

***Reinforced Concrete Design 9th edition Aghayere
Solutions Manual***

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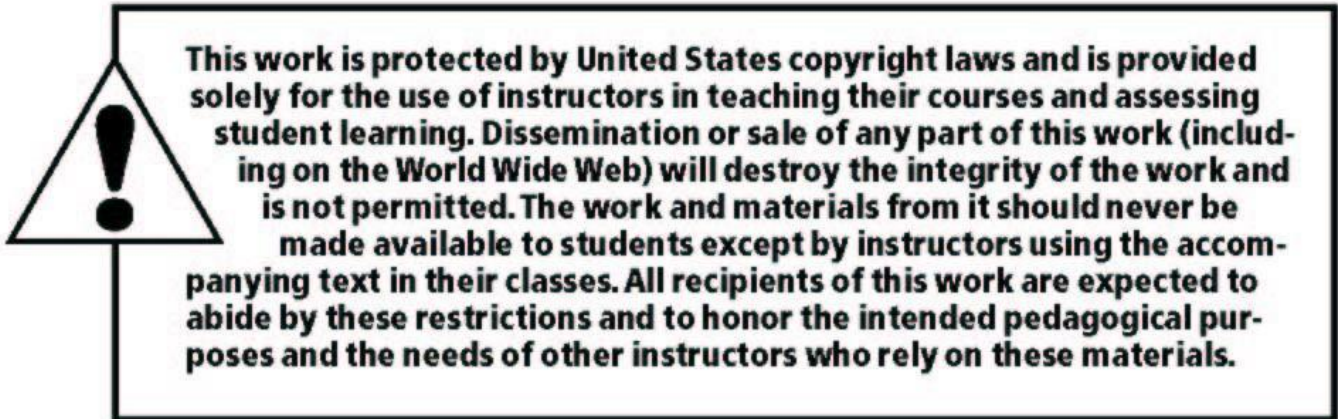
Instructor's Solutions Manual
To accompany

Reinforced Concrete Design

Ninth Edition

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NOTES:

This manual is intended solely as an aid for teachers and educators in their individual course preparation.

The solutions presented herein are, in general, somewhat abbreviated. The solutions follow, as closely as possible, the procedures developed in the examples in the text. They are satisfactory solutions within the scope of the text and are based on the limited tables and design aids furnished in the text.

The solutions for the design problems are generally not the only solutions, nor are they necessarily the most economical solutions.

Prob. 1-1

$$(a) \frac{16(28)}{144}(150) = 467 \text{ lb/ft}$$

$$(b) \frac{12(26-6)}{144}(150) + \frac{6(38)}{144}(150) = 488 \text{ lb/ft}$$

Prob. 1-2

Spreadsheet problem: $E_c = w_c^{1.5} 33 \sqrt{f'_c}$ Check value for $w_c = 145 \text{ lb/ft}^3$ and $f'_c = 4000 \text{ psi}$:
 $E_c = 3,644,000 \text{ psi}$

Prob. 1-3 $L = 24 \text{ in.}$ with 2100 lb load at midspan.

$$\text{Beam weight} = \frac{6(6)}{144}(0.145) = 0.036 \text{ kip/ft} \quad I = \frac{1}{12}(6)^4 = 108 \text{ in.}^4$$

$$M = \frac{0.036(2)^2}{8} + \frac{2.1(2)}{4} = 1.068 \text{ ft} \cdot \text{kips}$$

$$f = \frac{Mc}{I} = f_r = \frac{1.068(12)(3)}{108} = 0.356 \text{ ksi}$$

By ACI formula:

$$f_r = 7.5 \sqrt{f'_c} = 7.5 \sqrt{3000} = 411 \text{ psi}$$

Prob. 1-4 Simply supported beam of length L .

$$\text{Beam weight} = \frac{10(10)}{144} 145 = 100.7 \text{ lb/ft}; \quad f_r = 350 \text{ psi}; \quad I = \frac{10(10)^3}{12} = 833 \text{ in.}^4$$

$$M = \frac{100.7L^2}{8} = 12.59L^2$$

$$f = \frac{Mc}{I} = f_r = \frac{12.59(12)(5)L^2}{833} = 350$$

$$L = 19.65 \text{ ft}$$

Prob. 1-5

$$M = \frac{0.5(10)^2}{8} + \frac{2(10)}{4} = 11.25 \text{ ft} \cdot \text{kips}$$

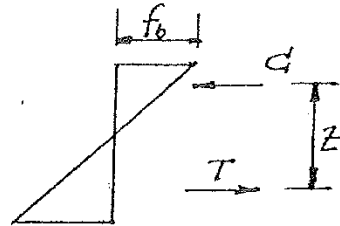
$$(a) C = \frac{f_b}{2}(8)(8) = 32f_b \text{ in.}^2$$

$$M = CZ$$

$$11.25 \text{ ft} \cdot \text{kips} = 32f_b (\text{in.}^2) \left(\frac{2}{3} \right) (16 \text{ in.})$$

$$f_b = \frac{11.25 \text{ ft} \cdot \text{kips} (12 \text{ in./ft})}{32 \text{ in.}^2 \left(\frac{2}{3} \right) (16 \text{ in.})} = 0.396 \text{ ksi}$$

$$(b) S_x = \frac{bh^2}{6} = \frac{8(16)^2}{6} = 341 \text{ in.}^3; \quad f_b = \frac{M}{S_x} = \frac{11.25(12)}{341} = 0.396 \text{ ksi} \quad (\text{O.K.})$$



Prob. 1-6

$$f_r = 7.5\sqrt{3000} = 411 \text{ psi} = 0.411 \text{ ksi}$$

$$(a) \text{ I.C. method: } Z = 16 - 2(2.67) = 10.67 \text{ in.}$$

$$C = T = 0.5(0.411)(8)(10) = 16.44 \text{ kips}$$

$$M_{cr} = CT = TZ = \frac{16.44(10.67)}{12} = 14.62 \text{ ft} \cdot \text{kips}$$

$$(b) \text{ Flexure formula check:}$$

$$S_x = \frac{10(16)^2}{6} = 427 \text{ in.}^3$$

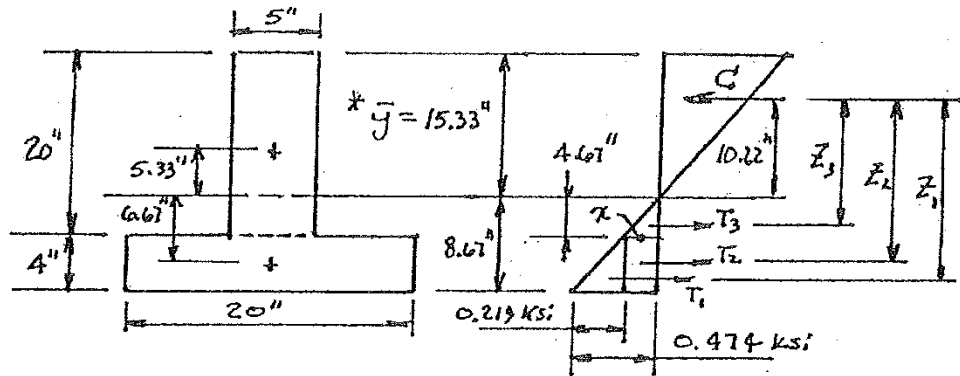
$$M_{cr} = f_r S_x = 0.411(427) = 175.5 \text{ in} \cdot \text{kips} = 14.62 \text{ ft} \cdot \text{kips} \quad (\text{O.K.})$$

Prob. 1-8

(a) $f'_c = 4000$ psi

$$f_r = \frac{7.5\sqrt{4000}}{1000} = 0.474 \text{ ksi}$$

$$x = 0.474 \left(\frac{4.67}{8.67} \right) = 0.255 \text{ ksi}$$



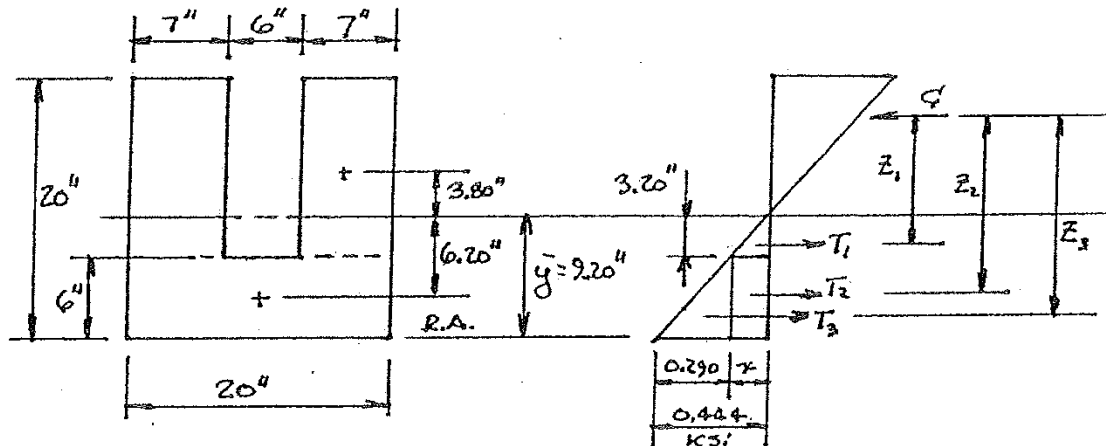
Force	Magnitude (kips)	Moment arm (in.)	I.C. (in.-kips)
T_1	$0.5(0.219)(20)(4)=8.76$	$10.22+(2/3)(4.67)=17.56$	153.8
T_2	$0.255(20)(4)=20.4$	$10.22+4.67+2=16.89$	344.6
T_3	$0.5(0.255)(4.67)=2.89$	$10.22+4.67+(2/3)(4)=12.33$	39.7
Total M_{cr}			538 in-kips

$$(b) I = \frac{20(4)^3}{12} + 20(4)(6.67)^2 + \frac{5(20)^3}{12} + 5(20)(5.33)^2 = 9840 \text{ in.}^2$$

$c = 8.67$ in. (to tension side.)

$$M_{cr} = \frac{0.474(9840)}{8.67} = 538 \text{ in-kips (O.K.)}$$

Prob. 1-9



$$f'_c = 3500 \text{ psi}; \quad f_r = 7.5\sqrt{3500} = 444 \text{ psi} = 0.444 \text{ ksi}$$

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{20(6)(3) + 2(7)(14)(13)}{20(6) + 2(7)(14)} = 9.20 \text{ in.}; \quad x = 0.444 \left(\frac{3.20}{9.20} \right) = 0.1544 \text{ ksi}$$

(a)

Force	Magnitude (kips)	Moment arm (in.)	I.C. (in.-kips)
T_1	$2(0.5)(0.1544)(7)(3.20)=3.46$	$7.20+(2/3)(3.20)=9.33$	32.3
T_2	$0.1544(20)(6)=18.53$	$7.20+3.20+3=13.40$	248.3
T_3	$0.5(0.290)(20)(6)=17.40$	$7.20+3.20+(2/3)(6)=14.40$	250.6
Total $M_{cr} =$			531 in-kips

$$(b) I = 2 \left(\frac{7(14)^3}{12} \right) + 2(7)(14)(3.80)^2 + \frac{20(6)^3}{12} + 6(20)(6.20)^2 = 11,004 \text{ in.}^4$$

