University of Washington

DEPARTMENT OF Construction Management

CM 420
TEMPORARY STRUCTURES

Winter Quarter 2007

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Formwork for Concrete

UNIVERSITY OF
WASHINGTON

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Department of Construction Management
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CM 420 - Temporary Structures

Lesson 1:

Introduction to Concrete Formwork and Vertical Formwork Design

Overview

The first lesson provides an overview on the basic structural wood design as it applies to concrete formwork. This lesson covers materials, methods and techniques associated with concrete formwork design and construction for walls (slab formwork design will be covered in lesson 2). This lesson intends to provide enough information to be able to design horizontal forms, which will be covered in step-by-step fashion.

Lesson Objectives

By the end of this lesson you will be able to:

* recognize the importance of temporary structures and their relationship to permanent structures;

* describe basic properties of wood and plywood;

* explain design considerations for concrete formwork;

* recognize the causes of failure in concrete formwork and plan to avoid them;

* identify formwork components, materials, and accessories;

* calculate loads on concrete formwork; and

* design wall forms.

Reading Assignment

Background reading: M.K. Hurd, Chapters 1 through 4.

Essential reading: M.K. Hurd, Chapter 4: 4-1 to 4-13 and 4-32 to 4-33, Chapter 5, and Chapter 6: 6-1 to 6-16.

NOTATIONS (used in Lessons 1 and 2)

A = Area of cross section, in.^{2}

ACI = American Concrete Institute

b = width of beam cross section, in.

C = allowable timber stress, psi, in compression || to grain

Δ = deflection, in.

Δ_{max} = maximum deflection, in.

E = modulus of elasticity, psi

f = allowable stress in extreme fiber in bending, psi

h = depth of beam cross section, in.

I = moment of inertia, in.^{4} ($I = bd^3/12$ for rectangular beam)

Ib/Q = rolling shear constant

L = span, ft.

l = span or length, in.

l/d = ratio of unsupported length to least dimension in compression member (shore)

M_{max} = Maximum induced bending moment, ft-lb or in.-lb as indicated

OSHA = Occupational Safety and Health Administration

S = section modulus, in.^{3}, ($S = bd^2/6$ for rectangular beam)

s = spacing of member, in.

v = average shearing stress, or average horizontal shearing stress, psi

V = maximum vertical shear, lb (same as end reaction for simple beam)

w = uniformly distributed load, lb per lineal ft

W = total uniformly distributed load, lb ($= wL$)

Introduction

CM 420 will deal with the materials, methods and techniques associated with temporary structures utilized in various construction operations, such as:

- concrete formwork construction;

- scaffolding;

- falsework/shoring;

- cofferdams;

- underpinning;

- diaphram/slurry walls;

- earth-retaining structures; and

- construction dewatering.

Temporary structures are critical elements of the overall construction plan. A temporary structure in construction affects the safety of the workers on the job and the general public and there is also the relationship of the temporary structure to the finished structure. Temporary structures are sometimes incorporated into the finished work or are removed at the end of the conclusion of their usefulness. In either case the contractor will have to deal with supervision work, code requirements, contract and legal requirements, and perhaps disputes with others over the work being performed. As far as design, drawings and specifications are concerned, they depend on the temporary structure under consideration. In extremely complex jobs involving such temporary work as cofferdams for bridge piers, the design of the temporary structure will often be done by the designer of the permanent structure. For simpler types of temporary structures, such as temporary ramps used by excavation contractor for a building projects, the excavation contractor will do the design. Between these two extremes is the type of temporary structure in which specialty contractors, who make a business of doing a specific type of temporary structure will be employed. The specifications for the temporary structure are usually drawn up by the temporary structure contractor and is required to obtain permits for any work done.

A major emphasis will be placed on concrete formwork construction covering detailed design analysis of both vertical and horizontal timber formwork systems.

Temporary Structures

Definition:

Any means or methods which provide temporary support, access, enhancement, or otherwise facilitate the construction of permanent structures.

Necessity:

Temporary structures form the interface between design and construction. Most permanent structures simply could not be built without temporary structures.

Impact on Schedule, Cost, and Quality

Losses in time and money will occur if the temporary structures are not planned and coordinated with the same degree of thoroughness as the permanent structures.

Safety

Failure of temporary structures have been responsible for hundreds of deaths on construction sites. Safety should be the overriding priority of contractors and designers responsible for implementing temporary structures.

Responsibility

The norm in the construction industry is to place the responsibility for temporary structures solely on the general contractor. However, architects and engineers must at least have formulated their own method of construction. Coordinating the design of permanent structure with the temporary structures that will be required can lead to more efficient and cost effective construction.

Design Considerations

Safety

Designers must place the first priority on safety. OSHA codes, as well as other codes in the industry, provide stringent performance specifications (how the system should work) regarding temporary structures.

