13

Concrete Form Design

13–1 DESIGN PRINCIPLES

The design of concrete formwork that has adequate strength to resist failure and will not deflect excessively when the forms are filled is a problem in structural design. Unless commercial forms are used, this will usually involve the design of wall, column, or slab forms constructed of wood or plywood. In such cases, after the design loads have been established, each of the primary form components may be analyzed as a beam to determine the maximum bending and shear stresses and the maximum deflection that will occur. Vertical supports and lateral bracing are then analyzed for compression and tension loads. The procedures and applicable equations are presented in this chapter.

13–2 DESIGN LOADS

Wall and Column Forms

For vertical forms (wall and column) forms, design load consists of the lateral pressure of the concrete against the forms. The maximum lateral pressure that the concrete exerts against a form has been found to be a function of the unit weight of the concrete, cement type or blend, temperature of the concrete, vertical rate of placing, and the height of the form. For ordinary internally vibrated concrete, the American Concrete Institute (ACI) recommends the use of the following formulas to determine the design lateral concrete pressure.

For all columns and for walls with a vertical rate of placement less than 7 ft/h (2.1 m/h) and a placement height of 14 ft (4.3m) or less:

$$p = C_w C_c\left(150 + \frac{9000\,R}{T}\right) \qquad \textbf{(13–1}\textbf{\textit{A}}\textbf{)}$$

$$\left[p = C_w C_c\left(7.2 + \frac{785\,R}{T + 18}\right)\right] \qquad \textbf{(13–1}\textbf{\textit{B}}\textbf{)}$$

where: C_w = unit weight coefficient (Table 13–1)
C_c = chemistry coefficient (Table 13–2)
p = lateral pressure (lb/sq ft or kPa)
R = rate of vertical placement (ft/h or m/h)
T = concrete temperature (°F or °C)
h = height of form (ft or m)
w = unit weight of concrete (lb/cu ft or kg/m^3)

Minimum pressure = 600 C_w lb/sq ft (28.7 C_w kPa)
Maximum pressure = wh

For walls with a vertical rate of placement of 7 to 15 ft/h (2.1 to 4.6 m/h) and walls with a rate of placement less than 7 ft/h (2.1 m/h) whose placement height exceeds 14ft (4.3 m):

$$p = C_w C_c\left(150 + \frac{43{,}400}{T} + \frac{2800\,R}{T}\right) \qquad \textbf{(13–2A)}$$

$$\left[p = C_w C_c\left(7.2 + \frac{1154}{T + 18} + \frac{244\,R}{T + 18}\right)\right] \qquad \textbf{(13–2B)}$$

Minimum pressure = 600 C_w lb/sq ft (28.7 C_w kPa)
Maximum pressure = wh

Table 13–1 Concrete unit weight coefficient (Courtesy of American Concrete Institute)

Unit Weight of Concrete	C_w
Under 140 lb/cu ft	$0.5\left(1 + \frac{w}{145}\right)$ but at least 0.80
[Under 2243 kg/m^3	$0.5\left(1 + \frac{w}{2323}\right)$ but at least 0.80]
140 to 150 lb/cu ft	1.0
[2243 to 2403 kg/m^3	1.0]
Over 150 lb/cu ft	$\left(\frac{w}{145}\right)$
[Over 2403 kg/m^3	$\left(\frac{w}{2323}\right)$]

Table 13–2 Concrete chemistry coefficient (Courtesy of American Concrete Institute)

Cement Type or Blend	C_c
Type I, II, or III without retarders	1.0
Type I, II, or III with a retarder	1.2
Other blends containing less than 70% slag or 40% fly ash without retarders	1.2
Other blends containing less than 70% slag or 40% fly ash with a retarder	1.4
Blends containing more than 70% slag or 40% fly ash	1.4

For walls with a vertical rate of placement greater than 15 ft/h (4.6 m/h) or when the forms will be filled before the concrete stiffens:

$$p = wh \tag{13–3}$$

When forms are vibrated externally, it is recommended that a design load twice that given by Equations 13–1 and 13–2 be used. When concrete is pumped into vertical forms from the bottom (both column and wall forms), Equation 13–3 should be used and a minimum additional pressure of 25% should be added to allow for pump surge pressure.

Floor and Roof Slab Forms

The design load to be used for elevated slabs consists of the weight of concrete and reinforcing steel, the weight of the forms themselves, and any live loads (equipment, workers, material, etc.). For normal reinforced concrete, the design load for concrete and steel is based on a unit weight of 150 lb/cu ft (2403 kg/m^3). The American Concrete Institute (ACI) recommends that a minimum live load of 50 lb/sq ft (2.4 kPa) be used for the weight of equipment, materials, and workers. When motorized concrete buggies are utilized, the live load should be increased to at least 75 lb/sq ft (3.6 kPa). Any unusual loads would be in addition to these values. ACI also recommends that a minimum design load (dead load plus live load) of 100 lb/sq ft (4.8 kPa) be used. This should be increased to 125 lb/sq ft (6.0 kPa) when motorized buggies are used. (Note: 1 kg/m^2 = 0.0098 kPa)

Lateral Loads

Formwork must be designed to resist lateral loads such as those imposed by wind, the movement of equipment on the forms, and the placing of concrete into the forms. Such forces are usually resisted by lateral bracing whose design is covered in Section 13–6. The minimum lateral design loads recommended for tied wall forms are given in Table 13–3. When form ties are not used, bracing must be designed to resist the internal concrete pressure as well as external loads.

Table 13–3 Recommended minimum lateral design load for wall forms

Wall Height, *h* (ft) [m]	Design Lateral Force Applied at Top of Form (lb/ft) [kN/m]
less than 8 [2.4]	$\frac{h \times wf^*}{2}$
8 [2.4] or over but less than 22 [6.7]	100 [1.46] but at least $\frac{h \times wf^*}{2}$
22 [6.7] or over	7.5 *h* [0.358 *h*] but at least $\frac{h \times wf^*}{2}$

**wf* = wind force prescribed by local code (lb/sq ft) [kPa] but minimum of 15 lb/sq ft [0.72 kPa]

For slab forms, the minimum lateral design load is expressed as follows:

$$H = 0.02 \times dl \times ws \tag{13–4}$$

where H = lateral force applied along the edge of the slab (lb/ft) [kN/m]; minimum value = 100 lb/ft [1.46 kN/m]
dl = design dead load (lb/sq ft) [kPa]
ws = width of slab perpendicular to form edge (ft) [m]

In using Equation 13–4, design dead load includes the weight of concrete plus formwork. In determining the value of *ws,* consider only that part of the slab being placed at one time.

13–3 METHOD OF ANALYSIS

Basis of Analysis

After appropriate design loads have been selected, the sheathing, joists or studs, and stringers or wales are analyzed in turn, considering each member to be a uniformly loaded beam supported in one of three conditions (single-span, two-span, or three-span or larger) and analyzed for bending, shear, and deflection. Vertical supports and lateral bracing must be checked for compression and tension stresses. Except for sheathing, bearing stresses must be checked at supports to ensure against crushing.