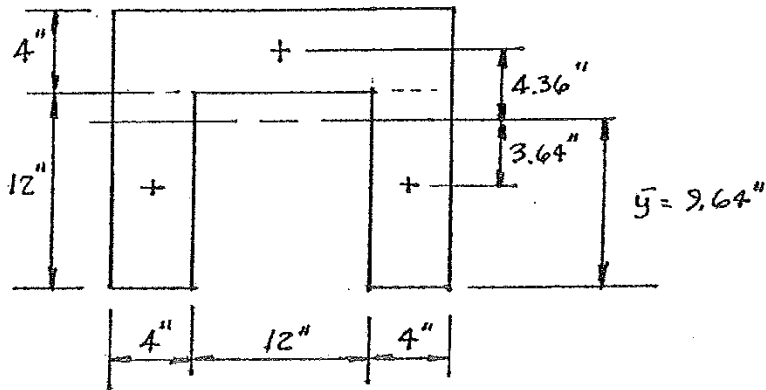
$$M_{cr} = \frac{f_r I}{c} = \frac{0.444(11,004)}{9.20} = 531 \text{ in-kips (O.K.)}$$

Prob. 1-10

$$f'_c = 3000 \text{ psi}$$

$$f_r = \frac{7.5\sqrt{3000}}{1000} = 0.411 \text{ ksi}$$

$$\begin{aligned} \bar{y} &= \frac{\sum Ay}{\sum A} \\ &= \frac{2(4)(12)(6) + 4(20)(14)}{2(4)(12) + 4(20)} \\ &= 9.64 \text{ in.} \end{aligned}$$



$$I = 2 \left(\frac{4(12)^3}{12} \right) + 2(4)(12)(3.64)^2 + \frac{20(4)^3}{12} + 4(20)(4.36)^2 = 4051 \text{ in.}^4$$

$$(a) M_{cr} = \frac{f_r I}{c} = \frac{0.411(4051)}{9.64} = 172.7 \text{ in.-kips}$$

$$(b) \text{ Beam weight} = \frac{4(20) + 2(4)(12)}{144} (0.145) = 0.1772 \text{ kip/ft}$$

$$\text{Beam weight moment} = \frac{0.1772(12)^2}{8} = 3.19 \text{ ft-kips} = 38.3 \text{ in.-kips}$$

$$\frac{PL}{4} = M_{cr} - 38.3 = 172.7 - 38.3 = 134.4 \text{ in.-kips}; \quad P = \frac{4(134.4 \text{ in.-k})}{12 \text{ ft} (12 \text{ in./ft})} = 3.73 \text{ kips}$$

Prob 1-11

Part (a): Dead Loads

Slab weight		
(8"/12)(150pcf)	=	100psf
½" light wt. floor finish	=	4psf
susp. ceiling	=	2psf
M/E (industrial)	=	20psf
Partitions	=	20psf

$$\Sigma_{DL} = 146\text{psf (with partitions)}$$

$$\Sigma_{DL} = 126\text{psf (without partitions)}$$

Part (b): Live Load = 100psf

$$1.4D = (1.4)(126) = 176.4\text{psf}$$

$$1.2D + 1.6L = (1.2)(126) + (1.6)(100) = \mathbf{311\text{psf}} \leftarrow \text{controls}$$

Part (c): Load on spandrel beam

$$\text{Trib. width} = (18 \text{ ft}/2) + 1.0 \text{ ft} = 10 \text{ ft}$$

Cladding:

$$\text{Block: } (55\text{psf})(10 \text{ ft.} - 2'-2'') = 431 \text{ plf}$$

$$\text{Brick: } (40\text{psf})(10 \text{ ft.}) = 400 \text{ plf}$$

Beam Stem:

$$(8''/12)(12''/12)(150\text{pcf}) = 225 \text{ plf}$$

Service Dead Load:

$$(126\text{psf})(10 \text{ ft}) + 431\text{plf} + 400 + 225 \text{ plf} = 2316 \text{ plf} = \mathbf{2.32 \text{ k/ft}}$$

Service Live Load:

$$(100\text{psf})(10 \text{ ft}) = 1000 \text{ plf} = \mathbf{1.0 \text{ k/ft}}$$

Part (d): Factored Load on spandrel beam:

$$1.2D + 1.6L = (1.2)(2.32) + (1.6)(1.0) = \mathbf{4.4\text{plf}}$$

Part (e): Load on interior beam

$$\text{Trib. width} = (18 \text{ ft}/2) + (18 \text{ ft}/2) + 1.0 \text{ ft} = 19 \text{ ft}$$

Beam Stem:

$$(8''/12)(12''/12)(150\text{pcf}) = 225 \text{ plf}$$

Service Dead Load:

$$(126\text{psf})(19 \text{ ft}) + 225 \text{ plf} = 2619 \text{ plf} = \mathbf{2.62 \text{ k/ft}}$$

Service Live Load:

$$(100\text{psf})(19 \text{ ft}) = 1900 \text{ plf} = \mathbf{1.9 \text{ k/ft}}$$

Part (f): Factored Load on interior beam:

$$1.2D + 1.6L = (1.2)(2.62) + (1.6)(1.9) = \mathbf{6.2\text{k/ft}}$$

Part (g):

Spandrel Beam:

$$V_u = \frac{w_u L}{2} = \frac{(4.4)(36)}{2} = \mathbf{79.2 \text{ k}}$$

$$M_u = \frac{w_u L^2}{8} = \frac{(4.4)(36)^2}{8} = \mathbf{713 \text{ k-ft}}$$

Prob 1-12

Roof Loads

Dead Load:

7" slab (7"/12 x 150 pcf)	= 87.5 psf
5 ply + gravel	= 6.5 psf
Insulation	= 3.5 psf
M & E	= 10 psf
<u>Suspended ceiling</u>	<u>= 2 psf</u>

Total roof dead load, D	= 109.5 psf
Roof live load (snow) S	= 35 psf

Typical Floor Loads

Dead Load:

7" slab (7"/12 x 150 pcf)	= 87.5 psf
0.5" light wt floor finish	= 4 psf
Partitions	= 20 psf
M & E	= 10 psf
<u>Suspended ceiling</u>	<u>= 2 psf</u>

Total roof dead load, D	= 123.5 psf
Floor live load (office) L	= 50 psf

A.

Roof: $w_{DL \text{ slab}} = 109.5 \text{ psf};$ $w_{LL \text{ slab}} = 35 \text{ psf}$

Using the load combinations, we find the maximum factored load to be,

$$w_{u \text{ slab}} = 1.2 (109.5 \text{ psf}) + 1.6 (35 \text{ psf}) = 188 \text{ psf}$$

Floor: $w_{DL \text{ slab}} = 123.5 \text{ psf};$ $w_{LL \text{ slab}} = 50 \text{ psf}$

Using the load combinations, we find the maximum factored load to be,

$$w_{u \text{ slab}} = 1.2 (123.5 \text{ psf}) + 1.6 (50 \text{ psf}) = 229 \text{ psf}$$

B.

(i) TYPICAL INTERIOR FLOOR BEAM

Tributary width of beam = 16 ft

$$\text{Beam stem weight} = (16"/12) [(26" - 7")/12] \times 150 \text{ pcf} = 317 \text{ lb/ft} = 0.32 \text{ kip/ft}$$

$$w_{DL \text{ beam}} = 123.5 \text{ psf} \times 16 \text{ ft} + 317 \text{ lb/ft} = 2293 \text{ lb/ft} = 2.3 \text{ kip/ft}$$

$$w_{LL \text{ beam}} = 50 \text{ psf} \times 16 \text{ ft} = 800 \text{ lb/ft} = 0.8 \text{ kip/ft}$$

$$w_{s \text{ beam}} = w_{DL \text{ beam}} + w_{LL \text{ beam}} = 3.1 \text{ kip/ft}$$

$$\begin{aligned} w_{u \text{ beam}} &= w_{u \text{ slab}} \times \text{beam trib width} + 1.2 \times \text{beam stem weight} \\ &= 229 \text{ psf} \times 16 \text{ ft} + 1.2 (317 \text{ lb/ft}) = 4045 \text{ lb/ft} = 4.1 \text{ kip/ft} \end{aligned}$$

For a beam subjected to a uniformly distributed load, the beam reactions, assuming simple supports, are:

$$R_{DL} = w_{DL \text{ beam}} \times L/2 = 2.3 \text{ kip/ft} \times 32 \text{ ft} / 2 = 37 \text{ kip}$$

$$R_{LL} = w_{LL \text{ beam}} \times L/2 = 0.8 \text{ kip/ft} \times 32 \text{ ft} / 2 = 12.8 \text{ kip}$$

$$R_u = w_{u \text{ beam}} \times L/2 \quad \text{or} \quad 1.2 (R_{DL}) + 1.6 (R_{LL})$$

$$= 4.1 \text{ kip/ft} \times 32 \text{ ft} / 2 = 66 \text{ kip} \quad \text{or} \quad 1.2 (37 \text{ kip}) + 1.6 (12.8 \text{ kip}) = 66 \text{ kip (same!)}$$

(ii) TYPICAL INTERIOR ROOF BEAM

Tributary width of beam = 16 ft

Beam stem weight = $(16''/12) [(26'' - 7'')/12] \times 150 \text{ pcf} = 317 \text{ lb/ft} = 0.32 \text{ kip/ft}$

$$w_{DL\text{beam}} = 109.5 \text{ psf} \times 16 \text{ ft} + 317 \text{ lb/ft} = 2070 \text{ lb/ft} = 2.1 \text{ kip/ft}$$

$$w_{LL\text{beam}} = 35 \text{ psf} \times 16 \text{ ft} = 560 \text{ lb/ft} = 0.56 \text{ kip/ft}$$

$$w_{s \text{ beam}} = w_{DL\text{beam}} + w_{LL\text{beam}} = \mathbf{2.7 \text{ kip/ft}}$$

$$\begin{aligned} w_{u \text{ beam}} &= w_{u \text{ slab}} \times \text{beam trib width} + 1.2 \times \text{beam stem weight} \\ &= 188 \text{ psf} \times 16 \text{ ft} + 1.2 (317 \text{ lb/ft}) = 3389 \text{ lb/ft} = \mathbf{3.4 \text{ kip/ft}} \end{aligned}$$

For a beam subjected to a uniformly distributed load, the beam reactions, assuming simple supports, are:

$$R_{DL} = w_{DL\text{beam}} \times L/2 = 2.1 \text{ kip/ft} \times 32 \text{ ft} / 2 = 34 \text{ kip}$$

$$R_{LL} = w_{LL\text{beam}} \times L/2 = 0.56 \text{ kip/ft} \times 32 \text{ ft} / 2 = 9 \text{ kip}$$

$$R_u = w_{u \text{ beam}} \times L/2 \quad \text{or } 1.2 (R_{DL}) + 1.6 (R_{LL})$$

$$= 3.4 \text{ kip/ft} \times 32 \text{ ft} / 2 = \mathbf{55 \text{ kip}} \quad \text{or } 1.2 (34 \text{ kip}) + 1.6 (9 \text{ kip}) = \mathbf{55 \text{ kip (same!)}}$$

(iii) TYPICAL FLOOR SPANDREL BEAM

Tributary width of beam = $(16 \text{ ft} / 2) + [(16''/12)/2] = 8.67 \text{ ft}$

Beam stem weight = $(16''/12) [(26'' - 7'')/12] \times 150 \text{ pcf} = 317 \text{ lb/ft} = 0.32 \text{ kip/ft}$

