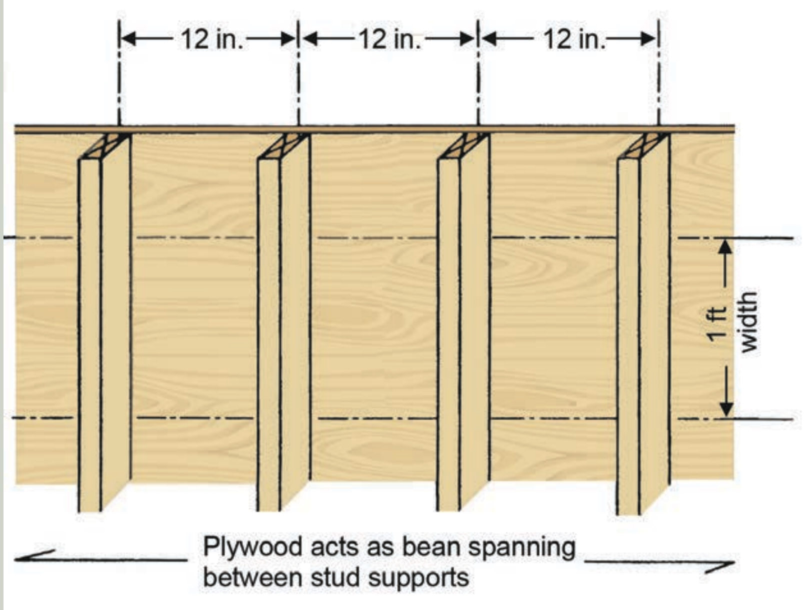

Fig. B: Wall form sheathing supported by vertical studs

This is the maximum allowable span length, center-to-center of supports, based on bending strength of the plywood. However, deflection must also be checked.

CHECK DEFLECTION, considering a 12 in. width of the plywood sheathing. Maximum allowable deflection of the sheathing is 1/360 of span length or 1/16 in., whichever is less. For Δ of ℓ/360 from Eq. (7.5)

$$\ell = 0.738\sqrt[3]{\frac{E'I}{w}}$$

For Δ of 1/16 in., substitute in Eq. (7.4c) to obtain

$$\ell = 1.735\sqrt[4]{\frac{E'I\,(\text{in.})}{w}} \qquad \text{(A)}$$

E′ for the plywood is 1,500,000 psi by Table 4.14, and *I* for plies parallel to the span is 0.199 in.^{4} (Table 4.13). *w* is 50 lb/in. as in the bending check. Substituting in Eq. (7.5)

$$\ell = 0.738\sqrt[3]{\frac{1{,}500{,}000\ \text{psi}\times 0.199\ \text{in.}^4}{50\ \text{lb/in.}}} = 0.738\sqrt[3]{5970\ \text{in.}^3} = 13.4\ \text{in.}$$

for Δ = ℓ/360

Substituting in Eq. (A)

$$\ell = 1.735\sqrt[4]{\frac{1{,}500{,}000\ \text{psi}\times 0.199\ \text{in.}^4(\text{in.})}{50\ \text{lb/in.}}} = 1.735\sqrt[4]{5970\ \text{in.}^4} = 15.2\ \text{in.}$$

for Δ = 1/16 in.

Refer to APA V345V and APA D510C for methods of calculating bending and shear deflections separately.

Both of the spans calculated to meet deflection requirements are longer than the span length permitted for bending; thus, bending still governs. However, a shear check must also be made.

CHECK ROLLING SHEAR: From Table 4.14, for 3/4 in. Class I Plyform with load duration < 7 days, the adjusted design value for rolling shear, F_{rs}'= 72 psi. From Table 4.13 for strong direction of use, *Ib/Q* = 7.187 in.^{2}

Using Eq. (7.14c), find the maximum clear span length for three-span continuous plywood sheathing

$$\ell_c = \frac{F_{rs}'(Ib/Q)}{0.6w} = \frac{72\ \text{psi}\times 7.187\ \text{in.}^2}{0.6\times 50\ \text{lb/in.}} = 17.25\ \text{in.}$$

At this point, the stud size may be unknown but is frequently a 2x4 so that ℓ_b = 1.5 in. and thus, ℓ =18.75 in.

SPACING OF THE STUDS: The span length based on bending moment span length is the smallest so it governs. Studs can be no farther than 13.2 in. apart. A 12 in. spacing of the studs is convenient for layout in Fig. B, but because the 8 ft plywood sheets should have studs for support at the joints, the first and last studs for each panel will be drawn in 3/4 in. This will result in a double stud at joints between adjacent panels.

Step 3. *Stud Size and Spacing of Wales* to support them (Fig. C). Assuming that 2x4 S4S studs will be used, find their maximum span length where the lateral pressure is greatest—600 lb/ft^2. Equivalent uniform load *w* for design of studs will be the maximum lateral pressure in pounds per square foot times the stud spacing, *s*. The stud spacing is the selected span length of the plywood, and the stud load is based on the simple tributary width.

w (studs) = $C_{CP} \times s$ = 600 lb/ft^2 × 12 in./12 in./ft = 600 lb/ft = 50 lb/in.

Assuming the studs act as uniformly loaded continuous beams, check allowable spans for bending, deflection, and shear.

BENDING CHECK: First, determine allowable bending stress. The reference design value for bending stress, F_b, for No. 2 Douglas Fir-Larch is 900 psi (Table 4.3). Possible adjustment factors from Table 4.4 are:

C_D = 1.25 for short-term loading less than 7 days;

C_M = 1.0 assuming the moisture content of the studs will not exceed 19% for an extended period of time;

C_t = 1.0 because ambient temperature is unlikely to exceed 100°F during concrete placement;

C_L = 1.0 from Table 7.3 and because nailing of plywood will provide lateral support to the studs;

C_F = 1.5 from Table 4.8;

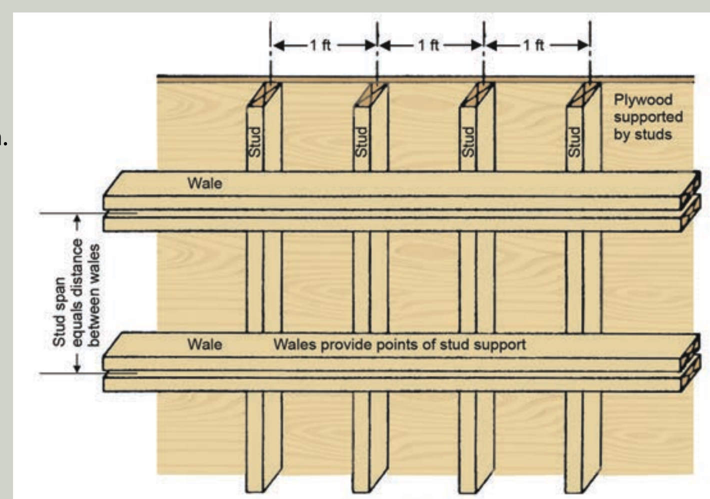

Fig. C: Wall form vertical studs supported by horizontal wales

C_r = 1.15 from Chapter 4 because there will be three or more studs spaced ≤ 24 in. on center;

C_{fu} does not apply because the studs are oriented for strong axis bending; and

C_i does not apply because the wood is not incised.

