UNIVERSITY AT BUFFALO
State University of New York
SCHOOL OF ENGINEERING AND APPLIED SCIENCES
Department of Civil, Structural, and Environmental Engineering
212 Ketter Hall, North Campus, Buffalo, NY 14260-4300 Fax: (716) 645-3733 Tel: (716) 645 2114, X 2400
http://www.civil.buffalo.edu

CIE429 – RC Design

http://civil.eng.buffalo.edu/cie429/

Lecture #15

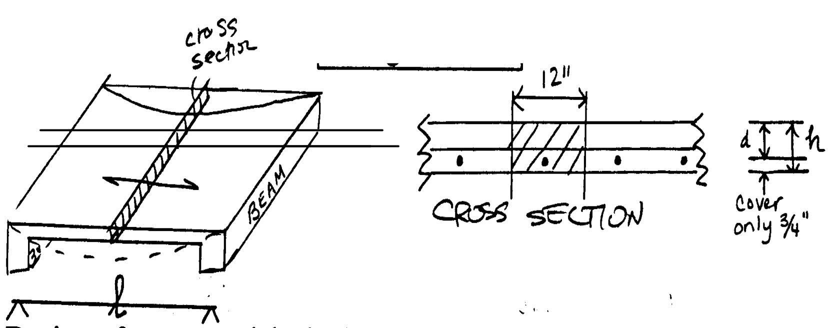
Ribbed (Joists) Slab

Instructor:

Andrei M. Reinhorn *P.E. Ph.D.*

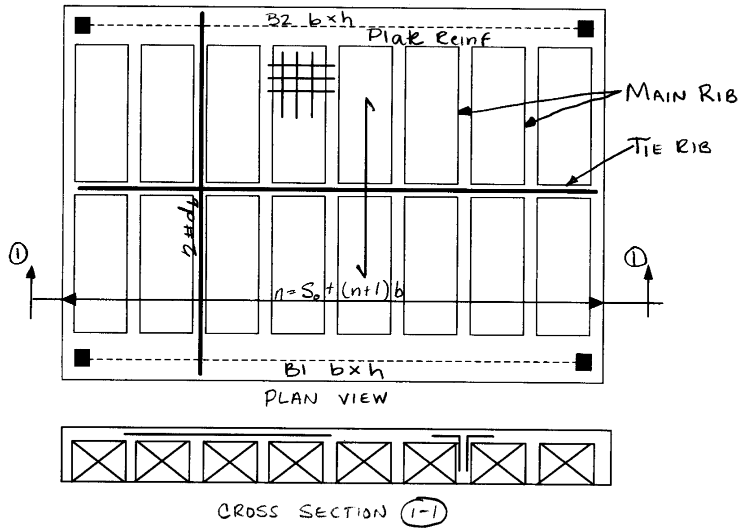
Professor of Structural Engineering

Design of Flat Slabs

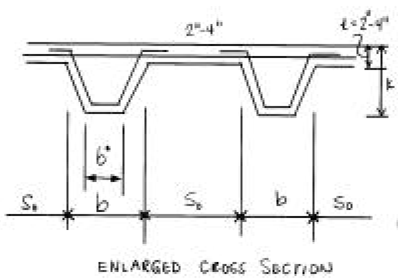


Design of one way slabs is like design of parallel 12" beams.

Slabs - Ribbed Slabs
One way joists



Details of Construction



(1) Continuous flat surface (2"-4" thick)

(2) Ribs spaced at 20" - 30" (face to face). Usually tapered 1:12 for form removal.

(3) Forms (between ribs) either removable boxes or hollowed concrete blocks. (Blocks should be included in weight computation.)

(4) Plate reinforcement is calculated for bending or min. reinforcement for shrinkage (0.2% for GR40 or 60) or (0.18% for GR60 >)

(5) Shear design of ribs:

(a) f_{vc} allowable is 10% higher than for regular beams.

i.e. for short design: $f_{vc} = 1.1 \times 2\sqrt{f_c'} = 2.2\sqrt{f_c'}$

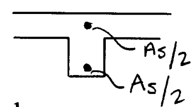
(b) If shear stress exceeds maximum, enlarge width of rib

Details of Construction (2)

(6) Ribs are designed as T sections.
Main positive reinforcement includes at least 2 bars.

(7) Tie beams are required as follows:

(1) One if span L is : 15'-0" ≤ L ≤ 20'-0"
(2) Two if span L is : 20'-0" ≤ L ≤ 30'-0"
(3) Three …..
Add Reinforcement :

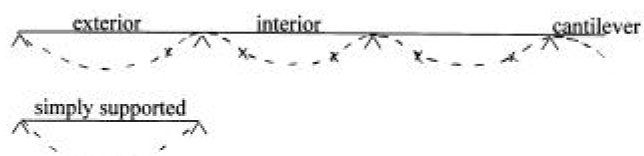


(8) Height of slab is approximated as:

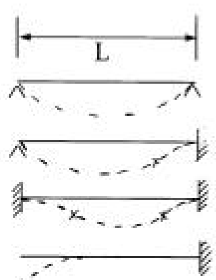
$$L \cong L_{eq}/20 \quad \text{[or ACI 9.5(a)]}$$