Cost

Temporary structures can be the most expensive part of some construction projects. Designing cost-effective solutions to temporary structures problems could easily be the competitive advantage a contractor has over others. The designer must have a thorough knowledge of all the options which will sufficiently solve the temporary structures problem.

Unique Design Challenges

Temporary structures are subject to unique loading conditions which do not apply to a permanent structure (fluctuating or dynamic loads, impact loads, and loads which change position). Working within spatial constraints and cramped sites requires the most efficient temporary structure so that workers still have room to maneuver safely.

It is always possible that an unforeseen condition could arise during an excavation due to uncertainty of soil conditions. Designers must include an appropriate factor of safety in their calculations or they may consider contingency plans for changing soil conditions.

The contractor

In many cases the contractor is the only member of the construction team with considerable experience and practical knowledge of temporary structures. The contractor must hire his or her own engineer, if the specifications or building codes require one, or self perform the design of temporary structures. The most complex temporary structures are often handled on a design-build basis (design-build approach is a construction technique which allows a single procurement for the design and construction of projects.) The design-build situation is optimal because it guarantees coordination between design and construction.

Anyone managing the construction process needs a basic understanding of the engineer's thinking process and the design intentions and the basic understanding of how a structure behaves. Constructor must be able to address a number of technical questions at the project site including structural issues that sometimes are not addressed by the design professionals. Since the safety of construction workers as

well as the strength and stability of structures during the construction phase is of paramount importance, construction managers need this knowledge.

Structural Design

Definition: Determination of overall proportions and dimensions of the supporting framework and the selection of individual members.

Responsibility: The structural engineer is responsible for structural design within the constraints imposed by the architect (number of stories, floor plan, etc.).

Important factors in design are:

- **safety** (the structure doesn't fall down);
- **serviceability** (how well the structure performs in term of appearance and deflection);
- **economy** (an efficient use of materials and labor); and

Several alternative designs should be prepared and their costs compared.

Types of load that structures support are:

- **dead loads** – permanent; including self-weight, floor covering, suspended ceiling, partitions, etc.
- **live loads** – not permanent; the location is not fixed; including furniture, equipment, and occupants of buildings
- **wind load** (exerts a pressure or suction on the exterior of a building);
- **earthquake loads** (the effects of ground motion are simulated by a system of horizontal forces);
- **snow load** (varies with geographical location and drift);
- **other loads** (hydrostatic pressure, soil pressure)
- If the load is applied suddenly, the effects of IMPACT must be accounted for.

Design specifications provide guidance for the design of structural members and their connections. They have no legal standing on their own, but they can easily be adopted, by reference, as part of a building code. i.e., ACI 318-99- Building Code Requirements for Structural Concrete. The Specifications for Design of Wood Members are by National Design Specifications for Wood Construction by American Forest and Paper Association.

Formwork for Concrete

Formwork development has paralleled the growth of concrete construction throughout the 20th century. The increasing acceptance of concrete as a major construction material presents the form builder a new range of problems in the development of appropriate sheathing materials and maintenance of rigid tolerances. Figure 1 shows a typical concrete wall formwork setup.

Wood forms

Wire snap ties

Walers
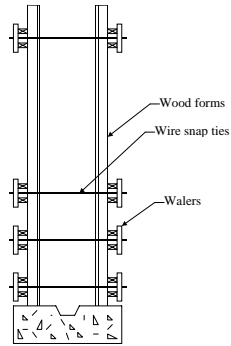
Figure 1 - Typical formwork setup for a concrete wall

Formwork is a classic temporary structure in the sense that it is erected quickly, highly loaded for a few hours during the concrete placement, and within a few days disassembled for future reuse. Also classic in their temporary nature are the connections, braces, tie anchorages, and adjustment devices which forms need.

For concrete formworks, the notion of "*Temporary Structures*" does not quite portray the reality. Forms, its hardware and accessories are used over and over again over their life time. Because of that it is necessary to use materials with high durability and easy to maintain. The form design should be such that it can be erected and disassembled efficiently in order to maximize productivity. The disassembly or stripping of forms depends on factors such as the bond between concrete and the form, rigidity and shrinkage of concrete. Forms should, whenever possible, be left in place for the entire curing period. Since early form removal is desirable for their reuse, a reliable basis for determining the earliest possible stripping time is necessary. Some of the early signs to look for during stripping are no excessive deflection or distortion and no evidence of cracking or other damage to the concrete due to the removal of the forms or the form supports. In any event, forms must not be stripped until the concrete has hardened enough to hold its own weight and any other weight it may be carrying. The surface must be hard enough to remain undamaged and unmarked when reasonable care is used in stripping the forms.

