This is a preview of a new chapter which will be included in the 2022 version (2nd edition) of the textbook.

Chapter 8 – Common Temporary Structures in Construction

8.1 Introduction

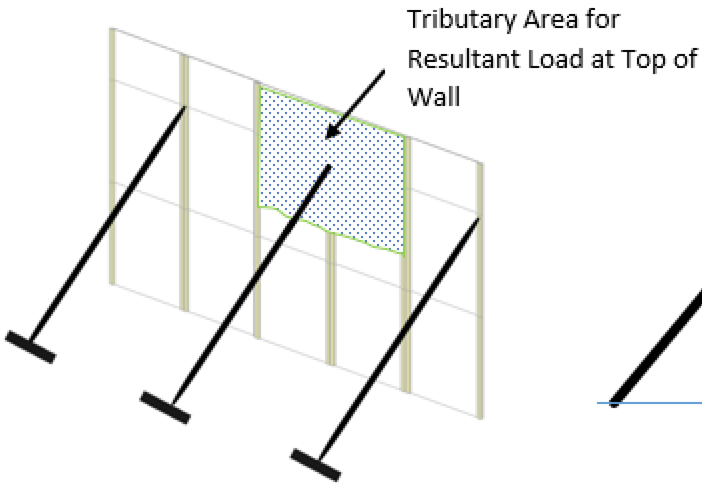
As previously mentioned, the construction process is unique in that it has loads and load combinations that are specific to the structure in its unfinished state. These loads and load combinations must be considered when acting upon temporary elements used in the building process or when acting upon the proposed structure in its unfinished state. Consideration of such loads during construction is important so that temporary elements can be properly designed. This chapter considers some common temporary structures used in construction.

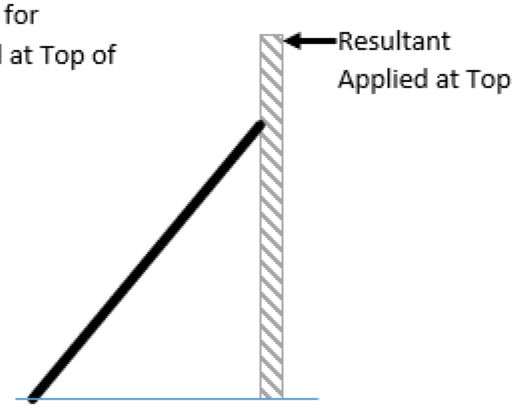
8.2 Bracing of Concrete Wall Formwork

The topic of bracing concrete wall formwork has previously been raised in the discussion of load paths and equilibrium earlier in this text. Typically, when concrete wall formwork is used the actual form ties will "connect" the two sides of the wall forms together and be responsible for resisting the lateral pressure of the concrete as it is being placed. ASCE 37 provides a series of equations to calculate this lateral pressure based on unit weight, rate of placement, and temperature of the concrete. The bracing that is used on wall formwork is typically there to resist wind pressure or other lateral working loads as the formwork is being erected. Strong winds have been known to blow down formwork that is in the process of being erected and formwork walls that are improperly braced can have serious safety consequences.
Bracing may be placed on both sides of the wall form, but if it is placed on one side then it has to be designed to resist both tension and compression. The American Concrete Institute's *Guide to Formwork for Concrete* (ACI 347) states the following:

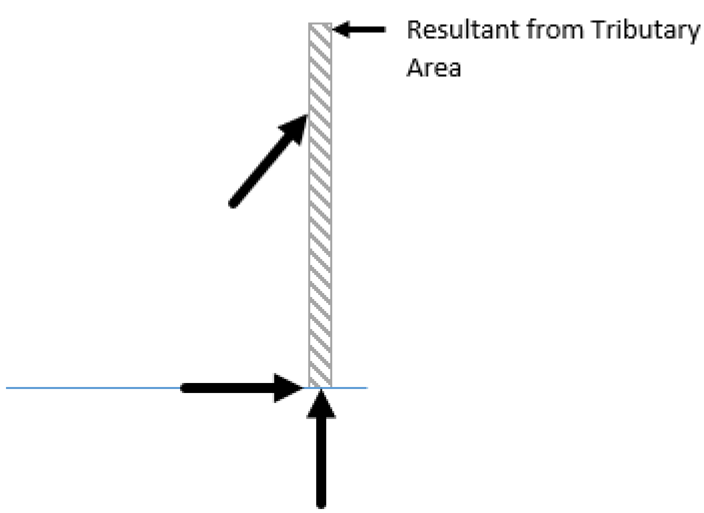
> "Wall form bracing should be designed to meet the minimum wind load requirements of the local building code…. For wall forms exposed to the elements, the minimum wind design load should be not less than 15 psf. Bracing for wall forms should be designed for a horizontal load of at least 100 lb./linear foot of wall length, applied at the top".

Typically, it is assumed that one-half of the lateral wind pressure will be distributed to the top of the wall form and the other one-half to the bottom of the wall form where the attachment to the footing will transfer the load. The load at the top of the wall form is assumed to be resisted by the brace. The first step is to determine the amount of lateral wind load which will ultimately be resisted by one brace. This load path distribution to one brace is shown as follows:





