

# *Chapter (4)*

## *Formwork*

### *Design Applications*

#### ***4.1 Design of Formwork for Slabs:***

Figure (4.1) shows a typical structural system for job-built forms for elevated slabs. The sequence of design is first to consider a strip of sheathing of the specified thickness and (12 in).in width. The maximum allowable span may then be determined based on allowable values of bending stress, shear stress, and deflection for the sheathing. The lower of the computed values will determine the maximum spacing of the joists. This span value, usually rounded down to some lower modular value, becomes the spacing of the joists.

Based on the joist spacing used, the joist itself is analyzed to determine its maximum allowable span. Each joist must support the load from the sheathing halfway over to the adjacent joist on either side. Therefore, the width of the load area carried by the joist is equal to the spacing of the joists. The joist span selected becomes the spacing of the stringers. Again, a modular value is selected for stringer spacing.

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. It is simpler and sufficiently accurate to treat the load on the stringer as a uniformly distributed load, however. Again, the width of the uniform design load applied to the stringer is equal to the stringer spacing.

The calculated stringer span must next be checked against the capacity of the shores used to support the stringers. The load on each shore is equal to the shore spacing multiplied by the load per foot of stringer.

Thus the maximum shore spacing (or stringer span) is limited to the lower span length as governed by stringer strength or shore strength. In addition, it is necessary to check the bearing at the point where each joist rest on the stringer. This is done by dividing the load at this point by the bearing area and comparing the resulting stress with the allowable unit stress in compression perpendicular to the grain. A similar procedure is applied at the point where each stringer rest on a vertical support.

The stringers shown in Figure (4.1) are supported by solid, rectangular wood shores that are columns (which we will assume are axially loaded). As with all axially loaded columns, the allowable load is a function of the slenderness ratio ( $\ell/d$ ), the ratio of the unbraced length of the member to its least lateral dimension (not to exceed 50). The slenderness ratio ( $\ell/d$ ) is further modified and expressed as ( $\ell_e/d$ ), where  $\ell_e$  represents an effective unbraced column length and is defined as:

$$\ell_e = k_e \ell$$

Where ( $k_e$ ) is an effective length factor based on column end conditions that affect rotation and translation. Factor  $k_e$  can be obtained from (Table 10).

The following design approach applies for the determination of the allowable stress for compression parallel to the grain for simple solid-sawn lumber columns.

- *Determination of allowable stress for compression parallel to the grain ( $F'_c$ ):*

Applicable adjustment factors for allowable stress for compression parallel to the grain are load duration, wet service, temperature, size, and column stability. This may be expressed as:

$$F'_c = F_c C_D C_M C_t C_F C_P$$

All other quantities have been previously defined. This allowable stress is applicable to all values of the slenderness ratio ( $\leq 50$ ) and replaces the previously used short-, intermediate-, and long-column equation of earlier specifications. Note that although the slenderness ratio ( $\ell_e/d$ ) for solid columns shall not exceed 50, during construction this limit is increased to 75.

Because wood shores (columns) are generally reused repeatedly, ACI Committee 347 does not recommend the use of any adjustment factor that provides increased stresses for short load duration. In addition, temperature and wet service adjustments for wood shores are generally not required or considered. Therefore

only the column size factor and the column stability factor are normally considered in shore design.

The size factor, ( $C_F$ ), for compression parallel to the grain for the two grades indicated can be taken from (Table 9).

The column stability factor, ( $C_P$ ), is a function of the effective slenderness ratio ( $le/d$ ) of the shore, the adjusted modulus of elasticity  $E'$ , and the adjusted base value of compression parallel to the grain before ( $C_P$ ) is applied.

The unadjusted modulus of elasticity is normally used for wood shore design as shores are rarely in the wet service condition. Therefore:

$$E' = E.$$

The column stability factor ( $C_P$ ) for solid-sawn lumber can be calculated from Equation (3.5):

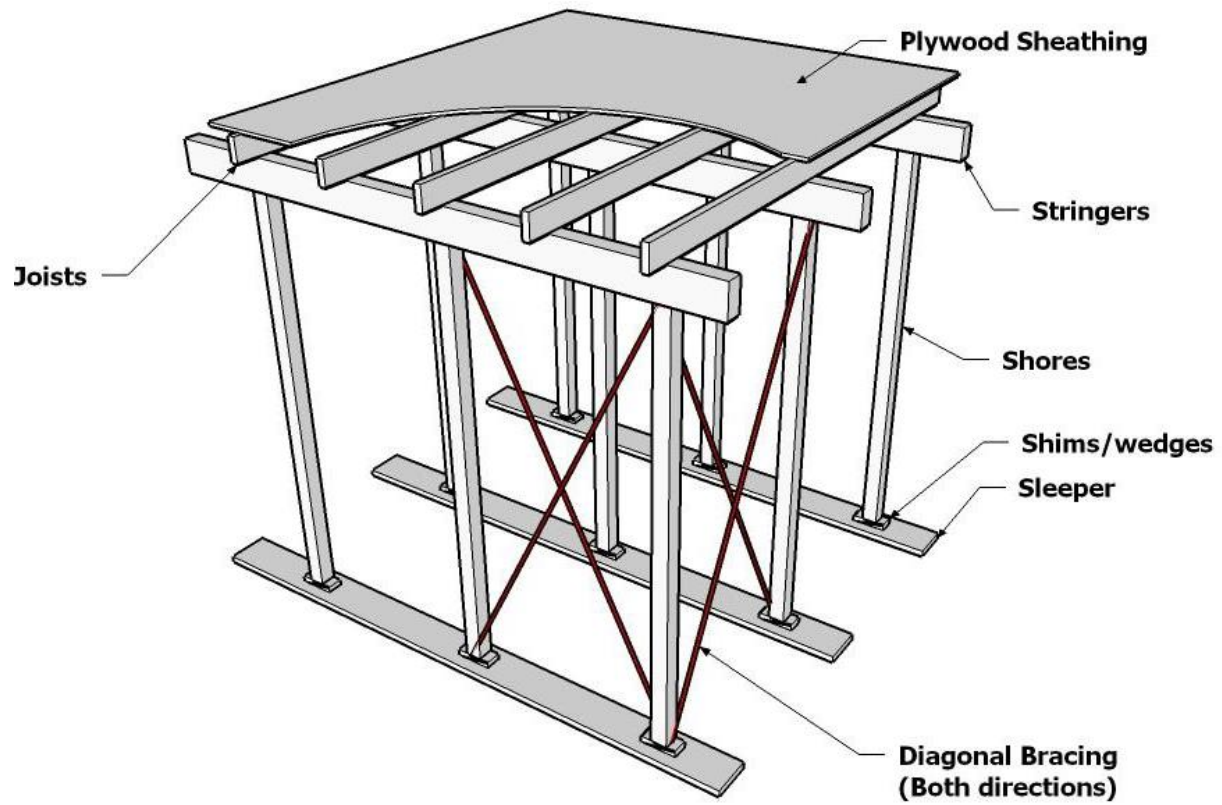
$$C_P = \frac{1 + \alpha}{1.6} - \sqrt{\frac{(1 + \alpha)^2}{1.6} - \frac{\alpha}{0.8}} \quad (4.1)$$

Where:

$$\alpha = \frac{0.3 E'}{\left(\frac{le}{d}\right)^2 \bar{F}_c}$$

$\bar{F}_c$  is the adjusted base value of compression stress parallel to the grain before application of  $C_P$ .

Various tables are available to simplify the design approach and to expedite the selection of the formwork structural member. These should be used with caution because of job-specific conditions and possible tabular limitations.



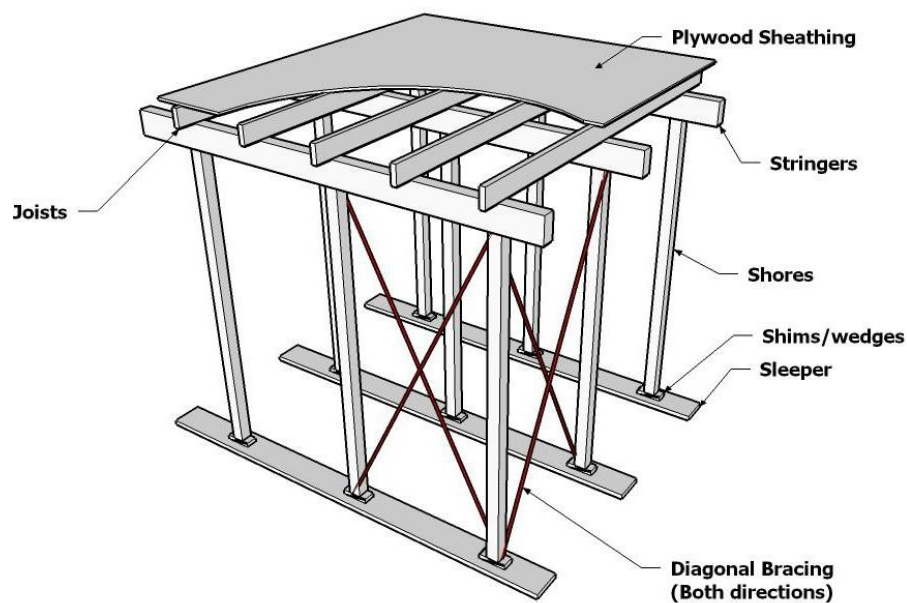
**Figure (4.1):** Typical job-built form for elevated slab.

### ***Example 4-1:***

Design the formwork for a 6-in. structural concrete floor slab. The floor system is of the type shown in figure (4.2). Use  $\frac{3}{4}$ -in. class I plyform for the sheathing and No. 2 Douglas fir-larch for the rest of the lumber. The maximum deflection for the sheathing may be taken as  $1/240$  of the span. The maximum deflection for other bending member is taken as  $1/360$  of the span. Based on end conditions, the effective un-braced length of the shores may be taken as 10 ft.