Using the methods of engineering mechanics, the maximum values expressed in customary units of bending moment, shear, and deflection developed in a uniformly loaded, simply supported beam of uniform cross section are given in Table 13–4.

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})

The maximum fiber stresses (expressed in conventional units) developed in bending, shear, and compression resulting from a specified load may be determined from the following equations:

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)

Since the grain of a piece of timber runs parallel to its length, axial compressive forces result in unit compressive stresses parallel to the grain. Thus, a compression force in a formwork brace (Figure 13–4) will result in unit compressive stresses parallel to the grain (f_c) in the member. Loads applied to the top or sides of a beam, such as a joist resting on a stringer (Figure 13–1b), will result in unit compressive stresses perpendicular to the grain ($f_{c\perp}$) in the beam. Equating allowable unit stresses in bending and shear to the maximum unit stresses developed in a beam subjected to a uniform load of w pounds per linear foot [kN/m] yields the bending and shear equations of Tables 13–5 and 13–5A.

When design load and beam section properties have been specified, these equations may be solved directly for the maximum allowable span. Given a design load and span length, the equations may be solved for the required size of the member. Design properties for Plyform® (plywood especially engineered for use in concrete formwork) are given in Table 13–6 and section properties for dimensioned lumber and timber are given in Table 13–7. The properties of plywood, lumber, and timber are described in Section 11–2. However, typical allowable stress values for lumber are given in Table 13–8. The allowable unit stress values in Table 13–8 (but not modulus of elasticity values) may be multiplied by a load duration factor of 1.25 (7-d load) when designing formwork for light construction and single use or very limited reuse of forms. However, allowable stresses for lumber sheathing (not Plyform®) should be reduced by the factors given in Table 13–8 for wet conditions. The values for Plyform® properties presented in Table 13–6 are based on wet strength and 7-d load duration, so no further adjustment in these values is required.

It should be noted that the deflection of plywood sheathing is precisely computed as the sum of bending deflection and shear deflection. While the deflection equations of Tables 13–5 and 13–5A consider only bending deflection, the modulus of elasticity values of Table 13–6 include an allowance for shear deflection. Thus, the deflection computed using these tables is sufficiently accurate in most cases. However, for very short spans (l/d ratio of less than 15) it is recommended that shear deflection be computed separately and added to bending deflection. See reference 3 for a recommended procedure.

13–4 SLAB FORM DESIGN

Method of Analysis

The procedure for applying the equations of Tables 13–5 and 13–5A to the design of a deck or slab form is first to consider a strip of sheathing of the specified thickness and 1 ft (or 1 m) wide (see Figure 13–1a). Determine in turn the maximum allowable span based on the

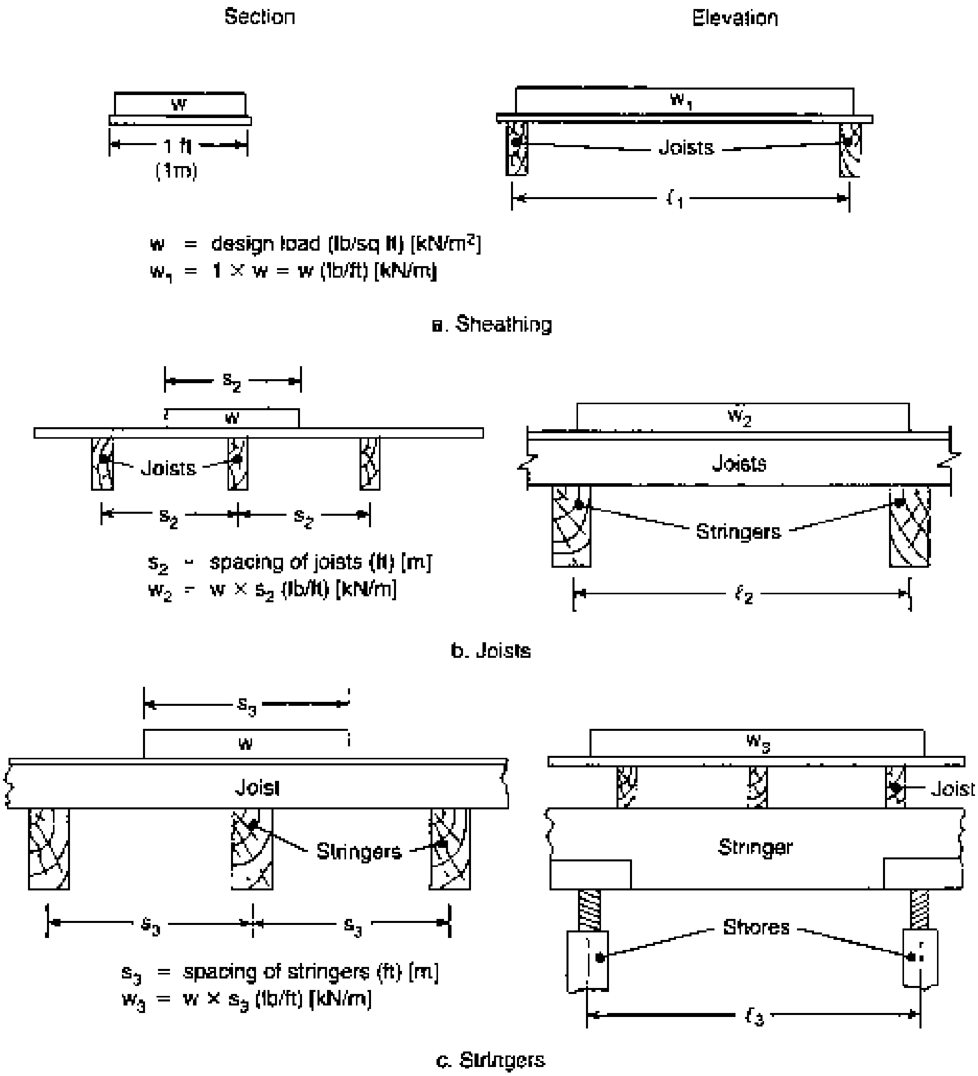


Figure 13–1 Design analysis of form members.

allowable values of bending stress, shear stress, and deflection. The lower of these values will, of course, determine the maximum spacing of the supports (joists). For simplicity and economy of design, this maximum span value is usually rounded down to the next lower integer or modular value when selecting joist spacing.

Based on the selected joist spacing, the joist itself is analyzed to determine its maximum allowable span. The load conditions for the joist are illustrated in Figure 13–1b. The joist span selected will be the spacing of the stringers. Again, an integer or modular value is selected for stringer spacing.