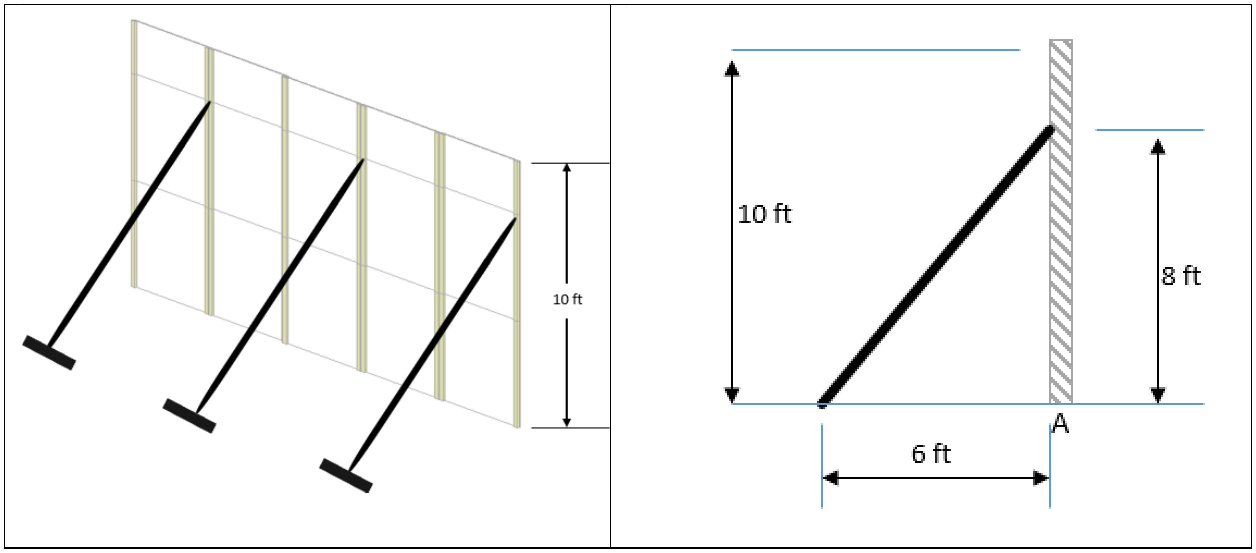
The determination of the load that must be carried by one brace follows the principles of equilibrium which were discussed earlier in this text. In this case the wall form acts as a beam (oriented vertically) and it can be assumed that the brace acts as a "link support" and the bottom of the wall form is pinned. So, the free body diagram would look like this:



The brace acts as a two-force member with the "reaction" force acting in the direction of the brace. Therefore, the x-component and y-component of the brace force are required to be in the same proportion as the geometry of the brace. By summing moments about the bottom of the wall form you can determine the x-component of the brace's force and based on the geometry of the brace you can determine the brace force. The following example shows this calculation.

Example 1

Calculate the force carried by a brace in the concrete wall form system shown below. Consider the wall to be 10 foot tall and the braces to be spaced at 6 feet on center. The brace is designed to resist a wind load of 20 psf.



It is typically assumed that one-half the wind load gets distributed to the top and the other one-half is distributed to the bottom of the form. The area of wall form that distributes to the top is:

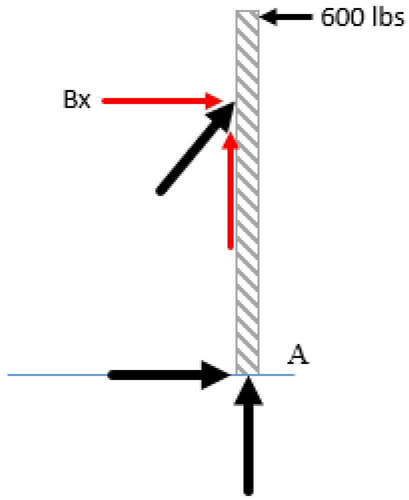
Area = (6 ft) (10/2 ft) = 30 ft^2

The load is then:

Load = 30 ft^2(20 psf) = 600 lb

Recall that ACI 347 requires that "Bracing for wall forms should be designed for a horizontal load of at least 100 lb./linear foot of wall length, applied at the top". In this case, since the braces are 6 feet apart, the minimum load per ACI 347 would be 100 lb/ft*6 feet = 600 lb per foot which is the same as the load calculated using the tributary area. Note that if the ACI minimum was greater than the calculated lateral load, you would use the ACI value.

The free-body diagram is as follows:



Summing Moments about point A will solve for the horizontal bracing component (i.e., the horizontal red arrow called Bx) as such:

$\sum M_A = 0$: $\quad - (600 \text{ lb}) (10 \text{ ft}) + (Bx) (8 \text{ ft}) = 0$ and solving Bx = 750 lb

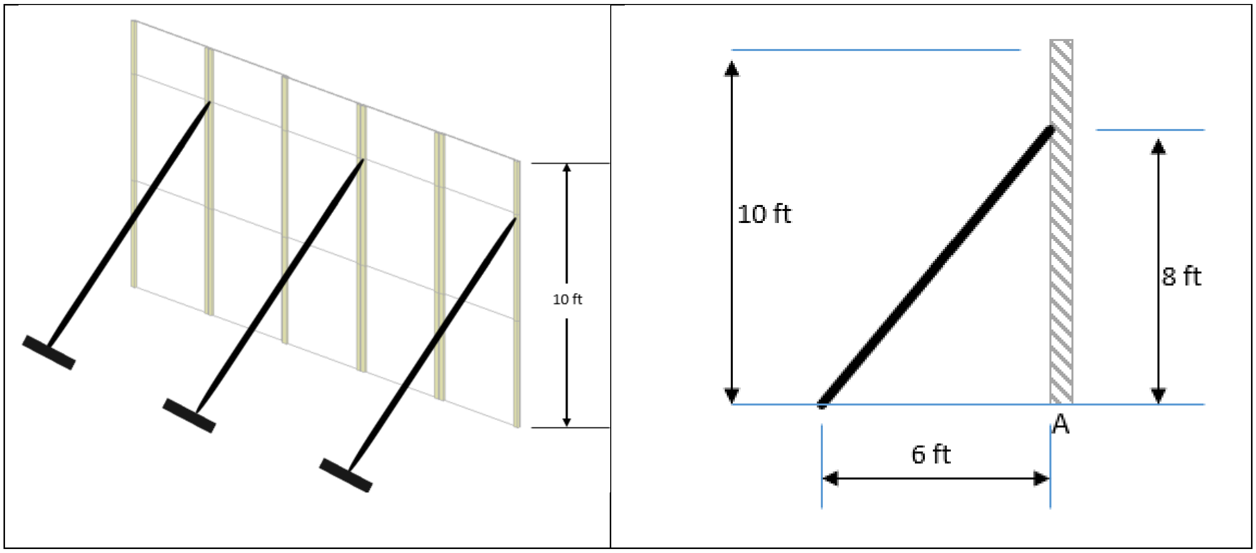
Since the geometry of the brace shows us that X = 6 ft and Y = 8 ft, by the Pythagorean theorem the length of the brace is 10 ft. Therefore if:

Bx = 750 lb then the force in the brace must be (10/6) (750 lb) = 1250 lbs.