The following conditions for design have been established:

- 1- Joists and stringers will be designed based on adequate lateral support.
- 2- Joists, stringer, and shores are used under dry service conditions.



**Figure (4.2):** Typical job-built form for elevated slab.

**EXAMPLE (1) Slabs Formwork Design** (ACI Committee 347)

REFERENCE	CALCULATION	OUTPUT
	<p><b><i>Solution:</i></b></p> <p>1. <b><i>Sheathing design</i></b> (find the joist spacing): Consider a 12-in-wide strip of sheathing perpendicular to the supporting joists. The sheathing acts as a beam and is continuous over three or more supports. Determine the maximum allowable span for the sheathing. This becomes the maximum spacing for the joists (which support the sheathing):</p> <p>a- The design values for 3/4-in class I plyform are:</p> $E = 1,650,000 \text{ psi} \quad (\text{modulus of elasticity}).$ $F_b = 1930 \text{ psi} \quad (\text{bending stress}).$ $F_v = 72 \text{ psi} \quad (\text{rolling shear stress}).$ <p>The plyform properties with face grain parallel to span (perpendicular to the joists) are:</p> $I = 0.199 \text{ in.}^4/\text{ft}$ $S = 0.455 \text{ in.}^3/\text{ft}$ $Ib/Q = 7.187 \text{ in.}^2/\text{ft} \quad (\text{rolling shear constant}).$ $ws = 2.2 \text{ psf}$ <p>(weight of sheathing: for a 12 in. trip, this will be lb/ft).</p>	
Table (5)		
Table (5)		

*b- The loading (w) on the sheathing is:*

$$DL \text{ (slab): } (6/12)(150) = 75 \text{ psf}$$

$$LL \text{ (min): } = 50 \text{ psf}$$

*\* Neglect sheathing weight.*

$$w = (75 \text{ psf} + 50 \text{ psf}) = 125 \text{ psf}$$

*c- The maximum joist spacing based on the bending moment formula is:*

$$\begin{aligned} l &= 10.95 \sqrt{\frac{F_b S}{w}} \\ &= 10.95 \sqrt{\frac{1930(0.455)}{125}} = 29 \text{ in.} \end{aligned}$$

*d- The maximum joist spacing based on shear is:*

$$\begin{aligned} l &= \frac{20 F_v (I_b/Q)}{w} + 2d \\ &= \frac{20(72) (7.187)}{125} + 2(0.75) = 84.3 \text{ in.} \end{aligned}$$

*e- The maximum joist spacing based on deflection (maximum deflection is  $1/240$  of the span) is:*

$$\begin{aligned} l &= 1.94 \sqrt[3]{\frac{EI}{w}} \\ &= 1.94 \sqrt[3]{\frac{1,650,000 (0.199)}{125}} = 26.8 \text{ in.} \end{aligned}$$

Table (4)

Table (4)

Table (4)



	<p># Deflection controls. The maximum spacing of supporting joists should not exceed (26.8 in). Use joists at (24 in.) o.c.</p>	24 in.
Table (7)	<p>2. <b>Joist design</b> (find the stringer spacing): Assume 2 in × 8 in (S4S) joists and a 7-day maximum duration of load. Consider the joists as uniformly loaded beams continuous over three or more spans (the supports are the stringers):</p> <p>a- Obtain allowable stresses using base design values and appropriate adjustment factor:</p> <p>(1) Bending: <math>F_b = 900 \text{ psi}</math>.</p> <p>Adjustment factors:</p> <p>a. Load duration factor <math>C_D = 1.25</math></p> <p>b. Size factor <math>C_F = 1.2</math></p> <p>Therefore:</p> $F'_b = F_b C_D C_F$ $= 900 (1.25) (1.2) = 1350 \text{ psi.}$ <p>(2) Shear: <math>F_v = 180 \text{ psi}</math>.</p> <p>Adjustment factors:</p> <ul style="list-style-type: none"> <li>Load duration factor <math>C_D = 1.25</math></li> </ul> <p>Therefore:</p> $F'_v = F_v C_D$ $= 180 (1.25) = 225 \text{ psi.}$	

(3) *Modulus of elasticity:*

$$E=1,600,000 \text{ psi}$$

*No adjustment factors apply. Thus:*

$$E' = E = 1,600,000 \text{ psi.}$$

Table (6)

*Properties for the 2 × 8 (S4S) are:*

$$S = 13.14 \text{ in.}^3$$

$$I = 47.63 \text{ in.}^4$$

$$A = 10.88 \text{ in.}^2$$

*b- Loading: because joists support 2-ft width of sheathing, the loading on the joist is:*

$$w = 125 \text{ psi (2)} = 250 \text{ lb/ft.}$$

*Assume the weight of the sheathing and joists to be (5 psf). Then:*

$$w = 250 + 5 (2) = 260 \text{ lb/ft.}$$

*c- The maximum stringer spacing based on bending moment is:*

$$\begin{aligned} l &= 10.95 \sqrt{\frac{F'_{bs} S}{w}} \\ &= 10.95 \sqrt{\frac{1350 (13.14)}{260}} = 90.4 \text{ in.} \end{aligned}$$

Table (4)

*d- The maximum stringer spacing based on shear is:*

Table (4)

$$\begin{aligned} l &= \frac{13.3 F'_{vs} A}{w} + 2d \\ &= \frac{13.3(225)(10.88)}{260} + 2(7.25) = 139.7 \text{ in.} \end{aligned}$$

Table (4)	<p>e- The maximum stringer spacing based on deflection (maximum deflection is <math>1/360</math> of the span) is:</p> $l = 1.69 \sqrt[3]{\frac{EI}{w}}$ $= 1.69 \sqrt[3]{\frac{1,600,000 (47.63)}{260}} = 112.3 \text{ in.}$ <p># Bending governs. Therefore the maximum spacing of supporting stringers cannot exceed (90.4 in.).</p> <p>Use a stringer spacing of 7 ft-0 in (84 in.).</p>	84 in.
Table (7)	<p>3. <b>Stringer design</b> (find the shore spacing): 2 in × 8 in. (S4S) stringers and a 7-day maximum duration of load. Consider the stringers to be uniformly loaded beams continuous over three or more supports. The supports are the shores. The loads from the joists are concentrated loads, but for simplicity, we will assume a uniformly distributed load:</p> <p>a- Obtain allowable stresses using base design values and appropriate adjustment factor:</p> <p>(1) Bending: <math>F_b = 900 \text{ psi}</math>.</p> <p>Adjustment factors:</p> <p>a. Load duration factor <math>C_D = 1.25</math></p> <p>b. Size factor <math>C_F = 1.3</math></p> <p>Therefore:</p>	
Table (7)		
Table (8)		

Table (7)

$$F'_b = F_b C_D C_F$$

$$= 900 (1.25) (1.3) = 1463 \text{ psi.}$$

(2) *Shear:  $F_v = 180 \text{ psi.}$*

*Adjustment factors:*

*Load duration factor  $C_D = 1.25$*

*Therefore:*

$$F'_v = F_v C_D$$

$$= 180 (1.25) = 225 \text{ psi.}$$

Table (6)

(3) *Modulus of elasticity:*

$$E = 1,600,000 \text{ psi}$$

*No adjustment factors apply. Thus:*

$$E' = E = 1,600,000 \text{ psi.}$$

*Properties for the  $2 \times 8$  (S4S) are:*

$$S = 30.66 \text{ in.}^3$$

$$I = 111.1 \text{ in.}^4$$

$$A = 25.38 \text{ in.}^2$$

b- *Loading: Each stringer supports 7 ft-0 in.-wide strip of design load. Assuming a formwork weight of (5 psf), the uniformly distributed load on a stringer is calculated from:*

$$w = 7.0 (125 + 5) = 910 \text{ lb/ft.}$$

Table (4)

c- The maximum shore spacing based on bending (of the stringers) is:

$$l = 10.95 \sqrt{\frac{F'_{bs}}{w}}$$

$$= 10.95 \sqrt{\frac{1463 (30.66)}{910}} = 76.9 \text{ in.}$$

Table (4)

d- The maximum shore spacing based on shear is:

$$l = \frac{13.3 F'_{vs} A}{w} + 2d$$

$$= \frac{13.3(225)(25.38)}{910} + 2(7.25) = 98.0 \text{ in.}$$

Table (4)

e- The maximum shore spacing based on deflection is:

$$l = 1.69 \sqrt[3]{\frac{EI}{w}}$$

$$= 1.69 \sqrt[3]{\frac{1,600,000 (111.1)}{910}} = 98.1 \text{ in.}$$

# Bending governs. Therefore the maximum spacing of supporting shores cannot exceed (76.9 in.). Use a shore spacing of 6 ft-0 in. (72.0 in.).

72 in.