Table 13–5 Concrete form design equations

Design Condition	Support Conditions: 1 Span	2 Spans	3 or More Spans
Bending			
Wood	$\ell = 4.0d\left(\frac{F_b b}{w}\right)^{1/2}$	$\ell = 4.0d\left(\frac{F_b b}{w}\right)^{1/2}$	$\ell = 4.46d\left(\frac{F_b b}{w}\right)^{1/2}$
	$\ell = 9.8\left(\frac{F_b S}{w}\right)^{1/2}$	$\ell = 9.8\left(\frac{F_b S}{w}\right)^{1/2}$	$\ell = 10.95\left(\frac{F_b S}{w}\right)^{1/2}$
Plywood	$\ell = 9.8\left(\frac{F_b KS}{w}\right)^{1/2}$	$\ell = 9.8\left(\frac{F_b KS}{w}\right)^{1/2}$	$\ell = 10.95\left(\frac{F_b KS}{w}\right)^{1/2}$
Shear			
Wood	$\ell = 16\,\frac{F_v A}{w} + 2d$	$\ell = 12.8\,\frac{F_v A}{w} + 2d$	$\ell = 13.3\,\frac{F_v A}{w} + 2d$
Plywood	$\ell = 24\,\frac{F_s Ib/Q}{w} + 2d$	$\ell = 19.2\,\frac{F_s Ib/Q}{w} + 2d$	$\ell = 20\,\frac{F_s Ib/Q}{w} + 2d$
Deflection	$\ell = 5.51\left(\frac{EI\Delta}{w}\right)^{1/4}$	$\ell = 6.86\left(\frac{EI\Delta}{w}\right)^{1/4}$	$\ell = 6.46\left(\frac{EI\Delta}{w}\right)^{1/4}$
If $\Delta = \ell/180$	$\ell = 1.72\left(\frac{EI}{w}\right)^{1/3}$	$\ell = 2.31\left(\frac{EI}{w}\right)^{1/3}$	$\ell = 2.13\left(\frac{EI}{w}\right)^{1/3}$
If $\Delta = \ell/240$	$\ell = 1.57\left(\frac{EI}{w}\right)^{1/3}$	$\ell = 2.10\left(\frac{EI}{w}\right)^{1/3}$	$\ell = 1.94\left(\frac{EI}{w}\right)^{1/3}$
If $\Delta = \ell/360$	$\ell = 1.37\left(\frac{EI}{w}\right)^{1/3}$	$\ell = 1.83\left(\frac{EI}{w}\right)^{1/3}$	$\ell = 1.69\left(\frac{EI}{w}\right)^{1/3}$
Compression	f_c or $f_{c\perp} = \frac{P}{A}$		
Tension	$f_t = \frac{P}{A}$		

Notation:

ℓ = length of span, center to center of supports (in.)
F_b = allowable unit stress in bending (psi)
$F_b KS$ = plywood section capacity in bending (lb × in./ft)
F_c = allowable unit stress in compression parallel to grain (psi)
$F_{c\perp}$ = allowable unit stress in compression perpendicular to grain (psi)
$F_s Ib/Q$ = plywood section capacity in rolling shear (lb/ft)
F_v = allowable unit stress in horizontal shear (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)
A = area of section (in.^{2})*
E = modulus of elasticity (psi)
I = moment of inertia (in.^{4})*
EI = plywood stiffness capacity (kPamm^4/m)
P = applied force (compression or tension) (lb)
S = section modulus (in.^{3})*
Δ = deflection (in.)
b = width of member (in.)
d = depth of member (in.)
w = uniform load per foot of span (lb/ft)

*For a rectangular member: $A = bd$, $S = bd^2/6$, $I = bd^3/12$

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	$f_t = \frac{P}{A}$		

Notation:

ℓ = length of span, center to center of supports (mm)
F_b = allowable unit stress in bending (kPa)
$F_b KS$ = plywood section capacity in bending (Nmm/m)
F_c = allowable unit stress in compression parallel to grain (kPa)
$F_{c\perp}$ = allowable unit stress in compression perpendicular to grain (kPa)
$F_s Ib/Q$ = plywood section capacity in rolling shear (N/m)
f_v = allowable unit stress in horizontal shear (kPa)
f_c = actual unit stress in compression parallel to grain (kPa)
$f_{c\perp}$ = actual unit stress in compression perpendicular to grain (kPa)
f_t = actual unit stress in tension (kPa)
A = area of section (mm^2)*
E = modulus of elasticity (kPa)
I = moment of inertia (mm^4)*
EI = plywood stiffness capacity ($kPamm^4/m$)
P = applied force (compression or tension) (N)
S = section modulus (mm^3)*
Δ = deflection (mm)
b = width of member (mm)
d = depth of member (mm)
w = uniform load per meter of span (kPa/m)
*For a rectangular member: $A = bd$, $S = bd^2/6$, $I = bd^3/12$

Table 13–6 Section properties of plywood.* (Created by author with data from APA—the Engineered Wood Assn.)

Thickness in. (mm)	Approx. Weight psf (kg/m²)	Face Grain Across Supports: EI $10^6 \frac{lbin.^2}{ft}$ ($10^9 \frac{kPamm^4}{m}$)	Face Grain Across Supports: F_bKS $10^3 \frac{lbin.}{ft}$ ($10^3 \frac{Nmm}{m}$)	Face Grain Across Supports: F_sIb/Q $10^3 \frac{lb}{ft}$ ($10^3 \frac{N}{m}$)	Face Grain Parallel to Supports: EI $10^4 \frac{lbin.^2}{ft}$ ($10^9 \frac{kPamm^4}{m}$)	Face Grain Parallel to Supports: F_bKS $10^3 \frac{lbin.}{ft}$ ($10^3 \frac{Nmm}{m}$)	Face Grain Parallel to Supports: F_sIb/Q $10^3 \frac{lb}{ft}$ ($10^3 \frac{N}{m}$)
Plyform Class I							
½ (12.7)	1.5 (7.3)	0.116 (1087)	0.517 (191)	0.371 (5.41)	0.036 (339)	0.251 (93)	0.197 (2.88)
⅝ (15.9)	1.8 (8.8)	0.195 (1836)	0.691 (256)	0.412 (6.01)	0.057 (537)	0.338 (125)	0.223 (3.25)
¾ (19.1)	2.2 (10.7)	0.298 (2810)	0.878 (326)	0.517 (7.55)	0.138 (1299)	0.591 (219)	0.293 (4.27)
⅞ (22.2)	2.6 (12.7)	0.444 (4180)	1.127 (418)	0.616 (8.99)	0.226 (2132)	0.814 (302)	0.434 (6.33)
1 (25.4)	3.0 (14.6)	0.641 (6030)	1.422 (527)	0.675 (9.85)	0.405 (3813)	1.224 (454)	0.505 (7.37)
1⅛ (28.6)	3.3 (16.1)	0.831 (7824)	1.639 (607)	0.751 (10.96)	0.597 (5621)	1.542 (572)	0.606 (8.85)
Plyform Class II							
½ (12.7)	1.5 (7.3)	0.097 (918)	0.355 (132)	0.352 (5.14)	0.026 (245)	0.222 (82.3)	0.196 (2.87)
⅝ (15.9)	1.8 (8.8)	0.169 (1591)	0.475 (176)	0.403 (5.88)	0.042 (392)	0.299 (111)	0.221 (3.23)
¾ (19.1)	2.2 (10.7)	0.257 (2423)	0.604 (224)	0.477 (6.97)	0.097 (918)	0.521 (193)	0.292 (4.25)
⅞ (22.2)	2.6 (12.7)	0.390 (3672)	0.786 (291)	0.575 (8.40)	0.160 (1505)	0.721 (267)	0.432 (6.30)
1 (25.4)	3.0 (14.6)	0.547 (5153)	1.003 (372)	0.620 (9.05)	0.286 (2693)	1.080 (400)	0.503 (7.34)
1⅛ (28.6)	3.3 (16.1)	0.736 (6928)	1.156 (428)	0.689 (10.06)	0.420 (3953)	1.361 (504)	0.604 (8.81)

