reqd}$

M_{cr} = cracking moment
f_r = modulus of rupture
b = thickness
$h = d_v$= depth of beam

Flexural Tension Stress Parallel to Bed Joint	Mortar Type			
	PCL or mortar cement		Masonry Cement	
	M or S	N	M or S	N
Running bond; fully grouted	267	200	160	100
Not laid in running bond; continuous grout section parallel to bed joints	335	335	335	335

Maximum Reinforcement

Strength Design Guide 6.3.1.2; TMS 402 Section 9.3.3.2

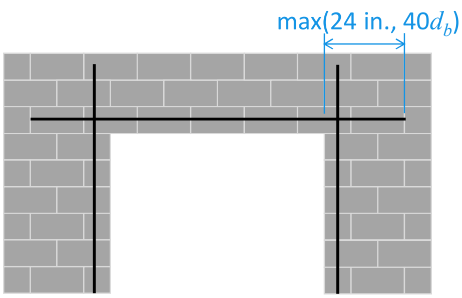
$$\rho_{max} = \frac{0.8(0.8)f'_m}{f_y}\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + 1.5\varepsilon_y}\right)$$

Steel Ratio	Grade 60 steel	
	Clay	CMU
ρ_{max} (f'_m in ksi)	$0.00565f'_m$	$0.00476f'_m$
f_y = 60 ksi	f'_m = 3 ksi 0.01697	f'_m = 2 ksi 0.00952

Beam Construction

Strength Design Guide 6.3.1.5

- d = depth to centroid of tension reinforcement
- For one layer of reinforcement, $d \sim (d_v - 4in.)$



Seismic Requirements (TMS 402 7.3.2.3.1)

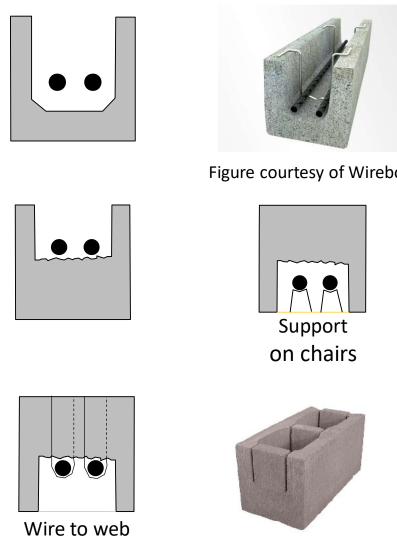


Figure courtesy of Wirebond

Deflections

- Effective Moment of Inertia
- Deflection Requirements

Effective Moment of Inertia

Strength Design Guide 6.3.1.1; TMS 402 Section 5.2.1.4

$$I_{eff} = I_n \left(\frac{M_{cr}}{M_a}\right)^3 + I_{cr}\left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right]$$

$$I_{cr} = \frac{bk^3d^3}{3} + nA_s(d - kd)^2$$

$$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho$$

$$\rho = \frac{A_s}{bd} \qquad n = \frac{E_s}{E_m}$$

I_n = net moment of inertia
I_{cr} = cracked moment of inertia
M_{cr} = cracking moment
M_a = Moment under allowable stress level loads
n = modular ratio, E_s/E_m
k = depth to neutral axis under allowable stress assumptions

23

Deflection Requirements

Strength Design Guide 6.3.1.1; TMS 402 Section 5.2.1.4

- Deflection of beam or lintels supporting unreinforced masonry is limited to $L/600$, where L is span length (TMS 402 5.2.1.4.1)
- Deflections of approximately $L/300$ needed to be visible.
- Deflections do not need to be checked when $L \leq 8d$ (TMS 402 5.2.1.4.3).