Exterior Cladding load = $55 \text{ psf block} \times 7.83' \text{ height} + 40 \text{ psf brick} \times 10' \text{ height} = 0.83 \text{ kip/ft}$

$$w_{DL\text{beam}} = 123.5 \text{ psf} \times 8.67 \text{ ft} + 317 \text{ lb/ft} + 830 \text{ lb/ft} = 2218 \text{ lb/ft} = 2.2 \text{ kip/ft}$$

$$w_{LL\text{beam}} = 50 \text{ psf} \times 8.67 \text{ ft} = 434 \text{ lb/ft} = 0.44 \text{ kip/ft}$$

$$w_{s \text{ beam}} = w_{DL\text{beam}} + w_{LL\text{beam}} = \mathbf{2.7 \text{ kip/ft}}$$

$$\begin{aligned} w_{u \text{ beam}} &= w_{u \text{ slab}} \times \text{beam trib width} + 1.2 \times \text{beam stem weight} \\ &= 229 \text{ psf} \times 8.67 \text{ ft} + 1.2 (317 \text{ lb/ft} + 830 \text{ lb/ft}) = 3362 \text{ lb/ft} = \mathbf{3.4 \text{ kip/ft}} \end{aligned}$$

For a beam subjected to a uniformly distributed load, the beam reactions, assuming simple supports, are:

$$R_{DL} = w_{DL\text{beam}} \times L/2 = 2.2 \text{ kip/ft} \times 32 \text{ ft} / 2 = 36 \text{ kip}$$

$$R_{LL} = w_{LL\text{beam}} \times L/2 = 0.44 \text{ kip/ft} \times 32 \text{ ft} / 2 = 7 \text{ kip}$$

$$R_u = w_{u \text{ beam}} \times L/2 \quad \text{or } 1.2 (R_{DL}) + 1.6 (R_{LL})$$

$$= 3.4 \text{ kip/ft} \times 32 \text{ ft} / 2 = \mathbf{55 \text{ kip}} \quad \text{or } 1.2 (36 \text{ kip}) + 1.6 (7 \text{ kip}) = \mathbf{55 \text{ kip (same!)}}$$

(iv) TYPICAL ROOF SPANDREL BEAM

Tributary width of beam = $(16 \text{ ft} / 2) + [(16''/12)/2] = 8.67 \text{ ft}$

Beam stem weight = $(16''/12) [(26'' - 7'')/12] \times 150 \text{ pcf} = 317 \text{ lb/ft} = 0.32 \text{ kip/ft}$

*On the roof beam and girder, there is usually a one to two feet high parapet. However, if we assume **NO PARAPET** in this example, the exterior cladding load on the roof spandrel beam will be:*

Exterior Cladding load = $55 \text{ psf block} \times 0' \text{ height} + 40 \text{ psf brick} \times 0' \text{ height} = 0 \text{ kip/ft}$

$W_{DL\text{beam}} = 109.5 \text{ psf} \times 8.67 \text{ ft} + 317 \text{ lb/ft} + 0 \text{ lb/ft} = 1267 \text{ lb/ft} = 1.3 \text{ kip/ft}$

$W_{LL\text{beam}} = 35 \text{ psf} \times 8.67 \text{ ft} = 303 \text{ lb/ft} = 0.3 \text{ kip/ft}$

$W_s \text{ beam} = W_{DL\text{beam}} + W_{LL\text{beam}} = \mathbf{1.6 \text{ kip/ft}}$

$W_u \text{ beam} = W_u \text{ slab} \times \text{beam trib width} + 1.2 \times \text{beam stem weight}$
 $= 188 \text{ psf} \times 8.67 \text{ ft} + 1.2 (317 \text{ lb/ft} + 0 \text{ lb/ft}) = 2010 \text{ lb/ft} = \mathbf{2.0 \text{ kip/ft}}$

For a beam subjected to a uniformly distributed load, the beam reactions, assuming simple supports, are:

$R_{DL} = W_{DL\text{beam}} \times L/2 = 1.3 \text{ kip/ft} \times 32 \text{ ft} / 2 = 21 \text{ kip}$

$R_{LL} = W_{LL\text{beam}} \times L/2 = 0.3 \text{ kip/ft} \times 32 \text{ ft} / 2 = 5 \text{ kip}$

$R_u = W_u \text{ beam} \times L/2$ or $1.2 (R_{DL}) + 1.6 (R_{LL})$
 $= 2.0 \text{ kip/ft} \times 32 \text{ ft} / 2 = \mathbf{32 \text{ kip}}$ or $1.2 (21 \text{ kip}) + 1.6 (5 \text{ kip}) = \mathbf{33 \text{ kip (same!)}$

C.

(i) TYPICAL INTERIOR FLOOR GIRDER

Girder stem weight = $(18''/12) [(26'' - 7'')/12] \times 150 \text{ pcf} = 357 \text{ lb/ft} = 0.36 \text{ kip/ft}$

This girder has TWO interior beams framing into it at the MIDSPA, in addition to a uniformly distributed loading (UDL) due to the girder stem weight. Therefore, the total concentrated loads acting on the girder are:

$P_{DL \text{ girder}} = 2 \text{ beams} \times R_{DL \text{ beam}} = 2 \times 37 \text{ kip} = 74 \text{ kip (acts at midspan)}$

$P_{LL \text{ girder}} = 2 \times R_{LL \text{ beam}} = 2 \times 12.8 \text{ kip} = 26 \text{ kip (acts at midspan)}$

$P_s \text{ girder} = P_{DL \text{ girder}} + P_{LL \text{ girder}} = 74 + 26 = \mathbf{100 \text{ kip (acts at midspan)}}$

$P_u \text{ girder} = 1.2 P_{DL \text{ girder}} + 1.6 P_{LL \text{ girder}} = 1.2(74) + 1.6(26) = \mathbf{131 \text{ kip (acts at midspan)}}$

The uniformly distributed load (UDL) on this girder is due to the girder stem weight. Thus,

$$W_{DL\text{ girder}} = \text{girder stem weight} = 357 \text{ lb/ft}$$

$$W_{LL\text{ girder}} = 0$$

The total UDL on the girder are:

$$W_{S\text{ girder}} = W_{DL\text{ girder}} + W_{LL\text{ girder}} = 357 \text{ lb/ft}$$

$$\begin{aligned} W_{U\text{ girder}} &= 1.2 W_{DL\text{ girder}} + 1.6 W_{LL\text{ girder}} \\ &= 1.2 (357 \text{ lb/ft}) + 1.6 (0) = 430 \text{ lb/ft} = \mathbf{0.43 \text{ kip/ft}} \end{aligned}$$

The REACTIONS for a GIRDER subjected to a uniformly distributed load, w , in addition to concentrated loads at MIDSPAN (see plan) from the beam reactions, assuming simple supports, are:

$$R_{DL} = W_{DL\text{ girder}} \times L/2 + P_{DL\text{ girder}}/2 = (0.36 \text{ kip/ft} \times 32 \text{ ft}/2) + (74 \text{ kip}/2) = 43 \text{ kip}$$

$$R_{LL} = W_{LL\text{ girder}} \times L/2 + P_{LL\text{ girder}}/2 = (0 \text{ kip/ft} \times 32 \text{ ft}/2) + (26 \text{ kip}/2) = 13 \text{ kip}$$

$$R_U = W_{U\text{ girder}} \times L/2 + P_{U\text{ girder}}/2 = (0.43 \text{ kip/ft} \times 32 \text{ ft}/2) + (131 \text{ kip}/2) = \mathbf{73 \text{ kip}}$$

(ii) TYPICAL INTERIOR ROOF GIRDER

$$\text{Girder stem weight} = (18''/12) [(26'' - 7'')/12] \times 150 \text{ pcf} = 357 \text{ lb/ft} = 0.36 \text{ kip/ft}$$

This girder has TWO interior beams framing into it at the MIDSPAN, in addition to a uniformly distributed loading (UDL) due to the girder stem weight. Therefore, the total concentrated loads acting on the girder are:

$$P_{DL\text{ girder}} = 2 \text{ beams} \times R_{DL\text{ beam}} = 2 \times 34 \text{ kip} = 68 \text{ kip (acts at midspan)}$$

$$P_{LL\text{ girder}} = 2 \times R_{LL\text{ beam}} = 2 \times 9 \text{ kip} = 18 \text{ kip (acts at midspan)}$$

$$P_{S\text{ girder}} = P_{DL\text{ girder}} + P_{LL\text{ girder}} = 68 + 18 = \mathbf{86 \text{ kip (acts at midspan)}}$$

$$P_{U\text{ girder}} = 1.2 P_{DL\text{ girder}} + 1.6 P_{LL\text{ girder}} = 1.2(68) + 1.6(18) = \mathbf{111 \text{ kip (acts at midspan)}}$$

The uniformly distributed load (UDL) on this girder is due to the girder stem weight. Thus,

$$W_{DL\text{ girder}} = \text{girder stem weight} = 357 \text{ lb/ft}; \quad W_{LL\text{ girder}} = 0$$

The total UDL on the girder are:

$$W_{S\text{ girder}} = W_{DL\text{ girder}} + W_{LL\text{ girder}} = 357 \text{ lb/ft}$$

$$\begin{aligned} W_{U\text{ girder}} &= 1.2 W_{DL\text{ girder}} + 1.6 W_{LL\text{ girder}} \\ &= 1.2 (357 \text{ lb/ft}) + 1.6 (0) = 430 \text{ lb/ft} = \mathbf{0.43 \text{ kip/ft}} \end{aligned}$$

The REACTIONS for a GIRDER subjected to a uniformly distributed load, w , in addition to concentrated loads at MIDSPAN (see plan) from the beam reactions, assuming simple supports, are:

$$R_{DL} = w_{DL\text{girder}} \times L/2 + P_{DL\text{girder}} / 2 = (0.36 \text{ kip/ft} \times 32 \text{ ft} / 2) + (68 \text{ kip} / 2) = 40 \text{ kip}$$

$$R_{LL} = w_{LL\text{girder}} \times L/2 + P_{LL\text{girder}} / 2 = (0 \text{ kip/ft} \times 32 \text{ ft} / 2) + (18 \text{ kip} / 2) = 9 \text{ kip}$$

$$R_u = w_u \text{ girder} \times L/2 + P_{ugirder} / 2 = (0.43 \text{ kip/ft} \times 32 \text{ ft} / 2) + (111 \text{ kip} / 2) = \mathbf{63 \text{ kip}}$$

(iii) TYPICAL FLOOR SPANDREL GIRDER

Girder stem weight = $(18''/12) [(26'' - 7'')/12] \times 150 \text{ pcf} = 357 \text{ lb/ft} = 0.36 \text{ kip/ft}$

Exterior Cladding load = $55 \text{ psf block} \times 7.83' \text{ height} + 40 \text{ psf brick} \times 10' \text{ height} = 0.83 \text{ kip/ft}$

This girder has ONE interior beam framing into it at the MIDSPAN, in addition to a uniformly distributed loading (UDL) due to the girder stem weight, the girder edge distance, AND the exterior cladding. Therefore, the total concentrated loads acting on the girder are:

$$P_{DL \text{ girder}} = 1 \text{ beam} \times R_{DL \text{ beam}} = 1 \times 37 \text{ kip} = 37 \text{ kip (acts at midspan)}$$

$$P_{LL \text{ girder}} = 1 \times R_{LL \text{ beam}} = 1 \times 12.8 \text{ kip} = 13 \text{ kip (acts at midspan)}$$

$$P_{s \text{ girder}} = P_{DL \text{ girder}} + P_{LL \text{ girder}} = 37 + 13 = \mathbf{50 \text{ kip (acts at midspan)}}$$

$$P_u \text{ girder} = 1.2 P_{DL \text{ girder}} + 1.6 P_{LL \text{ girder}} = 1.2(37) + 1.6(13) = \mathbf{65 \text{ kip (acts at midspan)}}$$