Table 13–6 *(Continued)*

Plyform Structural I							
½ (12.7)	1.5 (7.3)	0.117	0.523	0.501	0.043	0.344	0.278
		(1102)	(194)	(7.31)	(410)	(127)	(4.06)
⅝ (15.9)	1.8 (8.8)	0.196	0.697	0.536	0.067	0.459	0.313
		(1850)	(258)	(7.83)	(636)	(170)	(4.57)
¾ (19.1)	2.2 (10.7)	0.303	0.896	0.631	0.162	0.807	0.413
		(2853)	(332)	(9.21)	(1525)	(299)	(6.02)
⅞ (22.2)	2.6 (12.7)	0.475	1.208	0.769	0.268	1.117	0.611
		(4477)	(448)	(11.22)	(2528)	(414)	(8.92)
1 (25.4)	3.0 (14.6)	0.718	1.596	0.814	0.481	1.679	0.712
		(6765)	(592)	(11.88)	(4533)	(622)	(10.39)
1⅛ (28.6)	3.3 (16.1)	0.934	1.843	0.902	0.711	2.119	0.854
		(8798)	(683)	(13.16)	(6694)	(785)	(12.47)

*All properties adjusted to account for reduced effectiveness of plies with grain perpendicular to applied stress. Stresses adjusted for wet conditions, load duration, and experience factors.

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

Based on the selected stringer spacing, the process is repeated to determine the maximum stringer span (distance between vertical supports or shores). Notice in the design of stringers that the joist loads are actually applied to the stringer as a series of concentrated loads at the points where the joists rest on the stringer. However, it is simpler and sufficiently accurate to treat the load on the stringer as a uniform load. The width of the uniform design load applied to the stringer is equal to the stringer spacing as shown in Figure 13–1c. The calculated stringer span must next be checked against the capacity of the shores used to

Table 13–7 *(Continued)*

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}
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

support the stringers. The load on each shore is equal to the shore spacing multiplied by the load per unit length of stringer. Thus the maximum shore spacing (or stringer span) is limited to the lower of these two maximum values.

Although the effect of intermediate form members was ignored in determining allowable stringer span, it is necessary to check for crushing at the point where each joist rests on the stringer. This is done by dividing the load at this point by the bearing area and comparing the resulting stress to the allowable

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

unit stress in compression perpendicular to the grain. A similar procedure is applied at the point where each stringer rests on a vertical support.

To preclude buckling, the maximum allowable load on a rectangular wood column is a function of its unsupported length and least dimension (or *l/d* ratio). The *l/d* ratio must not exceed 50 for a simple solid wood column. For *l/d* ratios less than 50, the following equation applies:

$$F'_c = \frac{0.3E}{(l/d)^2} \leq F_c \tag{13–10}$$

where F_c = allowable unit stress in compression parallel to the grain (lb/sq in.) [kPa]
F'_c = allowable unit stress in compression parallel to the grain, adjusted for *l/d* ratio (lb/sq in.) [kPa]
E = modulus of elasticity (lb/sq in.) [kPa]
l/d = ratio of member length to least dimension

In using this equation, note that the maximum value used for F'_c may not exceed the value of F_c.

These design procedures are illustrated in the following example. Sheathing design employing plywood is illustrated in Example 13–2.

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 × 8 in. (50 × 200 mm) lumber will be used for joists. Stringers will be 4 × 8 in. (100 × 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:

	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 $\frac{\ell}{360}$. 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^3)

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

$$\left[\begin{aligned}
&\text{Pressure per m}^2\text{:}\\
\text{Concrete} = 1 \times 1 \times 0.152 \times 2403 \times 0.0098 &= 3.58 \text{ kPa}\\
\text{Formwork} &= 0.24 \text{ kPa}\\
\text{Live load} &= \underline{2.40 \text{ kPa}}\\
\text{Design load} &= 6.22 \text{ kPa}
\end{aligned}\right]$$

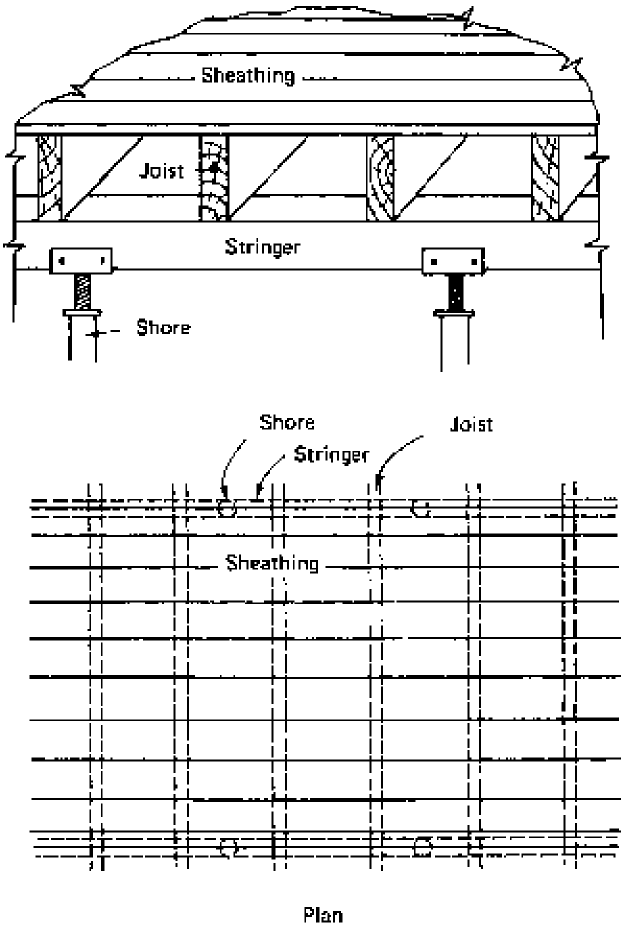
Deck Design

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 Tables 13–5 and 13–5A.

w = (1 sq ft/lin ft) × (130 lb/sq ft) = 130 lb/ft

[w = (1 m^2/m) × (6.22 kN/m^2) = 6.22 kN/m]

Figure 13–2 Slab form, Example 13–1.