The adjusted bending design value is

$$F_b' = F_b C_D C_M C_t C_L C_F C_{fu} C_i C_r$$

$$F_b' = 900 \text{ psi} \times 1.25 \times 1.5 \times 1.15 = 1940 \text{ psi}$$

Use Eq. (7.3b) based on a three or more span continuous beam to determine the maximum stud span length with S = 3.06 in.^{3} for an S4S 2x4 from Table 4.2

$$\ell = 3.16\sqrt{\frac{F_b' S}{w}} = 3.16\sqrt{\frac{1940 \text{ psi} \times 3.06 \text{ in.}^3}{50 \text{ lb/in.}}} = 3.16\sqrt{118.7 \text{ in.}^2} = 34.4 \text{ in.}$$

DEFLECTION CHECK: The allowable deflection Δ is required to be less than the smaller of 1/360 of the span length or 1/16 in. For the Douglas Fir-Larch 2x4, I = 5.36 in.^{4} from Table 4.2 and E = 1,600,000 psi from Table 4.3. Referring to Table 4.4

$$E' = E C_M C_t C_i = 1{,}600{,}000 \text{ psi}$$

Using Eq. (7.5) for $\Delta = \ell/360$, the maximum allowable span length is

$$\ell = 0.738\sqrt[3]{\frac{E'I}{w}} = 0.738\sqrt[3]{\frac{1{,}600{,}000 \text{ psi} \times 5.36 \text{ in.}^4}{50 \text{ lb/in.}}}$$

$$= 0.738\sqrt[3]{171{,}500 \text{ in.}^3} = 41.0 \text{ in.}$$

For Δ_{max} = 1/16 in., substitute in Eq. (A) and solve to get maximum allowable span length

$$\ell = 1.735\sqrt[4]{\frac{E'I(\text{in.})}{w}} = 1.735\sqrt[4]{\frac{1{,}600{,}000 \text{ psi} \times 5.36 \text{ in.}^4 (\text{in.})}{50 \text{ lb/in.}}}$$

$$= 1.735\sqrt[4]{171{,}500 \text{ in.}^4} = 35.3 \text{ in.}$$

SHEAR CHECK: The reference design value for horizontal shear of 180 psi from Table 4.3 is adjusted as follows (Table 4.4)

$$F_v' = F_v C_D C_M C_t C_i = 180 \text{ psi} \times 1.25 = 225 \text{ psi}$$

When determining the center-to-center span of the studs, length of bearing at the stud supports is often known or can be anticipated. In this case, double S4S 2x4 wales would probably be the minimum size so that the length of bearing is at least 1.5 in. at each end. For a three or more span continuous beam, the allowable span length from Eq. (7.11c) is

$$\ell = \frac{F_v' bd}{0.9w} + 1.67d + 0.83\ell_b =$$

$$\frac{225 \text{ psi} \times 1.5 \text{ in.} \times 3.5 \text{ in.}}{0.9 \times 50 \text{ lb/in.}} + 1.67 \times 3.5 \text{ in.} + 0.83 \times 2 \times 1.5 \text{ in.} = 34.6 \text{ in.}$$

SPACING OF THE WALES: Comparing the stud spans determined by allowable bending, deflection, and shear shows that the span length based on bending strength is the shortest at 34.4 in. The wales, which are the stud supports, can be spaced no more than 34.4 in. apart where maximum pressure of 600 lb/ft^2 exists.

Increased wale spacing is theoretically acceptable near the top of the form because the maximum design pressure declines from 600 lb/ft^2 at a depth of 4 ft to zero at the top. In this form, the top wale could be spaced at a greater distance than 34.4 in. from the adjacent one. However, construction considerations often determine the exact dimensions. The top and bottom wales are frequently set approximately 1 ft from top and bottom of wall forms.

With wales 12 in. from both top and bottom of the wall form, 12 ft or 144 in. remains for spacing the other wales, which can be no more than 34.4 in. apart. Even measurements or modular dimensions are convenient for field assembly, so by trial and error we arrive at a selection of four spaces of 30 in. each and one of 24 in. (Fig. D). We place the smaller span length near the bottom of the form where theoretically a higher pressure could occur if the assumptions on placing conditions did not match those in the field.

Step 4. *Wale Size and Tie Spacing*: Sketch the pressure diagram alongside the diagram of wale spacing, and determine equivalent uniform load per foot of wale. Assume that each wale carries load from the form for half the distance to each adjacent wale, a tributary width equal to the wale spacing. This gives the equivalent uniform load per foot of wale.

Because 3350 lb (safe working load) ties are to be used, the tie spacing will be determined as noted in Step 5, which follows. Wale design can then be developed using the calculated tie spacing of 2 ft. (Refer to Step 5, Tie Design.)

BENDING CHECK: Both span length and loading are known; thus, this is Case B. Substitute $F_b'S$ for M_{max} in Eq. (7.1b) and solve for the required section modulus, S. (The equivalent uniform load is used although actual wale loading is point loading applied where studs bear on wales.)

$$F_b' S = \frac{w\ell^2}{10}$$

$$S = \frac{w\ell^2}{10F_b'} \qquad \text{(B)}$$

The adjustments for bending stress in the wale are similar to those determined when checking the studs so long as we use lumber 4 in. wide or less, except that the wales are not repetitive members because the spacing exceeds 24 in.

$$F_b' = 900 \text{ psi} \times 1.25 \times 1.5 \times 1.0 = 1687 \text{ psi}$$

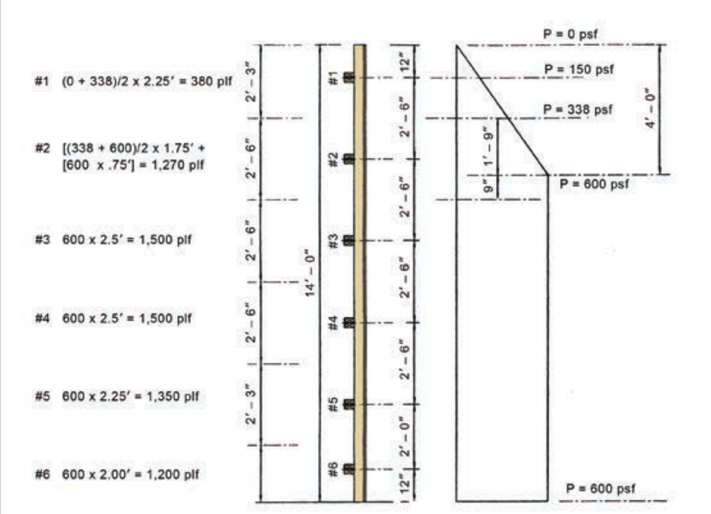

Fig. D: Spacing of wall form wales and tributary loads at each wale level

The wale span length, ℓ, is 24 in. and the maximum equivalent uniform load on the wales is determined from the simple tributary width corresponding to the stud span length and wale spacing, *s*, of 30 in.

$$w(wale) = C_{CP} \times s = 600 \text{ lb/ft}^2 \times \frac{30 \text{ in.}}{12 \text{ in./ft}}$$

$$= 1500 \text{ lb/ft} = 125 \text{ lb/in.}$$

Substitute these values in Eq. (B)

$$S = \frac{125 \text{ lb/in.} \times (24 \text{ in.})^2}{10 \times 1687 \text{ psi}} = 4.26 \text{ in.}^3$$

If a double-member wale is used, as is common to avoid drilling of timbers, the computed *S* is the required section modulus of two members. Use Table 4.2 to select an appropriate size. Double 2x4s ($S = 2 \times 3.06 \text{ in.}^3 = 6.12 \text{ in.}^3$) meet the requirements.

