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:**

- 2.4 kPa

In case of motorized concrete buggies are used

- 3.6 kPa

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

- 4.8 kPa

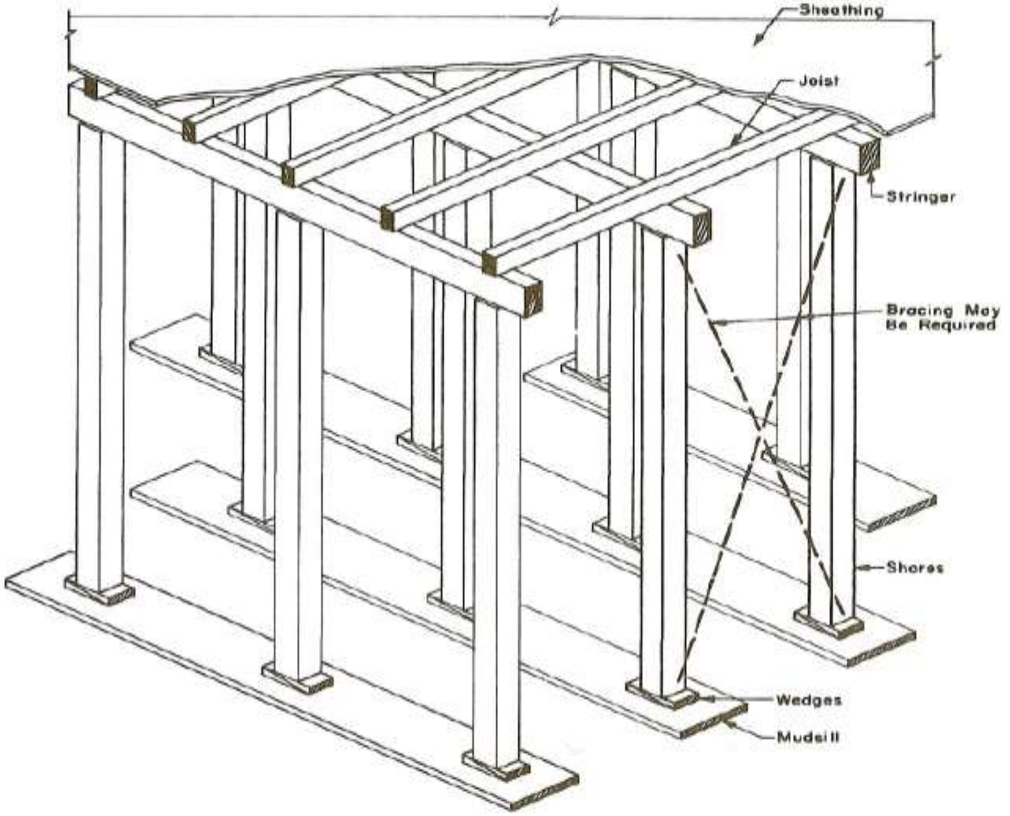
In case of motorized concrete buggies are used :

