

# **Chapter 13**

## **Concrete Form Design**

# Concrete Form Design

- SLAB FORM DESIGN Method
- <https://www.youtube.com/watch?v=jggeUUbPHZs>
- <https://www.youtube.com/watch?v=uGU8xgJykO0>

# Formwork Design

- **Floor and Roof formwork Design:**

The design load that acts on the slab form consist of :

- self-weight of the reinforced slab plus
- the live load and,
- the weight of the formwork themselves.

❖ The American concrete institute (ACI) recommended a **minimum live load** of:

2.4kpa

**In case of motorized concrete buggies are used**

3.6kpa

❖ ACI recommended a minimum **design load** (dead plus live) :

4.8kpa

**In case of motorized concrete buggies are used :**

6.0 kpa

## ➤ **Design steps:**

- ❑ Selecting the design load.
- ❑ Analyzing the sheeting, joist and stringers as beam under uniformly distributed load supported over one of the three conditions (single span - two spans – three spans or larger).
- ❑ Determining the allowable span for slab from table 13-5 & 13-5A by considering the smallest span based on the value of bending, shear and deflection.

Design the sheathing.

Design the joist.

Design the stringers.

Check the stringer spans and shore capacity.

Check the crushing between joist and stringer.

Maximum bending moment, shear force and deflection developed by uniformly distributed load can be obtained from table below:

**Table 13-4** Maximum bending, shear, and deflection in a uniformly loaded beam

Type	Support Conditions		
	1 Span	2 Spans	3 Spans
Bending moment (in.-lb)	$M = \frac{wl^2}{96}$	$M = \frac{wl^2}{96}$	$M = \frac{wl^2}{120}$
Shear (lb)	$V = \frac{wl}{24}$	$V = \frac{5wl}{96}$	$V = \frac{wl}{20}$
Deflection (in.)	$\Delta = \frac{5wl^4}{4608EI}$	$\Delta = \frac{wl^4}{2220EI}$	$\Delta = \frac{wl^4}{1740EI}$

*Notation:*

$l$  = length of span (in.)

$w$  = uniform load per foot of span (lb/ft)

$E$  = modulus of elasticity (psi)

$I$  = moment of inertia (in.<sup>4</sup>)

### **Bending**

$$f_b = \frac{M}{S} \quad (13-5)$$

### **Shear**

$$f_v = \frac{1.5V}{A} \text{ for rectangular wood members} \quad (13-6)$$

$$f_v = \frac{V}{Ib/Q} \text{ for plywood} \quad (13-7)$$

### **Compression**

$$f_c \text{ or } f_{c\perp} = \frac{P}{A} \quad (13-8)$$

### **Tension**

$$f_t = \frac{P}{A} \quad (13-9)$$

where  $f_b$  = actual unit stress for extreme fiber in bending (psi)  
 $f_c$  = actual unit stress in compression parallel to grain (psi)  
 $f_{c\perp}$  = actual unit stress in compression perpendicular to grain (psi)  
 $f_t$  = actual unit stress in tension (psi)



$f_v$  = actual unit stress in horizontal shear (psi)

$A$  = section area (sq in.)

$M$  = maximum moment (in.-lb)

$P$  = concentrated load (lb)

$S$  = section modulus (cu in.)

$V$  = maximum shear (lb)

$Ib/Q$  = rolling shear constant (sq in./ft)

The maximum fiber stress developed in bending, shear and compression resulting from a specified load can be determined from the upper equations.

**Table 13-8** Typical values of allowable stress for lumber

Species (No. 2 Grade, 4 × 4 [100 × 100 mm] or smaller)	Allowable Unit Stress (lb/sq in.)(kPa) (Moisture Content = 19%)					
	$F_b$	$F_v$	$F_{c\perp}$	$F_c$	$F_t$	$E$
Douglas fir—larch	1450 [9998]	185 [1276]	385 [2655]	1000 [6895]	850 [5861]	$1.7 \times 10^6$ [ $11.7 \times 10^6$ ]
Hemlock—fir	1150 [7929]	150 [1034]	245 [1689]	800 [5516]	675 [4654]	$1.4 \times 10^6$ [ $9.7 \times 10^6$ ]
Southern pine	1400 [9653]	180 [1241]	405 [2792]	975 [6723]	825 [5688]	$1.6 \times 10^6$ [ $11.0 \times 10^6$ ]
California redwood	1400 [9653]	160 [1103]	425 [2930]	1000 [6895]	800 [5516]	$1.3 \times 10^6$ [ $9.0 \times 10^6$ ]
Eastern spruce	1050 [7240]	140 [965]	255 [1758]	700 [4827]	625 [4309]	$1.2 \times 10^6$ [ $8.3 \times 10^6$ ]
Reduction factor for wet conditions	0.86	0.97	0.67	0.70	0.84	0.97
Load duration factor (7-day load)	1.25	1.25	1.25	1.25	1.25	1.00