SHEAR CHECK: Use Eq. (7.11c) to check horizontal shear in the proposed double 2x4 with the 2 in. wide tie wedge plate (Fig. E) for bearing length and assuming equivalent uniform load

$$\ell = \frac{F_v'bd}{0.9w} + 1.67d + 0.83\ell_b =$$

$$\frac{225 \text{ psi} \times 2 \times 1.5 \text{ in.} \times 3.5 \text{ in.}}{0.9 \times 125 \text{ lb/in.}} + 1.67 \times 3.5 \text{ in.} + 0.83 \times 2 \text{ in.} = 28.5 \text{ in.}$$

When the series of point loads are spaced at intervals greater than half the wale span length, a check using actual loads rather than equivalent uniform loading is advisable. Review of the similar cases in Fig. 7.1 where loads are spaced at half the span length suggests that the 24 in. wale span length, with stud reaction point loads at 12 in. spacing, should be adequate.

Step 5. *Tie Design:* Assume that ties of 3350 lb safe working load (allowable) capacity are available and will be used. Because the tie provides rigid support, the load should be determined considering continuity of the wale at the first interior support. When wale loading in lb per foot is known at the beginning of Step 4, the tie spacing can then be calculated. Not all wales, from form top to bottom, carry the same load. Tie spacing is often designed for the heaviest loading and used uniformly throughout the form for concrete surface aesthetic considerations and convenience in drilling plywood panels.

Maximum load on the wale is $w = 600 \text{ lb/ft}^2 \times 2.5 \text{ ft} = 1500 \text{ lb/ft} = 125 \text{ lb/in.}$ The maximum tie load from Table 7.2 for a three-span continuous uniformly loaded beam is

$$R_b = R_c = 1.1w\ell$$

Setting the maximum reaction to the tie capacity and solving for ℓ, the tie spacing

$$\text{tie spacing} = \frac{\text{tie capacity}}{\text{wale load}} = \frac{3350 \text{ lb}}{1.1 \times 125 \text{ lb/in.}} = 24.3 \text{ in.}$$

Use a 2 ft tie spacing. This agrees with the assumed tie spacing used to complete the wale design in Step 4.

Step 6. Bearing Check: Points to be investigated in this design would be bearing perpendicular to grain of studs on wales and bearing between the tie washers or tie holders and the wales. The reference design value for $F_{c\perp}$ is 625 psi for No. 2 Douglas Fir-Larch (Table 4.3).

TIES: In areas of maximum lateral pressure, maximum reaction at the tie is 1.1 × 125 lb/in. × 24 in., or 3300 lb. Assume the 2 x 6 in. tie wedge plate will have 2 x 3 in. = 6.0 in.[2] of actual contact with the double 2x4s that make up the wale (Fig. E) The length of bearing, ℓ_b, is 2 in. and C_b = 1.19 from Table 7.4. Other adjustment factors are either 1.0 or not applicable as before. From Table 4.4

$$F_{c\perp}' = F_{c\perp} C_M C_t C_i C_b = 625 \text{ psi} \times 1.19 = 743 \text{ psi}$$

The actual bearing stress is

$$\frac{\text{maximum reaction at tie}}{\text{bearing area}} = \frac{3300 \text{ lb}}{6.0 \text{ in.}^2} = 550 \text{ psi} < 743 \text{ psi}$$

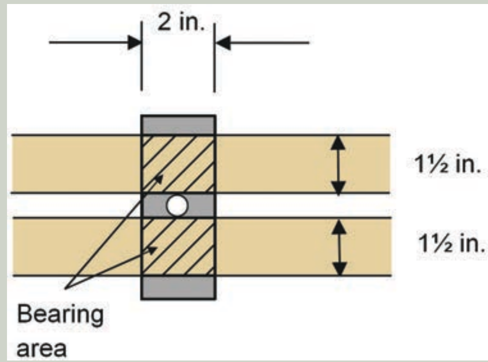

Fig. E: Bearing area between tie wedge plate and double wale

The actual bearing stress is within the allowable of 743 psi; thus, the bearing of the wedge plate on the wale is satisfactory.

STUDS ON WALES: The bearing area between 2x4 studs and double 2x4 S4S wales would be as shown in Fig. F.

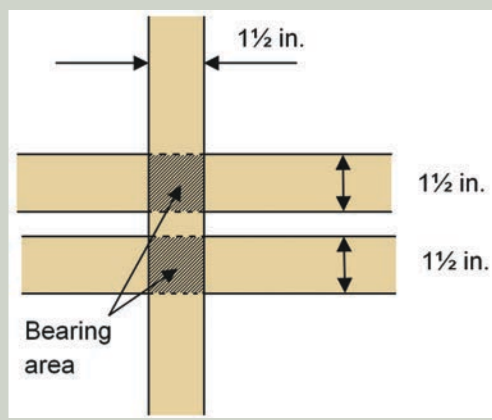

Fig. F: Bearing area between stud and wale

$A_b = 2 \times (1\text{-}1/2 \text{ in.} \times 1\text{-}1/2 \text{ in.}) = 4.5 \text{ in.}^2$

Maximum load transferred from studs to wales is in the region of maximum lateral pressure. Load transferred to the wale is the stud reaction on the wale.

$R_b = R_c = = w\ell = 50 \text{ lb/in.} \times 30 \text{ in.} = 1500 \text{ lb}$

Actual bearing stress is the contact stress between the stud and the double wale and is compared to the allowable bearing stress.

$$f_{c\perp} = \frac{R_b}{A_b} = \frac{1500 \text{ lb}}{4.5 \text{ in.}^2} = 333 \text{ psi}$$

The allowable bearing stress must consider both the length of bearing on the wale measured parallel to the grain of the wale, ℓ_b = 1-1/2 in., and the length of bearing on the stud measured parallel to the grain of the stud, ℓ_b = 2(1-1/2 in.) + 1/2 in. = 3.5 in. In the latter case, there are actually two areas of contact that are close together. Using the total length is a conservative assumption because the C_b factor is lower for greater bearing lengths. From Table 7.4, C_b = 1.11 and the allowable bearing stress is

$$F'_{c\perp} = F_{c\perp} C_M C_t C_i C_b = 625 \text{ psi} \times 1.11 = 694 \text{ psi}$$

The actual bearing stress of 333 psi is within the allowable 694 psi; thus, the design is satisfactory with respect to bearing.

Step 7. ***Lateral Bracing*** for this type of form is explained in Chapter 8.

Example summary:

Sheathing: 3/4 in. Class I BB Plyform;

Studs: 2x4 No. 2 S4S Douglas Fir-Larch spaced at 12 in. on center;

Wales: double 2x4 No. 2 S4S Douglas Fir-Larch spaced at 30 in. on center in maximum pressure region;

Ties: 3350 lb minimum capacity spaced at maximum of 24 in. with 2 in. x 6 in. wedge plate washers.