24

Shear

- Shear strength
- Stirrups
- Chapter 8: shear at $d/2$

Shear Strength

Strength Design Guide 6.2.4.1; TMS 402 Section 9.3.4.1.2

Assume $M_u/(V_u d_v)$ = 1.0

$$V_n = V_{nm} + V_{ns}$$

$$V_{nm} = 2.25 A_{nv} \sqrt{f'_m}$$

$$V_{ns} = 0.5 \left(\frac{A_v}{s}\right) f_y d_v$$

$$V_n \leq 4 A_{nv} \sqrt{f'_m}$$

- d_v = actual depth of masonry
- A_{nv} = net shear area = bd_v
  - Many designers use d instead of d_v for beams; clarified in 2022 TMS 402
- ϕ= 0.8

Stirrups

Strength Design Guide 6.3.1; TMS 402 Section 9.3.4.2.3

a) Single bar with 180-degree hook at each end
b) Hook shear reinforcement around flexural reinforcement
c) Minimum area of shear reinforcement is $0.0007bd_v$
   - Interpreted as $A_v/s \geq 0.0007b$
d) First bar within $d_v/4$
e) Maximum spacing is $d_v/2$ or 48 in.

$$d_{min} = \frac{V_u}{1.8b\sqrt{f'_m}}$$ for no shear reinforcement

Shear at *d*/2

Strength Design Guide 6.4.3.4; TMS 402 Section 8.3.5.4

Sections within $d/2$ from face of support can be designed for shear at $d/2$ (TMS 402 8.3.5.4): (moved to Chapter 5 in 2022 TMS 402)

A. Noncantilever beam
B. Reaction introduces compression into end region of member
C. No concentrated load between $d/2$ and face of support

Design Example

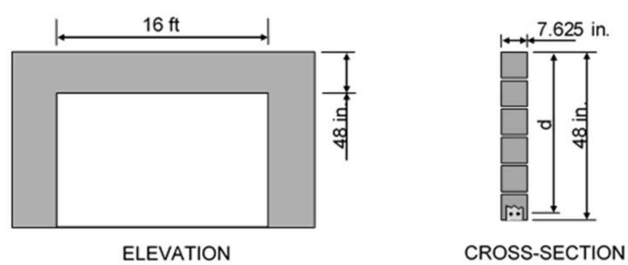
Beam Design

Design Example

Strength Design Guide Example 6.3.1.8

Given:

- superimposed dead load = 500 lb/ft
- live load = 450 lb/ft
- Grade 60 steel
- Type S PCL mortar
- 8 in. CMU
- f'_m = 2000 psi



Design Example: Load

Strength Design Guide Example 6.3.1.8

Span length: Assume not built integrally with support

Span length = 16 ft + 2(4 in.) = 16.67 ft

Beam Weight: Assume fully grouted, medium weight units; 81 psf

Weight = 81psf(4ft) = 324 lb/ft

Factored load, w_u

$$w_u = 1.2D + 1.6L = 1.2\left(500\frac{lb}{ft} + 324\frac{lb}{ft}\right) + 1.6\left(450\frac{lb}{ft}\right) = 1{,}709\frac{lb}{ft}$$

31

Design Example: Beam Depth

Strength Design Guide Example 6.3.1.8

Factored shear, V_u

$$V_u = \frac{1{,}709\frac{lb}{ft}(16.67ft)}{2} = 14{,}240lb$$

Minimum depth to avoid shear reinforcement

$$d_{min} = \frac{V_u}{1.8b\sqrt{f'_m}} = \frac{14{,}240lb}{1.8(7.625in.)\sqrt{2{,}000psi}} = 23.2in.$$

- 32 in. deep beam is needed
- Prescriptive seismic reinforcement would be needed in top course
- Rather than leave one course ungrouted, fully grout entire height of beam
- Shear strength OK by inspection

32

Design Example: Sharpen Pencil

Strength Design Guide Example 6.3.1.8

Try 24 in. deep beam, d = 20 in. Check V_u at $d/2$ from face of support

$d/2$ from face of support

$$(4in.+20in.)\frac{1ft}{12in.} = 2.0ft$$

Factored shear, V_u

$$V_u = 14{,}240lb\frac{8.33ft \quad .0ft}{8.33ft} = 14{,}240lb(0.76) = 10{,}820lb$$

$$d_{min} = \frac{V_u}{1.8b\sqrt{f'_m}} = \frac{10{,}820lb}{1.8(7.625in.)\sqrt{2{,}000psi}} = 17.6in.$$

- A 24 in. deep beam will work
- In this example, most engineers would still fully grout the beam, rather than have two ungrouted courses.