Uniform floor load on Girder edge distance:

$$w_{DL \text{ edge distance}} = (9''/12) \times 123.5 \text{ psf} = 93 \text{ lb/ft}$$

$w_{LL \text{ edge distance}} = 0$ (Because the WHOLE girder edge distance is occupied by the block wall cladding and thus there can be no live load in that edge distance)

The uniformly distributed load (UDL) on this girder is due to the girder stem weight, the load on the girder edge distance, AND the exterior cladding. Thus,

$$\begin{aligned} w_{DL\text{girder}} &= \text{girder stem weight} + \text{girder edge distance} + \text{exterior cladding} \\ &= 357 \text{ lb/ft} + 93 + 830 \text{ lb/ft} = 1280 \text{ lb/ft}; \quad w_{LL \text{ girder}} = 0 \end{aligned}$$

The total uniformly distributed load (UDL) on the girder is:

$$w_{s \text{ girder}} = w_{DL\text{girder}} + w_{LL\text{girder}} = 1280 \text{ lb/ft}$$

$$\begin{aligned} w_u \text{ girder} &= 1.2 w_{DL\text{girder}} + 1.6 w_{LL\text{girder}} \\ &= 1.2 (1280 \text{ lb/ft}) + 1.6 (0) = 1536 \text{ lb/ft} = \mathbf{1.54 \text{ kip/ft}} \end{aligned}$$

The REACTIONS for a GIRDER subjected to a uniformly distributed load, w , in addition to concentrated loads at MIDSPAN (see plan) from the beam reactions, assuming simple supports, are:

$$R_{DL} = w_{DL\text{girder}} \times L/2 + P_{DL\text{girder}} / 2 = (1.28 \text{ kip/ft} \times 32 \text{ ft} / 2) + (37 \text{ kip} / 2) = 39 \text{ kip}$$

$$R_{LL} = w_{LL\text{girder}} \times L/2 + P_{LL\text{girder}} / 2 = (0 \text{ kip/ft} \times 32 \text{ ft} / 2) + (13 \text{ kip} / 2) = 6.5 \text{ kip}$$

$$R_u = w_u \text{ girder} \times L/2 + P_{ugirder} / 2 = (1.54 \text{ kip/ft} \times 32 \text{ ft} / 2) + (65 \text{ kip} / 2) = \mathbf{57 \text{ kip}}$$

(iv) **TYPICAL ROOF SPANDREL GIRDER**

Girder stem weight = $(18''/12) [(26'' - 7'')/12] \times 150 \text{ pcf} = 357 \text{ lb/ft} = 0.36 \text{ kip/ft}$ (no parapet)

This girder has ONE interior beam framing into it at the MIDSPAN, in addition to a uniformly distributed loading (UDL) due to the girder stem weight, the load on the girder edge distance, AND the exterior cladding. Therefore, the total concentrated loads acting on the girder are:

$$P_{DL \text{ girder}} = 1 \text{ beam} \times R_{DL \text{ beam}} = 1 \times 34 \text{ kip} = 34 \text{ kip (acts at midspan)}$$

$$P_{LL \text{ girder}} = 1 \times R_{LL \text{ beam}} = 1 \times 9 \text{ kip} = 9 \text{ kip (acts at midspan)}$$

$$P_{s \text{ girder}} = P_{DL \text{ girder}} + P_{LL \text{ girder}} = 34 + 9 = \mathbf{43 \text{ kip}} \text{ (acts at midspan)}$$

$$P_{u \text{ girder}} = 1.2 P_{DL \text{ girder}} + 1.6 P_{LL \text{ girder}} = 1.2(34) + 1.6(9) = \mathbf{55 \text{ kip}} \text{ (acts at midspan)}$$

Uniform roof load on Girder edge distance:

$$W_{DL \text{ edge distance}} = (9''/12) \times 109.5 \text{ psf} = 82 \text{ lb/ft}$$

$$W_{LL \text{ edge distance}} = (9''/12) \times 35 \text{ psf} = 26 \text{ lb/ft}$$

The uniformly distributed load (UDL) on this girder is due to the girder stem weight, the load on the girder edge distance, AND the exterior cladding. Thus,

$$\begin{aligned} W_{DL \text{ girder}} &= \text{girder stem weight} + \text{girder edge distance} + \text{exterior cladding/parapet} \\ &= 357 \text{ lb/ft} + 82 \text{ lb/ft} + 0 \text{ lb/ft} = 439 \text{ lb/ft (assuming no parapet)} \end{aligned}$$

$$W_{LL \text{ girder}} = 26 \text{ lb/ft}$$

The total UDL on the girder are:

$$W_{s \text{ girder}} = W_{DL \text{ girder}} + W_{LL \text{ girder}} = 465 \text{ lb/ft}$$

$$\begin{aligned} W_{u \text{ girder}} &= 1.2 W_{DL \text{ girder}} + 1.6 W_{LL \text{ girder}} \\ &= 1.2 (439 \text{ lb/ft}) + 1.6 (26 \text{ lb/ft}) = 568 \text{ lb/ft} = \mathbf{0.57 \text{ kip/ft}} \end{aligned}$$

The REACTIONS for a GIRDER subjected to a uniformly distributed load, w , in addition to concentrated loads at MIDSPAN (see plan) from the beam reactions, assuming simple supports, are:

$$R_{DL} = W_{DL \text{ girder}} \times L/2 + P_{DL \text{ girder}}/2 = (0.44 \text{ kip/ft} \times 32 \text{ ft}/2) + (34 \text{ kip}/2) = 24 \text{ kip}$$

$$R_{LL} = W_{LL \text{ girder}} \times L/2 + P_{LL \text{ girder}}/2 = (0.026 \text{ kip/ft} \times 32 \text{ ft}/2) + (9 \text{ kip}/2) = 4.9 \text{ kip}$$

$$R_u = W_{u \text{ girder}} \times L/2 + P_{u \text{ girder}}/2 = (0.57 \text{ kip/ft} \times 32 \text{ ft}/2) + (55 \text{ kip}/2) = \mathbf{36.6 \text{ kip}}$$

General notes at beginning of Chapter 2 problem-set apply

Prob. 2-1

(a) 4#9, $A_s = 4.00 \text{ in.}^2$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4.00(60)}{0.85(3)(16)} = 5.88 \text{ in.}$$

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) = \frac{4.00(60) \left(24 - \frac{5.88}{2} \right)}{12} = 421 \text{ ft - kips}$$

(b) 4#10, $A_s = 5.08 \text{ in.}^2$

$$a = \frac{5.08(60)}{0.85(3)(16)} = 7.47 \text{ in.} \quad M_n = \frac{5.08(60) \left(24 - \frac{7.47}{2} \right)}{12} = 515 \text{ ft - kips}$$

% Increase: A_s : +27%; M_n : +22%

(c) 4#9, $A_s = 4.00 \text{ in.}^2$, $a = 5.88 \text{ in.}$ (from part (a))

$$M_n = \frac{4.00(60) \left(28 - \frac{5.88}{2} \right)}{12} = 501 \text{ ft - kips}$$

% Increase: d : +16.7 %; M_n : +19%

(d) $f'_c = 4000 \text{ psi}$

$$a = \frac{4(60)}{0.85(4)(16)} = 4.41 \text{ in.} \quad M_n = \frac{4.00(60) \left(24 - \frac{4.41}{2} \right)}{12} = 436 \text{ ft - kips}$$

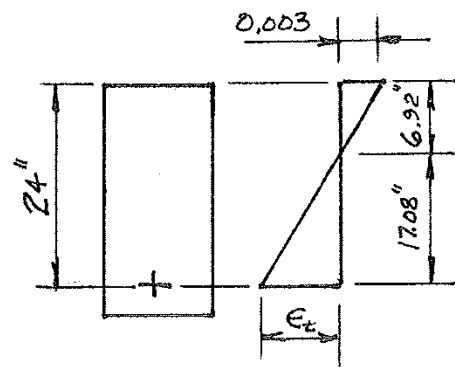
% Increase: f'_c : 33.3%; M_n : 3.6%

Prob. 2-2 Check ϵ_t for Prob. 2-1(a)

$$c = \frac{a}{\beta_1} = \frac{5.88}{0.85} = 6.92 \text{ in.} \quad \text{then, from a strain diagram :}$$

$$\frac{\epsilon_t}{(24 - 6.92)} = \frac{0.003}{6.92}$$

$$\epsilon_t = 0.0074 > \epsilon_y = 0.00207 \quad \therefore f_s = f_y$$



Prob. 2-3

(a) [4/40], 4#8, $A_s = 3.16 \text{ in.}^2$, $b = 13 \text{ in.}$, $d = 24 \text{ in.}$ $\rho = \frac{3.16}{13(24)} = 0.0101$

$A_{s,\min} = 0.005(13)(24) = 1.56 \text{ in.}^2 < 3.16 \text{ in.}^2$ (O.K.)

(Table A-9) $\bar{k} = 0.3800 \text{ ksi}$ and $\epsilon_t > 0.005$, $\therefore \phi = 0.90$

$$\phi M_n = \phi b d^2 \bar{k} = \frac{0.90(13)(24)^2(0.3800)}{12} = 213 \text{ ft - kips}$$

(b) [4/60], 4#8, $A_s = 3.16 \text{ in.}^2$, $b = 13 \text{ in.}$, $d = 24 \text{ in.}$ $\rho = \frac{3.16}{13(24)} = 0.0101$

$A_{s,\min} = 0.0033(13)(24) = 1.03 \text{ in.}^2 < 3.16 \text{ in.}^2$ (O.K.)

(Table A-10) $\bar{k} = 0.5520 \text{ ksi}$ and $\epsilon_t > 0.005$, $\therefore \phi = 0.90$

$$\phi M_n = \phi b d^2 \bar{k} = \frac{0.90(13)(24)^2(0.5520)}{12} = 310 \text{ ft - kips}$$

% Increase: f_y : +50%; ϕM_n : +45.5%

Prob. 2-4 [4/60]

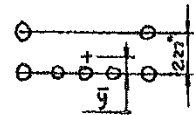
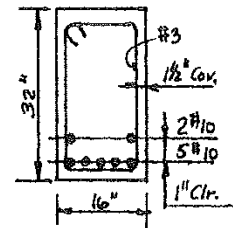
$$\bar{y} = \frac{2A(2.27)}{7A} = 0.649 \text{ in.}$$

$d = 32 - 1.5 - 0.375 - 1.27/2 - 0.649 = 28.8 \text{ in.}$

$$\rho = \frac{8.89}{16(28.8)} = 0.0193, \quad \bar{k} = 0.9609 \text{ ksi}, \quad \epsilon_t = 0.00449$$

$$\therefore \phi = 0.65 + (0.00449 - 0.002) \left(\frac{250}{3} \right) = 0.858$$

$$\phi M_n = \phi b d^2 \bar{k} = \frac{0.858(16)(28.8)^2(0.9609)}{12} = 912 \text{ ft - kips}$$