Example 7.3: Partial Wall Form Design—LRFD

As an example of LRFD design, the design for the span length of the studs for Example 7.2 will be repeated using LRFD.

From Eq. (5.14), the factored lateral pressure becomes $1.6C_{CP}$ and with the 12 in. spacing of the studs based on the plywood maximum span length from the ASD analysis

w (studs) = $1.6C_{CP} \times s = 1.6 \times 600$ lb/ft^2 $\times$ 12 in./12 in./ft = 960 lb/ft = 80 lb/in.

BENDING CHECK: First, determine LRFD adjusted bending stress. Reference design value for bending stress F_b for No. 2 Douglas Fir-Larch is 900 psi (Table 4.3).

Possible adjustment factors from Table 4.4 are:

C_M = 1.0 assuming the moisture content of the studs will not exceed 19% for an extended period of time;

C_t = 1.0 because ambient temperature is unlikely to exceed 100°F during a concrete placement;

C_L = 1.0 from Table 7.3 and because nailing of plywood will provide lateral support to the studs;

C_F = 1.5 from Table 4.8;

C_r = 1.15 from Chapter 4 because there will be three or more studs spaced ≤ 24 in. on center;

C_{fu} does not apply because the studs are oriented for strong axis bending;

C_i does not apply because wood is not incised;

K_F = 2.54 from Table 4.10;

ϕ_b = 0.85 from Table 4.10; and

λ = 0.9.

The selection of λ is related to the duration of the dominant loads as discussed in Chapter 4. In this case, the value of 0.9 reflects the load duration less than 7 days.

The adjusted bending design stress is

$$F'_b = F_b C_M C_t C_L C_F C_{fu} C_i C_r K_F \phi_b \lambda$$

$$F'_b = 900 \text{ psi} \times 1.5 \times 1.15 \times 2.54 \times 0.85 \times 0.9 = 3016 \text{ psi}$$

Use Eq. (7.3b) to determine the maximum stud span length as a continuous beam with S = 3.06 in.^{3} for a S4S 2x4 from Table 4.2

$$\ell = 3.16\sqrt{\frac{F'_b S}{w}} = 3.16\sqrt{\frac{3016 \text{ psi} \times 3.06 \text{ in.}^3}{80 \text{ lb/in.}}}$$
$$= 3.16\sqrt{115.4 \text{ in.}^2} = 33.9 \text{ in.}$$

SHEAR CHECK: The reference design value for horizontal shear of 180 psi from Table 4.3 is adjusted following Table 4.4 and Table 4.10 as follows

$$F'_v = F_v C_M C_t C_i K_F \phi_v \lambda = 180 \text{ psi} \times 2.88 \times 0.75 \times 0.9 = 350 \text{ psi}$$

When determining the center-to-center span of the studs, length of bearing at the stud supports is often known or anticipated. In this case, double S4S 2x4 wales would probably be the minimum size so that the length of bearing is at least 1.5 in. at each end. For a three or more span continuous beam, the allowable span length from Eq. (7.11c) is

$$\ell = \frac{F'_v bd}{0.9w} + 1.67d + 0.83\ell_b =$$
$$\frac{350 \text{ psi} \times 1.5 \text{ in.} \times 3.5 \text{ in.}}{0.9 \times 80 \text{ lb/in.}} + 1.67 \times 3.5 \text{ in.} + 0.83 \times 2 \times 1.5 \text{ in.} = 33.8 \text{ in.}$$

Deflection of the studs must still be based on actual loads rather than factored loads and therefore is calculated by ASD as before.

Comparing ASD and LRFD results for calculated span length of the 2x4 wall studs:

	ASD	LRFD
Bending	34.4 in.	33.9 in.
Shear	34.6 in.	33.8 in.
Deflection	41.0 in.	Same as ASD

In this case, there is very little difference between ASD and LRFD results.

7.7 Slab Form Design

Because conditions vary greatly from project to project, there is no single correct sequence for slab form design. Once the load applied to the forms is calculated, design may start at any one of several points. The general goal is a balanced form design—one that loads all the parts at or near their safe carrying capacity. Often, the preliminary design has to be adjusted to fit the module used in design of the structure, or it may be modified to improve the "balance" and use materials more efficiently. Where labor costs are high, changes may be introduced to save labor time instead of materials.

Following are the basic steps performed during a slab form design:

1. *Design load*—Determine the dead and live loads for which the forms must be designed, according to provisions cited in Chapter 5. Deflection limits are primarily to limit bulges in the formed concrete surface. Because the slab formwork members rebound when the temporary live load of workers is removed, formwork designers sometimes do not include live load in deflection calculations.

NOTE: The sequence of the following steps has been arbitrarily selected. On one project it might be entirely reversed; on another, the designer might preselect both shores and sheathing, in effect working from both ends toward the middle of the suggested sequence. Each individual step, however, is needed, and Example 7.4 shows how calculations can be made. The intended sheathing type and grade and the intended wood species and grade are often decided in advance. Plywood is assumed in the following step.

2. *Sheathing thickness and spacing of its supports (joist spacing)*—One of these will be predetermined by the form designer, based on economic considerations of material available for supply or reuse, or based on layout dimensional constraints, and the other will be calculated.

BENDING CHECK

A. If sheathing thickness is known, determine its maximum allowable span length, which is the maximum spacing of joists.

B. If the joist spacing is fixed, calculate the required section modulus *KS* of sheathing to carry the load, and select sheathing thickness to meet this requirement from Table 4.13.

DEFLECTION CHECK

A. If sheathing thickness is known, calculate maximum allowable span length that satisfies deflection requirements.

B. If the joist spacing is fixed, transpose the deflection equation to solve for the required moment of inertia *I*. Then select sheathing thickness to meet this requirement, using Table 4.13.

SHEAR CHECK

A. If sheathing thickness is known, calculate the maximum span length that satisfies shear-stress requirements.

B. If the joist spacing is fixed, solve the appropriate shear equation for the required plywood rolling shear constant, *Ib/Q*. Select thickness to meet the requirement (Table 4.13).

3. *Joist size and spacing of supports (stringer spacing)*—One of these must be selected arbitrarily and the other designed to correspond with it. In some cases, joists are supported directly on shores, but the allowable span length of joists will generally serve to determine the stringer spacing. Joists are often continuous over three or more spans. (In slab and beam formwork construction, the joist may span between two beam sides with one or no intermediate supports, in which case the appropriate beam formula corresponding to the number of spans should be used for design.)

BENDING CHECK

A. If the joist size is known, calculate its maximum allowable span length. This span length is the maximum allowable spacing of the stringers.

B. If the joist span length is fixed by other job conditions, transpose the basic equation to solve for the required section modulus *S*, and then select a joist with the required *S*.

DEFLECTION CHECK

A. If the joist size is known, calculate the maximum allowable span length that meets the deflection requirements.

B. If the joist span length is fixed by other job conditions, solve the deflection equation for *I* and select a member to meet this requirement.

SHEAR CHECK

When a combination of joist size and span length (stringer spacing) has been selected on the basis of deflection and bending requirements, investigate horizontal shear in the joist. If this exceeds the allowable stress, a modification to either size or span length must be made.