- 6.0 kPa

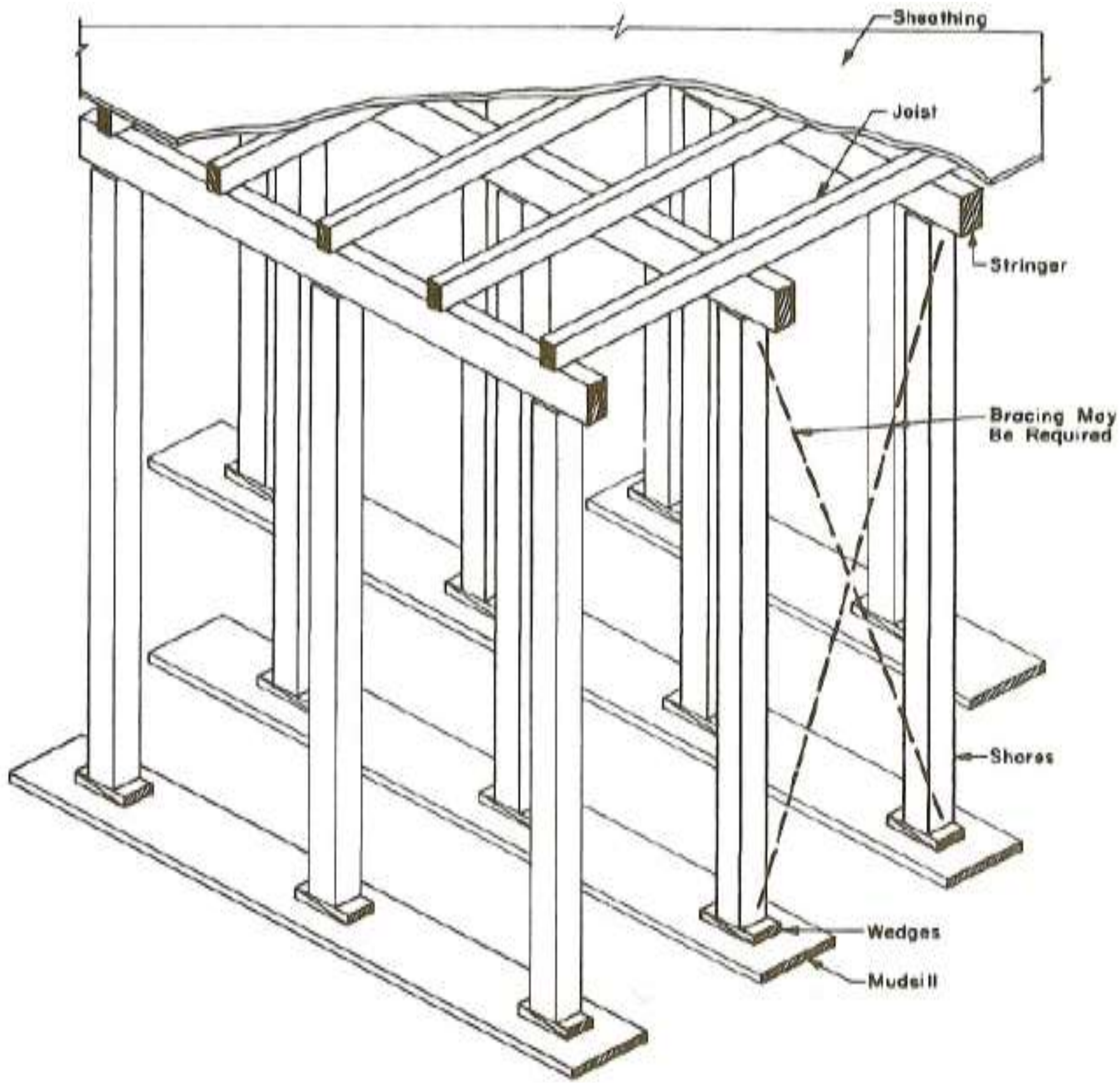
➢ Design steps:

- ❑ Specify the design load.
- ❑ Analyzing the sheathing, 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 steps (cont.)



- ❑ Design the sheathing.
- ❑ Design the joist.
- ❑ Design the stringers.
- ❑ Check the stringer spans and shore capacity.
- ❑ Check the crushing between joist and stringer.


Sheathing
Joist
Stringer
Bracing May Be Required
Shores
Wedges
Mudsill

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.^{4})

Bending

$$f_b = \frac{M}{S} \tag{13–5}$$

Shear

$$f_v = \frac{1.5V}{A} \text{ for rectangular wood members} \tag{13–6}$$

$$f_v = \frac{V}{Ib/Q} \text{ for plywood} \tag{13–7}$$

Compression

$$f_c \text{ or } f_{c\perp} = \frac{P}{A} \tag{13–8}$$

Tension

$$f_t = \frac{P}{A} \tag{13–9}$$

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]	1.7×10^6 [11.7×10^6]
Hemlock—fir	1150 [7929]	150 [1034]	245 [1689]	800 [5516]	675 [4654]	1.4×10^6 [9.7×10^6]
Southern pine	1400 [9653]	180 [1241]	405 [2792]	975 [6723]	825 [5688]	1.6×10^6 [11.0×10^6]
California redwood	1400 [9653]	160 [1103]	425 [2930]	1000 [6895]	800 [5516]	1.3×10^6 [9.0×10^6]
Eastern spruce	1050 [7240]	140 [965]	255 [1758]	700 [4827]	625 [4309]	1.2×10^6 [8.3×10^6]
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
Bending			
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	$\ell = 2.83 \left(\frac{F_b KS}{w}\right)^{1/2}$	$\ell = 2.83 \left(\frac{F_b KS}{w}\right)^{1/2}$	$\ell = 3.16 \left(\frac{F_b KS}{w}\right)^{1/2}$
Shear			
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	$\ell = 2.00 \frac{F_s Ib/Q}{w} + 2d$	$\ell = 1.60 \frac{F_s Ib/Q}{w} + 2d$	$\ell = 1.67 \frac{F_s Ib/Q}{w} + 2d$
Deflection	$\ell = \frac{526}{1000} \left(\frac{EI\Delta}{w}\right)^{1/4}$	$\ell = \frac{655}{1000} \left(\frac{EI\Delta}{w}\right)^{1/4}$	$\ell = \frac{617}{1000} \left(\frac{EI\Delta}{w}\right)^{1/4}$
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 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	
in.	*in.*	*mm*	*in.²*	*10³ mm²*	*in.³*	*10⁵ mm³*	*in.⁴*	*10⁶ mm⁴*
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