(a) Bending:

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

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

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

(b) Shear:

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

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

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

(c) Deflection:

$$l = 1.69\left(\frac{EI}{w}\right)^{1/3} = 1.69\left(\frac{Ebd^3}{w12}\right)^{1/3}$$

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

$$\left[\begin{aligned} l &= \frac{73.8}{1000}\left(\frac{EI}{w}\right)^{1/3} = \frac{73.8}{1000}\left(\frac{Ebd^3}{w12}\right)^{1/3} \\ &= \frac{73.8}{1000}\left(\frac{(9.4 \times 10^6)(1000)(19)^3}{(12)(6.22)}\right)^{1/3} = 703 \text{ mm} \end{aligned} \right]$$

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 × 8 in. (50 × 200 mm) lumber. Assume that the joists are continuous over three spans.

$$w = (2 \text{ ft}) \times (1) \times (130 \text{ lb/sq ft}) = 260 \text{ lb/ft}$$

$$[w = (0.610 \text{ m}) \times (1) \times (6.22 \text{ kPa}) = 3.79 \text{ kN/m}]$$

(a) Bending:

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

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

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

(b) Shear:

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

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

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

(c) Deflection:

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

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

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

Thus bending governs and the maximum joist span is 87 in. (2213 mm). We will select a stringer spacing (joist span) of 84 in. (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 × 8 (100 × 200 mm) lumber. Now analyze the stringer as a beam and determine the maximum allowable span.

$$w = (7)\,(130) = 910 \text{ lb/ft}$$

$$[w = (2.13)\,(1)\,(6.22) = 13.25 \text{ kN/m}]$$

(a) Bending:

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

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

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

(b) Shear:

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

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

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

(c) Deflection:

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

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

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

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

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.

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.

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

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

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

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

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

$$f_{c\perp} = \frac{809 \times 10^6}{3382} = 2392 \text{ kPa} < 2792 \text{ kPa } (f_{c\perp})$$

Final Design

Decking: nominal 1-in. (25-mm) lumber

Joists: 2 × 8's (50 × 200-mm) at 24-in. (610-mm) spacing

Stringers: 4 × 8's (100 × 200-mm) at 84-in. (2.13-m) spacing

Shore: 4000-lb (17.8-kN) commercial shores at 48-in. (1.22-m) intervals

13–5 WALL AND COLUMN FORM DESIGN

Design Procedures

The design procedure for wall and column forms is similar to that used for slab forms substituting studs for joists, wales for stringers, and ties for shores. First, the maximum lateral pressure against the sheathing is determined from the appropriate equation (Equation 13–1, 13–2, or 13–3). If the sheathing thickness has been specified, the maximum allowable span for the sheathing based on bending, shear, and deflection is the maximum stud spacing. If the stud spacing is fixed, calculate the required thickness of sheathing.

Next, calculate the maximum allowable stud span (wale spacing) based on stud size and design load, again considering bending, shear, and deflection. If the stud span has already been determined, calculate the required size of the stud. After stud size and wale spacing have been determined, determine the maximum allowable spacing of wale supports (tie spacing) based on wale size and load. If tie spacing has been preselected, determine the minimum wale size. Double wales are commonly used (see Figure 13–3) to avoid the necessity of drilling wales for tie insertion.

Next, check the tie's ability to carry the load imposed by wale and tie spacing. The load (lb) [kN] on each tie is calculated as the design load (lb/sq ft) [kPa] multiplied by the product of tie spacing (ft) [m] and wale spacing (ft) [m]. If the load exceeds tie strength, a stronger tie must be used or the tie spacing must be reduced.

The next step is to check bearing stresses (or compression perpendicular to the grain) where the studs rest on wales and where tie ends bear on wales. Maximum bearing stress must not exceed the allowable compression stress perpendicular to the grain or crushing will result. Finally, design lateral bracing to resist any expected lateral loads, such as wind loads.

EXAMPLE 13–2

Forms are being designed for an 8-ft (2.44-m) -high concrete wall to be poured at a rate of 4 ft/h (1.219 m/h), internally vibrated, at a temperature of 90° F (32° C). The concrete mixture will use Type I cement without retarders and is estimated to weigh 150 lb/cu.ft (2403 kg/m^3). Sheathing will be 4 × 8–ft (1.2 × 2.4–m) sheets of ¾ in. (19 mm) thick Class I Plyform with face grain perpendicular to studs (see Figure 13–3). Studs and double wales will be 2 × 4-in. (50 × 100-mm) lumber. Snap ties are 3000-lb (13.34-kN) capacity with 1½-in. (38-mm) -wide wedges bearing on wales. Deflection must not exceed $\ell/360$. Determine stud, wale, and tie spacing. Use Plyform section properties and allowable stress from Table 13–6 and lumber section properties from Table 13–7. Allowable stresses for the lumber being used for studs and wales are:

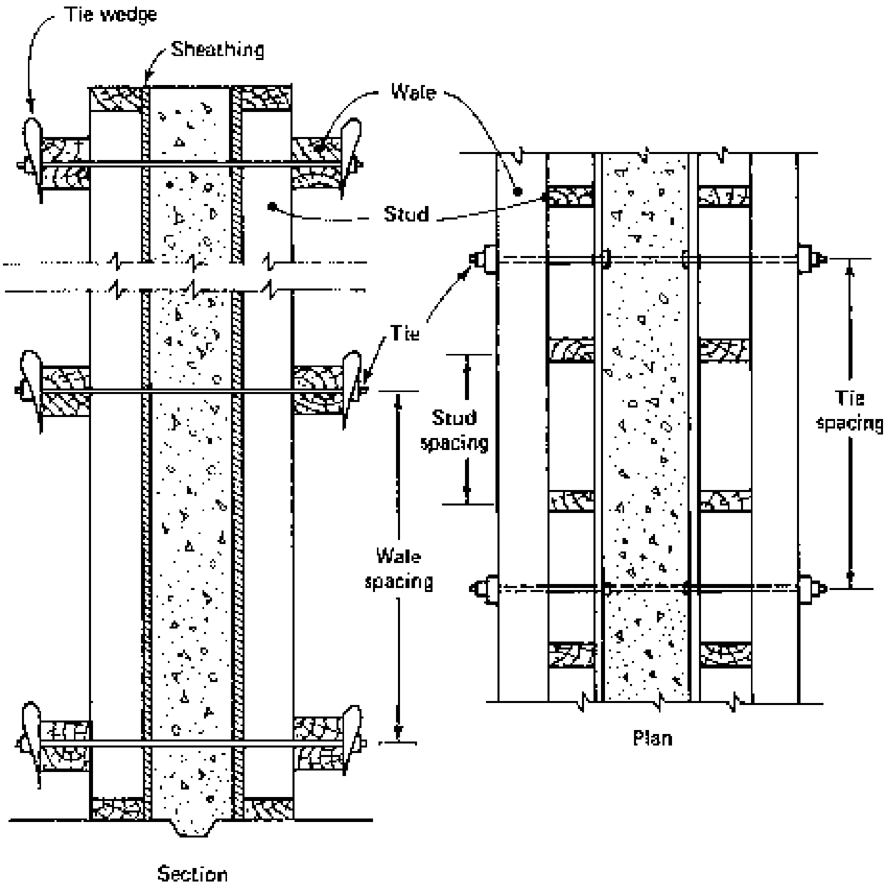


Figure 13–3 Wall form, Example 13–2.