**Table 13-5A** Metric (SI) concrete form design equations

Design Conditions	Support Conditions		
	1 Span	2 Spans	3 or More Spans
<b>Bending</b>			
Wood	$\ell = \frac{36.5}{1000} d \left( \frac{F_b b}{w} \right)^{1/2}$	$\ell = \frac{36.5}{1000} d \left( \frac{F_b b}{w} \right)^{1/2}$	$\ell = \frac{40.7}{1000} d \left( \frac{F_b b}{w} \right)^{1/2}$
	$\ell = \frac{89.9}{1000} \left( \frac{F_b S}{w} \right)^{1/2}$	$\ell = \frac{89.9}{1000} \left( \frac{F_b S}{w} \right)^{1/2}$	$\ell = \frac{100}{1000} \left( \frac{F_b S}{w} \right)^{1/2}$
Plywood	$\ell = 2.83 \left( \frac{F_b K S}{w} \right)^{1/2}$	$\ell = 2.83 \left( \frac{F_b K S}{w} \right)^{1/2}$	$\ell = 3.16 \left( \frac{F_b K S}{w} \right)^{1/2}$
<b>Shear</b>			
Wood	$\ell = \frac{1.34}{1000} \frac{F_v A}{w} + 2d$	$\ell = \frac{1.07}{1000} \frac{F_v A}{w} + 2d$	$\ell = \frac{1.11}{1000} \frac{F_v A}{w} + 2d$
Plywood	$\ell = 2.00 \frac{F_s I b / Q}{w} + 2d$	$\ell = 1.60 \frac{F_s I b / Q}{w} + 2d$	$\ell = 1.67 \frac{F_s I b / Q}{w} + 2d$
<b>Deflection</b>			
	$\ell = \frac{526}{1000} \left( \frac{EI \Delta}{w} \right)^{1/4}$	$\ell = \frac{655}{1000} \left( \frac{EI \Delta}{w} \right)^{1/4}$	$\ell = \frac{617}{1000} \left( \frac{EI \Delta}{w} \right)^{1/4}$
If $\Delta = \frac{1}{180}$	$\ell = \frac{75.1}{1000} \left( \frac{EI}{w} \right)^{1/3}$	$\ell = \frac{101}{1000} \left( \frac{EI}{w} \right)^{1/3}$	$\ell = \frac{93.0}{1000} \left( \frac{EI}{w} \right)^{1/3}$
If $\Delta = \frac{1}{240}$	$\ell = \frac{68.5}{1000} \left( \frac{EI}{w} \right)^{1/3}$	$\ell = \frac{91.7}{1000} \left( \frac{EI}{w} \right)^{1/3}$	$\ell = \frac{84.7}{1000} \left( \frac{EI}{w} \right)^{1/3}$
If $\Delta = \frac{1}{360}$	$\ell = \frac{59.8}{1000} \left( \frac{EI}{w} \right)^{1/3}$	$\ell = \frac{79.9}{1000} \left( \frac{EI}{w} \right)^{1/3}$	$\ell = \frac{73.8}{1000} \left( \frac{EI}{w} \right)^{1/3}$
Compression	$f_c \text{ or } f_{c\perp} = \frac{P}{A}$		
Tension	$f_t = \frac{P}{A}$		

**Table 13-7** Section properties of U.S. standard lumber and timber ( $b$  = width,  $d$  = depth)

Nominal Size ( $b \times d$ )	Actual Size (S4S)		Area of Section $A$		Section Modulus $S$		Moment of Inertia $I$	
	<i>in.</i>	<i>mm</i>	<i>in.</i> <sup>2</sup>	$10^3$ <i>mm</i> <sup>2</sup>	<i>in.</i> <sup>3</sup>	$10^5$ <i>mm</i> <sup>3</sup>	<i>in.</i> <sup>4</sup>	$10^6$ <i>mm</i> <sup>4</sup>
	<i>in.</i>	<i>mm</i>	<i>in.</i> <sup>2</sup>	$10^3$ <i>mm</i> <sup>2</sup>	<i>in.</i> <sup>3</sup>	$10^5$ <i>mm</i> <sup>3</sup>	<i>in.</i> <sup>4</sup>	$10^6$ <i>mm</i> <sup>4</sup>
1 × 3	0.75 × 2.5	19 × 64	1.875	1.210	0.7812	0.1280	0.9766	0.4065
1 × 4	0.75 × 3.5	19 × 89	2.625	1.694	1.531	0.2509	2.680	1.115
1 × 6	0.75 × 5.5	19 × 140	4.125	2.661	3.781	0.6196	10.40	4.328
1 × 8	0.75 × 7.25	19 × 184	5.438	3.508	6.570	1.077	23.82	9.913
1 × 10	0.75 × 9.25	19 × 235	6.938	4.476	10.70	1.753	49.47	20.59
1 × 12	0.75 × 11.25	19 × 286	8.438	5.444	15.82	2.592	88.99	37.04
2 × 3	1.5 × 2.5	38 × 64	3.750	2.419	1.563	0.2561	1.953	0.8129
2 × 4	1.5 × 3.5	38 × 89	5.250	3.387	3.063	0.5019	5.359	2.231
2 × 6	1.5 × 5.5	38 × 140	8.250	5.323	7.563	1.239	20.80	8.656
2 × 8	1.5 × 7.25	38 × 184	10.88	7.016	13.14	2.153	47.63	19.83
2 × 10	1.5 × 9.25	38 × 235	13.88	8.952	21.39	3.505	98.93	41.18
2 × 12	1.5 × 11.25	38 × 286	16.88	10.89	31.64	5.185	178.0	74.08
2 × 14	1.5 × 13.25	38 × 337	19.88	12.82	43.89	7.192	290.8	121.0
3 × 4	2.5 × 3.5	64 × 89	8.750	5.645	5.104	0.8364	8.932	3.718
3 × 6	2.5 × 5.5	64 × 140	13.75	8.871	12.60	2.065	34.66	14.43
3 × 8	2.5 × 7.25	64 × 184	18.12	11.69	21.90	3.589	79.39	33.04
3 × 10	2.5 × 9.25	64 × 235	23.12	14.91	35.65	5.842	164.9	68.63
3 × 12	2.5 × 11.25	64 × 286	28.12	18.14	52.73	8.642	296.6	123.5
3 × 14	2.5 × 13.25	64 × 337	33.12	21.37	73.15	11.99	484.6	201.7
3 × 16	2.5 × 15.25	64 × 387	38.12	24.60	96.90	15.88	738.9	307.5
4 × 4	3.5 × 3.5	89 × 89	12.25	7.903	7.146	1.171	12.50	5.205
4 × 6	3.5 × 5.5	89 × 140	19.25	12.42	17.65	2.892	48.53	20.20
4 × 8	3.5 × 7.25	89 × 184	25.38	16.37	30.66	5.024	111.1	46.26
4 × 10	3.5 × 9.25	89 × 235	32.38	20.89	49.91	8.179	230.8	96.08
4 × 12	3.5 × 11.25	89 × 286	39.38	25.40	73.83	12.10	415.3	172.8
4 × 14	3.5 × 13.25	89 × 337	46.38	29.92	102.4	16.78	678.5	282.4
4 × 16	3.5 × 15.25	89 × 387	53.38	34.43	135.7	22.23	1034	430.6
6 × 6	5.5 × 5.5	140 × 140	30.25	19.52	27.73	4.543	76.25	19.52
6 × 8	5.5 × 7.5	140 × 191	41.25	26.61	51.56	8.450	193.4	80.48
6 × 10	5.5 × 9.5	140 × 241	52.25	33.71	82.73	13.56	393.0	163.6
6 × 12	5.5 × 11.5	140 × 292	63.25	40.81	121.2	19.87	697.1	290.1
6 × 14	5.5 × 13.5	140 × 343	74.25	47.90	167.1	27.38	1128	469.4
6 × 16	5.5 × 15.5	140 × 394	85.25	55.00	220.2	36.09	1707	710.4