Therefore, the brace would need to be chosen to resist at least 1250 lbs.

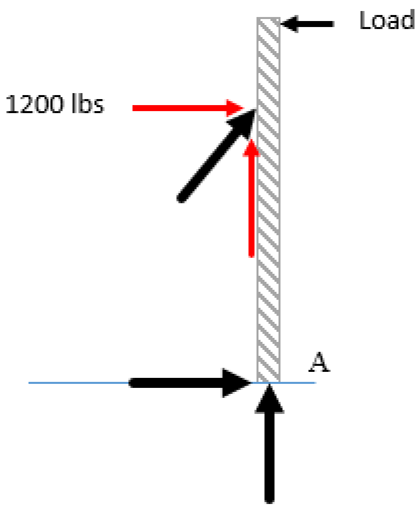
Example 2

Using the same wall form as in Example 1, calculate the maximum allowable spacing of the braces if a brace has an allowable load capacity of 2000 lb (i.e., the brace can safely carry 2000 lbs.) Consider the wall to be 10 foot tall and the brace is designed to resist a wind load of 20 psf.



In this case, if the brace can carry 2000 lb, then based on its geometry the x-component could carry (6/10) (2000 lb) = 1200 lb.

This would make the free-body diagram as follows:



Summing Moments about point A will solve for the horizontal bracing component (i.e., the horizontal red arrow called Bx) as such:

$\sum M_A = 0$: - (Load) (10 ft) + (1200 lb) (8 ft) = 0 and solving Load = 960 lb

Since that load of 960 pounds is a resultant of the wind pressure of 20 psf acting over an area which is (5 feet tall) *(spacing), then the spacing allowed can be solved as follows:

960 lb = (20 psf) (5 ft) (spacing)

Therefore, the maximum allowable spacing is 9.6 feet.

8.3 Shoring

The design of shoring and shoring systems are other important elements involved in concrete construction because they support formwork and construction loads. The design of shoring systems is typically done by a professional engineer and improper design, erection, or removal can lead to damaged concrete and safety issues. The coverage herein is meant to only minimally illustrate shoring concepts as actual design is handled by professionals.

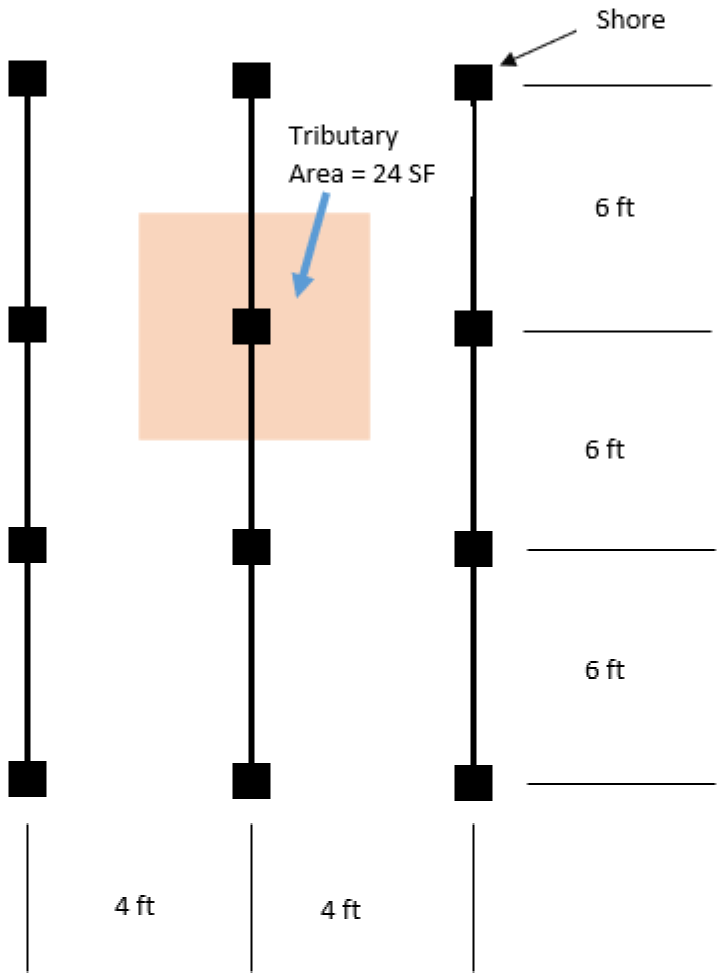
The typical loads supported by shoring include the weight of formwork, the weight of plastic concrete, equipment and other miscellaneous construction loads. Shoring systems may involve horizontal shoring and beams but our discussion here will be limited to vertical elements which carry vertical loads.

The American Concrete Institute's *Guide to Formwork for Concrete* (ACI 347) provides guidance on loads and states the following:

> The formwork should be designed for a live load of not less than 50 psf of horizontal projection. When motorized carts are used, the live load should not be less than 75 psf… The design load for combined dead and live loads should not be less than 100 psf or 125 psf if motorized carts are used.