Equivalent Span L_{eq}

Note:

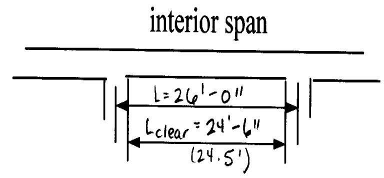


L_{eq} = span between contraflexure points.



simply supported	$L_{eq} = L$
exterior	$L_{eq} = 0.8L$
interior	$L_{eq} = 0.6L$
cantilever	L_{eq} 2.0L

Example of Joists Design



Design slab for an interior span of concrete joist floor using 30" forms; LL = 80 psf ; f_c` = 3 ksi ; f_y = 60 ksi. Use moment coefficients (No redistribution required) (Add 20 psf for ducts, tiles, ceilings)

The design has two steps:

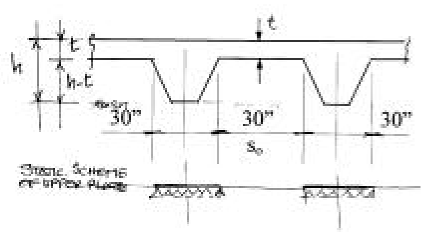
1. Design of the top flat plate supported by the ribs (joists)
2. Design of the joists as T beams (integrally connected with top flat slab

Upper Flat Plate Design

(1) Upper Plate Design: (design as unreinforced section)
Assume t = 3" for weight estimate.

$$M_n = f_r \cdot \frac{bt^2}{6}$$

$$M_d = \phi M_n \quad (\phi = 0.65 \text{ for unreinforced concrete })$$



$$M_d = \frac{w_d s_o^2}{12}$$

find t.

$$w_d = 1.4 \times \left(\frac{3.0}{12} \times 0.150\right) + 1.4 \times 20 + 1.7 \times 80 = 0.22 \text{ ksf}$$

(3.0 assumed)

$$M_d = 0.22 \times \left(\frac{30}{12}\right)^2 \times \frac{1}{12} = 0.115 \text{ ft} \cdot \text{kips}/\text{ft}$$

$$t = \sqrt{6M_d / \phi b f_r} = \sqrt{6 \times (0.115 \times 12000) / 0.65 \times 12 \times 5\sqrt{3000}} = 2.1" \quad \text{use } t = 3"$$

$$A_{s,req} = A_{s,min} = \frac{0.18}{100} \times 3 \times 12 = 0.065 \text{ in}^2/\text{ft} \rightarrow \#3 @ 20"$$

welded wire mesh → 4 × 12 - W2.5 × W1.4
from tables → (A_s = 0.075)

Design of Joists

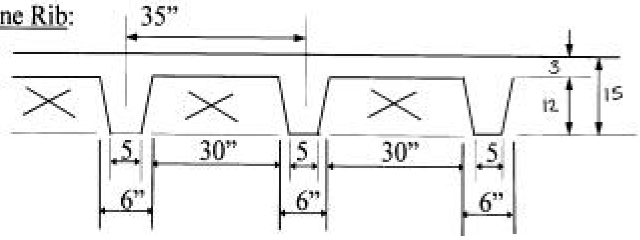
By ACI Table 9.5

$$h_{min} = \frac{L}{21} = \frac{26\times 12}{21} = 14.9 \text{ in} \qquad \text{say } 15"$$

Make joists 12” and slabs t = 3” , b = 5”

(Suggested practical dimensions b = 4”, t = 2” d = 16”)

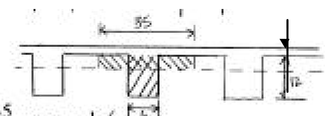
* Design One Rib:



Prof. Andrei M Reinhorn

Design for Flexure

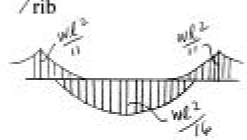
(d = 14”)



$$w_{d,rib} = 1.4\left[3 + \left(\frac{6\times 12}{35}\right) * \frac{.15}{12\times 12}\right] + \left[1.4\times\frac{20}{1000} + 1.7\times\frac{80}{1000}\right] * \frac{35}{12} = 0.744 \ {}^{k}\!/_{ft}$$

Negative Moment

$$M_d = \frac{1}{11}\cdot w_d L^2 = \frac{1}{11}\times 0.744\times 24.5^2 = 40.6 \ {}^{k\cdot ft}\!/_{rib}$$



d = 15” - ¾” - ¼” = 14 ”

cover $\quad {}^{d_s}\!/_2$

$$k_{min} = \frac{200}{0.85 f_c'} = \frac{200}{0.85\times 3000} = 0.078 \ ; \ k_{max} = 0.75\times 0.85\times\frac{87}{87+60} = 0.377$$

$$k = 1 - \sqrt{1 - \frac{2\times 40.6\times {}^{12}\!/_{0.90}}{0.85\times 3\times 5\times 14^2}} = 0.247 \qquad \begin{matrix} < k_{max} \\ > k_{min} \end{matrix} \qquad \text{OK}$$

$$A_{s,req} = \frac{40.6\times {}^{12}\!/_{0.9}}{60\times 14\left(1 - {}^{0.247}\!/_{2}\right)} = 0.74 \text{ in}^2 \rightarrow \begin{matrix} 2\#4 \\ 1\#5 \end{matrix}$$