F_b = 1810 lb/sq in. (12 480 kPa)
F_v = 120 lb/sq in. (827 kPa)
$F_{c\perp}$ = 485 lb/sq in. (3340 kPa)
$E = 1.7 \times 10^6$ lb/sq in. (11.7×10^6 kPa)

SOLUTION

Design Load

$C_w = 1.0$
$C_c = 1.0$

$$p = C_w C_c\left(150 + \frac{9000R}{T}\right) = (1)(1)\left\{150 + \frac{(9000)(4)}{90}\right\} = 550 \text{ lb/sq ft}$$

$$\left[p = C_w C_c\left(7.2 + \frac{785R}{T + 18}\right) = (1)(1)\left\{7.2 + \frac{(785)(1.219)}{32 + 18}\right\} = 26.3 \text{ kN/m}^2\right]$$

Use 600 lb/sq ft [28.7 kN/m^2].

Select Stud Spacing (Three or More Spans)

Material: ¾-in. (19-mm) Class I Plyform® (Table 13–4)
Consider a strip 12 in. wide (or 1 m wide):

$w = 1 \times 1 \times 600 = 600$ lb/ft

[$w = 1 \times 1 \times 28.7 = 28.7$ kN/m]

(a) Bending:

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

$$= 10.95\left(\frac{0.878 \times 10^3}{550}\right)^{1/2} = 13.2 \text{ in.}$$

$$\left[\begin{aligned} l &= 3.16\left(\frac{F_b KS}{w}\right)^{1/2} \\ &= 3.16\left(\frac{326 \times 10^3}{28.7}\right)^{1/2} = 337 \text{ mm}\end{aligned}\right]$$

(b) Shear:

$$l = 20\frac{F_s Ib/Q}{w} + 2d$$

$$= \frac{(20)(0.517 \times 10^3)}{600} + (2)(3/4)$$

$$= 17.2 + 1.5 = 18.7 \text{ in.}$$

$$\left[\begin{aligned} l &= 1.67\frac{F_s Ib/Q}{w} + 2d \\ &= \frac{(1.67)(7.55 \times 10^3)}{28.7} + (2)(.19) \\ &= 439 + 38 = 477 \text{ mm}\end{aligned}\right]$$

(c) Deflection:

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

$$= (1.69)\left(\frac{0.298 \times 10^6}{600}\right)^{1/3} = 13.4 \text{ in.}$$

$$\left[\begin{aligned} l &= \frac{73.8}{1000}\left(\frac{EI}{w}\right)^{1/3} \\ &= \frac{73.8}{1000}\left(\frac{2810 \times 10^9}{28.7}\right)^{1/3} = 340 \text{ mm}\end{aligned}\right]$$

Bending governs. Maximum span = 13.2 in. (337 mm). Use a 12-in. (304-mm) stud spacing.

Select Wale Spacing (Three or More Spans)

Since the stud spacing is 12 in. (304 mm), consider a uniform design load 1 ft (304 mm) wide resting on each stud.

$$w = 1 \times 1 \times 600 = 600 \text{ lb/ft}$$

$$\left[w = \frac{304}{1000} \times 1 \times 28.7 = 8.7 \text{ kN/m} \right]$$

(a) Bending:

$$l = 10.95 \left(\frac{F_b S}{2} \right)^{1/2}$$
$$= (10.95)\left(\frac{(1810)(3.063)}{600} \right)^{1/2} = 33.3 \text{ in.}$$

$$\left[\begin{aligned} l &= \frac{100}{1000} \left(\frac{F_b S}{w} \right)^{1/2} \\ l &= \frac{100}{1000} \left(\frac{(12\,480)(0.5019 \times 10^5)}{8.7} \right)^{1/2} = 849 \text{ mm} \end{aligned} \right]$$

(b) Shear:

$$l = 13.3 \frac{F_v A}{w} + 2d$$
$$= \frac{(13.3)(120)(5.25)}{600} + (2)(3.5) = 21.0 \text{ in.}$$

$$\left[\begin{aligned} l &= \frac{1.11}{1000} \frac{F_v A}{w} + 2d \\ &= \frac{(1.11)(827)(3.387 \times 10^3)}{(1000)(8.7)} + (2)(88.9) = 535 \text{ mm} \end{aligned} \right]$$

(c) Deflection:

$$l = 1.69 \left(\frac{El}{w} \right)^{1/3}$$
$$= 1.69 \left(\frac{1.7 \times 10^6)(5.359)}{600} \right)^{1/3} = 41.8 \text{ in.}$$

$$\left[\begin{aligned} l &= \frac{73.8}{1000} \left(\frac{El}{w} \right)^{1/3} \\ &= \frac{73.8}{1000} \left(\frac{(11.7 \times 10^6)(2.231 \times 10^6)}{8.7} \right) = 1064 \text{ mm} \end{aligned} \right]$$

Shear governs, so maximum span (wale spacing) is 22.23 in. (566 mm). Use a 16-in. (406-mm) wale spacing for modular units.

Select Tie Spacing (Three or More Spans, Double Wales)

Based on a wale spacing of 16 in. (406 mm):

$$w = \frac{16}{12} \times 600 = 800.0 \text{ lb/ft}$$

$$\left[w = \frac{406}{304} \times 8.7 = 11.6 \text{ kN/m} \right]$$

(a) Bending:

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

$$= 10.95 \left(\frac{(1810)(2 \times 3.063)}{800} \right)^{1/2} = 40.8 \text{ in.}$$

$$\left[\begin{aligned} l &= \frac{100}{1000} \left(\frac{F_b S}{w} \right)^{1/2} \\ &= \frac{100}{1000} \left(\frac{(12480)(2 \times 0.5019 \times 10^5)}{11.6} \right)^{1/2} = 1039 \text{ mm} \end{aligned} \right]$$

(b) Shear:

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

$$= \frac{(13.3)(120)(2 \times 5.25)}{800} + (2)(3.5) = 27.9 \text{ in.}$$

$$\left[\begin{aligned} l &= \frac{1.11}{1000} \frac{F_v A}{w} + 2d \\ &= \frac{(1.11)(827)(2 \times 3.387 \times 10^3)}{(1000)(11.6)} + (2)(88.9) = 714 \text{ mm} \end{aligned} \right]$$

(c) Deflection:

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

$$= 1.69 \left(\frac{(1.7 \times 10^6)(2 \times 5.359)}{800} \right)^{1/3} = 47.9 \text{ in.}$$

$$\left[\begin{aligned} l &= \frac{73.8}{1000} \left(\frac{EI}{w} \right)^{1/3} \\ &= \frac{73.8}{1000} \left(\frac{(11.7 \times 10^6)(2 \times 2.231 \times 10^6)}{11.6} \right)^{1/3} = 1218 \text{ mm} \end{aligned} \right.$$

Shear governs. Maximum span is 27.9 in. (714 mm). Select a 24-in. (610-mm) tie spacing for a modular value.

(d) Check tie load:

$$P = \text{Wale spacing} \times \text{Tie spacing} \times p$$

$$= \frac{16}{12} \times \frac{24}{12} \times (600) = 1600 \text{ lb/tie} < 3000 \text{ lb} \qquad OK$$

$$\left[P = \frac{(406)(610)}{(1000)(1000)} \times 28.7 = 7.11 \text{ kN} < 13.34 \text{ kN}\right]$$

Check Bearing

(a) Stud on wales:

Bearing area (A) (double wales) = (2)(1.5)(1.5) = 4.5 sq in.