As previously mentioned, a shore is a vertical member that supports loads such as the formwork, fresh concrete and construction loads above it. Many times, shoring will be a timber member, usually a 4 x 4 or a 6 x 6 that can have a metal fitting that helps attach it to the beams or stringers above. Others times shoring maybe adjustable steel-tube like members which are manufactured supports sometimes referred to as jack shores. All of these must be selected so that they can safely support the compressive loads that they are carrying. In timber design, the bearing capacity, or bearing perpendicular to the grain of the wood will also usually will also need to be checked to ensure the beam type members do not crush.

The load path to the vertical shoring starts with the principles of tributary area which was first discussed in Chapter 1 Section 3 which essentially considers the area of influence that a member has as it occupies in the structure. In essence, this is the "area" that a particular member is responsible for supporting which is generally thought to have a length and width that goes "halfway" to the next similar member. This is simplistically shown in the following illustration.



Therefore, the load carried by an individual shoring member in this illustration would be 24 square feet (i.e., its tributary area) multiplied by the load (formwork, concrete, construction, etc.) from above. The formula is simply:

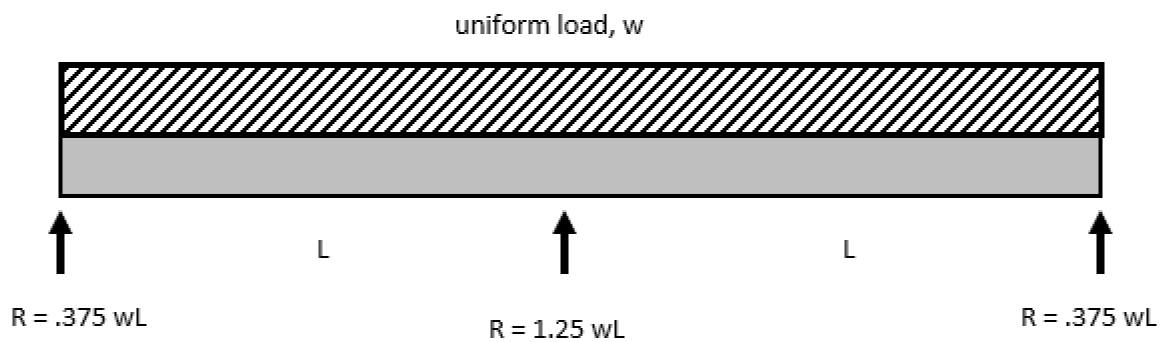
Load = Area x Floor Load

Were

Area = Tributary Area in ft^2

Floor Load = Loading in psf

It should be noted here that is a simplified method of establishing loading on a vertical shoring element and that, depending on the actual construction of the system, this load might be increased based on a "continuity factor". Typically, shores are placed under "beam-type" members (joists or stringers) that transfer the actual load into the shore. The simplified method outlined above considers these joists or stringers to be "simply supported" by the shores, which means is the individual "beam" is assumed only to span from shore to shore. In reality, these joists or stringers that the shore supports might be continuous over the interior shore and this would cause the interior shore to support more of a reaction. In a beam that is continuous over two-spans, the reactions from a uniformly loaded continuous beam would be as follows:



In this condition, the reaction of 1.25wL in the middle would mean the shore at the interior support would theoretically support 25% more load than the "simply supported" case. For simplicity, the discussion herein will not use any continuity factors but the construction management student should be aware of what they are and why they might be used in shoring system design.

If wooden shores are used the load capacity of the shoring in compression may follow the principles outlined in Chapter 7 Sections 12 through 14. It is worth remembering that the piece of lumber will be compressed parallel to the grain (which is in the longitudinal direction) and in wood this direction has much higher compressive strength than that which is "perpendicular to the grain".

In calculating the compressive strength of a piece of lumber, the effective length of the column, l_e, is found by multiplying the true unbraced length by the appropriate effective length factor, K_e. This can be expressed as follows:

$$l_e = K_e(L)$$

Where

L = the actual unbraced length of the wood column

K_e = effective length factor

However, shores are typically assumed to be “pinned” at both the top and bottom so the effective length, l_e, will just be the actual length of the shoring element. As mentioned previously in an unbraced wooden compression member having a rectangular cross-section the slenderness ratio can be expressed as follows:

$$\frac{l_e}{d}$$

Where

l_e = the effective length of the column

d = smallest dimension of the cross-section

In wood shoring the l_e / d ratio should be less than 50. The student will remember from chapter 7 that many times we follow the allowable stress methodology is simply stated as follows:

$$\text{Actual Stress} \leq \text{Allowable Stress}$$

For wood shoring, the allowable design compressive stress is calculated from a tabular value of compressive stress for a particular species which is parallel to the grain. This tabular value is typically denoted as Fc and can then be adjusted by applicable modifiers as needed. In fundamental terms, the design compressive stress is calculated as follows:

$$F_c^* = F_c \text{ x modifiers (except for } C_P)$$

$$F_c' = F_c^* \text{ x } C_P$$

Where

F_c = tabular allowable compressive stress value (parallel to the grain)

C_P = Column Stability Factor (see eq. 7-13 in Chapter 7)

You can review all modifiers in Chapter 7. For purposes of discussing temporary structure in this chapter we will consider all modifiers as 1.0 except for the load duration factor, C_D, which many times is taken as 1.25 for construction loads. We will also need to be calculating the column stability factor, C_P. The following example will demonstrate how the allowable load on a shore can be calculated using allowable stress principles.

Example 3

Determine the largest compressive load ($P_{allowable}$) that can be supported by a 4 x 4 shore which is 10 feet tall. A duration factor of 1.25 will be used. The column post is unbraced and made of Hem-Fir No. 2 and exists under normal moisture, temperature, and other conditions (all other modifiers = 1.0 but consider the stability factor). Assume the column to have an effective length factor of 1.0 and use ASD principles.