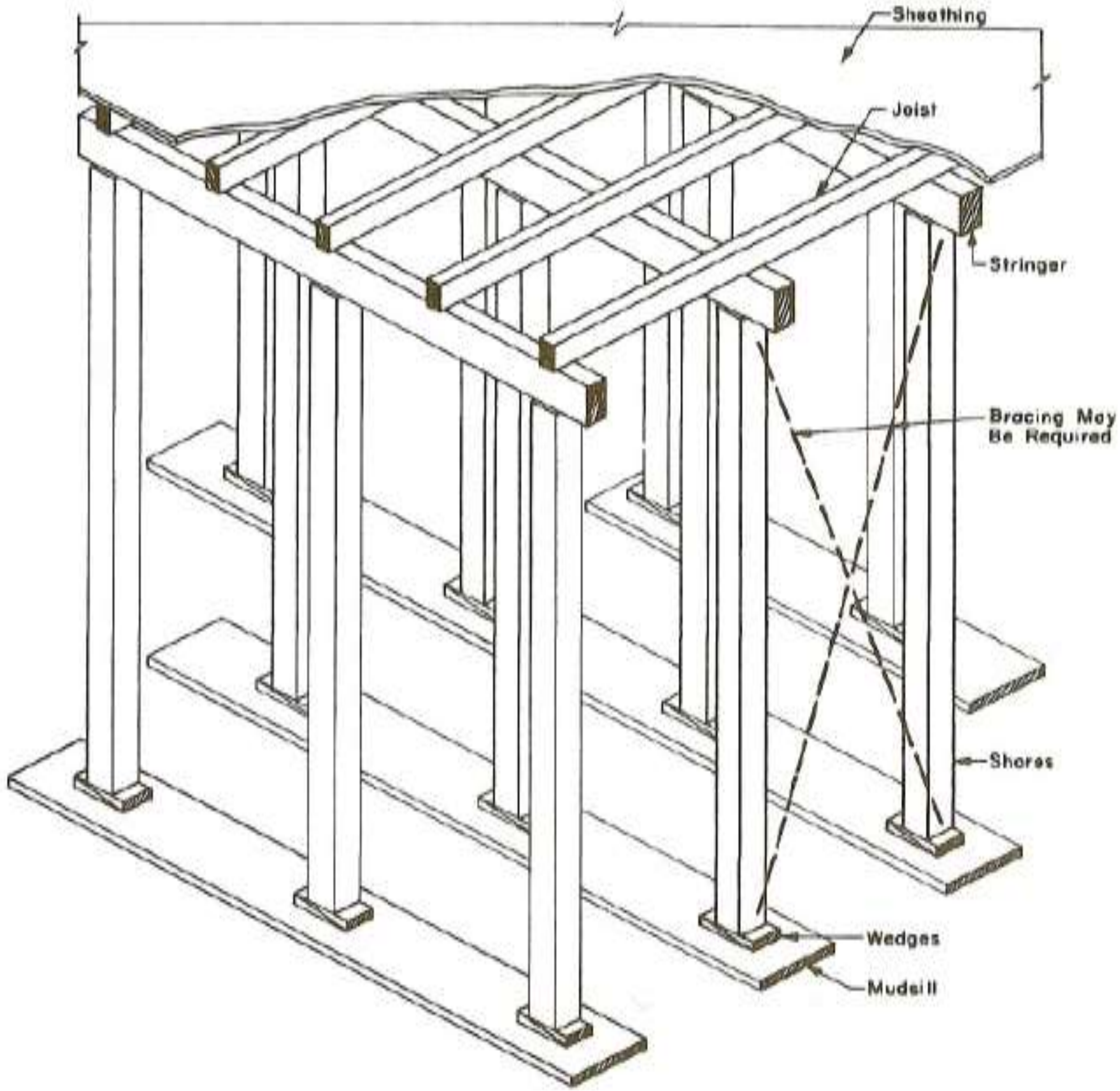
EXAMPLE 13-1

- 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
- A 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	1.36×10^6 [9.4×10^6]	1.40×10^6 [9.7×10^6]

- 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.


Sheathing
Joist
Stringer
Bracing May
Be Required
Shores
Wedges
Mudsill

Solution

Design Load. Assume concrete density is 150 lb/cu ft (2403 kg/m^3)