Prob. 2-5 [3/40], $b = 20$ in., $d = 42$ in., $h = 45$ in., $L = 28$ ft

Beam is adequate if $\phi M_n \geq M_u$

$$\text{Beam weight} = \frac{20(45)}{144}(0.150) = 0.938 \text{ kip/ft}$$

$$w_u = 1.2(0.938 + 2.20) + 1.6(3.60) = 9.53 \text{ kips/ft}; \quad M_u = \frac{9.53(28)^2}{8} = 939 \text{ ft - kips}$$

$$(a) \quad 6\#10, \quad A_s = 7.62 \text{ in.}^2, \quad \rho = \frac{7.62}{20(42)} = 0.00907$$

$$A_{s,\min} = 0.005(20)(42) = 4.20 \text{ in.}^2 < 7.62 \text{ in.}^2 \text{ (O.K.)}$$

$$(\text{Table A-7}) \quad \bar{k} = 0.3380 \text{ ksi and } \epsilon_t > 0.005, \therefore \phi = 0.90$$

$$\phi M_n = \phi b d^2 \bar{k} = \frac{0.90(20)(42)^2(0.3380)}{12} = 894 \text{ ft - kips} < 939 \text{ ft - kips (N.G.)}$$

$$(b) \quad 6\#11, \quad A_s = 9.36 \text{ in.}^2, \quad \rho = \frac{9.36}{20(42)} = 0.0111$$

$$A_{s,\min} = 4.20 \text{ in.}^2 < 9.36 \text{ in.}^2 \text{ (O.K.)}$$

$$(\text{Table A-7}) \quad \bar{k} = 0.4053 \text{ ksi and } \epsilon_t > 0.005, \therefore \phi = 0.90$$

$$\phi M_n = \phi b d^2 \bar{k} = \frac{0.90(20)(42)^2(0.4053)}{12} = 1072 \text{ ft - kips} > 939 \text{ ft - kips (O.K.)}$$

Prob. 2-7 [4/60] $b = 12$ in., $h = 20$ in., 3#8 ($A_s = 2.37 \text{ in.}^2$)

$$\text{Beam weight} = \frac{12(20)}{144}(0.150) = 0.250 \text{ k/ft}$$

$$d = 20 - 1.5 - 0.38 - 0.50 = 17.62 \text{ in.}; \quad A_{s,\min} = 0.0033(12)(17.62) = 0.700 \text{ in.}^2 \text{ (O.K.)}$$

$$\rho = \frac{2.37}{12(17.62)} = 0.0112; \quad \bar{k} = 0.6056 \text{ ksi, } \epsilon_t > 0.005, \quad \phi = 0.90$$

$$\phi M_n = \frac{0.90(12)(17.62)^2(0.6056)}{12} = 169 \text{ ft - kips}$$

$$M_u = \frac{[1.2(0.7 + 0.250) + 1.6(2.5)](16)^2}{8} = 164.5 \text{ ft - kips} < 169 \text{ ft - kips (O.K.)}$$

Prob. 2-8

[3/60] $b = 16$ in., $h = 38$ in., $L = 26.5$ ft simple span. Check moment adequacy.

$$\text{Beam weight} = \frac{16(38)}{144}(0.150) = 0.633 \text{ k/ft}$$

$$M_u = \frac{[1.2(1.80 + 0.633) + 1.6(3.20)]}{8}(26.5)^2 = 706 \text{ ft-kips}$$

(a) 5#9, $A_s = 5.00 \text{ in.}^2$, $d = 35$ in., $\rho = \frac{5.00}{16(35)} = 0.0089$

$$A_{s,\min} = 0.0033(16)(35) = 1.85 \text{ in.}^2 < 5.00 \text{ in.}^2 \text{ (O.K.)}$$

$$\bar{k} = 0.4781 \text{ ksi}, \quad \varepsilon_t > 0.005, \quad \phi = 0.90$$

$$\phi M_n = \frac{0.90(16)(35)^2(0.4781)}{12} = 703 \text{ ft-kips} < 706 \text{ ft-kips (N.G.)}$$

(a) 6#9, $A_s = 6.00 \text{ in.}^2$, $d = 34.4$ in., $\rho = \frac{6.00}{16(34.4)} = 0.0109$

$$A_{s,\min} = 0.0033(16)(34.4) = 1.82 \text{ in.}^2 < 6.00 \text{ in.}^2 \text{ (O.K.)}$$

$$\bar{k} = 0.5702 \text{ ksi}, \quad \varepsilon_t > 0.005, \quad \phi = 0.90$$

$$\phi M_n = \frac{0.90(16)(34.4)^2(0.5702)}{12} = 808 \text{ ft-kips} > 706 \text{ ft-kips (O.K.)}$$

Prob. 2-9 [3/60] 3#10, $A_s = 3.81 \text{ in.}^2$, $b = 14.5$ in., $h = 26$ in. check moment adequacy.

$$d = 26 - 1.5 - 0.38 - 1.27/2 = 23.5 \text{ in.}$$

$$\text{Calculated beam weight} = 0.393 \text{ k/ft}$$

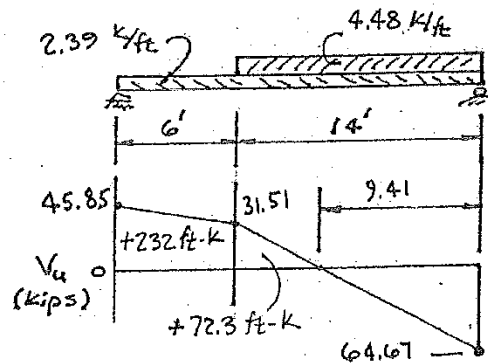
$$\text{Max. } M_u \text{ from diag.} = 304 \text{ ft-kips}$$

$$\rho = \frac{3.81}{14.5(23.5)} = 0.0112$$

$$A_{s,\min} = 0.0033(14.5)(23.5) = 1.12 \text{ in.}^2$$

$$\bar{k} = 0.5835, \quad \varepsilon_t > 0.005, \quad \phi = 0.90$$

$$\phi M_n = \frac{0.90(14.5)(23.5)^2(0.5835)}{12} = 350 \text{ ft-kips} > 304 \text{ ft-kips (O.K.)}$$



Prob. 2-10 [4/60] 4#9, $b = 14$ in., $h = 24$ in., find max simple span L

$$d = 24 - 1.5 - 0.38 - 1.13/2 = 21.6 \text{ in.}$$

$$\text{Beam wt.} = \frac{14(24)}{144}(0.150) = 0.350 \text{ k/ft; } \rho = \frac{4.00}{14(21.6)} = 0.0132$$

$$A_{s,\min} = 0.0033(14)(21.6) = 1.00 \text{ in.}^2$$

$$\bar{k} = 0.6998 \text{ ksi, } \epsilon_t > 0.005, \phi = 0.90$$

$$\phi M_n = \frac{0.90(14)(21.6)^2 0.6998}{12} = 343 \text{ ft - kips}$$

$$M_u = \frac{[1.2(0.60 + 0.35) + 1.6(1.4)]L^2}{8} = 343 \text{ ft - kips, from which } L = 28.5 \text{ ft}$$

Prob. 2-11 [3/60] One-way slab analysis. #7@6 in., $A_s = 1.20$ in.²/ft, $h = 10$ in., $L = 16$ ft

$$\text{Slab weight} = \frac{10(12)}{144}(0.150) = 0.125 \text{ k/ft;}$$

$$M_u = \frac{[1.2(0.125) + 1.6(0.600)]16^2}{8} = 35.5 \text{ ft - kips}$$

$$d = 10 - 0.75 - 0.875/2 = 8.81 \text{ in.; } \rho = \frac{1.20}{12(8.81)} = 0.0113$$

$$A_{s,\min} = 0.0018(12)(8.81) = 0.19 \text{ in.}^2/\text{ft (O.K.); } \bar{k} = 0.5879 \text{ ksi, } \epsilon_t > 0.005, \phi = 0.90$$

$$\phi M_n = \frac{0.90(12)(8.81)^2(0.5879)}{12} = 41.4 \text{ ft - kips} > 35.5 \text{ ft - kips (O.K.)}$$

Prob. 2-12 [3/40] One-way slab analysis, $h = 8$ in., #8@6 in., $A_s = 1.58$ in.²/ft, $L = 12$ ft

$$\text{Slab weight} = \frac{8(12)}{144}(0.150) = 0.100 \text{ k/ft;}$$

$$d = 8 - 0.75 - 1.00/2 = 6.75 \text{ in.; } A_{s,\min} = 0.0020(12)(6.75) = 0.16 \text{ in.}^2/\text{ft (O.K.)}$$

$$\rho = \frac{1.20}{12(6.75)} = 0.0195, \bar{k} = 0.6608 \text{ ksi, } \epsilon_t > 0.005, \phi = 0.90$$

$$\phi M_n = \frac{0.90(12)(6.75)^2(0.6608)}{12} = 27.1 \text{ ft - kips}$$

$$M_{u(D.L.)} = \frac{1.2(0.100)(12)^2}{8} = 2.16 \text{ ft - kips, } M_{u(L.L.)} = \frac{1.6w_{LL}L^2}{8} = 27.1 - 2.16 = 24.9 \text{ ft - kips}$$

$$\text{From which, } w_{LL} = 0.865 \text{ k/ft} = 865 \text{ psf}$$

Prob. 2-13 [4/60] One-way slab w/ construction errors.

As designed: #7@11, $A_s = 0.65 \text{ in.}^2/\text{ft}$, $d = 8.5 - 1 - 0.875/2 = 7.06 \text{ in.}$

$A_{s,\min} = 0.0018(12)(8.50) = 0.18 \text{ in.}^2/\text{ft}$ (O.K.)

$$\rho = \frac{0.65}{12(7.06)} = 0.0077; \quad \bar{k} = 0.4306 \text{ ksi}, \quad \varepsilon_t > 0.005, \quad \phi = 0.90$$

$$\phi M_n = \frac{0.90(12)(7.06)^2(0.4306)}{12} = 19.3 \text{ ft-kips}$$

As built: $d = 8.5 - 3.5 - 0.875/2 = 4.56 \text{ in.}$

$$\rho = \frac{0.65}{12(4.56)} = 0.0119; \quad \bar{k} = 0.6391 \text{ ksi}, \quad \varepsilon_t > 0.005, \quad \phi = 0.90$$

$$\phi M_n = \frac{0.90(12)(4.56)^2(0.6391)}{12} = 11.96 \text{ ft-kips} \quad (\% \text{ Change} = -38\%)$$

Prob. 2-14 Design. [3/60] $M_u = 133 \text{ ft-kips}$, $b = 11\frac{1}{2} \text{ in.}$, $h = 23 \text{ in.}$

Est. $d = 20 \text{ in.}$, Assume $\phi = 0.90$.

$$\text{Required } \bar{k} = \frac{133(12)}{0.90(11.5)(20)^2} = 0.3855 \text{ ksi}$$

$$\text{Required } \rho = 0.0070 \quad (\varepsilon_t > 0.005, \quad \phi = 0.90)$$

$$\text{Required } A_s = 0.007(11.5)(20) = 1.61 \text{ in.}^2, \quad A_{s,\min} = 0.0033(11.5)(20) = 0.76 \text{ in.}^2 \text{ (O.K.)}$$

Select 3#7, one layer ($A_s = 1.80 \text{ in.}^2$, $b_{\min} = 8.5 \text{ in.}$)

$$\text{Calculated } d = 23 - 1.5 - 0.38 - \frac{0.875}{2} = 20.7 \text{ in.} > 20 \text{ in. (O.K.)}$$

Prob. 2-15 Design. [4/60] $M_u = 400 \text{ ft-kips}$, $b = 16 \text{ in.}$, $h = 28 \text{ in.}$

Est. $d = 25 \text{ in.}$, Assume $\phi = 0.90$.

$$\text{Required } \bar{k} = \frac{400(12)}{0.90(16)(25)^2} = 0.5333 \text{ ksi}$$

$$\text{Required } \rho = 0.0098 \quad (\varepsilon_t > 0.005, \quad \phi = 0.90)$$

$$\text{Required } A_s = 0.0098(16)(25) = 3.92 \text{ in.}^2, \quad A_{s,\min} = 0.0033(16)(25) = 1.32 \text{ in.}^2 \text{ (O.K.)}$$

Select 4#9, one layer ($A_s = 4.00 \text{ in.}^2$, $b_{\min} = 12 \text{ in.}$)

$$\text{Calculated } d = 28 - 1.5 - 0.38 - \frac{1.13}{2} = 25.6 \text{ in.} > 25 \text{ in. (O.K.)}$$

Prob. 2-16 (Prob. 2-15 with incorrectly placed steel making $d = 24$ in.)