Use the following:

Area = 12.25 in^2
Width = Depth = 3.5 in
F_c = 575 psi
Modulus of Elasticity (E) = 1,100,000 psi
E_{min} = 400,000 psi

$F_c^* = F_c\,(C_D)\,(C_M)\,(C_F)\,(C_t)\,(C_i)$

F_c^* = 575 psi (1.25) (1.0) (1.0) (1.0) (1.0) = 719 psi

$l_e / d = 120/3.5 = 34.2 < 50$

$$F_{cE} = \frac{.822\ (400{,}000\ psi)}{\left(\frac{120\ in.}{3.5\ in.}\right)^2} = 280\ psi$$

$$\propto_c = \frac{F_{cE}}{F_c^*} = \frac{280\ psi}{719\ psi} = 0.389$$

$$C_P = \left(\frac{1 + .389}{2(.8)}\right) - \sqrt{\left(\frac{1 + .389}{2(.8)}\right)^2 - \frac{.389}{.8}}$$

$$C_P = 0.35$$

Fc' = Fc* (C_P) = 719 psi (.35) = 252 psi

$P_{allowable}$ = 252 psi (12.25 in^2) = 3,087 pounds

Therefore, the 4 x 4 could carry 3,087 lbs.

Wood shoring design would also consider bearing capacity on the area of the beam that rested on the actual shore itself. This is considered to be compression perpendicular to the grain. Another aspect of shoring design that may affect the capacity of the shore in compression is if the shore has lateral bracing. Any lateral bracing reduces the effective length and therefore could cause the capacity to increase. Finally, ASCE 37 provides guidance on continuous shoring and the removal of shoring. Section 4.7.3 states the following:

> "Where shores are required to support the load of newly placed concrete, these shores shall be maintained until the concrete has gained enough strength to support applicable dead and construction loads. Where shoring is continuous over several floors, the calculated loads on these shores shall be cumulative unless and until the shores have been released and reset to allow the slab in question to carry its own dead and construction loads. Such release shall not occur until the concrete is capable of carrying its own dead load."

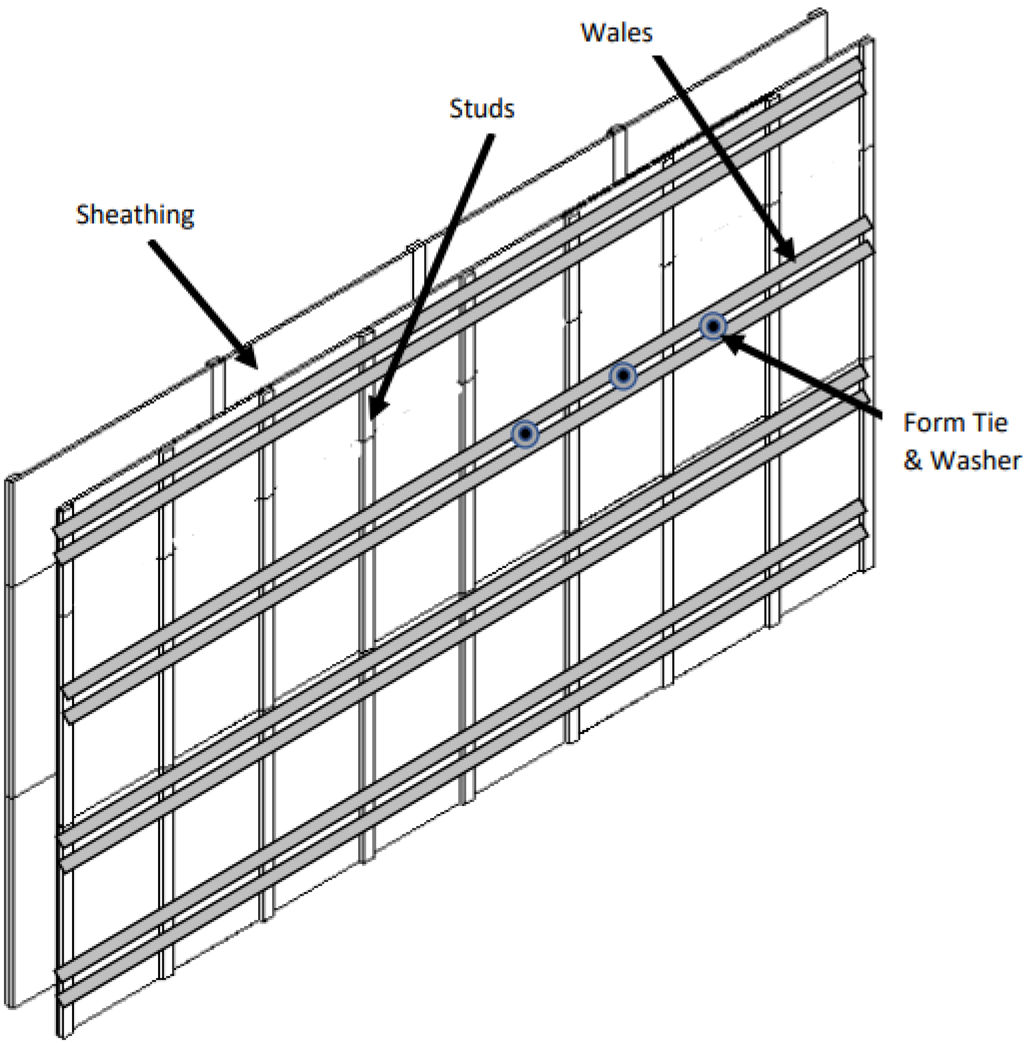
8.4 Wall Forms

Another common temporary structure seen on many construction projects is the wall form used in concrete wall construction. Although wall forms can be made from various materials, the discussion in the following pages will focus on wooden wall forms which are typically constructed on-site. The design of the various components of the wall form will follow the allowable stress design philosophy which has been previously introduced. The basic premise of that philosophy is actual stress in a member (or component) must be less than or equal to the allowable stress. The basic components of a wall form are as follows:

1. Sheathing – the component directly supported by the studs, and against which concrete is to be placed. This might be considered the "inside" of the wall form which comes into contact with the concrete. Although sheathing could be made out of various materials, for purposes in this text we will consider it to be plywood.
2. Studs – the vertical components that are directly attached to the sheathing. Again, for purposes of this text, the studs will be considered to be made from dimensional lumber.
3. Wales – the horizontal components that are directly support the studs. Wales are generally composed of two members with the form tie hardware inserted between. Again, for purposes of this text, the wales will be considered to be made from dimensional lumber.