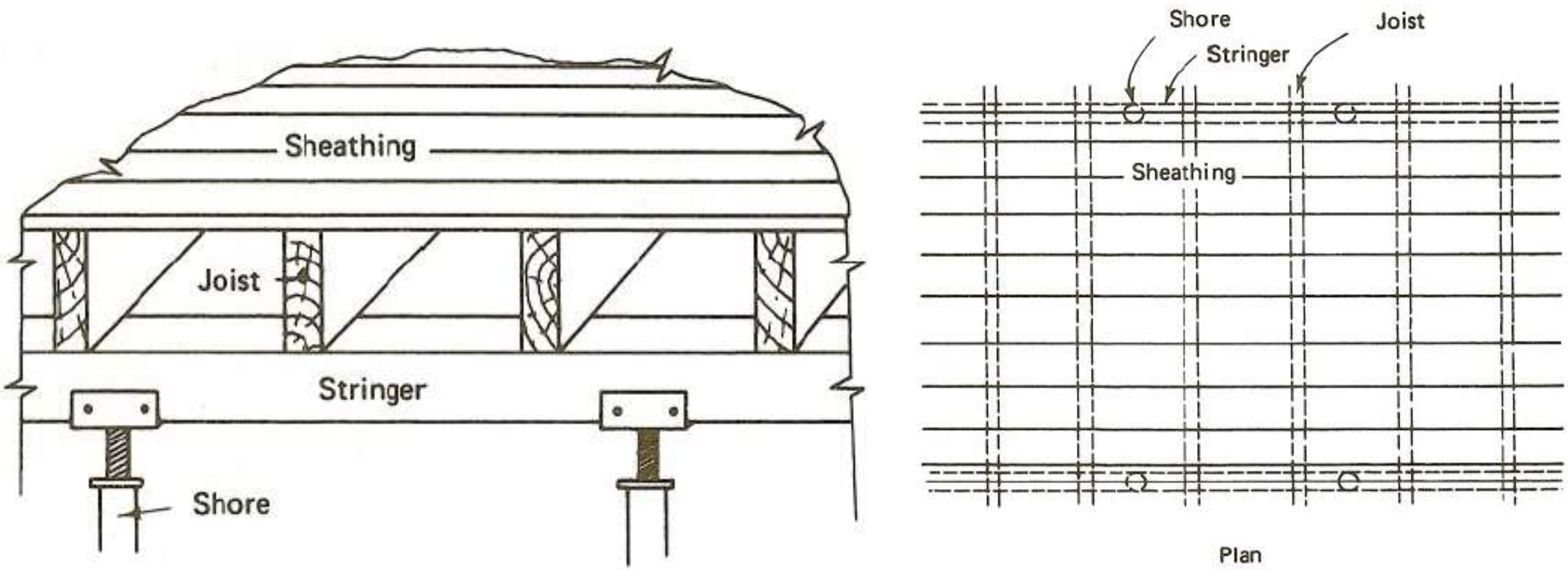
$$
\begin{aligned}
\text{Concrete} = 1 \text{ sq ft} \times 6/12 \text{ ft} \times 150 \text{ lb/cu ft} &= 75 \text{ lb/sq ft} \\
\text{Formwork} &= 5 \text{ lb/sq ft} \\
\text{Live load} &= \underline{50 \text{ lb/sq ft}} \\
\text{Design load} &= 130 \text{ lb/sq ft}
\end{aligned}
$$

[Pressure per m^2:

$$
\begin{aligned}
\text{Concrete} = 1 \times 0.152 \times 9.8 \times 2403/1000 &= 3.58 \text{ kPa} \\
\text{Formwork} \quad &= 0.24 \text{ kPa} \\
\text{Live load} \quad &= \underline{2.40 \text{ kPa}} \\
\text{Design load} &= 6.22 \text{ kPa}
\end{aligned}
$$
]