4. ***Stringer size and span length (shore spacing)***—Depending on job conditions, either the shore spacing or the size of the stringer will have been preselected. The other must be designed to correspond. Stringers are actually loaded with a series of point loads from the joists, but for most cases an equivalent uniform load is sufficiently accurate for design purposes. In cases of heavily loaded short spans, a shear check using point loads may be desirable.

BENDING CHECK

A. If the stringer size is known, calculate the maximum allowable span length. This establishes a maximum spacing of the shores.

B. If the span length of stringer is fixed by predetermined shore spacing, transpose the basic equation to solve for the section modulus *S*, and select a member (or members) with the required *S*.

DEFLECTION CHECK

A. If the stringer size is known, calculate the maximum allowable span length that meets deflection requirements.

B. If the span length of the stringers is fixed on the basis of shore spacing, solve the deflection equation for *I* and select a member to meet this requirement.

SHEAR CHECK

A. If the stringer size is known, calculate the maximum allowable span length that can be used to keep shear stress f_v below the allowable.

B. If the shore spacing is fixed, this in effect determines the span length of the stringers. Use the shear formula to determine the required stringer cross-sectional area *bd*.

When these three checks have been made, it will be obvious which criterion governs stringer design. If shear governs, it may be advisable to recheck the design using point loads on the stringers instead of equivalent uniform loads because maximum horizontal shear stress may be significantly affected by the location of the point loads.

5. ***Shore design to support stringers*** is based on principles given in Chapter 8. Shore design (both spacing and size) may precede selection of stringers, or it may be done after other formwork components have been designed. If available shores are not suitable to support stringers at the designed spacing, the spacing must be revised. This may necessitate other design changes.

6. ***Check bearing stresses*** as recommended in Section 7.4.5, wherever loads are transmitted perpendicular to the grain of a wood member.

7**.** ***Design lateral bracing*** for shores to carry minimum lateral force prescribed in Table 5.6. Design of bracing is explained in Chapter 8.

Several members of a slab form could alternately be designed using the LRFD method but presently not all can. Plywood design specifications are currently based on ASD only. Many formwork hardware devices, such as manufactured steel and aluminum shores, are rated for working (allowable) load capacity and rating methods have not been established by ACI 347R for LRFD use. LRFD design procedures for the joists designed by ASD in Example 7.4 are illustrated in Example 7.5.

Example 7.4: Slab Form Design—ASD

Design forms to support an 8 in. thick flat plate floor of normal-weight concrete, using construction grade Douglas Fir-Larch S4S framing members and steel shoring. Ceiling height is 8 ft and bays are 15 x 15 ft. Shoring towers with a 5 ft leg spacing each way are available and will be used so that the stringer spacing and typical span length will be 5 ft. Job conditions are such that the wood joists and stringers will not be subject to wet service. Because the forms will have substantial reuse, do not adjust reference design values for short-term load. Deflection of the sheathing is limited to the lesser of ℓ/360 or 1/16 in. between supporting form members and deflection of other members is limited to ℓ/360. Live load is temporary and need not be considered for vertical deflection.

Step 1: ***Estimate Loads***

Dead load concrete and reinforcing = 150 lb/ft³ × 8 in./(12 in./ft)	C_{VML} = 100 lb/ft²
Minimum construction live load on forms	C_{PE} = 50 lb/ft²
Weight of forms, estimated	C_{DL} = 8 lb/ft²
Total form vertical design load for strength	$q_s = C_{DL} + C_{VML} + C_{PE}$ = 158 lb/ft²
Total form vertical design load for deflection	$q_d = C_{DL} + C_{VML}$ = 108 lb/ft²

The load of the fresh concrete can be considered as fixed material load, C_{FML}, if it is placed without having temporary excess thickness. But it should be considered as variable material load, C_{VML}, if temporarily over-deposited in local areas. The construction live load must be included in strength calculations such as bending, shear, and bearing. However, it is reasonable not to include this live load in deflection calculations because loads from workers and equipment are not present during the setting of the concrete.

Step 2. *Sheathing Design:* Assuming 3/4-in. Structural I BB Plyform sheathing is to be used as shown in Fig. A, determine its maximum allowable span length, working with a 1 ft wide strip for convenience in design.

Allowable plywood stresses can be taken directly from Table 4.14, where plywood values are already adjusted for wet service, experience, and load duration

$F_b' = 1545$ psi; $F_{rs}' = 82$ psi; $E' = 1{,}500{,}000$ psi

Use plywood the strong way, with face grain parallel to span. From Table 4.13, select plywood cross-section properties needed for design

$KS = 0.464$ in.^{3}; $I = 0.202$ in.^{4}; $Ib/Q = 6.189$ in.^{2}

CHECK BENDING: For design purposes, we can look at a 1 ft wide strip of plywood. Uniform load on this strip is 158 lb/ft (1 ft × design load of $q_s = 158$ lb/ft^2) or 13.16 lb/in. By Eq. (7.3b) and with $S = KS$, the maximum allowable span length based on three or more continuous spans is

$$\ell = 3.16\sqrt{\frac{F_b'S}{w}} = 3.16\sqrt{\frac{1545 \text{ psi} \times 0.464 \text{ in.}^3}{13.16 \text{ lb/in.}}} = 23.3 \text{ in.}$$

This is the maximum span length based on bending strength of the plywood. However, deflection frequently governs sheathing design and it must be checked. A shear check is also shown in the following, but it generally does not govern for sheathing with loads as light as this slab.

CHECK DEFLECTION, considering a 12 in. width of sheathing. Specified maximum allowable deflection Δ is the lesser of 1/360 of span length or 1/16 in. Uniform load on this strip is 1 ft × design load of $q_d = 108$ lb/ft^2 or 108 lb/ft = 9.0 lb/in. Substituting in Eq. (7.5) for $\Delta_{max} = \ell/360$

$$\ell = 0.738\sqrt[3]{\frac{E'I}{w}} = 0.738\sqrt[3]{\frac{1{,}500{,}000 \text{ psi} \times 0.202 \text{ in.}^4}{9.0 \text{ lb/in.}}}$$

$$= 0.738\sqrt[3]{33{,}667 \text{ in.}^3} = 23.8 \text{ in.}$$

and substituting in Eq. (A) (from Example 7.2) for $\Delta_{max} = 1/16$ in.

$$\ell = 1.735\sqrt[4]{\frac{E'I \text{ (in.)}}{w}} = 1.735\sqrt[4]{33{,}667 \text{ in.}^4} = 23.5 \text{ in.}$$

Maximum allowable span length on the basis of the deflection limit of 1/16 in. would be 23.5 in.

CHECK ROLLING SHEAR using Eq. (7.14c) for sheathing over at least three continuous spans. The maximum allowable clear span length is

$$\ell_c = \frac{F_{rs}'(Ib/Q)}{0.6w} = \frac{82 \text{ psi} \times 6.189 \text{ in.}^2}{0.6 \times 13.16 \text{ lb/in.}} = 64.2 \text{ in.}$$

The center-to-center span can be obtained by adding the width of the support bearing but it is clear that rolling shear does not govern.