*4. Design of shores: the stringers are spaced at 6 ft-6 in. on center and are supported by shores at 6 ft-0 in. on center. Therefore each shore must support a floor area of:*

$$7.0 (6.0) = 42 \text{ ft}^2.$$

*Again, assuming formwork weight of (5psf), the load per shore is calculated as:*

$$42 (125+5) = 5460 \text{ lb}$$

*Although commercial shores are usually readily available to support this load, we will design 4 × 4 wood shores. The effective un-braced length of the shore,  $l_e$ , is 10 ft-0 in. in each direction. The capacity of the 4 × 4 (S4S) shore is calculated using:*

- a. The base design value for compression parallel to the grain is:*

$$F_c = 1350 \text{ psi}$$

- b. Adjustment factors:*

*(1) Size factor:  $C_F = 1.15$*

*(2) For the column stability factor  $C_p$ , initially the following items must be established:*

- i- For modulus of elasticity, there is no adjustment factor:*

$$E' = E = 1,600,000 \text{ psi.}$$

Table (7)

Table (9)

ii- Find  $F_c^* = F_c C_F$

$$F_c^* = 1350(1.15) = 1553 \text{ psi.}$$

iii-  $\frac{le}{d} = \frac{10(12)}{3.5} = 34.3 < 50 \text{ (O.K)}$

iv- Solve for  $\alpha$ :

$$\alpha = \frac{0.3 E'}{\left(\frac{le}{d}\right)^2 F_c^*}$$

$$\alpha = \frac{0.3(1,600,000)}{34.3^2(1553)} = 0.263$$

Solve for  $C_p$ :

$$C_p = \frac{1+\alpha}{1.6} - \sqrt{\left(\frac{1+\alpha}{1.6}\right)^2 - \frac{\alpha}{0.8}}$$

$$C_p = \frac{1+0.263}{1.6} - \sqrt{\left(\frac{1+0.263}{1.6}\right)^2 - \frac{0.263}{0.8}} = 0.247$$

c. Compute the allowable stress  $F'_c$ :

$$\begin{aligned} F'_c &= F_c C_F C_p \\ &= 1350(1.15)(0.247) = 383 \text{ psi.} \end{aligned}$$

Therefore the allowable load is:

$$\begin{aligned} P &= F'_c A \\ &= 383(3.5)^2 = 4690 \text{ lb} \quad (\text{N.G.}) \\ 4690 \text{ lb} &< 5460 \text{ lb} \end{aligned}$$

*There are several possible solutions. The shore size could be increased (try a 4× 4 [S4S]), use lateral bracing (horizontal lacing) at midheight to reduce the effective length of the shore, or reduce the shore spacing. The latter choice is the simplest approach.*

$$\text{New required spacing} = \frac{4690}{5460} (72\text{in}) = 61.8 \text{ in.}$$

*If a shore spacing of 5 ft-0 in. is used:*

$$\text{Load per shore} = 7.0(5.0)(125+5) = 4550 \text{ lb.}$$

$$4550 \text{ lb} < 4690 \text{ lb}$$

*(O.K).*

*Use 4× 4 (S4S) shores at 5 ft-0 in. on center.*

5ft-0 in.

### **5. Bearing stresses:**

*a- Where stringers bear on shore (4× 8 stringers on 4× 4 shores):*

$$\text{contact area} = 3.5(3.5) = 12.25\text{in.}^2$$

$$\text{Total load on shore} = 4550 \text{ lb.}$$

*The actual bearing stress perpendicular to the grain of the stringer is:*

$$\frac{4550}{12.25} = 371 \text{ psi.}$$

*Determine the allowable bearing stress perpendicular to the grain of the stringers.  $F_{c\perp} = 625 \text{ psi.}$*

Table (7)



*The adjustment factor applicable in this case is  $C_b$ , since the length of bearing  $lb$  is 3.5 in.:*

$$C_b = \frac{lb+0.375}{lb} = \frac{3.88}{3.5} = 1.107$$

*Therefore:*

$$F'_{c\perp} = 625 (1.107) = 692 \text{ psi.}$$

$$371 \text{ psi} < 692 \text{ psi.}$$

*(O.K)*

*b- Where joists bear on stringers (2× 8 joists on 4× 8 stringers):*

$$\text{contact area} = 1.5 (3.5) = 5.25 \text{ in.}^2$$

*Recalling from the design of the joists that the loading on each joist was 260 lb/ft, the load on the stringer from each joist is:*

$$260 (7.0) = 1820 \text{ lb.}$$

*Determine the allowable bearing stress perpendicular to the grain.*

*Because the calculated bearing stress is low relative to the 625 psi base design value for  $F'_{c\perp}$ , the increase in  $F'_{c\perp}$  due to  $C_b$  may be disregarded.*

*Thus:*

$$347 \text{ psi} < 625 \text{ psi.}$$

*(O.K)*

6. **Lateral bracing:** For floor systems, the minimum load to be used in designing lateral bracing is the greater of 100 lb per lineal foot of floor edge, or 2% of the total dead load of the floor. We will assume the slab to be 80 ft×100ft and placed in one operation. Guy wire bracing capable of carrying a load of 4000 lb each will be used on all four sides of the slab area attached at slab elevation and making a 45° angle with the ground. Guy wires can resist only tensile forces.

Calculating lateral load (H) as 2% of the dead load of the floor, again assuming the formwork to be 5 psf, yields:

$$H = 0.02 (75+5)(80)(100) = 12,800 \text{ lb.}$$

Distributing this load along the long side yield:

$$\frac{12,800}{100} = 128 \text{ lb/ft} > 100 \text{ lb/ft.}$$

and along the short side yields:

$$\frac{12,800}{80} = 160 \text{ lb/ft} > 100 \text{ lb/ft.}$$

These results are shown in figure (4.3-a). For determination of the guy wire spacing, the 160 lb/ft lateral load will be used. From figure (4.3-b), the tension in the guy wire (T) is calculated as:

$$\frac{T}{1.414} = \frac{160}{1}$$

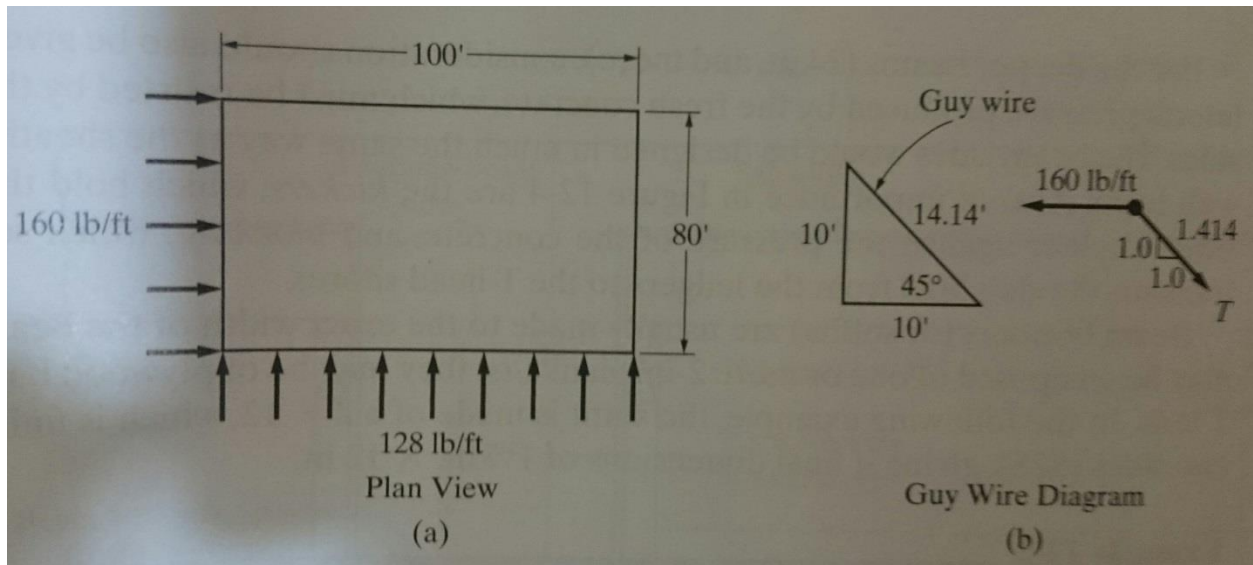
$$T = 226.2 \text{ lb per ft of slab bending braced.}$$

The maximum spacing for the guy wires is:

$$\frac{4000}{226.2} = 17.7 \text{ ft.}$$

Use guy wires spaced at 15 ft (max.) on center on all sides.

15 ft.



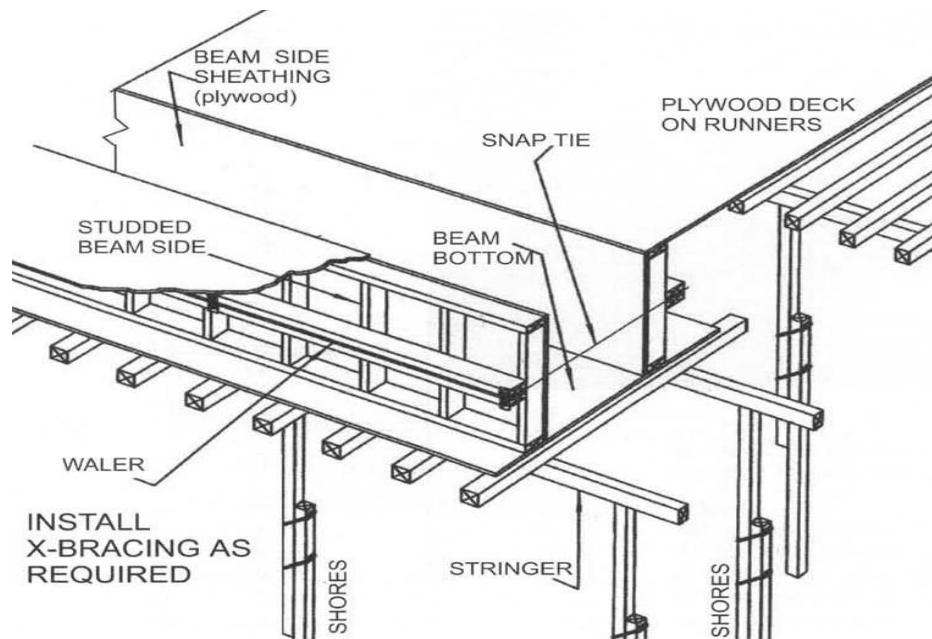
**Figure (4.3):** Floor slab lateral bracing design.

## 4.2 Design of formwork for beams:

Figure (4.4) shows one of several common types of beam forms. The usual design procedure involves consideration of the vertical loads, with the following components to be designed: the beam bottom, the ledger that supporting shores. Bearing stresses must also be checked.

For the deeper beams (24 in. and more), consideration should also be given to the lateral pressure produced by the fresh concrete, which must be resisted by the beam sides. The beam sides would be designed in much the same way as the sheathing in a wall form. Also of importance in figure (4.4) are the kickers, which hold the beam sides in place against the pressure of the concrete, and blocking, which serves to transmit the slab load from the ledgers to the T-head shores.