33

Design Example: Flexure

Strength Design Guide Example 6.3.1.8

Factored moment, M_u

$$M_u = \frac{1{,}709\frac{lb}{ft}(16.67ft)^2}{8}\frac{12in.}{ft} = 712{,}000in.\cdot lb$$

Depth of stress block, a

$$a = d - \sqrt{d^2 - \frac{2M_u}{0.8\phi f'_m b}}$$

$$= 44in. - \sqrt{44in.^2 - \frac{2(712{,}000in.\cdot lb)}{0.8(0.9)(2{,}000psi)(7.625in.)}} = 1.50in.$$

Required steel, $A_{s,reqd}$

$$A_{s,reqd} = \frac{0.8f'_m ba}{f_y} = \frac{0.8(2{,}000psi)(7.625in.)(1.50in.)}{60{,}000psi} = 0.305in.^2$$

Try 1 – No. 5

34

Design Example: Minimum Reinf.

Strength Design Guide Example 6.3.1.8

Modulus of rupture, Type S PCL mortar: 267 psi

Cracking moment, M_{cr}

$$M_{cr} = f_r S_n = f_r \frac{bh^2}{6} = 267psi \frac{7.625in.(48in.)^2}{6} = 782{,}000in.\cdot lb$$

Nominal moment, M_n

$$M_n = A_s f_y \left(d - \frac{1}{2}\frac{A_s f_y}{0.8 f'_m b}\right) =$$

$$(0.31in.^2)(60{,}000psi)\left(44in. - \frac{1}{2}\frac{(0.31in.^2)60{,}000psi}{0.8(2{,}000psi)(7.625in.)}\right) = 804{,}000in.\cdot lb$$

$M_n = 804in.\cdot kip < 1.3M_{cr} = 1.3(782)in.\cdot kip = 1{,}016\ in.\cdot kip$ No good

Try 2 – No. 4; $M_n = 1{,}032\ in\cdot kip$ OK

Design Example: Maximum Reinf.

Strength Design Guide Example 6.3.1.8

Reinf. Ratio, ρ

$$\rho = \frac{A_s}{bd} = \frac{2(0.20in.^2)}{7.625in.(44in.)} = 0.00119$$

0.00119 << 0.00952 OK

Design Example: Bracing

Strength Design Guide Example 6.3.1.8

$32b = 32(7.625in.) = 20.3ft$

$120b^2/d = 120\,(7.625in.)^2/44in. = 13.2ft$

- If top of beam were the roof, that would provide continuous lateral support
- It top of beam were a parapet, provide bracing at midspan and ends

37

Design Example: Deflections

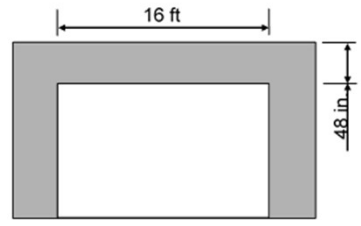
Strength Design Guide Example 6.3.1.8

- Beam is not supporting unreinforced masonry, so deflections do not need to be checked
- As a quick check: $l/d = 16.67ft\left(12\frac{in.}{ft}\right)/44in. = 4.5 \leq 8$ OK
- Deflection check will be illustrated in next example

38

Design Example

Strength Design Guide Example 6.3.1.9



Given: Modify load

- superimposed dead load = 2,000 lb/ft
- live load = 700 lb/ft

Factored load, w_u

$$w_u = 1.2D + 1.6L = 1.2\left(2{,}000\frac{lb}{ft} + 324\frac{lb}{ft}\right) + 1.6\left(700\frac{lb}{ft}\right) = 3{,}910\frac{lb}{ft}$$