1 kPa= 1 kN/ m^2

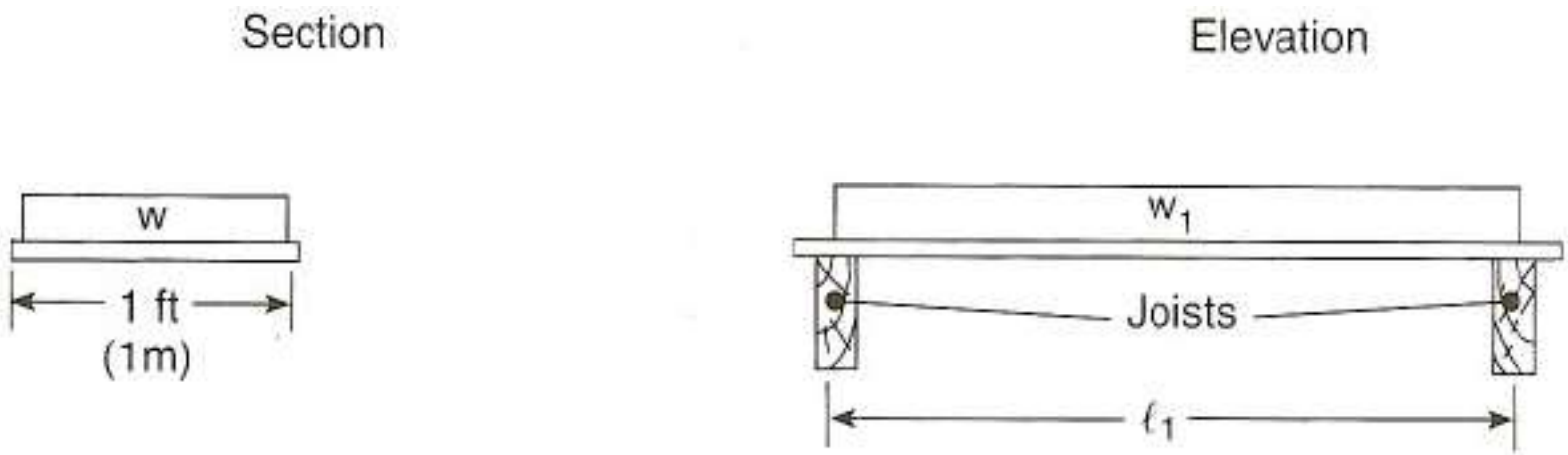
Figure 13-2 Slab form



Deck Design

- *Consider a uniformly loaded strip of decking (sheathing)* 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 sq ft/lin ft) x (130 lb/sq ft) = 130 lb/ft*
- *[w =(1 m^2/lin m) x (6.22 kN/m^2) =6.22 kN/m]*

Figure 13-1
Design Analysis for form member



w = design load (lb/sq ft) [kN/m^2]
w_1 = $1 \times w = w$ (lb/ft) [kN/m]

a. Sheathing

Deck Design

(a) Bending:

$$l = 4.46\, d \left(\frac{F_b b}{w}\right)^{1/2}$$

$$= (4.46)\,(0.75)\left(\frac{(1075)\,(12)}{130}\right)^{1/2} = 33.3 \text{ in.}$$

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

	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	1.36×10^6 [9.4×10^6]	1.40×10^6 [9.7×10^6]

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

Design Conditions	Support Conditions: 1 Span	Support Conditions: 2 Spans	Support Conditions: 3 or More Spans
Bending			
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	$\ell = 2.83 \left(\frac{F_b KS}{w}\right)^{1/2}$	$\ell = 2.83 \left(\frac{F_b KS}{w}\right)^{1/2}$	$\ell = 3.16 \left(\frac{F_b KS}{w}\right)^{1/2}$
Shear			
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	$\ell = 2.00 \frac{F_s Ib/Q}{w} + 2d$	$\ell = 1.60 \frac{F_s Ib/Q}{w} + 2d$	$\ell = 1.67 \frac{F_s Ib/Q}{w} + 2d$
Deflection	$\ell = \frac{526}{1000} \left(\frac{EI\Delta}{w}\right)^{1/4}$	$\ell = \frac{655}{1000} \left(\frac{EI\Delta}{w}\right)^{1/4}$	$\ell = \frac{617}{1000} \left(\frac{EI\Delta}{w}\right)^{1/4}$
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 or $f_{c\perp} = \frac{P}{A}$		
Tension	$f_t = \frac{P}{A}$		

Deck Design

(b) Shear:

$$l = 13.3 \, \frac{F_v A}{w} + 2d$$

$$= \frac{(13.3)\,(174)\,(12)\,(0.75)}{130} + (2)\,(0.75) = 161.7 \text{ in.}$$

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

	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	1.36×10^6 [9.4×10^6]	1.40×10^6 [9.7×10^6]

Deck Design

(c) Deflection:

$$l = 1.69\left(\frac{EI}{w}\right)^{1/3} = 1.69\left(\frac{Ebd^3}{w\ 12}\right)^{1/3}$$

$$= 1.69\left(\frac{(1.36\times10^6)\,(12)\,(0.75)^3}{(130)\,(12)}\right)^{1/3} = 27.7\text{ in.}$$

$$\left[l = \frac{73.8}{1000}\left(\frac{EI}{w}\right)^{1/3} = \frac{73.8}{1000}\left(\frac{Ebd^3}{w12}\right)^{1/3}\right.$$

$$\left.= \frac{73.8}{1000}\left(\frac{(9.4\times10^6)\,(1000)\,(19)^3}{(12)\,(6.22)}\right)^{1/3} = 703\text{ mm}\right]$$