Prof. Andrei M Reinhorn

Verification of Shear

$$V_d = w_d\left(\frac{L}{2} - d\right) = 0.744\left(12.25 - \left(\tfrac{14}{12}\right)\right) = 8.25 \text{ kips}$$

at d distance
away from support

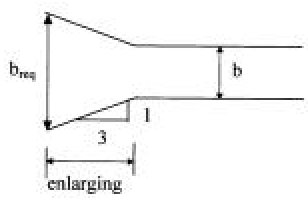
$$f_v = \frac{\left(\frac{8.25}{0.85}\right)}{5 \times 14} = 139 \text{ psi}$$

$$f_{vc,max} = 2.2\sqrt{f_c'} = 120 \text{ psi}$$

$f_v > f_{vc,max}$: (a) reinforcement required.
(b) enlarge rib at its end.

Use alternative (b) :

$$b_{req} = \frac{139}{120} \times 5 = 5.6 \text{ in.} \rightarrow 6"$$



commercial forms:

Design for Flexure

Positive Moment

$$M^+ = \frac{1}{16} w_d L^2 = \frac{1}{16} \times 0.744 \times 24.5 = 27.9 \text{ k-ft}$$

$$B = \min \begin{cases} 16t + b = 16 \times 3 + 5 & = 53 \\ s & = 35 \quad \leftarrow \text{governs} \\ \frac{L}{4} = 26 \times 12/4 & = 78 \end{cases}$$

$$k = 1 - \sqrt{1 - \frac{2 \times 27.9 \times 12/0.90}{0.85 \times 3 \times 35 \times 14^2}} = 0.021 \qquad (< k_{min})$$

$$\bar{t} = 3/14 = 0.21 > 0.021 \qquad \text{neutral axis in flange.}$$

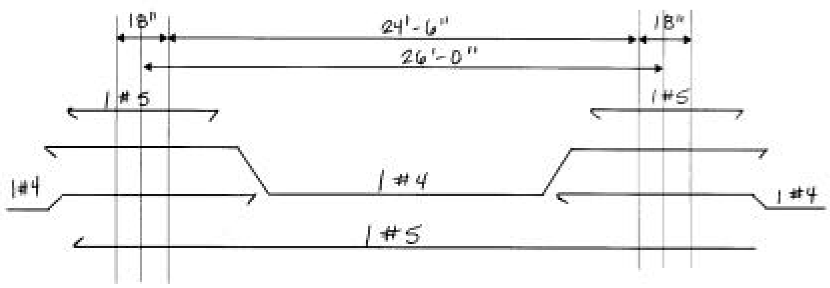
$$A_{s,req} = \frac{27.9 \times 12}{0.90 \times 60 \times 14\left(1 - 0.021/2\right)} = 0.45 \text{ in}^2/\text{rib} \quad \leftarrow 1\ \#4 + 1\ \#5$$

($> A_{s\,min}$) (0.2) (0.31)

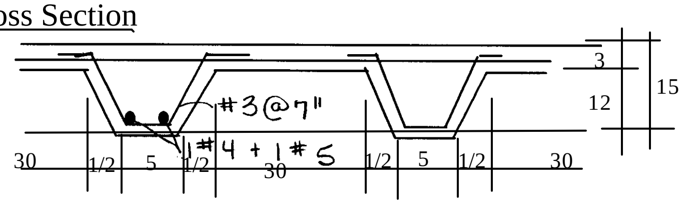
$$A_{s,min} = \frac{200}{f_y} bd = \frac{200}{60000} \times 5 \times 14 = 0.23 \text{ in}^2/\text{rib}$$

note - smaller b 4 × 12 or W2.5 × W1.25

Reinforcement



Cross Section



Area of Reinforcing Bars

Table 3-6 - Total Areas of Bars - A_s (in^2)

BAR SIZE	BAR DIAMETER	NUMBER OF BARS							
		1	2	3	4	5	6	7	8
# 3	0.375	0.11	0.22	0.33	0.44	0.55	0.66	0.77	0.88
# 4	0.500	0.20	0.40	0.60	0.80	1.00	1.20	1.40	1.60
# 5	0.625	0.31	0.62	0.93	1.24	1.55	1.96	2.17	2.48
# 6	0.750	0.44	0.88	1.32	1.76	2.20	2.64	3.08	3.52
# 7	0.875	0.60	1.20	1.80	2.40	3.00	3.60	4.20	4.80
# 8	1.000	0.79	1.58	2.37	3.16	3.95	4.74	5.53	6.32
# 9	1.128	1.00	2.00	3.00	4.00	5.00	6.00	7.00	8.00
#10	1.270	1.27	2.54	3.81	5.08	6.35	7.62	8.89	10.16
#11	1.410	1.56	3.12	4.68	6.24	7.80	9.36	10.92	12.48