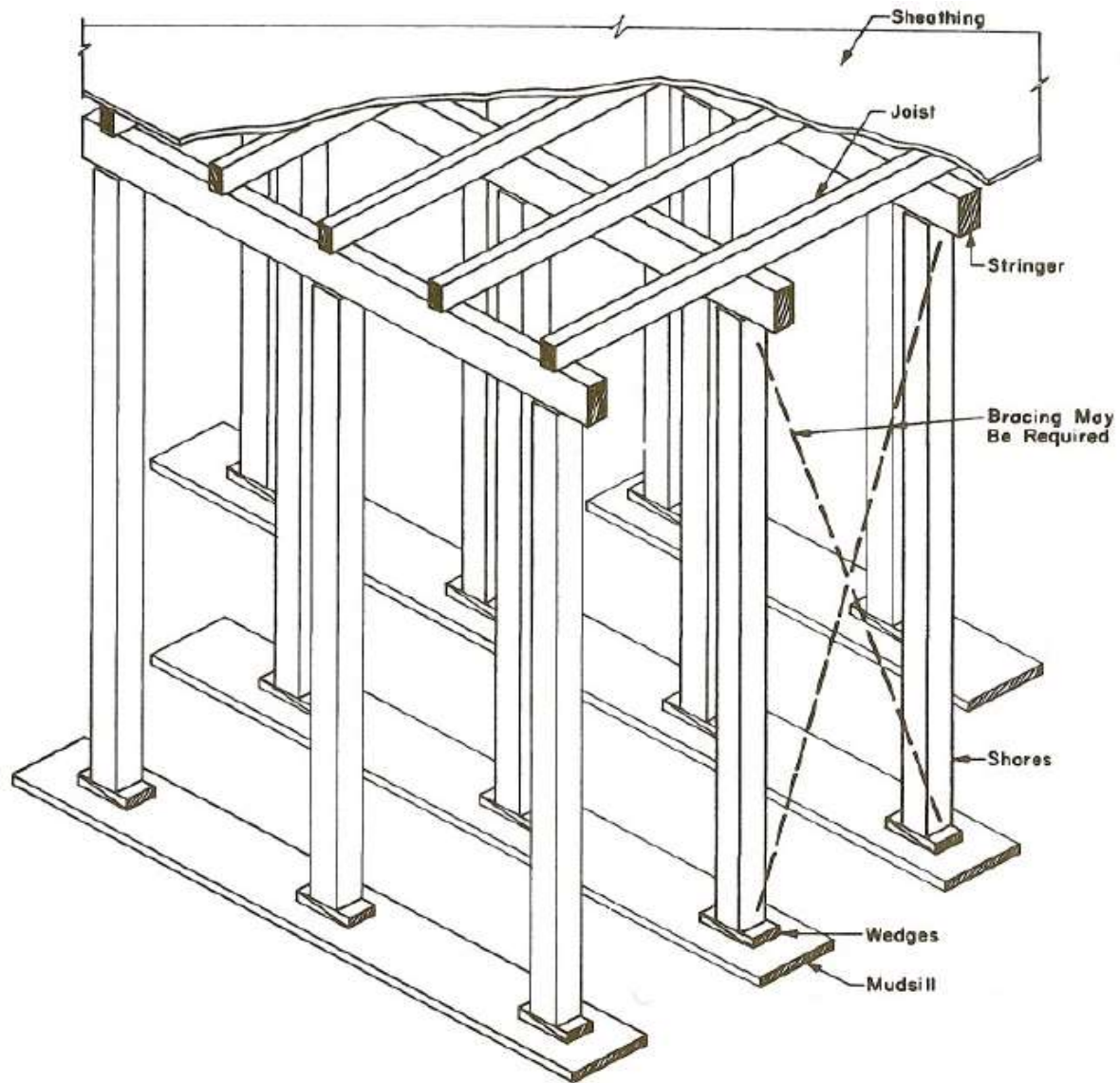
# EXAMPLE 13-1

- Design the formwork (Figure 13-2) for an elevated concrete floor slab 6 in. (152 mm) thick.
- Sheathing will be nominal 1 in. (25-mm) lumber while 2 x 8 in. (50 x 200 mm) lumber will be used for joists.
- Stringers will be 4 x 8 in. (100 x 200 mm) lumber.
- Assume that all members are continuous over three or more spans.
- Commercial 4000-lb (17.8-kN) shores will be used.
- It is estimated that the weight of the formwork will be 5 lb/sq ft (0.24 kPa).
- The adjusted allowable stresses for the lumber being used are as follows:

# EXAMPLE 13-1

	Sheathing psi [kPa]	Other Members psi [kPa]
$F_b$	1075 [7412]	1250 [8619]
$F_v$	174 [1200]	180 [1241]
$F_{c\perp}$		405 [2792]
$F_c$		850 [5861]
$E$	$1.36 \times 10^6$ [ $9.4 \times 10^6$ ]	$1.40 \times 10^6$ [ $9.7 \times 10^6$ ]

- Maximum deflection of form members will be limited to  $l/360$ .
- Use the minimum value of live load permitted by ACI.
- Determine joist spacing, stringer spacing, and shore spacing.