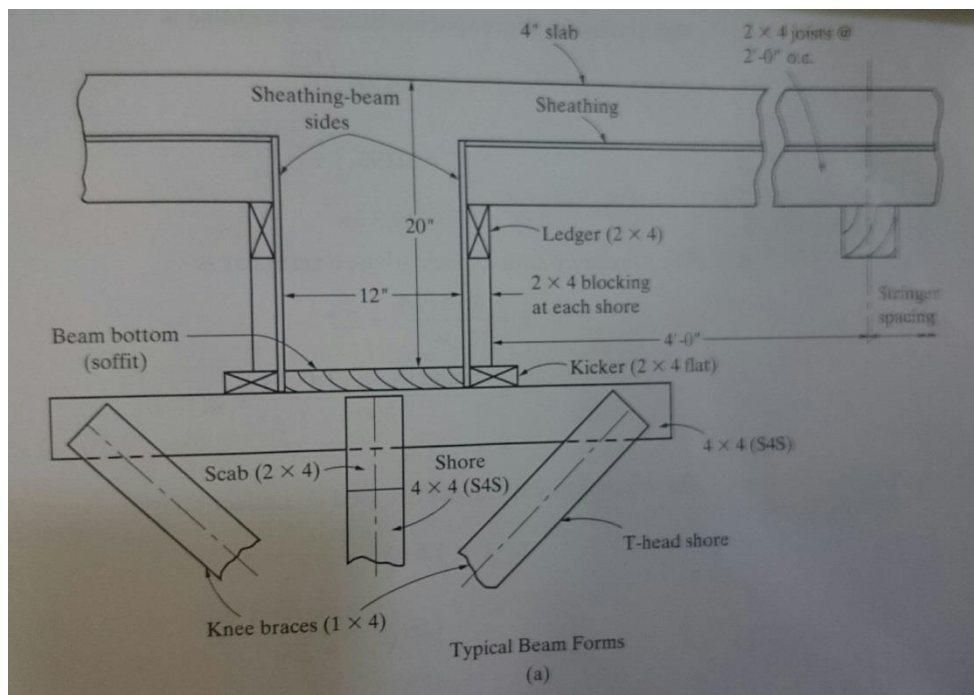
Beam bottoms (or soffits) are usually made to the exact width of the beam. They may be composed of one or more 2-in. planks, or they may be of plywood backed by 2 × 4s. In the following example, the soffit is made of a 2×12, which is finished on two sides (S2S), giving it final dimensions of 1½ in. ×12 in.



**Figure (4.4):** Typical beam form work.

### Example 4-2:

Design forms to support the 12 in. 20 in. beam shown in Figure (4.5). The beam is to support a 4-in.-thick reinforced concrete slab. Use Douglas fir-larch No. 2 grade. The maximum allowable deflection is to be  $1/360$  of the span for bending members. The unsupported shore height will be based on an assumed floor-to-floor height of 10 ft, from which the depth of the beam will be subtracted. All bending members are to be designed based on adequate lateral support.



**Figure (4.5):** Typical beam formwork.

**EXAMPLE (2) Beams Formwork Design** (ACI Committee 347)

REFERENCE	CALCULATION	OUTPUT
	<p><b><i>Solution:</i></b></p> <p>1. <b>Beam bottom design</b> (compute the maximum spacing between shores):</p> <p>Assume a 7-day maximum duration of load. Assume the plank to be continuous over three or more supports:</p> <p>a- <i>Obtain allowable stresses using base design values and appropriate adjustment factor:</i></p> <p>(1) <i>Bending: <math>F_b = 900</math> psi.</i></p> <p><i>Adjustment factors:</i></p> <ul style="list-style-type: none"> <li>• <i>Load duration factor <math>C_D = 1.25</math></i></li> <li>• <i>Size factor <math>C_F = 1.0</math></i></li> <li>• <i>Flat use factor <math>C_{fu} = 1.2</math></i></li> <li>• <i>Because <math>F_b C_F &lt; 1150</math> psi, wet service factor <math>C_M = 1.0</math></i></li> </ul> <p><i>Therefore:</i></p> $F'_b = F_b C_D C_F C_M C_{fu}$ $= 900(1.25)(1.0)(1.0)(1.2) = 1350 \text{ psi.}$ <p>(2) <i>Shear: <math>F_v = 180</math> psi.</i></p> <p><i>Adjustment factors:</i></p> <ul style="list-style-type: none"> <li>• <i>Load duration factor <math>C_D = 1.25</math></i></li> <li>• <i>Wet service factor <math>C_M = 0.97</math></i></li> </ul>	
Table (7)		
Table (7)		
Table (8)		
Table (8)		
Table (7)		
Table (7)		

Table (7)

*Therefore:*

$$F'_v = F_v C_D C_M$$

$$= 180(1.25)(0.97) = 218.3 \text{ psi.}$$

(3) *Modulus of elasticity:*

$$E = 1,600,000 \text{ psi.}$$

*Adjustment factor:*

- *Wet service factor*  $C_M = 0.9$

*Therefore:*

$$E' = E C_M = 1,600,000 (0.9)$$

$$= 1,440,000 \text{ psi.}$$

*Properties for the 2 × 12 (S2S) are:*

$$S = \frac{bh^2}{6} = \frac{12 (1.5)^2}{6} = 4.5 \text{ in.}^3$$

$$I = \frac{bh^3}{12} = \frac{12 (1.5)^3}{12} = 3.38 \text{ in.}^4$$

b- The loading on the beam soffit is calculated as:

$$\text{DL (reinforced concrete beam)} = \frac{12 (20)}{144} (150)$$

$$= 250 \text{ lb/ft.}$$

$$\text{LL (use 50 psf)} = 50 \text{ lb/ft.}$$

$$\text{Total load} = (250+50) = 300 \text{ lb/ft.}$$

c- *The maximum shore spacing based on bending is:*

$$l = 10.95 \sqrt{\frac{F'_b S}{w}}$$

$$= 10.95 \sqrt{\frac{1350 (4.5)}{300}} = 49.3 \text{ in.}$$

Table (4)

Table (4)

d- *The maximum shore spacing based on shear is:*

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

$$= \frac{13.3(218.3)(12)(1.5)}{300} + 2(1.5) = 177.2 \text{ in.}$$

Table (4)

e- *The maximum shore spacing based on deflection is:*

$$l = 1.69 \sqrt[3]{\frac{EI}{w}}$$

$$= 1.69 \sqrt[3]{\frac{1,440,000 (3.38)}{300}} = 42.8 \text{ in.}$$

# *Deflection governs. Try shore spacing at 42 in. o.c.*

42 in.

2. **Ledger design:** Use ¾-in. plyform sheathing (vertically) for the beam sides and 2 × 4 kickers as shown. The ledger is supported at each shore by a blocking piece. Because the shores are to be 42 in. o.c., the ledger will be continuous over three or more spans.

Use 2 × 4s (S4S) for the ledger as shown. Compute the required spacing for the ledger supports and compare with the 42-in. spacing previously determined. Neglect the connection of the ledger to the vertical sheathing:

Table (7)

a- *Obtain allowable stresses using base design values and appropriate adjustment factor:*



<p>Table (7)</p> <p>Table (8)</p>	<p>(1) <i>Bending: <math>F_b = 900 \text{ psi}</math>.</i></p> <p><i>Adjustment factors:</i></p> <ul style="list-style-type: none"> <li>• <i>Load duration factor <math>C_D = 1.25</math></i></li> <li>• <i>Size factor <math>C_F = 1.5</math></i></li> </ul> <p><i>Therefore:</i></p> $F'_b = F_b C_D C_F$ $= 900(1.25)(1.5) = 1688 \text{ psi.}$	
<p>Table (7)</p>	<p>(2) <i>Shear: <math>F_v = 180 \text{ psi}</math>.</i></p> <p><i>Adjustment factors:</i></p> <ul style="list-style-type: none"> <li>• <i>Load duration factor <math>C_D = 1.25</math></i></li> </ul> <p><i>Therefore:</i></p> $F'_v = F_v C_D$ $= 180(1.25) = 225 \text{ psi.}$	
<p>Table (6)</p>	<p>(3) <i>Modulus of elasticity:</i></p> <p><math>E = 1,600,000 \text{ psi.}</math> <i>No adjustment factors apply. Thus:</i></p> $E' = E = 1,600,000 \text{ psi.}$ <p><i>Properties for the <math>2 \times 4</math> (S4S) are:</i></p> $S = 3.06 \text{ in.}^3$ $I = 5.36 \text{ in.}^4$ $A = 5.25 \text{ in.}^2$	

b- Loading: The 2 × 4 joists are supported by the ledger, as shown in figure (4.3). Although the ledger is loaded with point loads, a uniform load will be assumed for simplicity. The loading on the slab sheathing is:

$$DL \text{ (slab)} = \left(\frac{4}{12}\right)(150) = 50 \text{ psf.}$$

$$LL \text{ (min.)} = 50 \text{ psf.}$$

Assume sheathing weight = 5 psf.

$$\text{Total load} = (50+50+5) = 105 \text{ psf.}$$

Because the span of the joists from the ledger to the adjacent stringer is 4 ft-0 in., the load to the ledger is calculated as:

$$w = \frac{1}{2} (4) (105) = 210 \text{ lb/ft.}$$

c- *The maximum blocking spacing based on bending is:*

$$\begin{aligned} l &= 10.95 \sqrt{\frac{F'_{bs}}{w}} \\ &= 10.95 \sqrt{\frac{1688 (3.06)}{210}} = 54.3 \text{ in.} \end{aligned}$$

d- *The maximum blocking spacing based on shear is:*

$$\begin{aligned} l &= \frac{13.3 F_v A}{w} + 2d \\ &= \frac{13.3(225)(5.25)}{210} + 2(3.5) = 81.8 \text{ in.} \end{aligned}$$

e- *The maximum blocking spacing based on deflection is:*

$$\begin{aligned} l &= 1.69 \sqrt[3]{\frac{EI}{w}} \\ &= 1.69 \sqrt[3]{\frac{1,600,000 (5.36)}{210}} = 58.2 \text{ in.} \end{aligned}$$