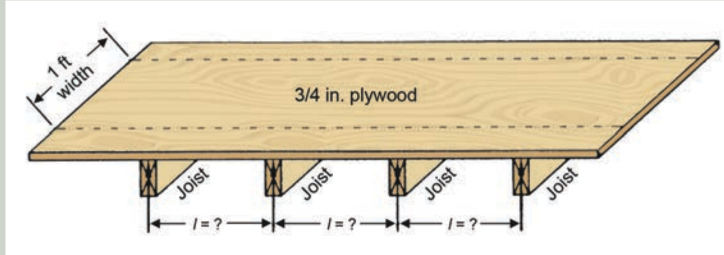

Fig. A: Slab form plywood sheathing supported by joists

The span length (23.3 in.) governed by bending is smallest. However, selection of the actual span length must consider bay sizes and the possible desire to divide bay dimensions into a number of equal dimensions. Another factor is that plywood sheets need support from the joists at both ends. Dividing the 8 ft panel length into five spans of 19.2 in. gives us the equal spacing closest to the controlling 23.3 in. length. The 19.2 in. spacing is selected.

Step 3. *Joist size and spacing of stringers to support the joists.* Because the stringers will be spaced at 5 ft on center, the joist must span 5 ft. A three-span continuous layout will fit the 15 ft bay. Because the load is relatively light, we will check 2x4s as joists. With construction grade Douglas-Fir-Larch, the size adjustment factor for bending, C_F, for a 2x4 is 1.0 from the Table 4.8 footnote. (Notice that if this were No. 1 or No. 2 lumber, a size adjustment factor from Table 4.8 would be applied and would vary with depth.) Because the forms will be used repeatedly, the cumulative load duration will be assumed >2 months so that $C_D = 1.0$. From Table 7.3, $C_L = 1.0$ for a 2x4. If a larger depth (2x6 or 2x8) is chosen, the joist ends will need to be restrained from rotation by full-depth blocking or other means for C_L to be 1.0.

The bending reference design value from Table 4.3 is F_b = 1000 psi. Assuming the temperature is below 100°F, no temperature adjustment for either bending or shear stress is needed. Based on stated job conditions, $C_M = 1.0$. The joists will be oriented for strong axis bending, not flat use, so $C_{fu} = 1.0$, and the wood is not indicated as being incised; thus, $C_i = 1.0$. Because the joist spacing is ≤ 24 in., there are more than three, and they are joined by the sheathing diaphragm, the repetitive member factor can be considered, $C_r = 1.15$. The adjusted bending stress is

$$F_b' = F_b C_D C_M C_t C_L C_F C_{fu} C_i C_r$$

$$F_b' = 1000 \text{ psi} \times 1.15 = 1150 \text{ psi}$$

From Table 4.3, the reference design value for shear stress F_v is 180 psi and for E is 1,500,000 psi. Because $C_D = 1.0$, $C_M = 1.0$, $C_t = 1.0$, and $C_i = 1.0$, the adjusted design values are

$$F_v' = F_v C_D C_M C_t C_i = 180 \text{ psi}$$

$$E' = E C_M C_t C_i = 1{,}500{,}000 \text{ psi}$$

The uniform load, w, on each joist using a tributary width equal to the joist spacing, $s = 19.2$ in. is for strength calculations

$$w = q_s \times s = \frac{158\ \text{lb/ft}^2}{144\ \text{in.}^2/\text{ft}^2} \times 19.2\ \text{in.} = 21.1\ \text{lb/in.} = 253\ \text{lb/ft}$$

for deflection calculations

$$w = q_d \times s = \frac{108\ \text{lb/ft}^2}{144\ \text{in.}^2/\text{ft}^2} \times 19.2\ \text{in.} = 14.4\ \text{lb/in.} = 173\ \text{lb/ft}$$

For the S4S 2x4s, $bd = 5.25$ in.^{2}, $I = 5.36$ in.^{4}, and $S = 3.06$ in.^{3} (values from Table 4.2).

CHECK BENDING: Determine required size for a continuous beam of three spans $\ell = 5$ ft. Combining Eq. (7.1b) and (7.2b), rearranging, and substituting

$$F_b'S = \frac{w\ell^2}{10}$$

Solving for the unknown section modulus

$$S = \frac{w\ell^2}{10F_b'} = \frac{21.1\ \text{lb/in.} \times (60\ \text{in.})^2}{10 \times 1150\ \text{psi}} = 6.60\ \text{in.}^3$$

CHECK DEFLECTION: For a continuous beam, with deflection limited to 1/360 of the span length, from Eq. (7.4c)

$$\frac{\ell}{360} = \frac{w\ell^4}{145E'I}$$

Solving for the unknown moment of inertia

$$I = \frac{360w\ell^3}{145E'} = \frac{360 \times 14.4\ \text{lb/in.} \times (60\ \text{in.})^3}{145 \times 1{,}500{,}000\ \text{psi}} = 5.15\ \text{in.}^4$$

Before checking shear, examine which sections satisfy the S and I requirements for bending and deflection. Comparing the 2x S4S sections in Table 4.2, a 2x4 is insufficient; however, a 2x6 meets the requirements, $S = 7.56$ in.$^3 > 6.60$ in.^{3} and $I = 20.08$ in.$^4 >> 5.15$ in.^{4}

CHECK SHEAR: From Eq. (7.10c) with $f_v = F_v'$

$$F_v' = \frac{0.9w}{bd}\left[\ell - (1.67d + 0.83\ell_b)\right]$$

Solving for bd directly is difficult because to fully benefit from designing for the shear force at a distance d from the face of the support, one needs to know the width of the supporting stringer and the depth of the joist, which might not be known at this point. There are two options:

- Ignore the benefit of the reduced shear force at distance d from the support face and see if shear does not control; or
- Use the knowledge of a possible size based on bending and deflection requirements, estimate ℓ_b based on experience, and verify size when the stringer is selected.

Trying the first option:

$$F_v' = \frac{0.9w\ell}{bd}$$

$$bd = \frac{0.9w\ell}{F_v'} = \frac{0.9 \times 21.1\ \text{lb/in.} \times 60\ \text{in.}}{180\ \text{psi}}$$

$$= 6.33\ \text{in.}^2 < A_{2x6} = 8.25\ \text{in.}^2 \quad \text{shear does not control}$$

Trying the second option just for the exercise with $d = 5.5$ in. and $\ell_b = 3.5$ in.

$$F_v' = \frac{0.9w}{bd}\left[\ell - (1.67d + 0.83\ell_b)\right]$$

$$180\ \text{psi} = \frac{0.9 \times 21.1\ \text{lb/in.}}{bd}\left[60\ \text{in.} - (1.67 \times 5.5\ \text{in.} + 0.83 \times 3.5\ \text{in.})\right]$$

$$bd = \frac{0.9 \times 21.1\ \text{lb/in.}}{180\ \text{psi}}\left[60\ \text{in.} - 12.1\ \text{in.}\right] = 5.05\ \text{in.}^2 < A_{2x6} = 8.25\ \text{in.}^2$$

In this example, shear does not control design of the joist, but the advantage of being able to calculate shear at distance d from the face of the support becomes apparent if there is a case where it does control.