# Solution

**Design Load.** Assume concrete density is 150 lb/cu ft (2403 kg/m<sup>3</sup>)

$$\text{Concrete} = 1 \text{ sq ft} \times 6/12 \text{ ft} \times 150 \text{ lb/cu ft} = 75 \text{ lb/sq ft}$$

$$\text{Formwork} = 5 \text{ lb/sq ft}$$

$$\text{Live load} = \underline{50 \text{ lb/sq ft}}$$

$$\text{Design load} = 130 \text{ lb/sq ft}$$

Pressure per m<sup>2</sup>:

$$\text{Concrete} = 1 \times 0.152 \times 9.8 \times 2403/1000 = 3.58 \text{ kPa}$$

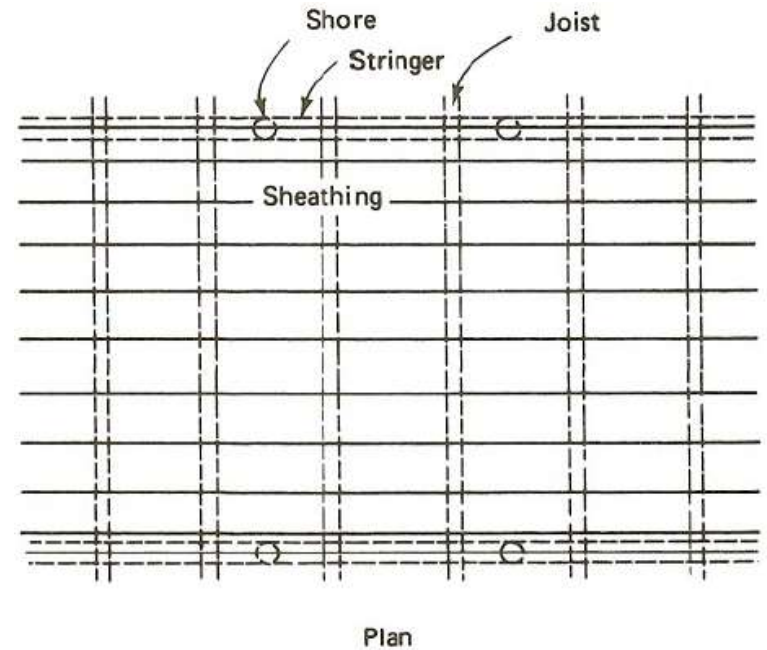
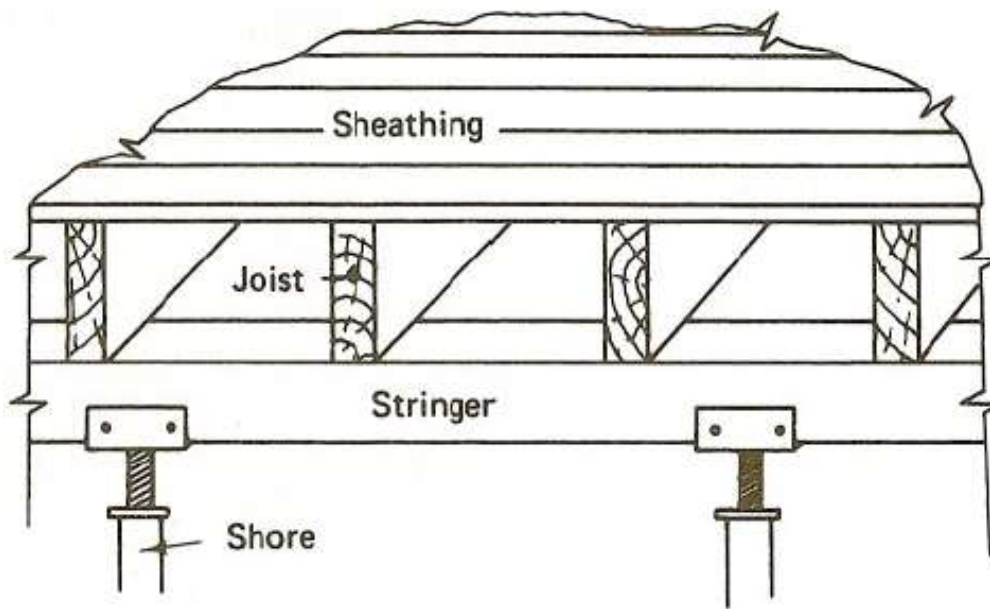
$$\text{Formwork} = 0.24 \text{ kPa}$$

$$\text{Live load} = \underline{2.40 \text{ kPa}}$$

$$\text{Design load} = 6.22 \text{ kPa}$$



# Figure 13-2 Slab form



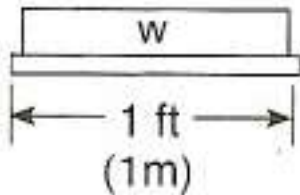
# Deck Design

- *Consider a uniformly loaded strip of decking (sheathing) 12 in.*
- (or 1 m) wide placed perpendicular to the joists (Figure 13-1a) and analyze it as a beam.
- Assume that the strip is continuous over three or more spans and use the appropriate equations of Table 13-5 and 13-5A.
- $w = (1 \text{ sq ft/lin ft}) \times (130 \text{ lb/sq ft}) = 130 \text{ lb/ft}$
- $[w = (1 \text{ m}^2/\text{lin m}) \times (6.21 \text{ kN/m}^2) = 6.21 \text{ kN/m}]$

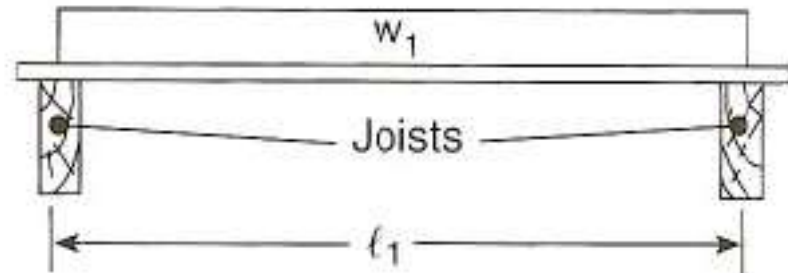
# Figure 13-1

## Design Analysis for form member

Section



Elevation



$w$  = design load (lb/sq ft) [kN/m<sup>2</sup>]

$w_1 = 1 \times w = w$  (lb/ft) [kN/m]

a. Sheathing

# Deck Design

(a) Bending:

$$l = 4.46 d \left( \frac{F_b b}{w} \right)^{1/2}$$
$$= (4.46) (0.75) \left( \frac{(1075) (12)}{130} \right)^{1/2} = 33.3 \text{ in.}$$

$$\left[ \begin{aligned} l &= \frac{40.7}{1000} d \left( \frac{F_b b}{w} \right)^{1/2} \\ &= \frac{(40.7) (19)}{1000} \left( \frac{(7412) (1000)}{6.22} \right)^{1/2} = 844 \text{ mm} \end{aligned} \right]$$