Table (4)

Table (4)

Table (4)

	<p><i># Because all the three foregoing spacings exceed 42 in., the 2 × 4 ledgers supported by blocking at 42 in. on center are satisfactory.</i></p>	42 in.
	<p><b>3. Design of the shores:</b> The shores are spaced 42 in. (or 3.5 ft) on center, and each must support a loading of:</p> <p>From beam bottom = <math>300 (3.5) = 1050 \text{ lb.}</math></p> <p>From slab forms (two sides) = <math>210 (3.5) (2) = 1470 \text{ lb.}</math></p> <p>Total load per shore = <math>(1050 + 1470) = 2520 \text{ lb.}</math></p> <p>Assume 4 × 4 (S4S) wood shores. The unsupported shore height will be based on an assumed floor-to-floor height of 10 ft-0 in., from which the depth of the beam will be subtracted. The unsupported height is:</p> <p><math>\ell = 10 (12) - 20 = 100 \text{ in.} = 8.33 \text{ ft.}</math></p> <p>We will assume the shores are pin-connected.</p> <p><math>K_e = 1.0</math></p> <p>Therefore, the effective unbraced length is:</p> <p><math>\ell_e = K_e \ell = 1.0 (8.33) = 8.33 \text{ ft.}</math></p> <p><i>a. The base design value for compression parallel to the grain is:</i></p>	
Table (7)	<p><math>F_c = 1350 \text{ psi}</math></p>	
Table (8)	<p><i>b. Adjustment factors:</i></p> <p>(1) Size factor: <math>C_F = 1.15</math></p>	

(2) For the column stability factor  $C_p$ , initially the following items must be established:

i- For modulus of elasticity, there is no adjustment factor:

$$E' = E = 1,600,000 \text{ psi.}$$

ii- Find  $F_c^* = F_c C_F$

$$F_c^* = 1350(1.15) = 1553 \text{ psi.}$$

iii-  $\frac{le}{d} = \frac{8.33 (12)}{3.5} = 28.6 < 50$

(O.K)

iv- Solve for  $\alpha$ :

$$\alpha = \frac{0.3 E'}{\left(\frac{le}{d}\right)^2 F_c^*}$$

$$\alpha = \frac{0.3 (1,600,000)}{28.6^2 (1553)} = 0.378$$

Solve for  $C_p$ :

$$C_p = \frac{1+\alpha}{1.6} - \sqrt{\left(\frac{1+\alpha}{1.6}\right)^2 - \frac{\alpha}{0.8}}$$

$$C_p = \frac{1+0.378}{1.6} - \sqrt{\left(\frac{1+0.378}{1.6}\right)^2 - \frac{0.378}{0.8}}$$

$$= 0.34$$

c. Compute the allowable stress  $F'_c$ :

$$F'_c = F_c C_F C_p \\ = 1350 (1.15) (0.342) = 531 \text{ psi.}$$

Therefore the allowable load is:

$$P = F'_c A \\ = 531 (3.5)^2 = 6500 \text{ lb} \\ 6500 \text{ lb} > 2520 \text{ lb.} \\ (O.K)$$

Use  $4 \times 4$  (S4S) shores spaced 42 in. on center.

42 in.

#### 4. Bearing stresses:

- (a) Assume  $4 \times 4$  (S4S) T-heads on the  $4 \times 4$  (S4S) shores. Actual bearing stress perpendicular to the grain of the T-head is calculated as :

$$\frac{\text{shore load}}{\text{contact area}} = \frac{2520}{3.5^2} = 206 \text{ psi.}$$

The base design value for  $F_{c\perp}$  is 625 psi. We will neglect the adjustment factor for bearing area (due to  $C_b > 1.0$  because  $3\frac{1}{2} \text{ in.} < 6 \text{ in.}$ ).

Therefore, use  $F'_{c\perp} = 625 \text{ psi.}$

$$206 \text{ psi} < 625 \text{ psi.} \\ (O.K)$$

Table (7)

(b) Check bearing stress between the 2 × 4 ledger and the 2 × 4 blocking. The load from the ledger to the blocking is:

$$210 (3.5) = 735 \text{ lb.}$$

*The actual bearing stress perpendicular to the grain of the ledger is:*

$$\frac{\text{load}}{\text{contact area}} = \frac{735}{1.5 (3.5)} = 140 \text{ psi.}$$

The allowable bearing stress, from part a, is 625 psi.  
Thus:

$$140 \text{ psi} < 625 \text{ psi.}$$

(O.K)

(c) Check bearing stress between the 2 × 4 joists and the 2 × 4 ledger. The joist loading is:

$$105 \text{ psf} (2 \text{ ft}) = 210 \text{ lb/ft.}$$

*And the span of the joist is 4 ft. Therefore, the load from the joist to the ledger is:*

$$210 \text{ lb/ft} (2 \text{ ft}) = 420 \text{ lb.}$$

*The actual bearing stress perpendicular to the grain of the ledger (and the joists) is:*

$$\frac{\text{load}}{\text{contact area}} = \frac{420}{1.5 (1.5)} = 186.7 \text{ psi.}$$

The allowable bearing stress is determined as in part a (the bearing area adjustment factor  $C_b$  is not applicable because this is end bearing):

$$186.7 \text{ psi} < 625 \text{ psi.}$$

(O.K)

(d) Check the bearing of the beam soffit on the  $4 \times 4$  (S4S) T-heads. The load from the beam soffit is:

$$(300 \text{ lb/ft}) (3.5 \text{ ft}) = 1050 \text{ lb.}$$

*The actual bearing stress perpendicular to the grain of the T-head is:*

$$\frac{\text{load}}{\text{contact area}} = \frac{1050}{12 (3.5)} = 25 \text{ psi.}$$

For allowable bearing stress, we will neglect the bearing area adjustment factor. The contact surface may be subjected to a wet condition, so this adjustment factor ( $C_M = 0.67$ ) will be used:

$$F'_{c\perp} = F_{c\perp} C_M$$

$$= 625 (0.67) = 419 \text{ psi}$$

$$25 \text{ psi} < 419 \text{ psi.}$$

(O.K)

Table (7)

### 4.3 Wall form design:

The design procedure for wall forms is similar to that used for slab forms, substituting studs for joists, wall for stringers, and ties for shores. See Figure (4.6) for location of these members.

The maximum lateral pressure against the sheathing must be determined first. We will assume conditions such that  $(C_c)$  and  $(C_w)$  are both (1.0). With the sheathing thickness specified, the maximum allowable span for the sheathing is computed based on bending, shear, and deflection. This will be the maximum stud spacing. (An alternative approach would be to establish the stud spacing and then calculate the required thickness of the sheathing).

Next, the maximum allowable stud span is calculated based on stud size and loading, considering bending, shear, and deflection. This will be maximum wale spacing. (An alternative approach would be to establish the wale spacing and then calculate the required size of the studs.)

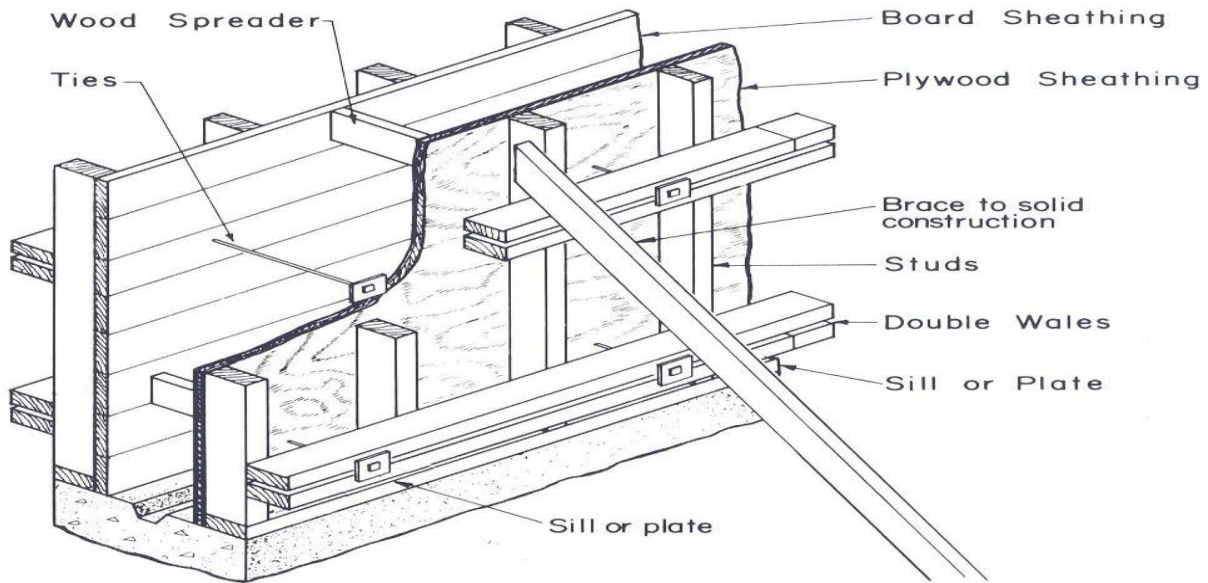
The next step is to determine the maximum allowable spacing of wale supports (tie spacing). This is calculated based on wale size and loading. (An alternative approach would be to preselect the tie spacing and then calculate the wale size.) Double wales are commonly used to avoid the necessity of drilling wales for tie insertion.

The load supported by each tie must be computed and compared with the tie capacity. The load on each tie is calculated as the design load (psf) multiplied by the tie spacing (ft) and wale spacing (ft). If the load exceeds the tie strength, a stronger tie must be used or the tie spacing must be reduced.