Comparison of the three limiting criteria calculated previously shows that bending governs design and the minimum required size is a 2x6 S4S. Some formwork contractors might prefer a 4×4 S4S (that also meets the section property requirements). Even though the larger cross-section area results in more wood volume and material cost, it might provide net savings through reduced labor and reduced materials due to not needing lateral support for buckling.

Step 4. *Stringer size and span length.* The stringers are spaced at $s = 5$ ft. Each stringer spans 5 ft from shore to shore and is three-span continuous, as shown in Fig. B. First, find the equivalent uniform load on the stringers:

for strength calculations

$$w = q_s \times s = 158\ \text{psf} \times 5\ \text{ft} = 790\ \text{lb/ft} = 65.8\ \text{lb/in.}$$

for deflection calculations

$$w = q_d \times s = 108\ \text{lb/ft}^2 \times 5\ \text{ft} = 540\ \text{lb/ft} = 45\ \text{lb/in.}$$

This equivalent uniform loading is usually sufficiently accurate. However, if the stringer design selected is close to limiting stresses or deflection, a recheck using point loading of joists on the stringers might be necessary. Rounding off span length values to get modular layout can result in extra capacity that can offset differences due to the assumed load distribution.

Using construction-grade Douglas Fir-Larch S4S stringers, adjusted design values will be the same as those for aforementioned joists except that the stringers are not repetitive members because their spacing > 24 in. Therefore, $F_b' = 1000$ psi. Obtain cross-sectional properties by the same process as used for the joists.

CHECK BENDING:

$$S = \frac{w\ell^2}{10F_b'} = \frac{65.8\ \text{lb/in.} \times (60\ \text{in.})^2}{10 \times 1000\ \text{psi}} = 23.7\ \text{in.}^3$$

CHECK DEFLECTION: Working with a deflection limit of 1/360 of the span length

$$I = \frac{360w\ell^3}{145E'} = \frac{360 \times 45\ \text{lb/in.} \times (60\ \text{in.})^3}{145 \times 1{,}500{,}000\ \text{psi}} = 16.1\ \text{in.}^4$$

CHECK SHEAR: Simplify the initial check by using the shear at the support to see if shear controls.

$$bd = \frac{0.9w\ell}{F_v'} = \frac{0.9 \times 65.8\ \text{lb/in.} \times 60\ \text{in.}}{180\ \text{psi}} = 19.74\ \text{in.}^2$$

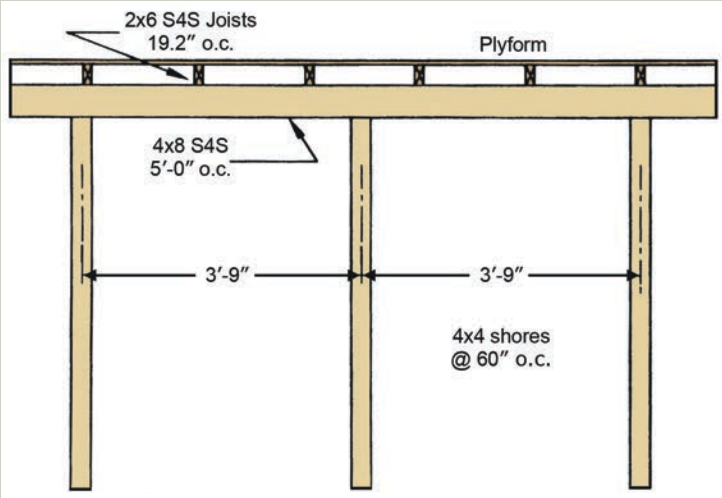

Fig. B: Slab form stringers supported by shores

Consulting Table 4.2, the objective is to find a size that meets all three requirements and will not have a lateral stability problem. A 4x8 S4S has the necessary section properties and with a *d/b* of 2 automatically satisfies lateral stability. Comparing properties to those needed:

$S_{req'd} = 23.7 \text{ in.}^3$ $\quad S_{4x8} = 30.66 \text{ in.}^3$

$I_{req'd} = 16.1 \text{ in.}^4$ $\quad I_{4x8} = 111.1 \text{ in.}^4$

$bd_{req'd} = 19.74 \text{ in.}^2$ $\quad A_{4x8} = 25.38 \text{ in.}^2$

A 16 ft long 4x8 S4S would be needed for three spans. The weight, 16 ft x 6.2 lb/ft = 100 lb, could be managed by two workers but some formwork engineers might also consider re-design using an aluminum beam with a 2x4 top nailer to reduce the handling weight.

Step 5. *Shore design:* The shore layout was predetermined by the planned use of an available shoring system. A check would be made to verify that the shores can carry the load within their rated capacity. Design of shores is discussed in Chapter 8. A check of bearing stresses on the underside of the stringer will be made in the next step.

Step 6. *Check bearing stresses* where stringers bear on shores and where joists bear on stringers.

Stringer bearing on shore: Assume the head piece of the adjustable steel shore is 11-1/2 x 3-5/8 in. The 4x8 stringer is actually 3-1/2 in. thick.

If the headpiece is placed parallel to the stringer as shown in Fig. C, bearing area is 3-1/2 x 11-1/2 in., or 40.25 in.^{2} Calculating the maximum reaction at the stringer first interior support

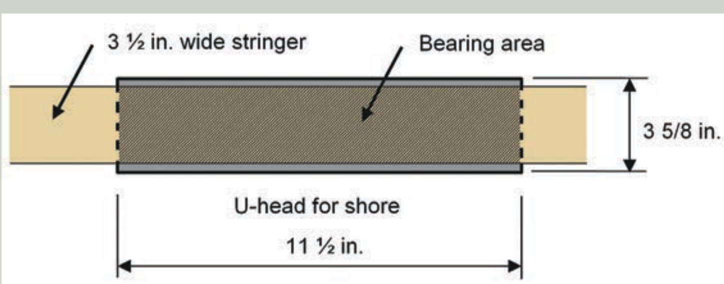

Fig. C: Bearing area of stringer on shore U-head

$$R_b = R_c = 1.1w\ell = 1.1 \times 790 \text{ lb/ft} \times 5 \text{ ft} = 4350 \text{ lb}$$

The length of bearing, ℓ_b, is 11.5 in. and C_b = 1.0 from Table 7.4. Other adjustment factors are either 1.0 or not applicable as before. From Table 4.3, $F_{c\perp}$ = 625 psi and with adjustments listed in Table 4.4

$$F'_{c\perp} = F_{c\perp} C_M C_t C_i C_b = 625 \text{ psi} \times 1.0 = 625 \text{ psi}$$

Actual bearing stress will be

$$f_{c\perp} = \frac{R}{\text{bearing area}} = \frac{4350 \text{ lb}}{40.25 \text{ in.}^2} = 108 \text{ psi}$$

which is well below the ASD adjusted design value for $F_{c\perp}'$ of 625 psi.

Joist bearing on stringer: The two members are 1-1/2 and 3-1/2 in. wide. Contact bearing area is 3-1/2 × 1-1/2 = 5.25 in.^{2}, as illustrated in Fig. D.