**Table 13-5A** Metric (SI) concrete form design equations

Design Conditions	Support Conditions		
	1 Span	2 Spans	3 or More Spans
<b>Bending</b>			
Wood	$\ell = \frac{36.5}{1000} d \left( \frac{F_b b}{w} \right)^{1/2}$	$\ell = \frac{36.5}{1000} d \left( \frac{F_b b}{w} \right)^{1/2}$	$\ell = \frac{40.7}{1000} d \left( \frac{F_b b}{w} \right)^{1/2}$
	$\ell = \frac{89.9}{1000} \left( \frac{F_b S}{w} \right)^{1/2}$	$\ell = \frac{89.9}{1000} \left( \frac{F_b S}{w} \right)^{1/2}$	$\ell = \frac{100}{1000} \left( \frac{F_b S}{w} \right)^{1/2}$
Plywood	$\ell = 2.83 \left( \frac{F_b KS}{w} \right)^{1/2}$	$\ell = 2.83 \left( \frac{F_b KS}{w} \right)^{1/2}$	$\ell = 3.16 \left( \frac{F_b KS}{w} \right)^{1/2}$
<b>Shear</b>			
Wood	$\ell = \frac{1.34}{1000} \frac{F_v A}{w} + 2d$	$\ell = \frac{1.07}{1000} \frac{F_v A}{w} + 2d$	$\ell = \frac{1.11}{1000} \frac{F_v A}{w} + 2d$
Plywood	$\ell = 2.00 \frac{F_s I b / Q}{w} + 2d$	$\ell = 1.60 \frac{F_s I b / Q}{w} + 2d$	$\ell = 1.67 \frac{F_s I b / Q}{w} + 2d$
<b>Deflection</b>			
	$\ell = \frac{526}{1000} \left( \frac{EI \Delta}{w} \right)^{1/4}$	$\ell = \frac{655}{1000} \left( \frac{EI \Delta}{w} \right)^{1/4}$	$\ell = \frac{617}{1000} \left( \frac{EI \Delta}{w} \right)^{1/4}$
If $\Delta = \frac{1}{180}$	$\ell = \frac{75.1}{1000} \left( \frac{EI}{w} \right)^{1/3}$	$\ell = \frac{101}{1000} \left( \frac{EI}{w} \right)^{1/3}$	$\ell = \frac{93.0}{1000} \left( \frac{EI}{w} \right)^{1/3}$
If $\Delta = \frac{1}{240}$	$\ell = \frac{68.5}{1000} \left( \frac{EI}{w} \right)^{1/3}$	$\ell = \frac{91.7}{1000} \left( \frac{EI}{w} \right)^{1/3}$	$\ell = \frac{84.7}{1000} \left( \frac{EI}{w} \right)^{1/3}$
If $\Delta = \frac{1}{360}$	$\ell = \frac{59.8}{1000} \left( \frac{EI}{w} \right)^{1/3}$	$\ell = \frac{79.9}{1000} \left( \frac{EI}{w} \right)^{1/3}$	$\ell = \frac{73.8}{1000} \left( \frac{EI}{w} \right)^{1/3}$
Compression	$f_c \text{ or } f_{c\perp} = \frac{P}{A}$		
Tension	$f_t = \frac{P}{A}$		

# Deck Design

(b) Shear:

$$l = 13.3 \frac{F_v A}{w} + 2d$$
$$= \frac{(13.3) (174) (12) (0.75)}{130} + (2) (0.75) = 161.7 \text{ in.}$$

$$\left[ \begin{aligned} l &= \frac{1.11}{1000} \frac{F_v A}{w} + 2d \\ &= \frac{(1.11) (1200) (1000) (19)}{(1000) (6.22)} + (2) (19) = 4107 \text{ mm} \end{aligned} \right]$$

# Deck Design

(c) Deflection:

$$l = 1.69 \left( \frac{EI}{w} \right)^{1/3} = 1.69 \left( \frac{Ebd^3}{w 12} \right)^{1/3}$$
$$= 1.69 \left( \frac{(1.36 \times 10^6) (12) (0.75)^3}{(130) (12)} \right)^{1/3} = 27.7 \text{ in.}$$

$$\left[ \begin{aligned} l &= \frac{73.8}{1000} \left( \frac{EI}{w} \right)^{1/3} = \frac{73.8}{1000} \left( \frac{Ebd^3}{w 12} \right)^{1/3} \\ &= \frac{73.8}{1000} \left( \frac{(9.4 \times 10^6) (1000) (19)^3}{(12) (6.22)} \right)^{1/3} = 703 \text{ mm} \end{aligned} \right]$$

- Deflection governs in this case and the maximum allowable span is 27.7 in. (703 mm).
- We will select a 24-in. (610-mm) joist spacing as a modular value for the design.

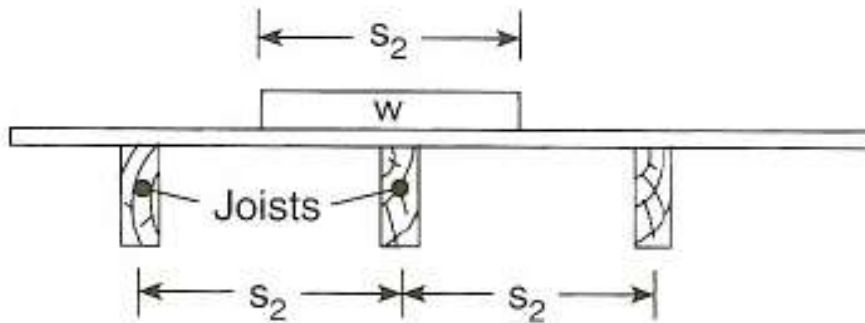


# Joist design

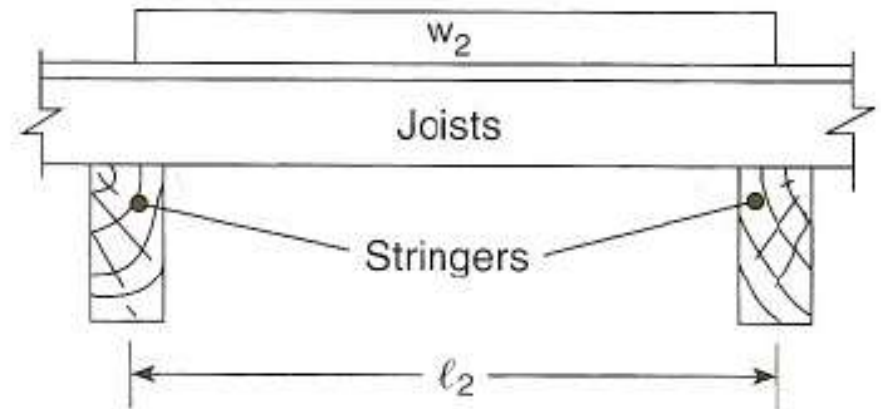
- *Consider the joist as a uniformly loaded beam supporting a strip of design load 24 in. (610 mm) wide (same as joist spacing; see Figure 13-1b).*
- Joists are 2 x 8 in. (50 x 200 mm) lumber.
- Assume that the joists are continuous over three spans.
- $w = (2 \text{ ft}) \times (1) \times (130 \text{ lb/sq ft}) = 260 \text{ lb/ft}$
- $[ w = (0.610 \text{ m}) \times (1) \times (6.22 \text{ kPa}) = 3.79 \text{ kN/m} ]$