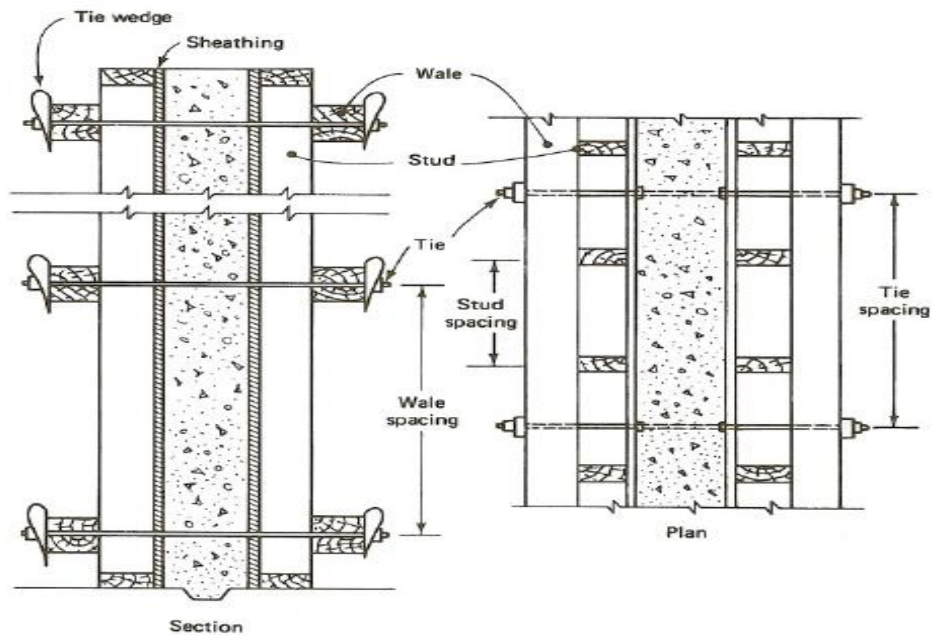
Bearing stresses must also be checked where the studs bear on the wales and where the tie ends bear on the wales. Maximum bearing stress must not exceed the allowable compression stress perpendicular to the grain or crushing will result.

Finally, lateral bracing must be designed to resist any expected lateral loads, such as wind loads.





**Figure (4.6-a):** typical wall form.



**Figure (4.6-b):** typical wall form.

### ***Example 4-3:***

Design formwork for an 8-ft-high wall. The concrete is to be placed at a rate of 4 ft/h and will be internally vibrated. Concrete temperature is expected to be 90° F. The maximum allowable deflection on bending members is to be 1/360 of span. Use 3/4-in. class I plyform for the sheathing and No. 2 Douglas fir-larch for the rest of the lumber. Assume all bending members to be supported on three or more spans.

The following conditions for design have been established:

- 1- Studs and wales are to be designed based on adequate lateral support.
- 2- Stud, wales, and bracing are to be used under dry service condition.

**EXAMPLE (3) Walls Formwork Design** (ACI Committee 347)

REFERENCE	CALCULATION	OUTPUT
Table (5)	<p><b><i>Solution:</i></b></p> <p>1. <b>Sheathing design</b> (find the stud spacing): Consider a 12-in.-wide strip of sheathing perpendicular to the supporting studs and acting as a beam continuous over three or more spans. The sheathing span horizontally.</p> <p>Place the face grain perpendicular to studs:</p> <p>a- The design values <math>\frac{3}{4}</math>-in. class I plyform are:</p> $E = 1,650,000 \text{ psi.}$ $F_b = 1930 \text{ psi.}$ $F_v = 72 \text{ psi.}$	
Table (6)	<p>Plyform properties for face grain parallel to the span are:</p> $I = 0.199 \text{ in.}^4$ $S = 0.455 \text{ in.}^3$ $Ib/Q = 7.187 \text{ in.}^2/\text{ft}$ <p>b- Loading: the sheathing will be designed for concrete pressure, which is the lesser of 150(h) (where h will be taken as 8 ft), 2000 psf, or as determined by formula (nothing than <math>R &lt; 7 \text{ ft/h}</math> and <math>h &lt; 14 \text{ ft}</math>):</p> $P = 150 + \frac{9000 R}{T}$ $= 150 + \frac{9000 (4)}{90} = 550 \text{ psf} < 600 \text{ psf.}$ $150 (h) = 150 (8) = 1200 \text{ psf.}$	

Use the ACI-recommended minimum of 600 psf for the sheathing design load, although the pressure could be decreased near the top of the form where 150(h) controls. The length in which the decreased pressure could be used may be calculated as:

$$\frac{600}{150} = 4.0 \text{ ft.} \quad (\text{from the top of the form}).$$

It is conservative to design the full height for 600 psf, however.

c- *The maximum stud spacing based on bending is:*

$$\begin{aligned} l &= 10.95 \sqrt{\frac{F_{bs}}{w}} \\ &= 10.95 \sqrt{\frac{1930(0.455)}{600}} = 13.25 \text{ in.} \end{aligned}$$

d- *The maximum stud spacing based on shear is:*

$$\begin{aligned} l &= \frac{20 F_v (Ib/Q)}{w} + 2d \\ &= \frac{20(72)(7.187)}{600} + 2(0.75) = 18.75 \text{ in.} \end{aligned}$$

e- *The maximum stud spacing based on deflection is:*

$$\begin{aligned} l &= 1.94 \sqrt[3]{\frac{EI}{w}} \\ &= 1.94 \sqrt[3]{\frac{1,650,000 (0.199)}{600}} = 13.82 \text{ in.} \end{aligned}$$

# Bending is critical. Use a stud spacing of 12 in. o.c.

12 in.

2. **Stud design** (compute the wale spacing):

*Assume  $2 \times 4$  (S4S) studs and a 7-day maximum duration of load:*

Table (7)

*a- Obtain allowable stresses using base design values and appropriate adjustment factor:*

*(1) Bending:  $F_b = 900$  psi.*

*Adjustment factors:*

Table (7)

- *Load duration factor  $C_D = 1.25$*

Table (8)

- *Size factor  $C_F = 1.5$*

*Therefore:*

$$\begin{aligned} F'_b &= F_b C_D C_F \\ &= 900 (1.25) (1.5) = 1688 \text{ psi.} \end{aligned}$$

*(2) Shear:  $F_v = 180$  psi.*

*Adjustment factors:*

Table (7)

- *Load duration factor  $C_D = 1.25$*

*Therefore:*

$$\begin{aligned} F'_v &= F_v C_D \\ &= 180 (1.25) = 225 \text{ psi.} \end{aligned}$$

*(3) Modulus of elasticity:*

$$E = 1,600,000 \text{ psi}$$

*No adjustment factors apply. Thus:*

$$E' = E = 1,600,000 \text{ psi.}$$

Table (6)

*Properties for the 2 × 4 lumber are:*

$$S = 3.06 \text{ in.}^3$$

$$I = 5.36 \text{ in.}^4$$

$$A = 5.25 \text{ in.}^2$$

*b- Loading: Since the stud spacing is 12in. o.c., the load (w) will be 600 lb/ft. (see step 1, part [b]).*

*c- The maximum wale spacing based on bending moment is:*

Table (4)

$$l = 10.95 \sqrt{\frac{F'_b S}{w}}$$

$$= 10.95 \sqrt{\frac{1688 (3.06)}{600}} = 32.1 \text{ in.}$$

Table (4)

*d- The maximum wale spacing based on shear is:*

$$l = \frac{13.3 F'_v A}{w} + 2d$$

$$= \frac{13.3(225)(5.25)}{600} + 2(3.5) = 33.2 \text{ in.}$$

Table (4)

*e- The maximum stringer spacing based on deflection is:*

$$l = 1.69 \sqrt[3]{\frac{EI}{w}}$$

$$= 1.69 \sqrt[3]{\frac{1,600,000 (5.36)}{600}} = 41.0 \text{ in.}$$

*Therefore, bending governs. Use a wale spacing of 24 in. o.c. (maximum).*

24 in.

3. **Wale design** (compute the tie spacing):

Double  $2 \times 4$  (S4S) wales will be assumed, and the allowable stresses will be adjusted for a 7-day maximum duration of load:

a- Design values: Allowable stresses and (E) will be the same as for the studs. Properties for the wales will be twice those for the studs because the wales are doubled.

Thus:

$$S = 6.12 \text{ in.}^3$$

$$I = 10.72 \text{ in.}^4$$

$$A = 10.5 \text{ in.}^2$$

b- *Loading: Each wale will support a strip of wall form that has a height equal to the spacing of the wales:*

$$w = \frac{24}{12} (600) = 1200 \text{ lb/ft.}$$

c- *The maximum tie spacing based on bending moment is:*

$$\begin{aligned} l &= 10.95 \sqrt{\frac{F'_{bs} S}{w}} \\ &= 10.95 \sqrt{\frac{1688 (6.12)}{1200}} = 32.1 \text{ in.} \end{aligned}$$

d- *The maximum tie spacing based on shear is:*

$$\begin{aligned} l &= \frac{13.3 F'_{rv} A}{w} + 2d \\ &= \frac{13.3(225)(10.5)}{1200} + 2(3.5) = 33.2 \text{ in.} \end{aligned}$$

Table (6)

Table (4)

Table (4)

Table (4)

*e- The maximum tie spacing based on deflection is:*

$$l = 1.69 \sqrt[3]{\frac{EI}{w}}$$

$$= 1.69 \sqrt[3]{\frac{1,600,000 (10.72)}{1200}} = 41.0 \text{ in.}$$

*# Bending governs. A modular spacing of 24 in. would be desirable.*

*Use tie spacing of 24 in. o.c.*

24 in.