[4/60] $M_u = 400$ ft-kips, $b = 16$ in.,

$d = 24$ in., Assume $\phi = 0.90$.

$$\rho = \frac{4.00}{16(24)} = 0.0104$$

$$A_{s,\min} = 0.0033(16)(24) = 1.27 \text{ in.}^2$$

$$\bar{k} = 0.5667, \quad \epsilon_t > 0.005, \quad \phi = 0.90$$

$$\phi M_n = \frac{0.90(16)(24)^2 0.5667}{12} = 392 \text{ ft-kips} < 400 \text{ ft-kips (N.G.)}$$

Prob. 2-17 [4/60] $L = 32$ ft, $b = 11\frac{1}{2}$ in., $h = 26$ in.

$$\text{Beam weight} = \frac{11.5(26)}{144}(0.150) = 0.312 \text{ kip/ft} \quad \text{Assume } \phi = 0.90$$

$$M_u = \frac{[1.2(0.85 + 0.312) + 1.6(1.0)](32)^2}{8} = 383 \text{ ft-kips}$$

Estimated $d = 23$ in.

$$\text{Required } \bar{k} = \frac{383(12)}{0.90(11.5)(23)^2} = 0.8394 \text{ ksi} \quad (\epsilon_t > 0.005, \phi = 0.90)$$

Required $\rho = 0.0164$

$$\text{Required } A_s = 0.0164(11.5)(23) = 4.34 \text{ in.}^2 \quad A_{s,\min} = 0.0033(11.5)(23) = 0.87 \text{ in.}^2$$

Select 3#11 in one layer ($A_s = 4.68 \text{ in.}^2$, $b_{\min} = 11$ in.)

Calculated $d = 26 - 1.5 - 0.38 - 1.41/2 = 23.4$ in. > 23 in. (O.K.)

Check ϕM_n :

$$\rho = \frac{4.68}{11.5(23.4)} = 0.0174, \quad \bar{k} = 0.8838 \text{ ksi}, \quad (\epsilon_t > 0.005, \phi = 0.90)$$

$$\phi M_n = \frac{0.90(11.5)(23.4)^2 (0.8838)}{12} = 417 \text{ ft-kips} > 383 \text{ ft-kips (O.K.)}$$

Prob. 2-18 [5/60] $L = 30$ ft, $b = 12$ in., $h = 27$ in.

$$\text{Beam weight} = \frac{12(27)}{144} = 0.338 \text{ k/ft}$$

Estimated $d = 24$ in., assume $\phi = 0.90$

$$M_u = \frac{[1.2(0.338) + 1.6(1.35)]30^2}{8} = 289 \text{ ft-kips}$$

$$\text{Required } \bar{k} = \frac{289(12)}{0.90(12)(24)^2} = 0.5575 \text{ ksi, required } \rho = 0.0100, (\epsilon_t > 0.005, \phi = 0.90)$$

$$\text{Required } A_s = 0.0100(12)(24) = 2.88 \text{ in.}^2, A_{s,\min} = 0.0035(12)(24) = 1.01 \text{ in.}^2 \text{ (O.K.)}$$

$$\text{Select 3\#9 } (A_s = 3.00 \text{ in.}^2, b_{\min} = 9.5 \text{ in.})$$

$$\text{Calculated } d = 27 - 1.5 - 0.38 - 1.13/2 = 24.6 \text{ in.} > 24 \text{ in. (O.K.)}$$

Check ϕM_n :

$$\rho = \frac{3.00}{12(24.6)} = 0.0102, \bar{k} = 0.5679 \text{ ksi, } (\epsilon_t > 0.005, \phi = 0.90)$$

$$\phi M_n = \frac{0.90(12)(24.6)^2(0.5679)}{12} = 309 \text{ ft-kips} > 289 \text{ ft-kips (O.K.)}$$

Prob. 2-19 (Redo Prob. 2-18 using superimposed loads: L.L. = 1.75 k/ft, D.L. = 1.0 k/ft)

$$M_u = \frac{[1.2(1.0 + 0.338) + 1.6(1.75)]30^2}{8} = 496 \text{ ft-kips}$$

Est. $d = 24 \text{ in.}$, assume $\phi = 0.90$

$$\text{Required } \bar{k} = \frac{496(12)}{0.90(12)(24)^2} = 0.9568 \text{ ksi, required } \rho = 0.0184, (\epsilon_t > 0.005, \phi = 0.90)$$

$$\text{Required } A_s = 0.0184(12)(24) = 5.30 \text{ in.}^2, A_{s,\min} = 0.0035(12)(24) = 1.01 \text{ in.}^2 \text{ (O.K.)}$$

$$\text{Select 6\#9, two layers, 1 in. clear } (A_s = 6.00 \text{ in.}^2, b_{\min} = 9.5 \text{ in.})$$

$$\text{Calculated } d = 27 - 1.5 - 0.38 - 1.13 - 0.5 = 23.5 \text{ in.} < 24 \text{ in. (Check } \phi M_n)$$

$$\rho = \frac{6.00}{12(23.5)} = 0.0213, \bar{k} = 1.0859 \text{ ksi, } (\epsilon_t > 0.005, \phi = 0.90)$$

$$\phi M_n = \frac{0.90(12)(23.5)^2(1.0859)}{12} = 540 \text{ ft-kips} > 496 \text{ ft-kips (O.K.)}$$

Prob. 2-20 [3/60] $L = 22 \text{ ft}$, $b = 15 \text{ in.}$, h : full inches.

$$M_u = \frac{[1.2(1.6) + 1.6(1.4)](22)^2}{8} = 252 \text{ ft-kips (Estimated beam weight included.)}$$

$$\text{Try } \rho = 0.0090, \bar{k} = 0.4828 \text{ ksi } (\epsilon_t > 0.005, \phi = 0.90)$$

$$\text{Req'd } d = \sqrt{\frac{252(12)}{0.90(15)(0.4828)}} = 21.5 \text{ in.} \quad \left(\frac{d}{b} = \frac{21.5}{15} = 1.4 \text{ (Say O.K.)} \right)$$

$$\text{Required } A_s = 0.009(15)(21.5) = 2.90 \text{ in.}^2, A_{s,\min} = 0.0033(15)(21.5) = 1.06 \text{ in.}^2 \text{ (O.K.)}$$

$$\text{Select 3\#9 } (A_s = 3.00 \text{ in.}^2, b_{\min} = 9.5 \text{ in.})$$

$$\text{Req'd } h = 21.5 + 1.13/2 + 0.38 + 1.5 = 23.9 \text{ in. Use } 24 \text{ in.}$$

$$\text{Check } \phi M_n: d = 21.6 \text{ in., } \rho = \frac{3.00}{15(21.6)} = 0.0093, \bar{k} = 0.4970 \text{ ksi, } (\epsilon_t > 0.005, \phi = 0.90)$$

$$\phi M_n = \frac{0.90(15)(21.6)^2(0.4970)}{12} = 261 \text{ ft-kips} > 252 \text{ ft-kips (O.K.)}$$

Prob. 2-21 Design, [3/60], $L = 30$ ft, $b \leq 16$ in., beam wt. not incl. in given loads.

Neglecting beam weight: $M_u = \frac{[1.2(1.0) + 1.6(2.0)]30^2}{8} = 495$ ft - kips

Try $\rho = 0.0090$, $\bar{k} = 0.4828$ ksi ($\epsilon_t > 0.005$, $\phi = 0.90$)

For $b = 16$ in.: Req'd $d = \sqrt{\frac{495(12)}{0.90(16)(0.4828)}} = 29.2$ in. $\left(\frac{d}{b} = \frac{29.2}{16} = 1.8 \text{ (O.K.)}\right)$

Estimate h assuming #8 bars: $h = 29.2 + 0.5 + 0.38 + 1.5 = 31.6$ in. Use $h = 32$ in.

Beam wt. = $\frac{16(32)}{144}(0.150) = 0.533$ kips/ft, new $M_u = 495 + \frac{1.2(0.533)(30)^2}{8} = 567$ ft - kips

With $b = 16$ in., $d = 29$ in., solve for A_s

Required $\bar{k} = \frac{567(12)}{0.90(16)(29)^2} = 0.5618$ ksi, required $\rho = 0.0108$, ($\epsilon_t > 0.005$, $\phi = 0.90$)

Required $A_s = 0.0108(16)(29) = 5.01$ in.², $A_{s,\min} = 0.0033(16)(29) = 1.53$ in.² (O.K.)

Select 5#9 ($A_s = 5.00$ in.², $b_{\min} = 14$ in.)

Calculated $d = 32 - 1.5 - 0.38 - 1.13/2 = 29.6$ in. > 29 in. (O.K.)

Use $b = 16$ in., $h = 32$ in., 5#9 bars.

Prob. 2-22 Redo Prob. 2-21 with $h \leq 30$ in. and no limitation on b .

Try estimated beam wt. of 0.533 k/ft and $M_u = 567$ ft-kips from Prob. 2-21.

Assume $h = 30$ in., $d = 27$ in.; try $\rho = 0.0090$, $\bar{k} = 0.4828$ ksi and $\phi = 0.90$

Required $b = \frac{567(12)}{0.90(27)^2(0.4828)} = 21.5$ in. Use $b = 21$ in.

$\frac{d}{b} = \frac{27}{21} = 1.29$ Low due to restricted h . (Use)

Beam weight = $\frac{30(21)}{144}(0.150) = 0.656$ kips/ft

New $M_u = 567 + \frac{1.2(0.656 - 0.533)(30)^2}{8} = 584$ ft - kips

Required $\bar{k} = \frac{584(12)}{0.90(21)(27)^2} = 0.5086$ ksi, req'd $\rho = 0.0096$, $\epsilon_t > 0.005$, $\phi = 0.90$

Required $A_s = 0.0096(21)(27) = 5.44$ in.²

Select 7#8 ($A_s = 5.53$ in.², $b_{\min} = 17$ in.) $A_{s,\min} = 0.0033(21)(27) = 1.87$ in.² (O.K.)

Calculated $d = 30 - 1.5 - 0.38 - 0.5 = 27.6$ in. > 27 in. (O.K.)

Use $b = 21$ in., $h = 30$ in., 7#8 bars

Prob. 2-23 Design, [3/60], $L = 32$ ft, $b \leq 18$ in.

Neglect beam weight: $M_u = \frac{[1.2(1.5) + 1.6(2.0)](32)^2}{8} = 640$ ft-kips

Try $b = 18$ in., $\rho = 0.0090$, $\bar{k} = 0.4828$ ksi ($\varepsilon_t > 0.005$, $\phi = 0.90$)

Req'd $d = \sqrt{\frac{640(12)}{0.90(18)(0.4828)}} = 31.3$ in. $\left(\frac{d}{b} = \frac{31.3}{18} = 1.74 \text{ (O.K.)}\right)$

Estimated $h = 31.3 + 0.5 + 0.38 + 1.5 = 33.7$ Use $h = 35$ in.

Beam weight = $\frac{18(35)}{144}(0.150) = 0.656$ kip/ft

New $M_u = 640 + \frac{1.2(0.656)(32)^2}{8} = 741$ ft-kips

Estimated $d = 32$ in.