Flexural reinforcement is 4 – No. 4

2 bars in each of bottom two courses

39

Design Example: Shear

Strength Design Guide Example 6.3.1.9

Factored shear, V_u

$$V_u = \frac{3{,}910\frac{lb}{ft}(16.67ft)}{2} = 32{,}600lb$$

Shear area, A_{nv}

$$A_{nv} = 7.625in.(40in.) = 305in.^2$$

Check max V_n

$$\phi(V_n)_{max} = 4A_{nv}\sqrt{f'_m} = 0.8(4)(305in.^2)\sqrt{2000psi} = 43{,}600lb$$

OK

Nominal strength masonry, V_{nm}

$$V_{nm} = 2.25A_{nv}\sqrt{f'_m} = 2.25(305in.^2)\sqrt{2{,}000psi} = 30{,}700lb$$

Req'd strength reinf., V_{ns}

$$V_{ns,reqd} = \frac{V_u}{\phi} - V_{nm} = \frac{32{,}600lb}{0.8} - 30{,}700 = 10{,}100lb$$

40

Design Example: Shear

Strength Design Guide Example 6.3.1.9

Try No. 3 bars for stirrups (deformed wire could also be used).

Spacing, s $$V_{ns} = 0.5\left(\frac{A_v}{s}\right) f_y d_v \;\Rightarrow\; s = \frac{0.5 A_v f_y d_v}{V_{ns}} = \frac{0.5(0.11in.^2)(60{,}000psi)\,(48in.)}{10{,}100lb} = 15.7in.$$

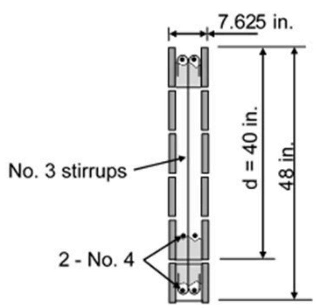
Try No. 3 at 8 in.

Detailing requirements

- $A_v/s = 0.11in.^2/8in. = 0.0138in. \geq 0.0007b = 0.0007(7.625in.) = 0.0053in.$
- First bar located not more than $d_v/4 = 48in./4 = 12in.$
- Maximum spacing of $\min\{d_v/2\,, 48in.\} = \min\{48in./2{,}48in.\} = 24in.$

Design Example: Shear Details

Strength Design Guide Example 6.3.1.9



One stirrup per cell; alternate side every other cell

Parameter	Width
Face shell; 1.25 in. each	2.50 in.
Block taper; assume to be 0.25 in. each side	0.50 in.
Thickness of grout between reinforcement and masonry; 0.50 in. (TMS 402 6.1.3.5)	1.00 in.
Stirrup diameter: 2 at 0.375 in.	0.75 in.
Longitudinal reinforcement diameter; 2 at 0.50 in.	1.00 in.
Space between bars; 1.00 in. (TMS 402 6.1.3.1)	1.00 in.
TOTAL	6.75 in.

≤ 7.625 in. OK

Design Example: Deflections

Strength Design Guide Example 6.3.1.9; Illustration

Modulus ratio $n = \frac{E_s}{E_m} = \frac{29000ksi}{900\ {}'_m} = \frac{29000ksi}{900(2ksi)} = 16.11$

Reinforcement ratio $\rho = \frac{A_s}{bd} = \frac{4(0.20in.^2)}{7.625in.(40in.)} = 0.002623$ $\quad n\rho = 0.04225$

Find k

$$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho$$
$$= \sqrt{(0.04225)^2 + 2(0.04225)} - 0.04225 = 0.252$$

$kd = 0.252(40in.) = 10.06in.$

Net moment of inertia, I_n $\quad I_n = \frac{bd_v^3}{12} = \frac{7.625in.(48in.)^3}{12} = 70{,}270in.^4$

Cracked moment of inertia, I_{cr}

$$I_{cr} = \frac{bk^3d^3}{3} + nA_s(d - kd)^2$$
$$= \frac{7.625in.(10.06in.)^3}{3} + 16.11(0.80in.^2)(40in. - 10.06in.)^2 = 14{,}140in.^4$$