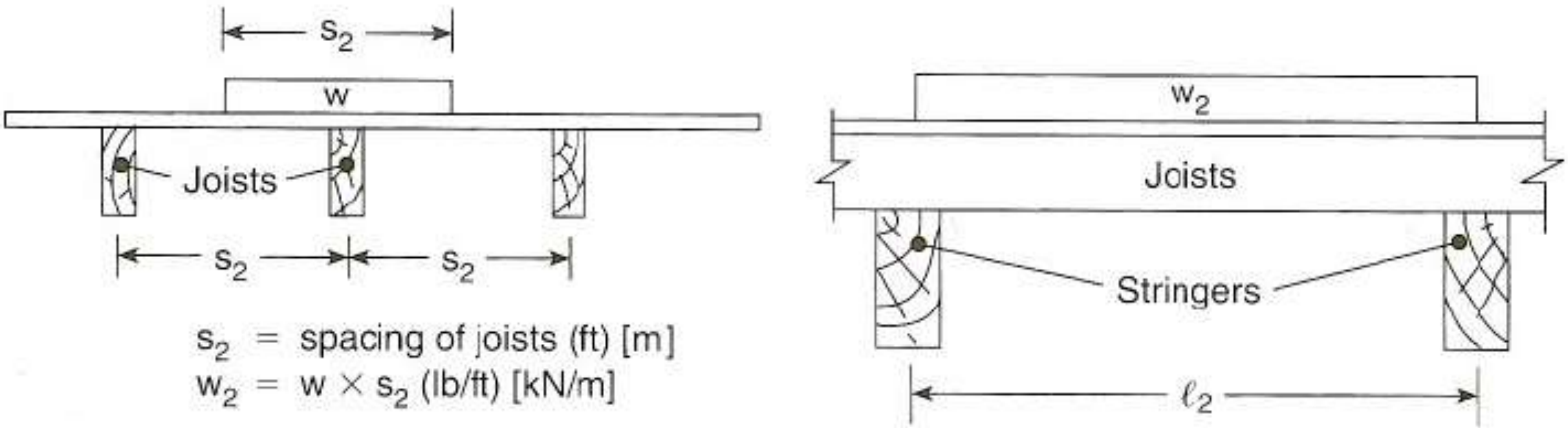
	Sheathing psi [kPa]	Other Members psi [kPa]
F_b	1075 [7412]	1250 [8619]
F_v	174 [1200]	180 [1241]
$F_{c\perp}$		405 [2792]
F_c		850 [5861]
E	1.36×10^6 [9.4×10^6]	1.40×10^6 [9.7×10^6]

- 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 ft) x (1) x (130 lb/sq ft) =260 lb/ft*
- [*w =(0.610 m) x (1) x (6.22 kPa) = 3.79 kN/m*]

Figure 13-1
Design Analysis for form member



b. Joists

Joist design

(a) Bending:

$$l = 10.95\left(\frac{F_bS}{w}\right)^{1/2}$$

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

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

	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	1.36×10^6 [9.4×10^6]	1.40×10^6 [9.7×10^6]

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	
in.	*in.*	*mm*	*in.*^{2}	10^3 *mm*^{2}	*in.*^{3}	10^5 *mm*^{3}	*in.*^{4}	10^6 *mm*^{4}
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
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	135.7	22.23	1034	430.6

Joist design

(b) Shear:

$$l = 13.3 \frac{F_v A}{w} + 2d$$

$$= \frac{(13.3)\,(180)\,(10.88)}{260} + (2)\,(7.25) = 114.7 \text{ in.}$$

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

	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	1.36×10^6 [9.4×10^6]	1.40×10^6 [9.7×10^6]

Joist design

(c) Deflection:

$$l = 1.69\left(\frac{EI}{w}\right)^{1/3}$$

$$= 1.69\left(\frac{(1.4\times10^6)(47.63)}{260}\right)^{1/3} = 107.4 \text{ in.}$$

$$\left[\begin{aligned} l &= \frac{73.8}{1000}\left(\frac{EI}{w}\right)^{1/3} \\ &= \frac{73.8}{1000}\left(\frac{(9.7\times10^6)\,(19.83\times10^6)}{3.79}\right)^{1/3} = 2732 \text{ mm} \end{aligned}\right]$$

	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	1.36×10^6 [9.4×10^6]	1.40×10^6 [9.7×10^6]