# Figure 13-1

## Design Analysis for form member



$s_2$  = spacing of joists (ft) [m]  
 $w_2$  =  $w \times s_2$  (lb/ft) [kN/m]



b. Joists

# Joist design

(a) Bending:

$$l = 10.95 \left( \frac{F_b S}{w} \right)^{1/2}$$
$$= 10.95 \left( \frac{(1250)(13.14)}{260} \right)^{1/2} = 87.0 \text{ in.}$$

$$\left[ \begin{aligned} l &= \frac{100}{1000} \left( \frac{F_b S}{w} \right)^{1/2} \\ &= \frac{100}{1000} \left( \frac{(8619)(2.153 \times 10^5)}{3.79} \right)^{1/2} = 2213 \text{ mm} \end{aligned} \right]$$

**Table 13-7** Section properties of U.S. standard lumber and timber ( $b$  = width,  $d$  = depth)

Nominal Size ( $b \times d$ )	Actual Size (S4S)		Area of Section $A$		Section Modulus $S$		Moment of Inertia $I$	
	<i>in.</i>	<i>mm</i>	<i>in.</i> <sup>2</sup>	$10^3$ <i>mm</i> <sup>2</sup>	<i>in.</i> <sup>3</sup>	$10^5$ <i>mm</i> <sup>3</sup>	<i>in.</i> <sup>4</sup>	$10^6$ <i>mm</i> <sup>4</sup>
1 × 3	0.75 × 2.5	19 × 64	1.875	1.210	0.7812	0.1280	0.9766	0.4065
1 × 4	0.75 × 3.5	19 × 89	2.625	1.694	1.531	0.2509	2.680	1.115
1 × 6	0.75 × 5.5	19 × 140	4.125	2.661	3.781	0.6196	10.40	4.328
1 × 8	0.75 × 7.25	19 × 184	5.438	3.508	6.570	1.077	23.82	9.913
1 × 10	0.75 × 9.25	19 × 235	6.938	4.476	10.70	1.753	49.47	20.59
1 × 12	0.75 × 11.25	19 × 286	8.438	5.444	15.82	2.592	88.99	37.04
2 × 3	1.5 × 2.5	38 × 64	3.750	2.419	1.563	0.2561	1.953	0.8129
2 × 4	1.5 × 3.5	38 × 89	5.250	3.387	3.063	0.5019	5.359	2.231
2 × 6	1.5 × 5.5	38 × 140	8.250	5.323	7.563	1.239	20.80	8.656
2 × 8	1.5 × 7.25	38 × 184	10.88	7.016	13.14	2.153	47.63	19.83
2 × 10	1.5 × 9.25	38 × 235	13.88	8.952	21.39	3.505	98.93	41.18
2 × 12	1.5 × 11.25	38 × 286	16.88	10.89	31.64	5.185	178.0	74.08
2 × 14	1.5 × 13.25	38 × 337	19.88	12.82	43.89	7.192	290.8	121.0
3 × 4	2.5 × 3.5	64 × 89	8.750	5.645	5.104	0.8364	8.932	3.718
3 × 6	2.5 × 5.5	64 × 140	13.75	8.871	12.60	2.065	34.66	14.43
3 × 8	2.5 × 7.25	64 × 184	18.12	11.69	21.90	3.589	79.39	33.04
3 × 10	2.5 × 9.25	64 × 235	23.12	14.91	35.65	5.842	164.9	68.63
3 × 12	2.5 × 11.25	64 × 286	28.12	18.14	52.73	8.642	296.6	123.5
3 × 14	2.5 × 13.25	64 × 337	33.12	21.37	73.15	11.99	484.6	201.7
3 × 16	2.5 × 15.25	64 × 387	38.12	24.60	96.90	15.88	738.9	307.5
4 × 4	3.5 × 3.5	89 × 89	12.25	7.903	7.146	1.171	12.50	5.205
4 × 6	3.5 × 5.5	89 × 140	19.25	12.42	17.65	2.892	48.53	20.20
4 × 8	3.5 × 7.25	89 × 184	25.38	16.37	30.66	5.024	111.1	46.26
4 × 10	3.5 × 9.25	89 × 235	32.38	20.89	49.91	8.179	230.8	96.08
4 × 12	3.5 × 11.25	89 × 286	39.38	25.40	73.83	12.10	415.3	172.8
4 × 14	3.5 × 13.25	89 × 337	46.38	29.92	102.4	16.78	678.5	282.4
4 × 16	3.5 × 15.25	89 × 387	53.38	34.43	135.7	22.23	1034	430.6
6 × 6	5.5 × 5.5	140 × 140	30.25	19.52	27.73	4.543	76.25	19.52
6 × 8	5.5 × 7.5	140 × 191	41.25	26.61	51.56	8.450	193.4	80.48
6 × 10	5.5 × 9.5	140 × 241	52.25	33.71	82.73	13.56	393.0	163.6
6 × 12	5.5 × 11.5	140 × 292	63.25	40.81	121.2	19.87	697.1	290.1
6 × 14	5.5 × 13.5	140 × 343	74.25	47.90	167.1	27.38	1128	469.4
6 × 16	5.5 × 15.5	140 × 394	85.25	55.00	220.2	36.09	1707	710.4