4. **Check the load on the ties (P<sub>tie</sub>) with the capacity of the ties:** Assume the tie capacity to be 3000 lb (tie of various capacities are widely available). Also assume that the ties have 1½-in. wedges bearing on the wales. Then:

$$P_{\text{tie}} = (\text{wale spacing}) \times (\text{tie spacing}) \times (\text{pressure})$$

$$= \frac{24}{12} \left( \frac{24}{12} \right) (600) = 2400 \text{ lb.}$$

$$2400 \text{ lb} < 3000 \text{ lb.}$$

(O.K)

*# Therefore, the capacity of the tie is satisfactory.*



### 5. Check bearing stresses:

- a- Where tie wedges bear on wales (wedges are 1½-in. wide):

$$P_{\text{tie}} = 2400 \text{ lb.}$$

$$\text{Bearing contact area} = 2(1.5)(1.5) = 4.5 \text{ in.}^2$$

$$\text{Bearing stress (actual)} = \frac{2400}{4.5} = 533 \text{ psi.}$$

The allowable compressive stress perpendicular to the grain is:

$F'_{c\perp} = 625 \text{ psi}$  (neglect bearing area adjustment factor). Thus:

$$533 \text{ psi} < 625 \text{ psi}$$

(O.K.)

- b- Where studs bear on wales (double wales):

$$\text{Bearing contact area} = 2(1.5)(1.5) = 4.5 \text{ in.}^2$$

The load on the wale from the stud is:

$$P = (\text{load/ft on stud}) \times (\text{wale spacing})$$

$$= 600 \left( \frac{24}{12} \right) = 1200 \text{ lb.}$$

$$\text{Actual bearing stress} = \frac{1200}{4.5} = 267 \text{ psi.}$$

As in part (a),  $F'_{c\perp} = 625 \text{ psi}$ . Thus:

$$267 \text{ psi} < 625 \text{ psi}$$

(O.K.)

Table (7)

Table (7)

6. **Lateral bracing:** Should be designed for wall forms based on the greater of wind load (using 15 psf as a minimum) or 100 lb/ft applied at the top of the wall. Calculate wind load on wall forms using the minimum 15 psf:

$$\text{wind load} = (15 \text{ psf})(1 \text{ ft})(8 \text{ ft}) = 120 \text{ lb per ft of wall.}$$

This load would be considered to act at mid-height of the wall, 4 ft above the base, and would create an overturning moment about the base of :

$$\begin{aligned} M_{OT} &= 120 \text{ lb/ft} (4 \text{ ft}) \\ &= 480 \text{ ft-lb} \quad (\text{per ft of wall}). \end{aligned}$$

The equivalent force, acting at the top of the wall, that would create the same overturning moment is:

$$\begin{aligned} \text{force} &= \frac{480 \text{ ft-lb}}{8 \text{ ft}} = 60 \text{ lb} \quad (\text{per ft of wall}) \\ 60 \text{ lb/ft} &< 100 \text{ lb/ft.} \end{aligned}$$

*Therefore use 100 lb/ft. This load, assumed to act at the top of the wall, can act in either direction. If guy wires are used, they must be placed on both sides of the wall. If wooden strut bracing is used, it can resist tension or compression and therefore single-side bracing may be used.*

*In this problem use single-side strut bracing, as shown in figure (4.7), and design for compression. The horizontal load  $H$  on the strut at point A is calculated by considering moment taken at the base of the wall:*

$$H(6.5) = 100 (8)$$

$$H = 123 \text{ lb.}$$

*The force  $F$  in the strut, using the slope triangle shown, is determined from:*

$$\frac{F}{8.2} = \frac{123}{5}$$

*Therefore:*

$$F = 202 \text{ lb} \quad (\text{per foot of wall}).$$

*Use double  $2 \times 4$  (S4S) lumber for the strut and compute the capacity as a compression member. (This will adequate for tension also). For No. 2 grade Douglas fir-larch:*

- a. The base design value for compression parallel to the grain is:*

$$F_c = 1350 \text{ psi}$$

- b. Adjustment factors:*

*(1) Size factor:  $C_F = 1.15$*

- (2) For the column stability factor  $C_p$ , initially the following items must be established:*

- i- For modulus of elasticity, there is no adjustment factor:*

$$E' = E = 1,600,000 \text{ psi.}$$

- ii- Find  $F_c^* = F_c C_F$*

$$F_c^* = 1350(1.15) = 1553 \text{ psi.}$$

- iii- Assume that the ends are pin connected.*

*Therefore,  $K_e = 1.0$  and  $l_e = 8.2 \text{ ft.}$  Then:*

Table (7)

Table (9)

Table(12)

$$\frac{le}{d} = \frac{8.2 (12)}{3.0} = 32.8 < 50 (O.K)$$

iv- Solve for  $\alpha$ :

$$\alpha = \frac{0.3 E'}{\left(\frac{le}{d}\right)^2 F_{c*}}$$

$$\alpha = \frac{0.3 (1,600,000)}{32.8^2 (1553)} = 0.287$$

Solve for  $C_p$ :

$$C_p = \frac{1+\alpha}{1.6} - \sqrt{\left(\frac{1+\alpha}{1.6}\right)^2 - \frac{\alpha}{0.8}}$$

$$C_p = \frac{1+0.287}{1.6} - \sqrt{\left(\frac{1+0.287}{1.6}\right)^2 - \frac{0.287}{0.8}}$$

$$= 0.267$$

c. Compute the allowable stress  $F'_c$ :

$$F'_c = F_c C_F C_p$$

$$= 1350 (1.15) (0.267) = 415 \text{ psi.}$$

Therefore the allowable load is:

$$P = F'_c A$$

$$= 415 (2)(5.25) = 4360 \text{ lb.}$$

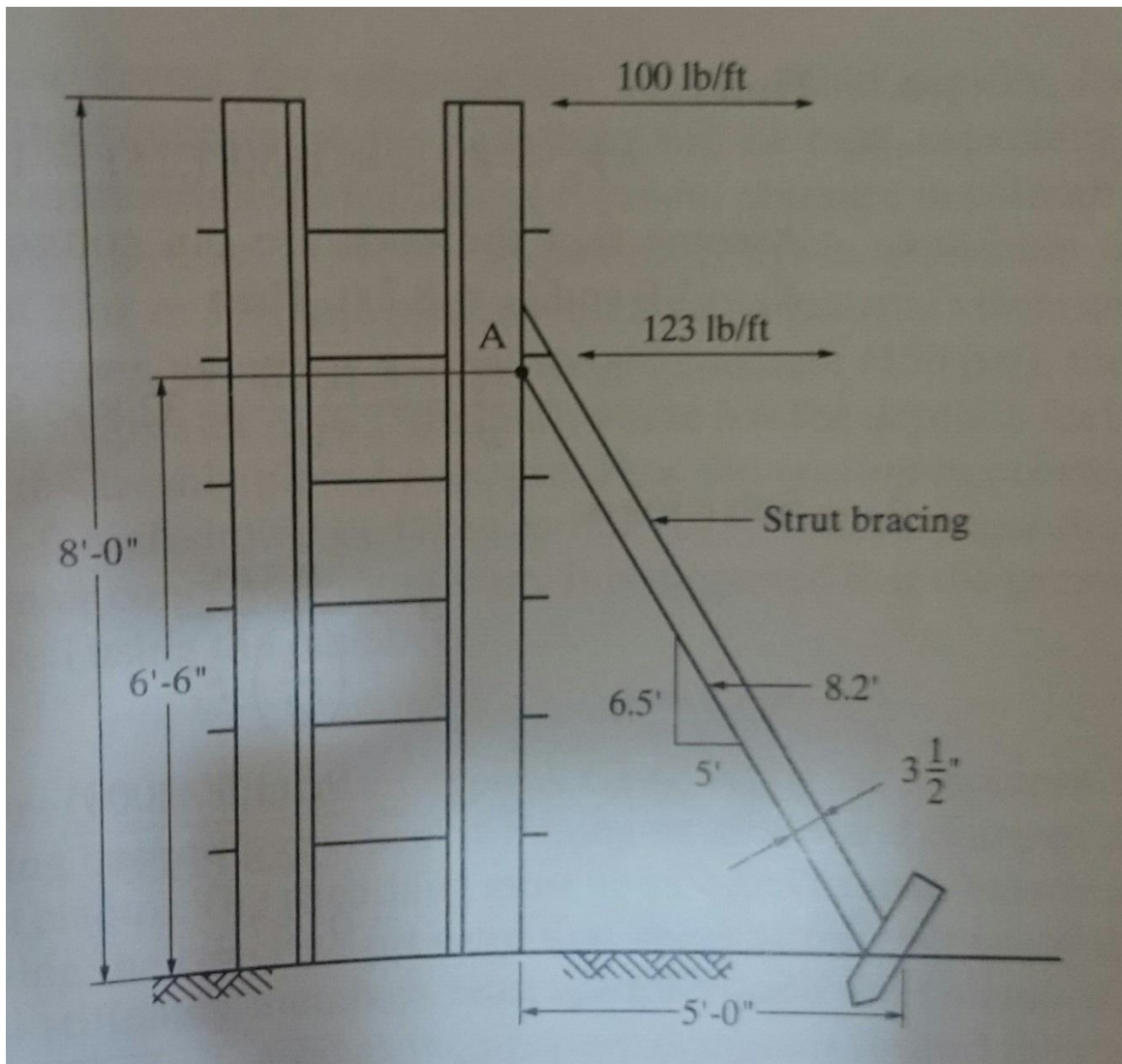
The maximum allowable strut spacing is calculated from:

$$\frac{4360 \text{ lb}}{202 \text{ lb/ft}} = 21.6 \text{ ft.}$$

Use struts at 21 ft-0 in. on center.

Single 2 × 4 struts could have been used if an intermediate brace were used to reduce the unbraced length.

21ft-0  
in.



**Figure (4.7):** Lateral bracing for wall form.

#### ***4.4 Forms for columns:***

Concrete column are usually one of five shapes: square, rectangular, L-shaped, octagonal, or round. Forms for the first four shapes are generally made of sheathing, consisting of vertical planks or plywood, with wood yokes and steel bolts, patented steel clamps, or steel bands used to resist the concrete pressure acting on the sheathing. Forms for round columns may be wood, steel plate, or patented fiber tubes.

Because forms for columns are usually filled rapidly, frequently in less than 60 min, the pressure on the sheathing will be high, especially for tall columns.