Traditionally, formwork was erected in place and wrecked after only one time of usage. In the United States, due to high labor costs, it is more efficient and profitable to prefabricate forms, assemble them in large units using mechanical devices, such as cranes to erect the forms and reuse them as much as possible.

Lumber was once the predominant form material, but developments in the use of plywood, metal, plastics, and other materials, together with the increasing use of specialized accessories, have changed the picture. In 1908 the use of wood versus steel formwork was debated at the American Concrete Institute (ACI) convention, the advantages of modular panel formed with its own connecting hardware and good for extensive reuse were also realized. By 1910 steel forms for paving were being produced commercially and used in the field (Figure 2).

Figure 2 - A 1909 construction scene shows the first application of steel forms for street paving (From M.K. Hurd)

Today modular panel forming is the norm. Figure 3 shows steel forms being used for concrete pavement construction.

Figure 3 - Steel modular forms being used in concrete pavement construction

Objectives of Form Building

Forms mold the concrete to desired size and shape and control its position and alignment. But formwork is more than a mold; it is a temporary structure that supports its own weight, plus the freshly placed concrete, plus construction live loads (including materials, equipment, and personnel).

Basic objectives in form building are:

1. Quality −   In terms of strength, rigidity, position, and dimensions of the forms

2. Safety −   for both the workers and the concrete structure

3. Economy − the least cost consistent with quality and safety requirements

Cooperation and coordination between engineer/architect and the contractor are necessary to achieve these goals.

Economy is a major concern since formwork costs constitute up to 60 percent of the total cost of concrete work in a project (Figure 4).
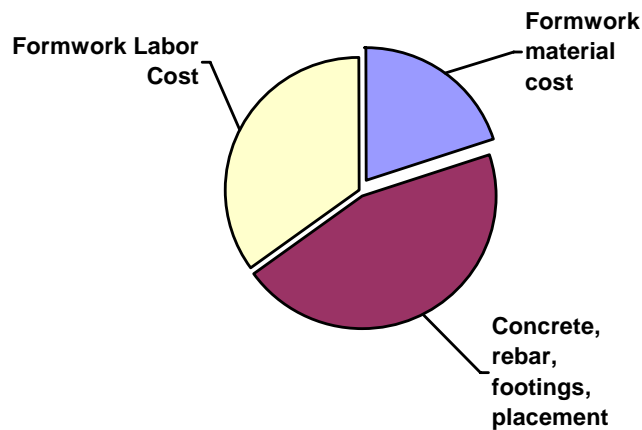
Figure 4 - Pie chart of cost components in a typical concrete construction

How Formwork Affects Concrete Quality

In designing and building formwork, the contractor should aim for maximum economy without sacrificing quality or safety. Size, shape, and alignment of slabs, beams, and other concrete structural elements depend on accurate construction of the forms.

The forms must be:

- Sufficiently rigid under the construction loads to maintain the designed shape of the concrete,

- Stable and strong enough to maintain large members in alignment, and

- Substantially constructed to withstand handling and reuse without losing their dimensional integrity.

The formwork must remain in place until the concrete is strong enough to carry its own weight, or the finished structure may be damaged.

Causes of Formwork Failure

Formwork failures are the cause of many accidents and building failures that occur during concrete construction, usually when fresh concrete is being placed. Generally some unexpected event causes one member to fail, then others become overloaded or misaligned and the entire formwork structure collapses. The main causes of formwork failure are:

1. improper stripping and shore removal

2. inadequate bracing

3. vibration

4. unstable soil under mudsills (A plank, frame, or small footing on the ground used as a base for a shore or post in formwork), shoring not plumb

5. inadequate control of concrete placement

6. lack of attention to formwork details.

Please read the case studies presented in M.K. Hurd in Chapter 2.

Safety

Formwork must be:

- strong to carry the full load and side pressure from freshly placed concrete, together with construction traffic and equipment, and

- sound (made of good quality, durable materials)

To ensure that forms are correctly designed and strong enough for the expected load OSHA (Occupational Safety and Health Administration) regulations, American Concrete Institute (ACI) recommendations, and local code requirements for formwork should be followed.

Planning for Formwork

The contractor should plan for formwork at the time of making bid considering the following factors:

- placing schedule and stripping time requirements;

- capacity of equipment available to handle form sections and materials;

- capacity of mixing and placing equipment;

- construction joints;

- reuse of forms as affected by stripping time;

- relative merits of job-built, shop-built and ready-made forms; and

- weather (protection requirements and stripping time)

Compare alternative methods to determine the most efficient plan.

Areas of Cost Reduction

1. Planning for maximum reuse − A form designed for max reuse is stronger and more expensive, but it can save on the total form cost.