4. Form ties – the components which “ties” one side of the form to the other. These ties are placed in tension and because of this are generally made from steel (which is good at resisting tension). Form ties have load capacities set by the manufacturer. Although form ties and associated hardware are an integral component of the wall form system, for the purposes of this text will only be mentioned.

The following illustration show the various components of a wall form system.



Previously we discussed wall forms that were subjected to wind loading prior to concrete placement. Now we are interested in understanding wall form performance with concrete placed inside the form. When concrete is placed into the wall forms it produced a lateral pressure that can be quite high. In order for the finished walls to be of high quality, the formwork has to be stiff enough and strong enough so that the shape and position of the wall falls within acceptable tolerances. Wall forms that deflect excessively, or worse experience a structural failure, will cause costly rework and delays. ASCE 37 provides formulas for the lateral pressure produced by wet concrete. These are as follows:

For concrete with a slump of 7 inches or less, placed with normal internal vibration to a depth of 4 ft or less, lateral form pressure (C_C) can be calculated as follows for walls with a rate of placement of less than 7 feet per hour and a placement height not exceeding 14 feet:

$$C_C = F_C\, F_W\, (150 + 9000\ R/T)$$

Where

F_C = Chemistry Factor from table below
F_W = Unit Weight Factor from table below
R = Rate of placement (feet per hour)
T = Temperature of concrete in the form (°F)

For walls where the placement height exceeds 14 feet (with a rate of placement of less than 7 feet per hour) and for all walls with a placement rate of 7 to 15 feet per hour, the lateral form pressure (C_C) can be calculated as follows:

$$C_C = F_C\, F_W\, (150 + 43{,}400/T + 2{,}800\ R/T)$$

F_c Table

Cement type of blend	**F_c**
Type I, II, III w/o retarders	1.0
Type I, II, III with a retarder	1.2
Other types containing less than 70% slag of 40% fly ash w/o retarders	1.2
Other types containing less than 70% slag or 40% fly ash with retarders	1.4
Blends with more than 70% slag or 40% fly ash	1.4

F_W Table

Unit Weight of Concrete	**F_W**
Less than 140 pcf	0.5[1 + (w/145 pcf)] But not less than 0.8
140 – 150 pcf	1.0

More than 150 pcf	w / 145 pcf

ASCE 37 also specifies a minimum form pressure as $600F_W$ (psf) as well. The maximum form pressure is

$$C_C = wh$$

Where

C_C = lateral pressure in pounds per square foot (psf)
w = unit weight of the fresh concrete in pounds per cubic foot (pcf)
h = depth of the wet concrete from top of placement to point of consideration (feet)

The design of wooden wall forms generally starts with the sheathing, followed by the studs, and finally ending with the wales (again this section will not include wall tie design). All of these components are considered to act as "beam-type" members that span between supports. The general load-carrying nature of the system is that the sheathing spans between the vertical studs and the vertical studs are supported by the wales. Therefore, the spacing of the studs is the span length of the sheathing and the spacing of the wales is the span length of the studs. Similarly, the span length of the wales is the spacing of the ties. The behavior of all these components is governed by the bending stress equation (although checking for shear capacity and deflection is always wise):

$f_b = M / S$

where

M = applied moment to the component

S = the elastic section modulus of the component (remember this can be found in Appendix C for components made from dimensional lumber)

If designing the size of the component, remember the bending stress equation can be rearranged to solve for the required elastic section modulus if the allowable bending stress is known. This could be expressed as follows:

$S_{required} = M / F_b$

where

M = applied moment to the component

F_b = the allowable bending stress of the component

Because the length of these components (sheathing, studs, and wales) is large compared to typical spacing of supports these components are not considered simply supported as that would be a very conservative assumption. Instead, they are considered to be continuous over three spans which makes the bending moment produced by a three-span continuous beam under uniform load $\mathbf{wl^2/10}$.

The following example will illustrate the basics of how wall forms might be evaluated for adequacy.

Example 4

Determine the adequacy of a wooden wall form that is 10 feet tall. The properties of the sheathing, studs and wales are given as follows. The concrete will be placed at a rate of 4 feet per hour and the temperature will be assumed as 80°. The concrete will be placed with normal internal vibration to a depth of 4 ft and Type I cement is used without retarders. The concrete is assumed to have a unit weight of 145 pcf. The adjustment modifiers used for dimensional lumber will only consider duration factor for simplicity. Vertical studs are spaced every 12 inches and wales are spaced at 24 inches on center. Sheathing is ¾" thick Class I Plyform (see APA hyperlink for properties).

First, calculate the maximum lateral load from the ASCE 37 formula. Using Type I cement the F_C coefficient is 1.0 and with a unit weight of 145 pcf the F_W coefficient is 1.0 also. The formula for lateral pressure is then:

$$C_C = F_C\, F_W(150 + 9000\ R/T)$$

$$C_C = (1.0)(1.0)(150 + 9000\ (4)/(80)) = 600\text{ psf}$$

This actual lateral pressure increases with the depth of the concrete from the top of the wall to this maximum value. For simplicity's sake the forms are typically designed for the calculated uniform pressure of 600 psf.