**Table 13-7 (Continued)**

Nominal Size ( <i>b</i> × <i>d</i> )	Actual Size (S4S)		Area of Section <i>A</i>		Section Modulus <i>S</i>		Moment of Inertia <i>I</i>	
	<i>in.</i>	<i>mm</i>	<i>in.</i> <sup>2</sup>	<i>10</i> <sup>3</sup> <i>mm</i> <sup>2</sup>	<i>in.</i> <sup>3</sup>	<i>10</i> <sup>5</sup> <i>mm</i> <sup>3</sup>	<i>in.</i> <sup>4</sup>	<i>10</i> <sup>6</sup> <i>mm</i> <sup>4</sup>
6 × 18	5.5 × 17.5	140 × 445	96.25	62.10	280.7	46.00	2456	1022
6 × 20	5.5 × 19.5	140 × 495	107.2	69.19	348.6	57.12	3398	1415
6 × 22	5.5 × 21.5	140 × 546	118.2	76.29	423.7	69.44	4555	1896
6 × 24	5.5 × 23.5	140 × 597	129.2	83.39	506.2	82.96	5948	2476
8 × 8	7.5 × 7.5	191 × 191	56.25	36.29	70.31	11.52	263.7	109.8
8 × 10	7.5 × 9.5	191 × 241	71.25	45.97	112.8	18.49	535.9	223.0
8 × 12	7.5 × 11.5	191 × 292	86.25	55.65	165.3	27.09	950.5	395.7
8 × 14	7.5 × 13.5	191 × 343	101.2	65.32	227.8	37.33	1538	640.1
8 × 16	7.5 × 15.5	191 × 394	116.2	75.00	300.3	49.21	2327	968.8
8 × 18	7.5 × 17.5	191 × 445	131.2	84.68	382.8	62.73	3350	1394
8 × 20	7.5 × 19.5	191 × 495	146.2	94.36	475.3	77.89	4634	1929
8 × 22	7.5 × 21.5	191 × 546	161.2	104.0	577.8	94.69	6211	2585
8 × 24	7.5 × 23.5	191 × 597	176.2	113.7	690.3	113.1	8111	3376
10 × 10	9.5 × 9.5	241 × 241	90.25	58.23	142.9	23.42	678.8	282.5
10 × 12	9.5 × 11.5	241 × 292	109.2	70.48	209.4	34.31	1204	501.2
10 × 14	9.5 × 13.5	241 × 343	128.2	82.74	288.6	47.29	1948	810.7
10 × 16	9.5 × 15.5	241 × 394	147.2	95.00	380.4	62.34	2948	1227
10 × 18	9.5 × 17.5	241 × 445	166.2	107.3	484.9	79.46	4243	1766
10 × 20	9.5 × 19.5	241 × 495	185.2	119.5	602.1	98.66	5870	2443
10 × 22	9.5 × 21.5	241 × 546	204.2	131.8	731.9	119.9	7868	3275
10 × 24	9.5 × 23.5	241 × 597	223.2	144.0	874.4	143.3	10274	4276
12 × 12	11.5 × 11.5	292 × 292	132.2	85.32	253.5	41.54	1458	594.2
12 × 14	11.5 × 13.5	292 × 343	155.2	100.2	349.3	57.24	2358	981.4
12 × 16	11.5 × 15.5	292 × 394	178.2	115.0	460.5	75.46	3569	1485
12 × 18	11.5 × 17.5	292 × 445	201.2	129.8	587.0	96.19	5136	2138
12 × 20	11.5 × 19.5	292 × 495	224.2	144.7	728.8	119.4	7106	2958
12 × 22	11.5 × 21.5	292 × 546	247.2	159.5	886.0	145.2	9524	3964
12 × 24	11.5 × 23.5	292 × 597	270.2	174.4	1058	173.4	12437	4276
14 × 14	13.5 × 13.5	343 × 343	182.2	117.5	410.1	67.20	2768	1152
14 × 16	13.5 × 15.5	343 × 394	209.2	135.0	540.6	88.58	4189	1744
14 × 18	13.5 × 17.5	343 × 445	236.2	152.4	689.1	112.9	6029	2510
14 × 20	13.5 × 19.5	343 × 495	263.2	169.8	855.6	140.2	8342	3472
14 × 22	13.5 × 21.5	343 × 546	290.2	187.3	1040	170.4	11181	4654
14 × 24	13.5 × 23.5	343 × 597	317.2	204.7	1243	203.6	14600	6077

# Joist design

(b) Shear:

$$l = 13.3 \frac{F_v A}{w} + 2d$$
$$= \frac{(13.3) (180) (10.88)}{260} + (2) (7.25) = 114.7 \text{ in.}$$

$$\left[ \begin{aligned} l &= \frac{1.11}{1000} \frac{F_v A}{w} + 2d \\ &= \frac{1.11}{1000} \frac{(1241) (7016)}{3.79} + (2)(184) = 2918 \text{ mm} \end{aligned} \right]$$

# Joist design

(c) Deflection:

$$l = 1.69 \left( \frac{EI}{w} \right)^{1/3}$$
$$= 1.69 \left( \frac{(1.4 \times 10^6)(47.63)}{260} \right)^{1/3} = 107.4 \text{ in.}$$

$$\left[ \begin{aligned} l &= \frac{73.8}{1000} \left( \frac{EI}{w} \right)^{1/3} \\ &= \frac{73.8}{1000} \left( \frac{(9.7 \times 10^6) (19.83 \times 10^6)}{3.79} \right)^{1/3} = 2732 \text{ mm} \end{aligned} \right]$$

# Joist design

- Thus bending governs and the maximum joist span is 87 in. (2213 mm).
- We will select a stringer spacing (joist span) of 84 in (7 ft). (2134 mm).