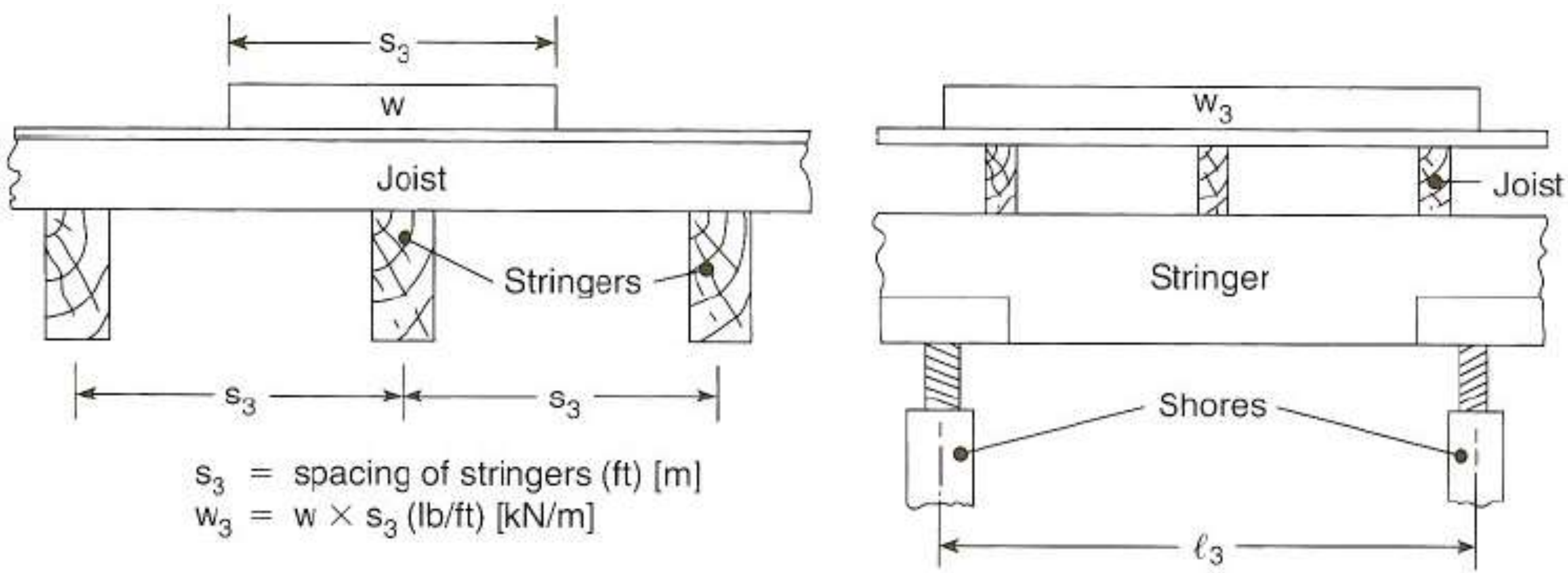
Joist design

- 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 lb/ft*
- *[w =(2.13) (1) (6.22) = 13.25 kN/m]*

Figure 13-1
Design Analysis for form member



c. Stringers

Stringer Design

(a) Bending:

$$l = 10.95 \left(\frac{F_b S}{w}\right)^{1/2}$$

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

$$\left[l = \frac{100}{1000} \left(\frac{F_b S}{w}\right)^{1/2} \right.$$

$$\left. = \frac{100}{1000} \left(\frac{(8619)\,(5.024 \times 10^5)}{13.25}\right)^{1/2} = 1808 \text{ mm} \right]$$

	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	1.36×10^6 [9.4×10^6]	1.40×10^6 [9.7×10^6]

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	
in.	in.	mm	in.^{2}	10^3 mm^2	in.^{3}	10^5 mm^3	in.^{4}	10^6 mm^4
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
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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

Stringer Design

(b) Shear:

$$l = \frac{13.3\,F_v\,A}{w} + 2d$$

$$= \frac{(13.3)\,(180)\,(25.38)}{910} + (2)\,(7.25) = 81.3 \text{ in.}$$

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

	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	1.36×10^6 [9.4×10^6]	1.40×10^6 [9.7×10^6]

Stringer Design

(c) Deflection:

$$l = 1.69 \left(\frac{EI}{w}\right)^{1/3}$$

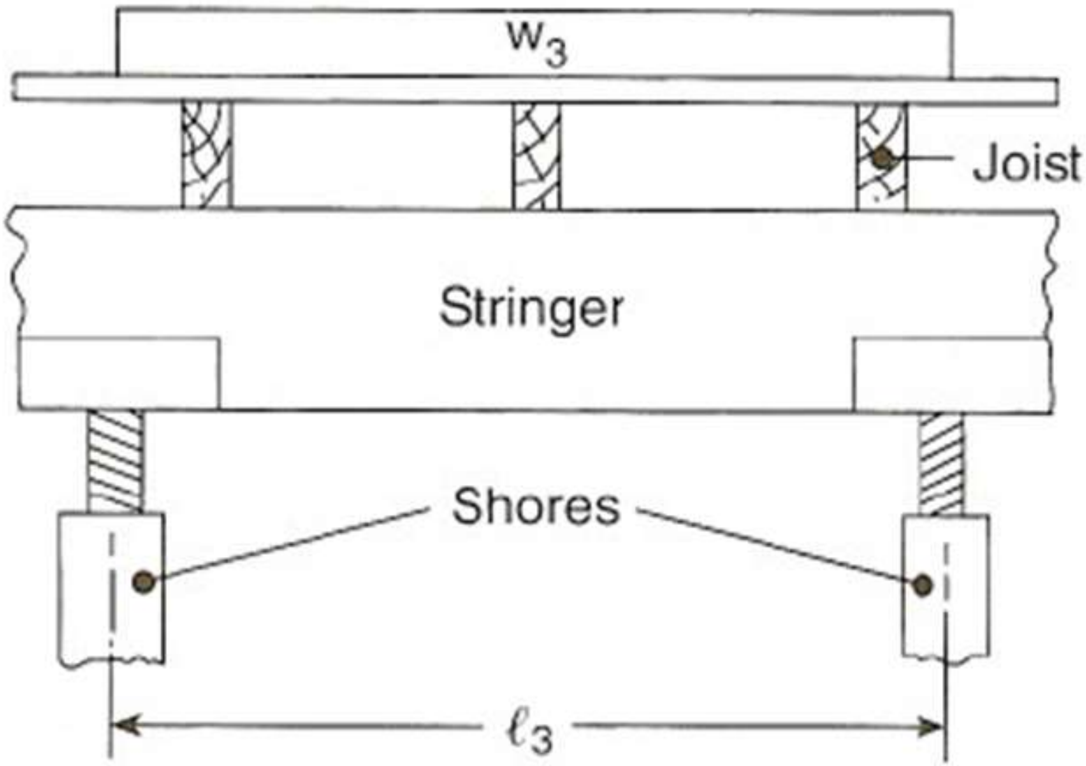
$$= 1.69 \left(\frac{(1.4 \times 10^6)\,(111.1)}{910}\right)^{1/3} = 93.8 \text{ in.}$$