$[A = (2)(38)(38) = 2888 \text{ mm}^2]$

Load at each panel point (P) = Load/ft(m) of stud × Wale spacing (ft/m)

$$P = (600)\frac{16}{12} = 800 \text{ lb}$$

$$\left[P = (8.7)\frac{406}{1000} = 3.53 \text{ kN}\right]$$

$$f_c = \frac{P}{A} = \frac{800}{4.5} = 178 \text{ lb/sq in.} < 485 \text{ lb/sq in.} (F_c) \qquad OK$$

$$\left[f_c = \frac{3.53 \times 10^6}{2888} = 1222 \text{ kPa} < 3340 \text{ kpa}(F_c)\right]$$

(b) Tie wedges on wales:

$$\text{Tie load } (P) = 1600 \text{ lb } [7.11 \text{ kN}]$$

$$\text{Bearing area } (A) = (1.5)(1.5)(2) = 4.5 \text{ sq in.}$$

$$[A = (38)(38)(2) = 2888 \text{ mm}^2]$$

$$f_{c_\perp} = \frac{P}{A} = \frac{1600}{4.5} = 356 \text{ lb/sq in.} < 485 \text{ lb/sq in.} (F_{c_\perp})\perp \qquad OK$$

$$\left[f_{c_\perp} = \frac{7.11 \times 10^6}{2888} = 2462 \text{ kPa} < 3340 \text{ kPa } (F_{c_\perp})\right]$$

Final Design

Sheathing: 4 × 8 ft (1.2 × 2.4 m) sheets of ¾-in. (19-mm) Class I Plyform placed with the long axis horizontal.

Studs: 2 × 4's (50 × 100 mm) at 12 in. (304 mm) on center.

Wales: Double 2 × 4's (50 × 100 mm) at 16 in. (406 mm) on center.

Ties: 3000-lb (13.34-kN) snap ties at 24 in. (610 mm) on center.

13–6 DESIGN OF LATERAL BRACING

Many failures of formwork have been traced to omitted or inadequately designed lateral bracing. Minimum lateral design load values were given in Section 13–2. Design procedures for lateral bracing are described and illustrated in the following paragraphs.

Lateral Braces for Wall and Column Forms

For wall and column forms, lateral bracing is usually provided by inclined rigid braces or guy-wire bracing. Since wind loads, and lateral loads in general, may be applied in either direction perpendicular to the face of the form, guy-wire bracing must be placed on both sides of the forms. When rigid braces are used they may be placed on only one side of the form if designed to resist both tension and compression forces. When forms are placed on only one side of a wall with the excavation serving as the second form, lateral bracing must be designed to resist the lateral pressure of the concrete as well as other lateral forces.

Inclined bracing will usually resist any wind uplift forces on vertical forms. However, uplift forces on inclined forms may require additional consideration and the use of special anchors or tiedowns. The strut load per foot of form developed by the design lateral load can be calculated by the use of Equation 13–11. The total load per strut is then P′ multiplied by strut spacing.

$$P' = \frac{H \times h \times l}{h' \times l'} \qquad \textbf{(13–11)}$$

$$l = (h'^2 + l'^2)^{1/2} \qquad \textbf{(13–12)}$$

where P' = strut load per foot of form (lb/ft) [kN/m]
H = lateral load at top of form (lb/ft) [kN/m]
h = height of form (ft) [m]
h' = height of top of strut (ft) [m]
l = length of strut (ft) [m]
l' = horizontal distance from form to bottom of strut (ft) [m]

If struts are used on only one side of the form, the allowable unit stress for strut design will be the lowest of the three possible allowable stress values (F_c, F'_c, or F_t).

EXAMPLE 13–3

Determine the maximum spacing of nominal 2 × 4-in. (50 × 100-mm) lateral braces for the wall form of Example 13–2 placed as shown in Figure 13–4. Assume that local code wind requirements are less stringent than Table 13–3. Allowable stress values for the braces are as follows.

	Allowable Stress	
	lb/sq in.	***kPa***
F_c	850	5861
F_t	725	4999
E	1.4×10^6	9.7×10^6

SOLUTION

Determine the design lateral force per unit length of form.

$$H = 100 \text{ lb/ft (Table 13–3)}$$
$$[H = 1.46 \text{ kN/m (Table 13–3)}]$$

Determine the length of the strut using Equation 13–12.

$$l = (h'^2 + l'^2)^{1/2}$$
$$= (6^2 + 5^2)^{1/2} = 7.81 \text{ ft}$$
$$[l = (1.83^2 + 1.53^2)^{1/2} = 2.38 \text{ m}]$$

The axial concentrated load on the strut produced by a unit length of form may now be determined from Equation 13–1.

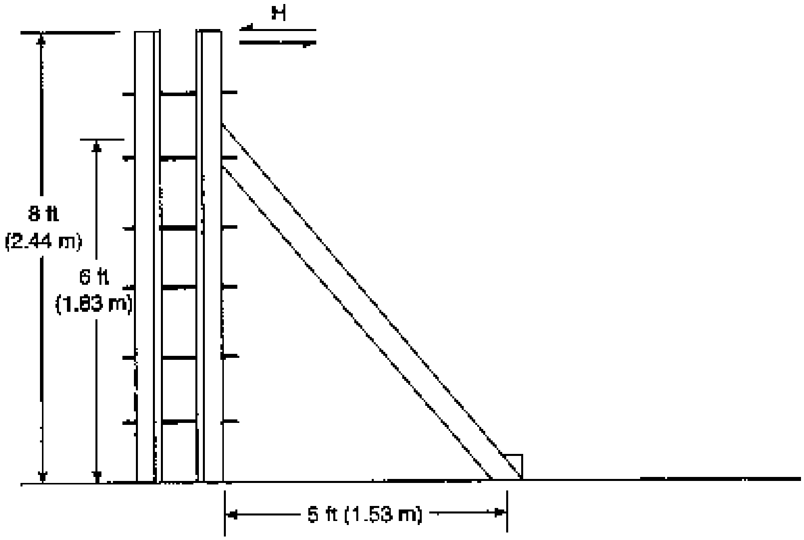
$$P' = \frac{H \times h \times l}{h' \times l'} = \frac{(100)(8)(7.81)}{(6)(5)} = 208.3 \text{ lb/ft of form}$$
$$\left[P' = \frac{(1.46)(2.44)(2.38)}{(1.83)(1.53)} = 3.03 \text{ kN/m}\right]$$

Next, we determine the allowable compressive stress for each strut using Equation 13–10. To do this, we must determine the *l/d* ratio of the strut.

$$l/d = \frac{(7.81)(12)}{1.5} = 62.5 > 50$$
$$\left[l/d = \frac{(2.38)(1000)}{(38.1)} = 62.5 > 50\right]$$

Since the *l/d* ratio exceeds 50, each strut must be provided lateral bracing to reduce its unsupported length. Try a single lateral support located at the midpoint of each strut, reducing ℓ to 46.9 in. (1.19 m).