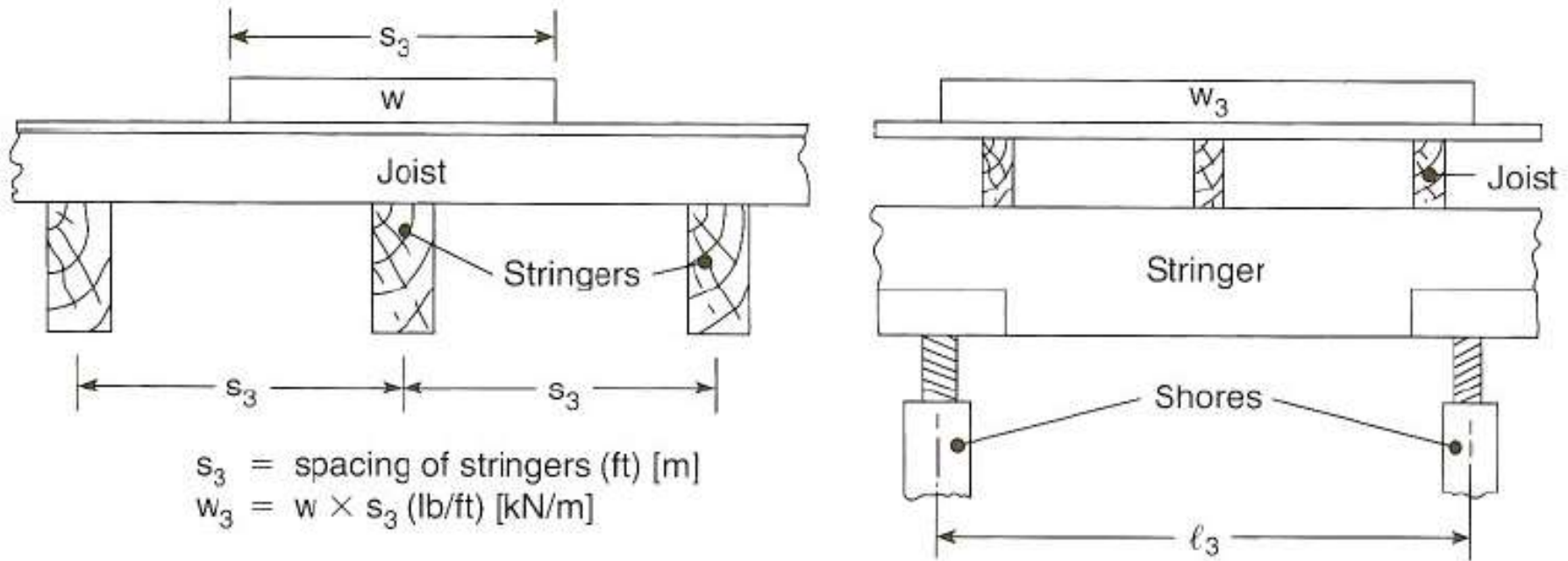


# Stringer Design

- *To analyze stringer design, consider a strip of design load 7 ft (2.13 m) wide (equal to stringer spacing) as resting directly on the stringer (Figure 13-1c).*
- Assume the stringer to be continuous over three spans.
- Stringers are 4 x 8 (100 x 200 mm) lumber.
- Now analyze the stringer as a beam and determine the maximum allowable span.
- $w = (7) (130) = 910 \text{ lb/ft}$
- $[w = (2.13) (1) (6.22) = 13.25 \text{ kN/m}]$

# Figure 13-1

## Design Analysis for form member



c. Stringers

# Stringer Design

(a) Bending:

$$l = 10.95 \left( \frac{F_b S}{w} \right)^{1/2}$$
$$= 10.95 \left( \frac{(1250)(30.66)}{910} \right)^{1/2} = 71.1 \text{ in.}$$

$$\left[ \begin{aligned} l &= \frac{100}{1000} \left( \frac{F_b S}{w} \right)^{1/2} \\ &= \frac{100}{1000} \left( \frac{(8619)(5.024 \times 10^5)}{13.25} \right)^{1/2} = 1808 \text{ mm} \end{aligned} \right]$$

# Stringer Design

(b) Shear:

$$l = \frac{13.3 F_v A}{w} + 2d$$
$$= \frac{(13.3) (180) (25.38)}{910} + (2) (7.25) = 81.3 \text{ in.}$$

$$\left[ \begin{aligned} l &= \frac{1.11 F_v A}{1000 w} + 2d \\ &= \frac{1.11 (1241) (16.37 \times 10^3)}{1000 \cdot 13.25} + (2) (184) = 2070 \text{ mm} \end{aligned} \right]$$

# Stringer Design

(c) Deflection:

$$l = 1.69 \left( \frac{EI}{w} \right)^{1/3}$$
$$= 1.69 \left( \frac{(1.4 \times 10^6) (111.1)}{910} \right)^{1/3} = 93.8 \text{ in.}$$

$$\left[ \begin{aligned} l &= \frac{73.8}{1000} \left( \frac{EI}{w} \right)^{1/3} \\ &= \frac{73.8}{1000} \left( \frac{(9.7 \times 10^6) (46.26 \times 10^6)}{13.25} \right)^{1/3} = 2388 \text{ mm} \end{aligned} \right]$$

- Bending governs and the maximum span is 71.1 in. (1808 mm).

# Check Shore Strength

- Now we must check shore strength before selecting the stringer span (shore spacing).
- The maximum stringer span based on shore strength is equal to the shore strength divided by the load per unit length of stringer.

$$l = \frac{4000}{910} \times 12 = 52.7 \text{ in.}$$

$$\left[ l = \frac{17.8}{13.25} = 1.343 \text{ m} \right]$$

- Thus the maximum stringer span is limited by shore strength to 52.7 in. (1.343 m).
- We select a shore spacing of 4 ft (1.22 m) as a modular value.

# Check for Crushing

- Before completing our design, we should check for crushing at the point where each joist rests on a stringer.
- The load at this point is the load per unit length of joist multiplied by the joist span.

$$P = (260) (84/12) = 1820 \text{ lb}$$

$$[P = (3.79) (2.134) = 8.09 \text{ kN}]$$

$$\text{Bearing area (A)} = (1.5)(3.5) = 5.25 \text{ sq in.}$$

$$[A = (38) (89) = 3382 \text{ mm}^2]$$

$$f_{c\perp} = \frac{P}{A} = \frac{1820}{5.25} = 347 \text{ psi} < 405 \text{ psi } (F_{c\perp})$$

OK

$$\left[ f_{c\perp} = \frac{8.09 \times 10^6}{3382} = 2392 \text{ kPa} < 2792 \text{ kPa } (f_{c\perp}) \right]$$

# *Final Design*

- Decking: nominal 1-in. (25-mm) lumber
- Joists: 2 x 8's (50 x 200-mm) at 24-in. (610-mm) spacing
- Stringers: 4 x 8's (100 x 200-mm) at 84-in. (2.13-m) spacing
- Shore: 4000-lb (17.8-kN) commercial shores at 48-in. (1.22-m) intervals