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

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

Check Shore Strength

- Bending governs , The maximum stringer span is 71.1 in. (1808 mm).



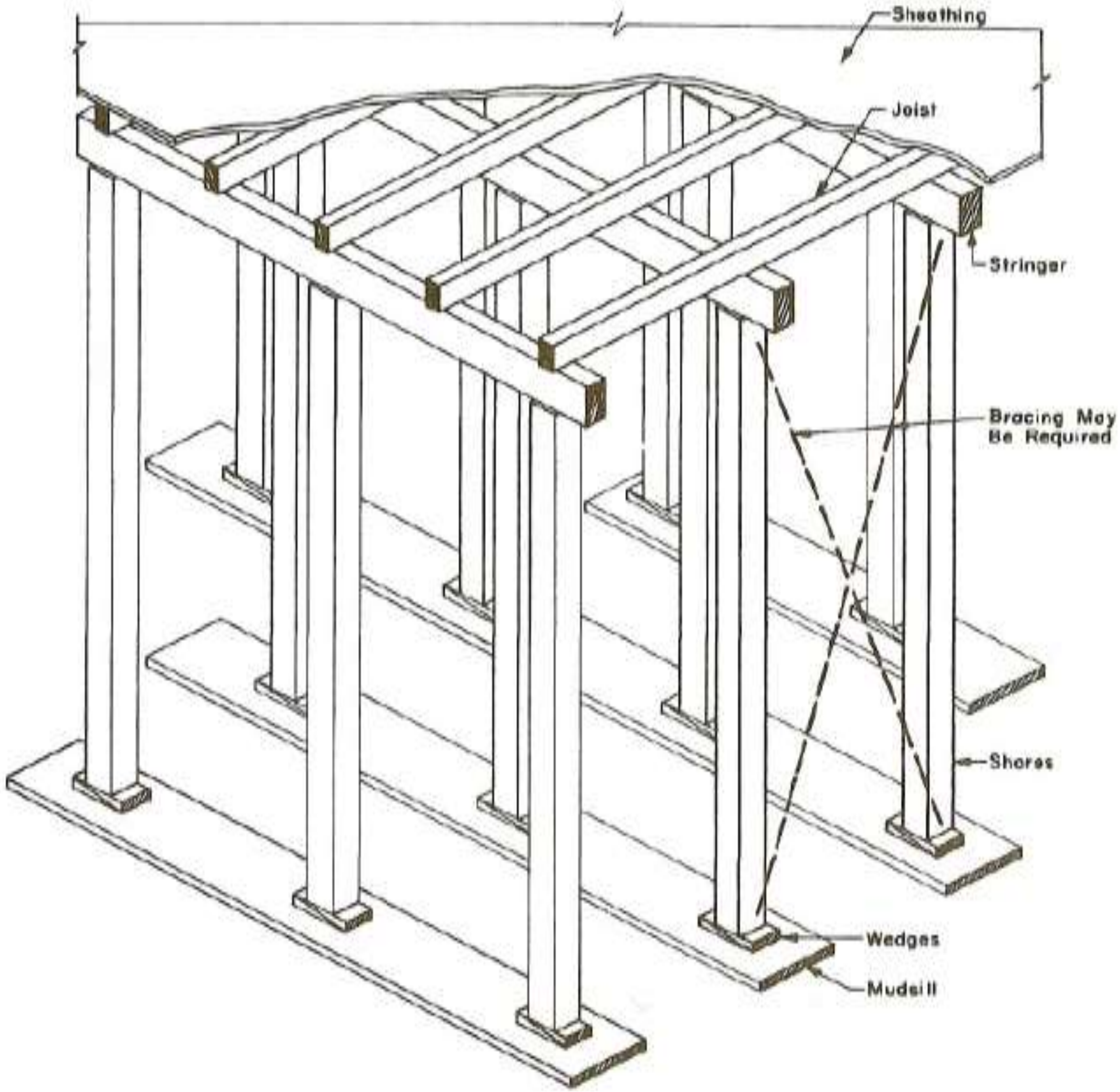
Check Shore Strength

- 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.


Sheathing
Joist
Stringer
Bracing May
Be Required
Shores
Wedges
Mudsill

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.

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

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

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

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

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

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

Final Design

- Decking: nominal1-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