Required $\bar{k} = \frac{741(12)}{0.90(18)(32)^2} = 0.5361$ ksi, required $\rho = 0.0102$, $\varepsilon_t > 0.005$, $\phi = 0.90$

Required $A_s = 0.0102(18)(32) = 5.87$ in.², $A_{s,\min} = 0.0033(18)(32) = 1.90$ in.² (O.K.)

Select 6#9 ($A_s = 6.00$ in.², $b_{\min} = 16.5$ in.)

Calculated $d = 35 - 1.5 - 0.38 - 1.13/2 = 32.6$ in. > 32 in. (O.K.)

Use $b = 18$ in., $h = 35$ in., 6#9 bars

Prob. 2-24 Redo Prob. 2-23 with $h \leq 32$ in. and no limitation on b .

Neglect beam weight. $M_u = 640$ ft-kips from Prob. 2-23.

Assume $h = 32$ in., $d = 29$ in.; try $\rho = 0.0090$, $\bar{k} = 0.4828$ ksi and $\phi = 0.90$

Required $b = \frac{640(12)}{0.90(29)^2(0.4828)} = 21$ in. Use $b = 21$ in.

$\frac{d}{b} = \frac{29}{21} = 1.38$ Low due to restricted h . (Use)

Beam weight = $\frac{32(21)}{144}(0.150) = 0.700$ kips/ft

New $M_u = 640 + \frac{1.2(0.700)(32)^2}{8} = 748$ ft-kips

Required $\bar{k} = \frac{748(12)}{0.90(21)(29)^2} = 0.5647$ ksi, req'd $\rho = 0.0108$, $\varepsilon_t > 0.005$, $\phi = 0.90$

Required $A_s = 0.0108(21)(29) = 6.58$ in.²

Select 7#9 ($A_s = 7.00$ in.², $b_{\min} = 18.5$ in.) $A_{s,\min} = 0.0033(21)(29) = 2.01$ in.² (O.K.)

Calculated $d = 32 - 1.5 - 0.38 - 1.13/2 = 29.6$ in. > 29 in. (O.K.)

Use $b = 21$ in., $h = 32$ in., 7#9 bars

Prob. 2-25 [4/60] Design, $L = 40$ ft

Neglect beam weight: $M_u = \frac{[1.2(0.8) + 1.6(1.4)](40)^2}{8} = 640$ ft - kips

Try $\rho = 0.0120$, $\bar{k} = 0.6438$ ksi ($\epsilon_t > 0.005$, $\phi = 0.90$,) and $d/b \approx 2$.

Required $d = \sqrt[3]{\frac{2(640)(12)}{0.90(0.6438)}} = 29.8$ in. (make $b = 16$ in. and $h = 34$ in.)

Beam weight $= \frac{16(34)}{144}(0.150) = 0.567$ kip/ft

New $M_u = 640 + \frac{1.2(0.567)(40)^2}{8} = 776$ ft - kips

Estimated $d = 31$ in.

Required $\bar{k} = \frac{776(12)}{0.90(16)(31)^2} = 0.6729$ ksi, req'd. $\rho = 0.0127$, $\epsilon_t > 0.005$, $\phi = 0.90$

Required $A_s = 0.0127(16)(31) = 6.30$ in.²

Select 5#10 ($A_s = 6.35$ in.², $b_{\min} = 15.5$ in.) $A_{s,\min} = 0.0033(16)(31) = 1.64$ in.² (O.K.)

Calculated $d = 34 - 1.5 - 0.38 - 1.27/2 = 31.5$ in. > 31 in. (O.K.)

Use $b = 16$ in., $h = 34$ in., 5#10 bars

Prob. 2-26 [5/60] Design. Unsymmetrical load.

From V_u diag. (neg. bm. wt.): $M_{u(\max)} = 504$ ft-kips

Try $\rho = 0.0150$, $\bar{k} = 0.8047$ ksi and $d/b \approx 2$.

($\epsilon_t > 0.005$, $\phi = 0.90$,)

Required $d = \sqrt[3]{\frac{2(504)(12)}{0.90(0.8047)}} = 25.6$ in.

Make $b = 13$ in. and $h = 28$ in.

Beam weight $= \frac{13(28)}{144}(0.150) = 0.379$ kip/ft

New $w_{uDL} = 2.40 + 1.2(.379) = 2.86$ k/ft :

new diagram yields $M_{u(\max)} = 553$ ft-kips.

Estimated $d = 25$ in.

Required $\bar{k} = \frac{553(12)}{0.90(13)(25)^2} = 0.9075$ ksi, req'd. $\rho = 0.0173$, $\epsilon_t > 0.005$, $\phi = 0.90$

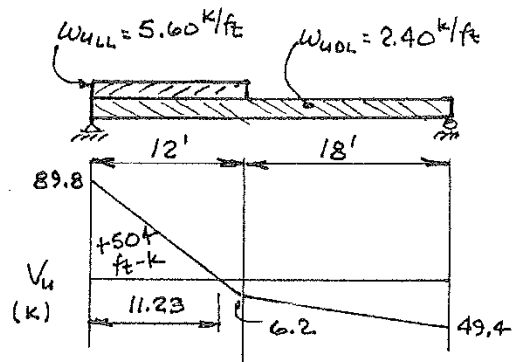
Required $A_s = 0.0173(13)(25) = 5.62$ in.²

Select 6#9 (two layers, 3 each, 1 in. clear) ($A_s = 6.00$ in.², $b_{\min} = 9.5$ in.)

$A_{s,\min} = 0.0035(13)(25) = 1.14$ in.² (O.K.)

Calculated $d = 28 - 1.5 - 0.38 - 1.13 - 0.50 = 24.5$ in. < 25 in. (N.G., check ϕM_n)

$\rho = \frac{6.00}{13(24.5)} = 0.0188$, $\bar{k} = 0.9783$ ksi, ($\epsilon_t > 0.005$, $\phi = 0.90$)



Prob. 2-26 (cont.)

$$\phi M_n = \frac{0.90(13)(24.5)^2(0.9783)}{12} = 572 \text{ ft-kips} > 553 \text{ ft-kips (O.K.)}$$

Use $b = 13 \text{ in.}$, $h = 28 \text{ in.}$, 6#9 bars in two layers

Prob. 2-27 [5/60] Design, $L = 28 \text{ ft}$

Neglect beam weight: $M_u = \frac{1.6(0.8)(28)^2}{8} + \frac{[1.2(10) + 1.6(14)](28)}{4} = 366 \text{ ft-kips}$

Try $\rho = 0.0150$, $\bar{k} = 0.8047 \text{ ksi}$ ($\epsilon_t > 0.005$, $\phi = 0.90$), and $d/b \approx 2$.

Required $d = \sqrt[3]{\frac{2(366)(12)}{0.90(0.8047)}} = 23 \text{ in.}$ (try $b = 12 \text{ in.}$ and $h = 27 \text{ in.}$, est $d = 24 \text{ in.}$)

Beam weight $= \frac{12(27)}{144}(0.150) = 0.338 \text{ kip/ft}$

New $M_u = 366 + \frac{1.2(0.338)(28)^2}{8} = 406 \text{ ft-kips}$

Required $\bar{k} = \frac{406(12)}{0.90(12)(24)^2} = 0.7832 \text{ ksi}$, req'd. $\rho = 0.0146$, $\epsilon_t > 0.005$, $\phi = 0.90$

Required $A_s = 0.0146(12)(24) = 4.20 \text{ in.}^2$

Select 3#11 ($A_s = 4.68 \text{ in.}^2$, $b_{\min} = 11 \text{ in.}$) $A_{s,\min} = 0.0035(12)(24) = 1.01 \text{ in.}^2$ (O.K.)

Calculated $d = 27 - 1.5 - 0.38 - 1.41/2 = 24.4 \text{ in.} > 24 \text{ in.}$ (O.K.)

Use $b = 12 \text{ in.}$, $h = 27 \text{ in.}$, 3#11 bars

Prob. 2-28 [3/60]

$P_u = 1.2(8) + 1.6(10) = 25.6 \text{ kips}$

$w_u = 1.2(0.3) + 1.6(0.5) = 1.16 \text{ kips/ft}$

V_u diag. (neg. bm. wt.): $M_{u(\max)} = 184.2 \text{ ft-kips}$

Try $\rho = 0.0090$, $\bar{k} = 0.4828 \text{ ksi}$ and $d/b \approx 2$.

($\epsilon_t > 0.005$, $\phi = 0.90$),

Required $d = \sqrt[3]{\frac{2(184.2)(12)}{0.90(0.4828)}} = 21.7 \text{ in.}$

Make $b = 11 \text{ in.}$ and $h = 25 \text{ in.}$

Beam weight $= \frac{11(25)}{144}(0.150) = 0.287 \text{ kip/ft}$

New $w_{uDL} = 1.16 + 1.2(0.287) = 1.504 \text{ k/ft}$:

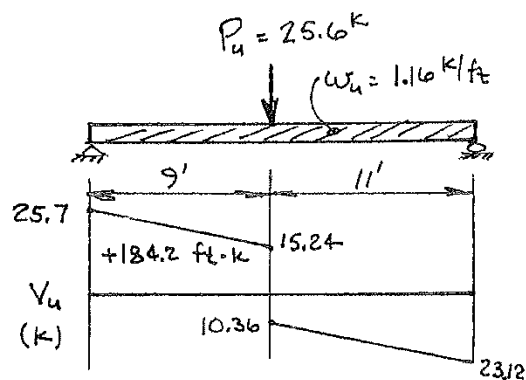
new diagram yields $M_{u(\max)} = 201 \text{ ft-kips}$.

Estimated $d = 22 \text{ in.}$

Required $\bar{k} = \frac{201(12)}{0.90(11)(22)^2} = 0.5034 \text{ ksi}$, req'd. $\rho = 0.0095$, $\epsilon_t > 0.005$, $\phi = 0.90$

Required $A_s = 0.0095(11)(22) = 2.30 \text{ in.}^2$

Select 2#10 ($A_s = 2.54 \text{ in.}^2$, $b_{\min} = 8.0 \text{ in.}$)



Prob. 2-28 (cont.)

$$A_{s,min} = 0.0033(11)(22) = 0.80 \text{ in.}^2 \text{ (O.K.)}$$

$$\text{Calculated } d = 25 - 1.5 - 0.38 - 1.27/2 = 22.5 \text{ in.} > 22 \text{ in. (O.K.)}$$

Use $b = 11 \text{ in.}$, $h = 25 \text{ in.}$, 2#10 bars

Prob. 2-29 [3/60] Rework Prob. 2-28 w/ addition of an overhanging span.

Assume bm. wt. = 0.287 k/ft from Prob. 2-28.