Sheathing:

The American Plywood Association (APA) publishes a APA Construction/Design Guide Concrete Formwork which contains various properties of the plywood we will be using herein. If the student is interested, they can download this guide by registering at the following site https://www.apawood.org/concrete-form-panels

The sheathing to be used in this problem is 3/4-inch-thick Class I plyform which has an effective section modulus of 0.455 in^3 per foot of width (stresses applied parallel to face grain). This type of sheathing has a stated allowable bending stress value of 1930 psi. This sheathing is supported by vertical studs that are spaced every 12 inches on center and (as discussed previously) this is considered to be continuous sheet of sheathing over three spans. The actual bending stress of the sheathing is calculated as:

$f_b = M / S$

where

$M = wl^2/10 = 600 \text{ psf } (1.0 \text{ ft})^2/10 = 60 \text{ lb-ft}$ (note the 12-inch span was changed to 1.0 feet)

So,

$f_b = M / S = 60 \text{ lb-ft } (12 \text{ in/ft}) / .455 \text{ in}^3/\text{ft} = 1582 \text{ psi}$

Since the actual bending stress of 1582 psi is less than the allowable stress of 1930 psi – the sheathing is okay in bending.

Deflection of the sheathing based on bending can also be checked, usually the allowable deflection is given in terms of a fraction of the span length. It is typical to have the actual deflection of the sheathing not to exceed 1/360 of the span length. The actual span length in this example is 12" (i,e, the vertical stud spacing) so the allowable deflection limit would be:

1/360(12 in.) = .033 in.

The actual deflection based on a three-span continuous beam that is uniformly loaded is:

$\Delta = 0.0069wl^4/EI$

For the sheathing we are using:

E = 1,650,000 psi (found in the previous APA design guide)

$I = .199 \text{ in}^4$ (found in the previous APA design guide)

w = 600 plf /12 = 50 lb per in. (based on a 1-foot strip of sheathing)

Therefore, the actual deflection is:

$\Delta = 0.0069wl^4/EI = \Delta = 0.0069(50 \text{ lb/in})(12 \text{ in.})^4/(1{,}650{,}000 \text{ psi})(.199 \text{ in}^4) = .022 \text{ in.}$ which is less than the allowable deflection of .033 inch so the deflection based on bending is okay.

Shear stresses in the plyform sheathing can also be checking using allowable stress principles. This type of sheathing has a stated allowable bending stress value of 72 psi and the actual shear stress is based on the following formula:

$f_v = VQ / Ib$

For a three-span continuous beam that is uniformly loaded is:

$V = .60wl – w(1.5 \text{ in})$ (note: shear calculation uses a shear occurring at the "clear span" between vertical studs)

$V = .60(50 \text{ lb/in})(12) – (50 \text{ lb/in})(.75 \text{ in}) = 322 \text{ lb}$

For sheathing the APA guide provides a rolling shear constant for plywood which is Ib/Q. For the type of plyform we are using:

$Ib/Q = 7.187\ in^2$ (found in the APA design guide)

Therefore:

$f_v = VQ / Ib = (322\ lb) / (7.187\ in^2) = 45$ psi which is less than the allowable of 72 psi, so shear stress is okay as well.

Vertical Studs:

Checking the vertical studs follows the allowable stress principles also, however the major difference is that the studs are dimensional lumber. The sectional properties of dimensional lumber as well as the reference design values for certain species (and grades) of lumber can be found in Appendix C. The vertical studs used in this problem will be 2 x 4s made from Douglas Fir Larch No. 2. This type of lumber has a stated reference value for bending stress value of 900 psi which we will increase by a load duration factor of 1.25 to 1125 psi. We will assume all other potential modifiers to be 1.0. Each stud is supported by the wales that are spaced every 24 inches on center (2 ft.) which means that they span 24 inches. Also, we will consider the studs continuous over three spans. Remember that there are studs every 12 inches (1 ft.) so the lateral pressure of 600 psf converts to a load of 600 plf. The actual bending stress of the sheathing is calculated as:

$f_b = M / S$

where

$M = wl^2/10 = 600\ plf\ (2.0\ ft)^2/10 = 240$ lb-ft (note the 24-inch span was changed to 2.0 feet)

So,

$f_b = M / S = 240\ lb\text{-}ft\ (12\ in/ft) / 3.06\ in^3 = 941$ psi

Since the actual bending stress of 941 psi is less than the allowable stress of 1125 psi – the vertical studs are okay in bending.

Deflection of the studs based on bending can also be checked, usually the allowable deflection is given in terms of a fraction of the span length. We will use an allowable deflection of 1/360 of the span length. The actual span length in this example is 24" (i,e, the spacing of the wales) so the allowable deflection limit would be:

1/360(24 in.) = .066 in.

The actual deflection based on a three-span continuous beam that is uniformly loaded is:

$\Delta = 0.0069wl^4/EI$

For the studs we are using:

E = 1,600,000 psi (found in Appendix C, assumed no modifiers are applied)

$I = 5.359\ in^4$ (found in Appendix C)

w = 600 plf /12 = 50 lb per in.

Therefore, the actual deflection is:

$\Delta = 0.0069wl^4/EI = \Delta = 0.0069(50 \text{ lb/in})(24 \text{ in.})^4/(1{,}600{,}000 \text{ psi})(5.359 \text{ in}^4) = .013$ in. which is less than the allowable so the deflection and this checks out to be okay.