Design Example: Deflections

Strength Design Guide Example 6.3.1.8

ASD load $\quad w = D + L = \left(2{,}000\frac{lb}{ft} + 324\frac{lb}{ft}\right) + 700\frac{lb}{ft} = 3{,}020\frac{lb}{ft}$

ASD Moment, M_a $\quad M_a = \frac{wL^2}{8} = \frac{3{,}020\frac{lb}{ft}(16.67ft)^2}{8}\frac{12in.}{ft} = 1{,}259{,}000in.\cdot lb$

Cracking Moment, M_{cr} $\quad M_{cr} = 782{,}000in.\cdot lb$ (from minimum reinforcement calc)

Effective Moment of Inertia, I_{eff}

$$I_{eff} = I_n\left(\frac{M_{cr}}{M_a}\right)^3 + I_{cr}\left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right]$$
$$70{,}270in.^4\left(\frac{782in.\cdot k}{1{,}259in.\cdot k}\right)^3 + 14{,}140in.^4\left[1 - \left(\frac{782in.\cdot k}{1{,}259in.\cdot k}\right)^3\right] = 27{,}590in.^4$$

Design Example: Deflections

Strength Design Guide Example 6.3.1.8

Deflection, δ $\delta = \frac{5wL^4}{384EI} = \frac{5\left(3{,}020\frac{lb}{ft}\right)(16.67ft)^4}{384(1{,}800{,}000psi)(27{,}590in.^4)} \frac{1728in.^3}{1ft^3} = 0.106in.$

Allowable δ $\frac{L}{600} = \frac{16.67ft}{600}\frac{12in.}{ft} = 0.333in.$ OK

Quick check using cracked moment of inertia, δ = 0.206 in.

Deep Beams

- Internal Lever Arm
- Miscellaneous Requirements
- Example

Internal Lever Arm

Strength Design Guide 6.3.3.6; TMS 402 Section 5.2.2.1, 5.2.2.2

Definition (TMS 402 2.2)

$$\frac{l_{eff}}{d_v} \leq \begin{cases} 3 & \text{continuous span} \\ 2 & \text{simple span} \end{cases}$$

Effective span length, l_{eff}, smaller of:

- center-to-center distance between supports
- 1.15 multiplied by the clear span

z – internal lever arm	
Simple spans	Continuous spans
$1 \leq \frac{l_{eff}}{d_v} < 2 \quad z = 0.2(l_{eff} + 2d_v)$	$1 \leq \frac{l_{eff}}{d_v} < 3 \quad z = 0.2(l_{eff} + 1.5d_v)$
$\frac{l_{eff}}{d_v} < 1 \quad z = 0.6l_{eff}$	$\frac{l_{eff}}{d_v} < 1 \quad z = 0.5l_{eff}$

Miscellaneous Requirements

Strength Design Guide 6.3.3.6; TMS 402 Section 5.2.2.3, 5.2.2.4, 5.2.2.5

- Flexural reinforcement
  - distributed flexural reinforcement for half beam depth
  - maximum spacing of one-fifth d_v or 16 in.
  - joint reinforcement can be used as flexural reinforcement
  - horizontal reinforcement anchored to develop yield strength at face of supports
- Shear reinforcement (when required)
  - minimum area of vertical reinforcement is $0.0007bd_v$
  - horizontal shear reinforcement area ≥ half vertical shear reinforcement
  - maximum spacing of shear reinforcement one-fifth d_v or 16 in.
- Total reinforcement: sum of horizontal and vertical reinforcement at least $0.001bd_v$.