V_u diag.: $+M_{u(max)} = 167.2 \text{ ft-kips}$

$$-M_{u(max)} = 75 \text{ ft-kips}$$

Design for positive moment:

Try $\rho = 0.0090$, $\bar{k} = 0.4828 \text{ ksi}$ and $d/b \approx 2$.

$$(\epsilon_t > 0.005, \phi = 0.90)$$

$$\text{Required } d = \sqrt[3]{\frac{2(167.2)(12)}{0.90(0.4828)}} = 21 \text{ in. (Use)}$$

Make $b = 11 \text{ in.}$ and $h = 24 \text{ in.}$

$$\text{Beam weight} = \frac{11(24)}{144}(0.150) = 0.275 \text{ kip/ft}$$

0.275 k/ft < 0.287 k/ft (O.K.)

$$\text{Required } \bar{k} = \frac{167.2(12)}{0.90(11)(21)^2} = 0.4596 \text{ ksi}$$

$$\text{req'd. } \rho = 0.0086, \epsilon_t > 0.005, \phi = 0.90$$

$$\text{Required } A_s = 0.0086(11)(21) = 1.99 \text{ in.}^2$$

Select 2#9 ($A_s = 2.00 \text{ in.}^2$, $b_{min} = 7.5 \text{ in.}$)

$$A_{s,min} = 0.0033(11)(21) = 0.76 \text{ in.}^2 \text{ (O.K.)}$$

$$\text{Calculated } d = 24 - 1.5 - 0.38 - 1.13/2 = 21.6 \text{ in.} > 21 \text{ in. (O.K.)}$$

Design for negative moment: Use $b = 11 \text{ in.}$, $h = 24 \text{ in.}$, est. $d = 21 \text{ in.}$

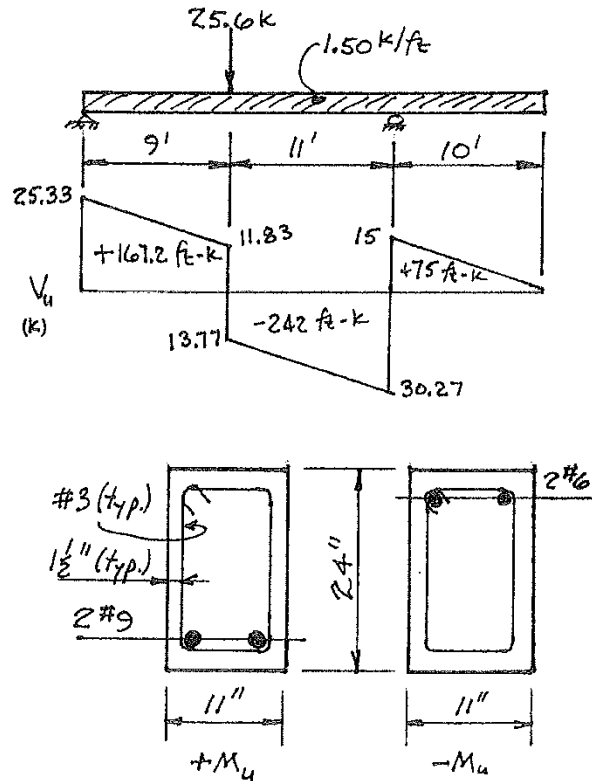
$$\text{Required } \bar{k} = \frac{75(12)}{0.90(11)(21)^2} = 0.2061 \text{ ksi} \quad \text{req'd. } \rho = 0.0036, \epsilon_t > 0.005, \phi = 0.90$$

$$\text{Required } A_s = 0.0036(11)(21) = 0.83 \text{ in.}^2$$

Select 2#6 ($A_s = 0.88 \text{ in.}^2$, $b_{min} = 6.5 \text{ in.}$) $A_{s,min} = 0.0033(11)(21) = 0.76 \text{ in.}^2 \text{ (O.K.)}$

$$\text{Calculated } d = 24 - 1.5 - 0.38 - 0.75/2 = 21.7 \text{ in.} > 21 \text{ in. (O.K.)}$$

See design sketches.



Prob. 2-30 [3/60] One-way slab design for $h_{\min}$. $L = 8$ ft

ACI $h_{\min} = L/20 = [8(12)]/20 = 4.8$ in. Use 5 in.

$$\text{Slab weight} = \frac{5(12)}{144}(0.150) = 62.5 \text{ psf}, \quad w_u = \frac{1.2(62.5) + 1.6(300)}{1000} = 0.555 \text{ kip/ft}$$

$$M_u = \frac{0.555(8)^2}{8} = 4.44 \text{ ft} \cdot \text{kip}$$

Estimated $d = 5 - 0.75 - 0.75/2 = 3.88$ in. (Assume #6 bars)

$$\text{Required } \bar{k} = \frac{4.44(12)}{0.90(12)(3.88)^2} = 0.3277 \text{ ksi}, \quad \rho = 0.0059, \quad \epsilon_t > 0.005, \quad \phi = 0.90$$

Required $A_s = 0.0059(3.88)(12) = 0.27 \text{ in.}^2/\text{ft}$

$A_{s,\min} = 0.0018(5)(12) = 0.11 \text{ in.}^2/\text{ft}$ (O.K.)

Main steel: select #5@14 in. o/c

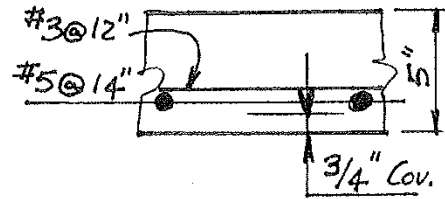
($A_s = 0.27 \text{ in.}^2/\text{ft}$, $s_{\max} = 3h = 15 \text{ in.} \leq 18 \text{ in.}$ (O.K.))

Calc. $d = 5 - 0.75 - 0.625/2 = 3.94 \text{ in.} > 3.88 \text{ in.}$ (O.K.)

($A_{s,\min} =$ required S&T steel)

S&T steel: select #3@12 in. o/c

($A_s = 0.11 \text{ in.}^2/\text{ft}$, $s_{\max} = 5h = 25 \text{ in.} \leq 18 \text{ in.}$ (O.K.))



Prob. 2-31 [3/60] One-way slab design. $L = 10$ ft

(a) Design for $h_{\min}$.

ACI $h_{\min} = L/20 = [10(12)]/20 = 6$ in. Use 6 in.

$$\text{Slab weight} = \frac{6(12)}{144}(0.150) = 75 \text{ psf}, \quad w_u = \frac{1.2(25 + 75) + 1.6(175)}{1000} = 0.400 \text{ kip/ft}$$

$$M_u = \frac{0.400(10)^2}{8} = 5.00 \text{ ft} \cdot \text{kips}$$

Estimated $d = 6 - 0.75 - 0.75/2 = 4.88$ in. (Assume #6 bars)

$$\text{Required } \bar{k} = \frac{5.00(12)}{0.90(12)(4.88)^2} = 0.2333 \text{ ksi}, \quad \rho = 0.0041, \quad \epsilon_t > 0.005, \quad \phi = 0.90$$

Required $A_s = 0.0041(4.88)(12) = 0.24 \text{ in.}^2/\text{ft}$

$A_{s,\min} = 0.0018(6)(12) = 0.13 \text{ in.}^2/\text{ft}$ (O.K.)

Main steel: select #5@15 in. o/c

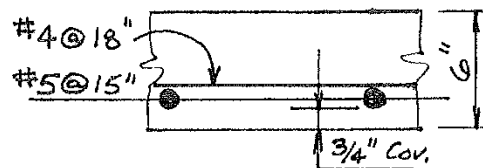
($A_s = 0.25 \text{ in.}^2/\text{ft}$, $s_{\max} = 3h = 18 \text{ in.} \leq 18 \text{ in.}$ (O.K.))

Calc. $d = 6 - 0.75 - 0.625/2 = 4.94 \text{ in.} > 4.88 \text{ in.}$ (O.K.)

($A_{s,\min} =$ required S&T steel)

S&T steel: select #4@18 in. o/c

($A_s = 0.13 \text{ in.}^2/\text{ft}$, $s_{\max} = 5h = 30 \text{ in.} \leq 18 \text{ in.}$ (O.K.))



(b) Design thinnest slab based on moment strength (max. ρ with $\epsilon_t \geq 0.005$)

Estimated slab weight: assume $h = 5$ in. and slab weight = 62.5 psf

$$w_u = \frac{1.2(62.5 + 25) + 1.6(175)}{1000} = 0.385 \text{ k/ft}$$

Prob. 2-31(b) (cont.)

$$M_u = \frac{0.385(10)^2}{8} = 4.81 \text{ ft} \cdot \text{kips}$$

Use $\rho = 0.01355$ ($\bar{k} = 0.6835 \text{ ksi}$, $\epsilon_t = 0.005$, $\phi = 0.90$)

$$\text{Required } d = \sqrt{\frac{4.81(12)}{0.90(12)(0.6835)}} = 2.79 \text{ in.}$$

Est. $h = 2.79 + 0.75/2 + 0.75 = 3.92 \text{ in.}$ (Use 4 in., neglect decrease in D.L.)

Est. $d = 4.00 - 0.75/2 - 0.75 = 2.88 \text{ in.}$

$$\text{Required } \bar{k} = \frac{4.81(12)}{0.90(12)(2.88)^2} = 0.6443 \text{ ksi}, \quad \rho = 0.0127, \quad \epsilon_t > 0.005, \quad \phi = 0.90$$

$$\text{Required } A_s = 0.0127(2.88)(12) = 0.44 \text{ in.}^2/\text{ft}$$

$$A_{s,\min} = 0.0018(4)(12) = 0.09 \text{ in.}^2/\text{ft} \text{ (O.K.)}$$

Main steel: select #6@12 in. o/c

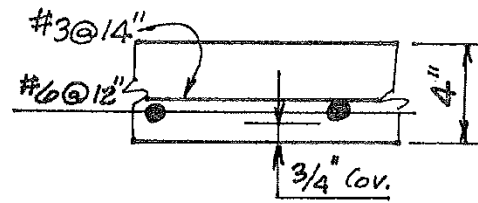
$$(A_s = 0.44 \text{ in.}^2/\text{ft}, \quad s_{\max} = 3h = 12 \text{ in.} \leq 18 \text{ in.} \text{ (O.K.)})$$

$$\text{Calc. } d = 4 - 0.75 - 0.75/2 = 2.88 \text{ in. (O.K.)}$$

$$(A_{s,\min} = \text{required S\&T steel})$$

S\&T steel: select #3@14 in. o/c

$$(A_s = 0.09 \text{ in.}^2/\text{ft}, \quad s_{\max} = 5h = 20 \text{ in.} \leq 18 \text{ in.} \text{ (O.K.)})$$



Prob. 2-32 [3/60] One-way slab design. $L = 13 \text{ ft}$

ACI $h_{\min} = L/20 = [13(12)]/20 = 7.8 \text{ in.}$ Use 8 in.

$$\text{Slab weight} = \frac{8(12)}{144}(0.150) = 100 \text{ psf}, \quad w_u = \frac{1.2(100) + 1.6(200)}{1000} = 0.440 \text{ kip/ft}$$

$$M_u = \frac{0.440(13)^2}{8} = 9.30 \text{ ft} \cdot \text{kip}$$

Estimated $d = 8 - 0.75 - 0.75/2 = 6.88 \text{ in.}$ (Assume #6 bars)

$$\text{Required } \bar{k} = \frac{9.30(12)}{0.90(12)(6.88)^2} = 0.2183 \text{ ksi}, \quad \rho = 0.0039, \quad \epsilon_t > 0.005, \quad \phi = 0.90$$

$$\text{Required } A_s = 0.0039(6.88)(12) = 0.32 \text{ in.}^2/\text{ft}$$

$$A_{s,\min} = 0.0018(8)(12) = 0.17 \text{ in.}^2/\text{ft} \text{ (O.K.)}$$

Main steel: select #6@16 in. o/c

$$(A_s = 0.33 \text{ in.}^2/\text{ft}, \quad s_{\max} = 3h = 24 \text{ in.} \leq 18 \text{ in.} \text{ (O.K.)})$$

$$\text{Calc. } d = 8 - 0.75 - 0.75/2 = 6.88 \text{ in. (O.K.)}$$

$$(A_{s,\min} = \text{required S\&T steel})$$

S\&T steel: select #4@14 in. o/c

$$(A_s = 0.17 \text{ in.}^2/\text{ft}, \quad s_{\max} = 5h = 40 \text{ in.} \leq 18 \text{ in.} \text{ (O.K.)})$$