Figure 13–4 Wall form bracing, Example 13–3.



$$Fc' = \frac{0.3\,E}{(l/d)^2} = \frac{(0.3)(1.4 \times 10^6)}{(46.9/1.5)^2} = 430 \text{ psi}$$

$$\left[Fc' = \frac{(0.3)(9.7 \times 10^6)}{(1190/38)^2} = 2.97 \text{ kPa}\right]$$

As $Fc' < Ft < Fc$, the value of Fc' governs.
The maximum allowable compressive force per strut is:

$$P = (1.5)(3.5)(430) = 2257 \text{ lb}$$

$$\left[P = \frac{(38)(89)(2967)}{10^6} = 10.03 \text{ kN}\right]$$

Thus maximum strut spacing is:

$$s = \frac{P}{P'} = \frac{2257}{208.3} = 10.8 \text{ ft}$$

$$\left[s = \frac{10.03}{3.03} = 3.31 \text{ m}\right]$$

Keep in mind that this design is based on providing lateral support to each strut at the midpoint of its length.

Lateral Braces for Slab Forms

For elevated floor or roof slab forms, lateral bracing may consist of cross braces between shores or inclined bracing along the outside edge of the form similar to that used for wall forms. The following example illustrates the method of determining the design lateral load for slab forms.

EXAMPLE 13–4

Determine the design lateral force for the slab form 6 in. (152 mm) thick, 20 ft (6.1 m) wide, and 100 ft (30.5 m) long shown in Figure 13–5. The slab is to be poured in one pour. Assume concrete density is 150 lb/cu ft (2403 kg/m^3) and that the formwork weighs 15 lb/sq ft (0.72 kPa).

SOLUTION

$$\text{Dead load} = (1/2)(1)(150) + 15 = 90 \text{ lb/sq ft}$$

$$\left[dl = \frac{(0.152)(1)(2403)(9.8)}{1000} + 0.72 = 4.30 \text{ kPa} \right]$$

$$H = 0.02 \times dl \times ws$$

For the 20-ft (6.1-m) face, the width of the slab is 100 ft (30.5 m).

$$H_{20} = (0.02)(90)(100) = 180 \text{ lb/lin ft}$$

$$[H = (0.02)(4.30)(30.5) = 2.62 \text{ kN/m}]$$

For the 100-ft (30.5-m) face, the width of the slab is 20 ft (6.1 m).

$$H_{100} = (0.02)(90)(20) = 36 \text{ lb/ft} < 100 \text{ lb/ft (minimum value)}$$

$$[H = (0.02)(4.29)(6.1) = 0.52 \text{ kN/m} < 1.46 \text{ kN/m}]$$

Therefore, use H_{100} = 100 lb/ft (1.46 kN/m) = minimum load.

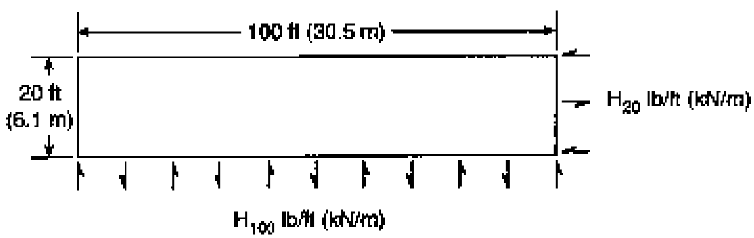


Figure 13–5 Slab form bracing design, Example 13–4.

PROBLEMS

1. Derive the equations for bending given in Table 13–5 using the relationships of Table 13–4 and Equation 13–5.
2. Calculate the maximum allowable span for ¾-in. (19-mm) Class I Plyform decking with face grain across supports carrying a design load of 150 lb/sq ft (7.2 kPa). Assume that the decking is continuous over three or more spans and limit deflection to $\frac{1}{240}$ of span length.
3. Design the formwork for a wall 8 ft (2.44 m) high to be poured at the rate of 5 ft/h (1.53 m/h) at a temperature of 77° F (25° C). The concrete mixture will use Type I cement without retarders and is estimated to weigh 150 lb/cu ft (2403 kg/m^3). Sheathing will be ¾-in. (19-mm) Class I Plyform with face grain across supports. All lumber is Southern Pine. Use nominal 2 × 4-in (50 × 100-mm) studs, double 2 × 4-in (50 × 100-mm) wales, and 3000-lb (13.3-kN) snap ties. Lateral wood bracing will be attached at a height of 5 ft (1.53 m) above the form bottom and anchored 4 ft (1.22 m) away from the bottom of the form. Use the allowable stresses of Table 13–8 adjusted for a 7-d load. Limit deflection to $\frac{1}{360}$ of the span length. Design wind load is 25 lb/sq ft (1.2 kPa).
4. What should be the design load for a column form 18 ft (5.5 m) high that is to be filled by pumping concrete weighing 150 lb/cu ft (2403 kg/m^3) from the bottom?
5. Determine the maximum allowable spacing of nominal 2 × 4-in (50 × 100-mm) studs for a wall form sheathed with nominal 1-in. (25-mm) lumber. Assume that the sheathing is continuous over three or more spans and is Hem-Fir. Limit deflection to $\frac{1}{240}$ of the span length. The design load is 600 lb/sq ft (28.7 kPa).
6. Determine the maximum allowable span of nominal 2 × 4-in. (50 × 100-mm) wall form studs carrying a design load of 1000 lb/sq ft (47.9 kPa). Tie stud spacing is 16 in. (406 mm) on center. Use the allowable stresses for Douglas Fir from Table 13–8 and 7-d load duration. Assume that studs are continuous over three or more spans. Limit deflection to $\frac{1}{360}$ of span length. Based on the maximum allowable span, check for crushing of studs on double 2 × 4 in. (50 × 100 mm) wales.
7. Design the formwork for a concrete slab 8 in. (203 mm) thick whose net width between beam faces is 15 ft (4.58 m). Concrete will be placed using a crane and bucket. The formwork is estimated to weigh 5 lb/sq ft (24.1 kg/m^2). Decking will be ¾ in. (19 mm) Class I Plyform with face grain across supports. All lumber will be Southern Pine. Joists will be nominal 2-in. (50-mm) -wide lumber. One 4-in. (100-mm) -wide stringer will be placed between the beam faces. Limit deflection to $\frac{1}{360}$ of span length. Commercial shores of 4500 lb (20 kN) capacity will be used. Lateral support will be provided by the beam forms.
8. What loads must formwork for elevated slabs be designed to resist?
9. Calculate the design load for the form of a floor slab 8 in. (203 mm) thick if the hand buggies are to be used and the formwork weighs 10 lb/sq ft (48.8 kg/m^2).
10. Develop a computer program to calculate the maximum span of a plywood deck used as an elevated slab form based on the equations of Table 13–5 and the Plyform

properties given in Table 13–6. Input should include design load, support conditions, allowable deflections, Plyform type and thickness, and whether face grain is across supports or parallel to supports.

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