The ACI recommendations for lateral concrete pressure in column forms were discussed in Section (3.2.2). ACI 347-04 (Equation [3.2]) is applicable for concrete having a slump of 7 in. or less and placed with normal internal vibration to a depth of 4 ft or less. However, assuming normal weight concrete (150 pcf), the pressure should not be taken as greater than  $150h$  (psf), where  $h$  is the depth in feet below the upper surface of the freshly placed concrete. Thus the maximum pressure at the bottom of a form 10-ft high should be taken as  $150(10) = 1500$  psf regardless of the rate of filling the form or concrete temperature. It is suggested that the pressure be conservatively calculated using the equation:

$$P = 150h$$

Figure (4.7-a) illustrates typing construction of a column form using plywood sheathing backed by vertical stiffening members and clamped with adjustable metal column clamps. The sheathing must be selected to span between the stiffening members using the concrete pressure that exists at the bottom of the column form. The vertical stiffening members must span between the column clamps, the spacing of which can be increased as the pressure decreases toward the top of the form.

Figure (4.7-b) illustrates a method suitable for forming smaller columns where no vertical stiffening members are required and the plywood. Sheathing is backed directly by battens that are part of a wood and bolt column yoke.

Column clamps can be used in this situation as well. If the thickness of the sheathing is selected, the design consists of determining the maximum safe spacing

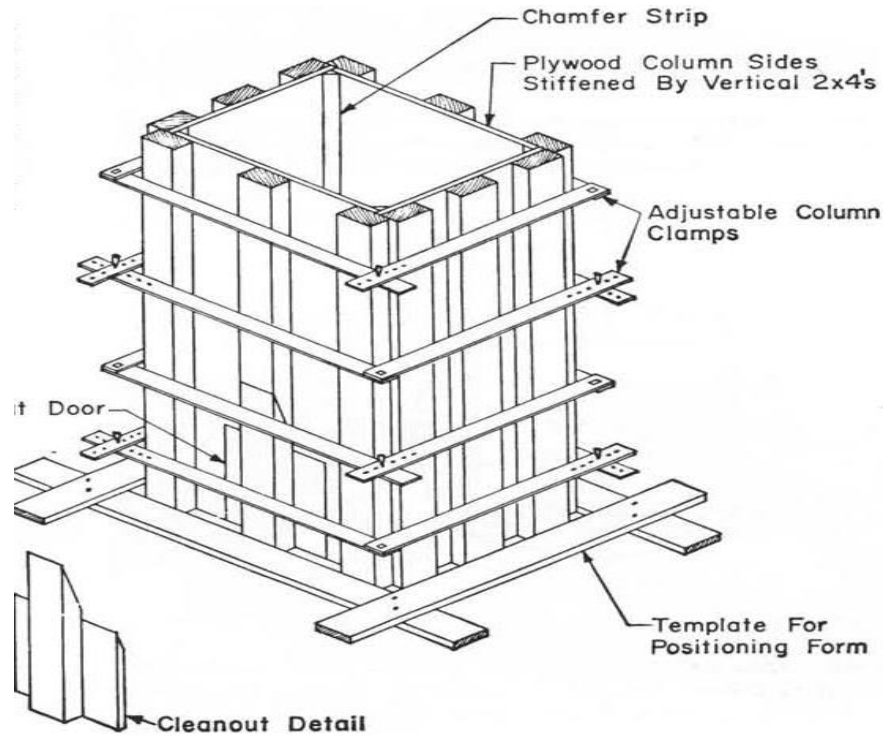
of the column clamps considering the pressure from the concrete as well as the permissible deflection, allowable bending stress, and allowable shearing stress.

The sheathing span length  $\ell$  may be calculated for moment, shear, and deflection, with the shortest of these span lengths being the controlling value.

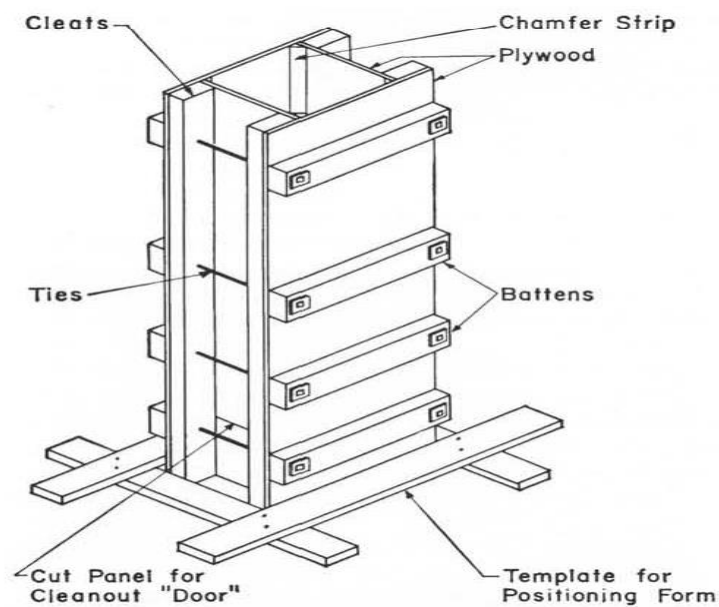
Because the pressure against the forms varies with height, however, the determination of the optimum clamp spacing becomes laborious. As a result, tables have been developed that aid in quick determination of support (clamp) spacing. (Table 11) is an example of one such table specifically set up for plywood sheathing. The tabular values are based on the assumption that the lateral pressure is uniform between clamps and of an intensity equal to that at the lower clamp.

Clamps must also be investigated to determine if they can resist the applied loads. The manufacturer usually has recommended capacities for steel column clamps. Wood-yoke-type clamps with tie rods must be designed.

Generally, the type of forming system is a function of column size and height. As column size increases, either the thickness of the sheathing must be increased or vertical stiffeners must be added to prevent sheathing deflection. If vertical supports or stiffeners are used (see Figure 4.7) in combination with a plywood sheathing, the sheathing should span between the vertical supports and the plywood face grain should be horizontal (in the direction of the span) for maximum strength. The clamp spacing is then a function of the vertical support member strength. If plywood sheathing spans between clamps (without vertical supports), the face grain should be vertical (in the direction of the span) for maximum strength.



**Figure (4.8-a):** Typical construction for larger column forms.



**Figure (4.8-b):** Typical construction for smaller column forms.



#### ***Example 4-4:***

Determine a clamp spacing pattern for column form sheathing made up of  $\frac{3}{4}$ -in.-thick plywood. The column height is to be 12 ft-0 in. Assume the sheathing continuous over four or more supports and its face grain parallel to the span (vertical). Use class I plyform design values of:

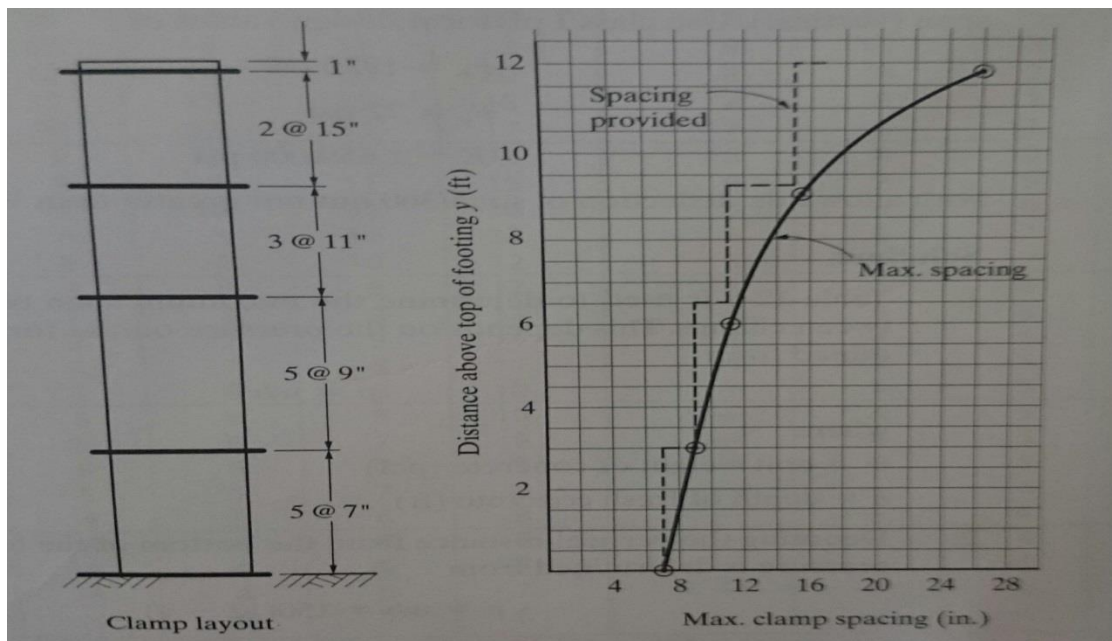
$$F_b = 1930 \text{ psi.}$$

$$F_v = 72 \text{ psi.}$$

$$E = 1,650,000 \text{ psi.}$$

with allowable deflection of (span/360) but not greater than 1/16 in.

Figure (4.9): Sketch for Example 4-4:



**EXAMPLE (4) columns Formwork Design (ACI Committee 347)**

REFERENCE	CALCULATION	OUTPUT
Table (10)	<p><b><i>Solution:</i></b></p> <p>1. To determine the maximum span of the plywood between clamps use Table (10). This depends on the pressure on the form, which is determined from:</p> $P = wh$	
Table (11)	<p>2. Denoting the vertical distance from the bottom of the form as y (ft), the pressure is determined from:</p> $P = wh = 150 (12 - y)$ <p>For some arbitrary values of y, the calculated pressures and the maximum spans (clamp spacings) from Table (10) are shown in Table (11).</p> <p>3. A plot of maximum clamp spacing as a function of distance above top-of-footing is shown in Figure (4.8). The final clamp layout, also shown in Figure (4.8), is determined by trial and error. This procedure is similar to stirrup design. One should attempt to minimize the number of clamps without having too many different-size spacings.</p>	