Design Example

Strength Design Guide Example 6.3.1.7

Given:

- 10 ft opening
- 6 ft deep beam
- superimposed dead load = 3.0 kip/ft
- live load = 2.0 kip/ft
- Grade 60 steel
- Type S masonry cement mortar
- 8 in. CMU
- f'_m = 2000 psi

C/C between supports = 10 ft + 2(4 in.) = 10.67 ft

1.15(clear span) = 1.15(10 ft) = 11.5 ft

Effective span length, l_{eff} = min(10.67, 11.5) = 10.67 ft

Span ratio, l_{eff}/d_v $\quad \frac{l_{eff}}{d_v} = \frac{10.67ft}{6ft} = 1.78 \leq 2$

Therefore a deep beam

Design Example: Flexure

Strength Design Guide Example 6.3.1.7

Beam Weight: Assume fully grouted, medium weight units; 81 psf

Weight = 81psf(6ft) = 0.486 k/ft

Factored load, w_u $\quad w_u = 1.2D + 1.6L = 1.2\left(3.0\frac{k}{ft} + 0.486\frac{k}{ft}\right) + 1.6\left(2.0\frac{k}{ft}\right) = 7.38\frac{k}{ft}$

Factored moment, M_u $\quad M_u = \frac{7.38\frac{k}{ft}(10.67ft)^2}{8} = 105.0k \cdot ft$

Internal lever arm, z $\quad z = 0.2(l_{eff} + 2d_v) = 0.2(10.67ft + 2(6ft)) = 4.53ft$

Design Example: Flexure

Strength Design Guide Example 6.3.1.7

Req'd A_s $$A_{s,reqd} = \frac{M_u/\phi}{zf_y} = \frac{105.0k \cdot ft/0.9}{4.53ft(60ksi)} = 0.429in.^2$$

Using standard beam theory would have $A_{s,reqd}$ underestimated by 14%.

Although 1-#6 could be used, use 2-#5, one in each of bottom two courses

- reduces development length and extension of bars beyond face of support
- helps with requirement of distributed reinforcement

Design Example: Reinforcement

Strength Design Guide Example 6.3.1.7

ρ_{max}

$$d = 72in. - 8in. = 64in.$$ distance to centroid of reinforcement

$$\rho = \frac{A_s}{bd} = \frac{2(0.31in.^2)}{7.625in.(64in.)} = 0.00127 \leq 0.00952$$

ρ_{min} $$A_s = 0.62in.^2 \geq \frac{4}{3}A_{s,reqd} = \frac{4}{3}(0.43in.^2) = 0.57in.^2$$

Distributed reinforcement: required over bottom half of beam at a spacing of 1/5d_v = 1/5(72in.) = 14.4in., but not greater than 16 in.

Use W1.7 (9 gage) joint reinforcement every 8 in. in bottom five bed joints.

Design Example: Development

Strength Design Guide Example 6.3.1.7

Development length, l_{de}

$$l_{de} = \frac{0.13 d_b^2 f_y \gamma}{K\sqrt{f'_m}} = \frac{0.13(0.625in.)^2(60{,}000psi)(1.0)}{\min\{9(0.625in.), 3.81in. - 0.625in./2\}\sqrt{2{,}000psi}} = 19.5in.$$

- Extend bars 20 in. beyond face of support
- Details of development length will be covered in Session 6

Design Example: Shear

Strength Design Guide Example 6.3.1.7

Factored shear, V_u

$$V_u = \frac{7.38\frac{k}{ft}(10.67ft)}{2} = 39.4kip$$

Design shear strength from masonry, ϕV_{nm}

$$\phi V_{nm} = \phi 2.25 A_{nv}\sqrt{f'_m}$$
$$= 0.8(2.25)(7.625in.)(68in.)\sqrt{2000psi} = 41.7k$$

Since $\phi V_{nm} \geq V_u$ no shear reinforcement required

Design Example: Total Reinforcement

Strength Design Guide Example 6.3.1.7

Total Reinforcement $0.001bd_v = 0.001(7.625in.)(72\ in.) = 0.55in.^2$

2 - #5 meets total reinforcement by inspection.

55

This concludes The American Institute of Architects Continuing Education Systems Course

The Masonry Society

Dr. Richard Bennett
rmbennett@utk.edu