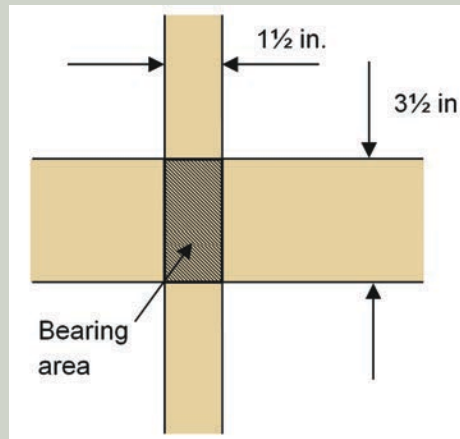

Fig. D: Bearing area between joist and stringer

Average load transmitted by a joist to a stringer is joist spacing × joist span length × form load

$$\frac{19.2 \text{ in.}}{12 \text{ in./ft}} \times \frac{60 \text{ in.}}{12 \text{ in./ft}} \times 158 \frac{\text{lb}}{\text{ft}^2} = 1264 \text{ lb}$$

The corresponding bearing stress is

$$f_{c\perp} = \frac{1264 \text{ lb}}{5.25 \text{ in.}^2} = 241 \text{ psi}$$

Bearing at this point is also low relative to the 625 psi reference design value for $F_{c\perp}$. Because actual bearing stress is so low, there is no need to check for the C_b adjustment for $F'_{c\perp}$ that could result from the short bearing length of 3-1/2 in. on the joist and 1-1/2 in. on the stringer.

Step 7. Refer to Chapter 8 for design of lateral bracing for this type of form.

Example 7.5: Partial Slab Form Design—LRFD

For this example, the design for the span length of the joists for Example 7.4 will be repeated using LRFD. Fresh concrete can be considered fixed material load, C_{FML}, if it is placed without having temporary excess thickness. But it should be considered variable material load, C_{VML}, if temporarily over-deposited in local areas.

From Eq. (5.8), the controlling factored vertical load with $C_{FML} = 0$ and $C_H = 0$ becomes

$q = 1.2C_{DL} + 1.2C_{FML} + 1.4C_{VML} + 1.6C_{PE} + 1.6C_H$
$= 1.2 \times 8\ \text{lb/ft}^2 + 1.4 \times 100\ \text{lb/ft}^2 + 1.6 \times 50\ \text{lb/ft}^2 = 230\ \text{lb/ft}^2$

With the 19.2 in. spacing of the studs based on the plywood maximum span length from the ASD analysis

w (joists) $= q \times s = 230\ \text{lb/ft}^2 \times 19.2\ \text{in./12 in./ft} = 368\ \text{lb/ft} = 30.7\ \text{lb/in.}$

The bending reference design value from Table 4.3 is F_b = 1000 psi. As in ASD Example 7.4, $C_M = 1.0$, $C_t = 1.0$, $C_L = 1.0$, $C_F = 1.0$, $C_{fu} = 1.0$, $C_i = 1.0$, and $C_r = 1.15$. From Table 4.10, $K_F = 2.54$ and $\phi_b = 0.85$. For significant reuse, take $\lambda = 0.8$.

$$F_b' = F_b C_M C_t C_L C_F C_{fu} C_i C_r K_F \phi_b \lambda \quad \text{for LRFD}$$

$$F_b' = 1000\ \text{psi} \times 1.15 \times 2.54 \times 0.85 \times 0.8 = 1986\ \text{psi}$$

From Table 4.3, the reference design value for shear stress, F_v, is 180 psi. Because $C_M = 1.0$, $C_t = 1.0$, $C_i = 1.0$, $K_F = 2.88$, and $\phi_v = 0.75$, the adjusted design values are

$$F_v' = F_v C_M C_t C_i K_F \phi_v \lambda = 180\ \text{psi} \times 2.88 \times 0.75 \times 0.8 = 311\ \text{psi}$$

BENDING CHECK: Determine required size for a continuous beam of three spans with $\ell = 5$ ft. The section modulus is the unknown to be determined. Combining Eq. (7.1b) and (7.2b), rearranging and substituting

$$F_b' S = \frac{w\ell^2}{10}$$

$$S = \frac{w\ell^2}{10F_b'} = \frac{30.7\ \text{lb/in.} \times (60\ \text{in.})^2}{10 \times 1986\ \text{psi}} = 5.56\ \text{in.}^3$$

CHECK SHEAR: For purposes of comparison, calculate using the same two options considered in ASD Example 7.4. Trying the first option (ignore the benefit of the reduced shear force at distance d from the support face)

$$f_v = \frac{0.9w\ell}{bd}$$

$$bd = \frac{0.9w\ell}{F_v'} = \frac{0.9 \times 30.7\ \text{lb/in.} \times 60\ \text{in.}}{311\ \text{psi}} = 5.33\ \text{in.}^2$$

Also trying the second option (estimating ℓ_b based on experience and verifying the stringer is properly sized) with $d = 5.5$ in. and $\ell_b = 3.5$ in.

$$F_v' = \frac{0.9w}{bd}\left[\ell - (1.67d + 0.83\ell_b)\right]$$

$$311\ \text{psi} = \frac{0.9 \times 30.7\ \text{lb/in.}}{bd}\left[60\ \text{in.} - (1.67 \times 5.5\ \text{in.} + 0.83 \times 3.5\ \text{in.})\right]$$

$$bd = \frac{0.9 \times 30.7\ \text{lb/in.}}{311\ \text{psi}}\left[60\ \text{in.} - 12.1\ \text{in.}\right] = 4.25\ \text{in.}^2$$

Deflection of the joists must still be based on actual loads rather than factored loads and must be calculated by ASD as before. Comparing ASD and LRFD results for calculated section properties

	ASD	LRFD	
Bending (S)	6.60 in.^{3}	5.56 in.^{3}	
Shear (bd)	6.33 in.^{2}	5.33 in.^{2}	based on V_{max}
Shear (bd)	5.05 in.^{2}	4.25 in.^{2}	based on V_{des}
Deflection (I)	5.15 in.^{4}	Same as ASD	

In this case, there is some difference between ASD and LRFD results, but it is not sufficient to change the required 2x6 S4S size.

7.8 Beam Form Design

Beam forms, like slab forms, carry a vertical load, and they are also subject to lateral pressure of the fresh concrete just as wall forms are. Furthermore, where slabs frame into beams, part of the load from the slab forms may be carried by the beam form to the supporting shores. Because there are several ways of forming beam and slab intersections, a review of the beam form construction methods in Chapter 11 will be helpful in understanding and applying the principles of design. It is important to note how and how much of the slab load (if any) is to be carried by the beam form, and judging by the details of the form construction (Fig. 7.6), how that load is transmitted to the supporting shores.

Following are the basic steps performed during a beam form design:

1. *Beam bottom:* Determine the load on the beam bottom, following principles outlined in Chapter 5. Dead and live loads are estimated similarly as for slabs, but where the beam is heavily reinforced, some allowance must be made for the extra weight of steel. If the beam side is supported directly on the beam bottom, any additional load transmitted from the slab through the beam side is also included.

After the loading on the beam bottom has been estimated, check for bending, deflection, and shear. The allowable span length of the beam bottom material generally determines the required shore spacing under the beam.

2. *Beam sides:* The beam side is subject to lateral pressure from the freshly placed concrete, and may also carry vertical load from the slab forms framing into it. It is