Shear stresses in the studs can also be checking using allowable stress principles. This type of lumber has a reference design shear stress value of 180 psi which we will increase by 1.25 (for the duration factor) to 225 psi (assuming all other modifiers are = 1.0). The actual shear stress is based on the following formula:

$f_v = 3\ V / 2\ A$

Maximum shear force (for a three-span continuous beam that is uniformly loaded) is normally taken at a distance "d" from the face of support. In this case "d" is the depth of the 2 x 4 which is 3 ½ inches:

$V = .60wl - w(3.5 \text{ in} + .75 \text{ in})$

$V = .60(50 \text{ lb/in})(24) - (50 \text{ lb/in})(4.25 \text{ in}) = 508$ lb

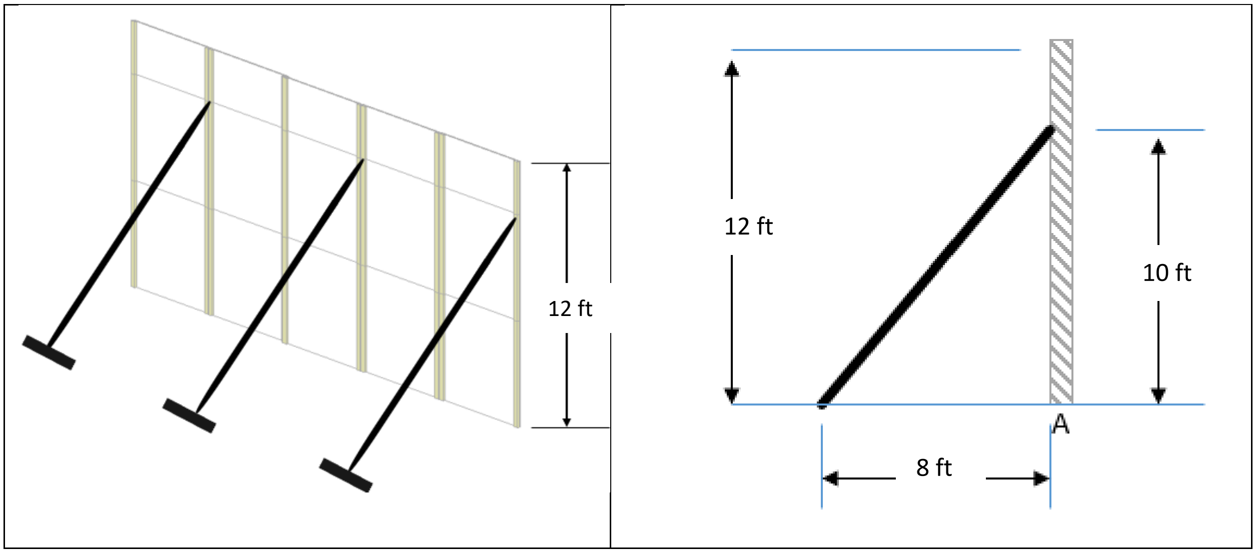
A 2 x 4 has an area of 5.25 in^2 and therefore:

$f_v = 3\ V / 2\ A = 3\ (508 \text{ lb}) / 2\ (5.25 \text{ in}^2) = 145$ psi which is less than the allowable so shear stress checks out as well.

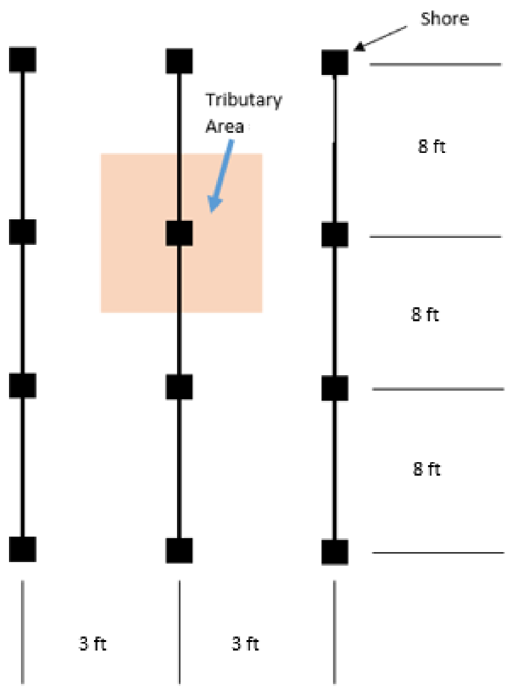
The check of the wales would follow the exact same process as seen with the studs, the only difference is the wales are generally "double" members (for instance two 2 x 4s might be used) and therefore parameters like area and section modulus will be doubled).

Homework Exercises

1. Calculate the force carried by a brace in the concrete wall form system shown below. Consider the wall form to be 12 foot tall and the braces to be spaced at 6 feet on center. The brace is designed to resist a wind load of 20 psf and remember ½ the load is applied at the top of the wall.



2. Redo the previous problem is the braces are spaced at 8 feet on center.
3. Determine the tributary area of the single shore shown in the following layout.



4. Determine the largest compressive load ($P_{allowable}$) that can be supported by a 4 x 4 shore which is 12 feet tall. A duration factor of 1.25 will be used. The column post is unbraced and made of Hem-Fir No. 2 and exists under normal moisture, temperature, and other conditions (all other modifiers = 1.0 but consider the stability factor). Assume the column to have an effective length factor of 1.0 and use ASD principles.
5. Determine the adequacy of the sheathing in a wooden wall form that is 12 feet tall. The properties of the sheathing, studs and wales are given as follows. The concrete will be placed at a rate of 4 feet per hour and the temperature will be assumed as 80°. The concrete will be placed with normal internal vibration to a depth of 4 ft and Type I cement is used without retarders. The concrete is assumed to have a unit weight of 145 pcf. The adjustment modifiers used for dimensional lumber will only consider duration factor for simplicity. Vertical studs are spaced every 12 inches and wales are spaced at 24 inches on center. Sheathing is ¾" thick Class I Plyform (see APA document for properties).
6. From the previous problem, check the adequacy of the vertical studs if the stud spacing is now 15 inches. The sectional properties of dimensional lumber as well as the reference design values for certain species (and grades) of lumber can be found in Appendix C. The vertical studs used in this problem will be 2 x 4s made from Douglas Fir Larch No. 2.