2. Economical form construction

   - use shop-built-forms− provides greatest efficiency in working conditions and in the purchase and use of materials and tools;

   - create shop area on the site− to form sections too large or transportation cost too high;

   - use job-built− for small jobs, or where forms must be fitted to terrain;

   - buy prefabricated forms(large number of reuses)

   - rent prefab forms(better flexibility in regulating volume of work)

3. Setting and stripping

   - repeat the same functions to increase the crew efficiency as the job progresses

   - use metal clamp or special wedge pin connections that are secure, yet easy to assemble and dismantle; and

   - add extra features that make handling, erection, and stripping easier such as handles, lifting eyes.

4. Cranes and Hoists

   - Size of form sections should be limited to the capacity of the largest crane planned for the job.

   - Stair towers may be completed early in the schedule to be used for moving men and materials.

   - Leave one bay open to permit mobile crane and concrete truck movement.

5. Bar Setting

   - Form design can permit the rebar to be pre assembled before installation (more favorable condition)

6. Concrete Placement

   - High lifts in wall construction make placing and vibration difficult.

   - Placing rate is limited by form design.

Form Materials and Accessories

Practically all formwork jobs require some lumber. A local supplier will advise what material and sizes are in stock or promptly obtainable, and the designer or builder can proceed accordingly. Southern yellow pine and Douglas fir, sometimes called Oregon pine are widely used in structural concrete forms. They are easily worked and are the strongest in the softwood group. Both hold nails well and are durable. They are used in sheathing, studs, and wales. Figure 5 shows a typical wall form with its components.
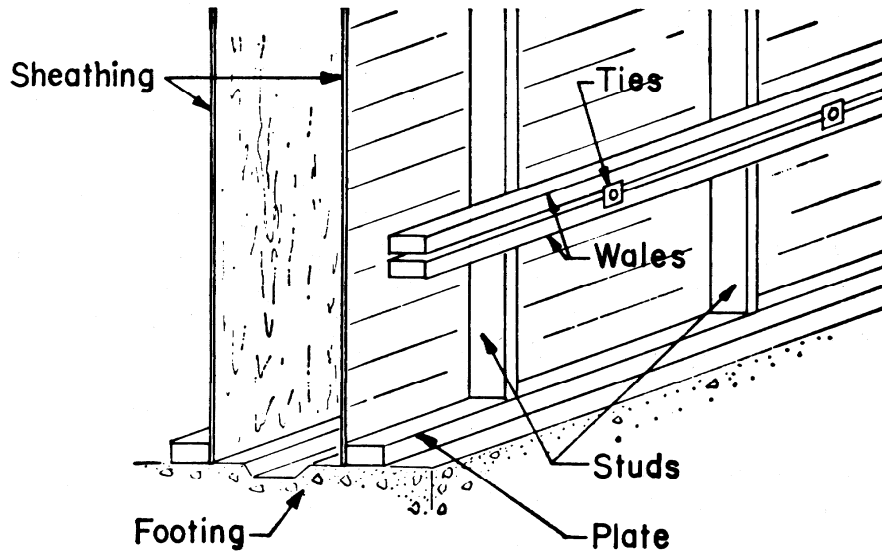
Figure 5 - Typical wall form with components identified. Plywood sheathing is more common than board sheathing material

Figure 6 shows parts of a typical wall form–
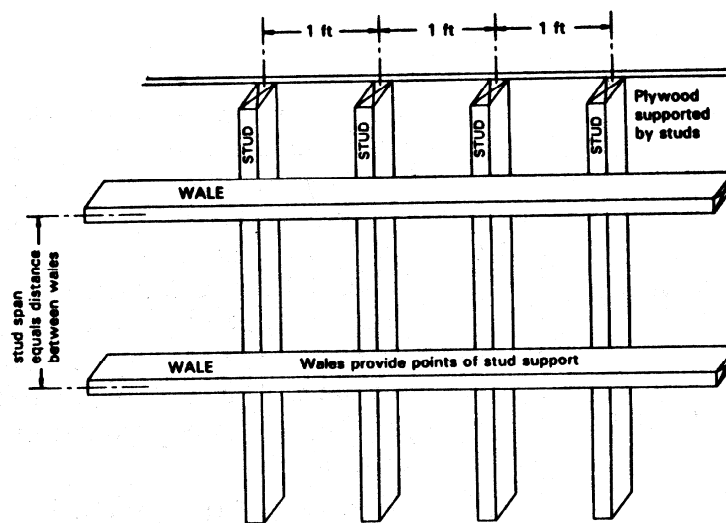
**Figure 6** - Parts of a typical wall form

Ties

▪ In order to secure concrete forms against the lateral pressure of unhardened concrete, a tensile unit called concrete form tie is used (they are also referred to as form clamps, coil ties, rod clamps, snap ties, etc.). They are ready-made units with safe load ratings ranging from 1000 lb to more than 50000 lb and have an internal tension unit and an external holding device. Figure 7 shows a typical single member tie.
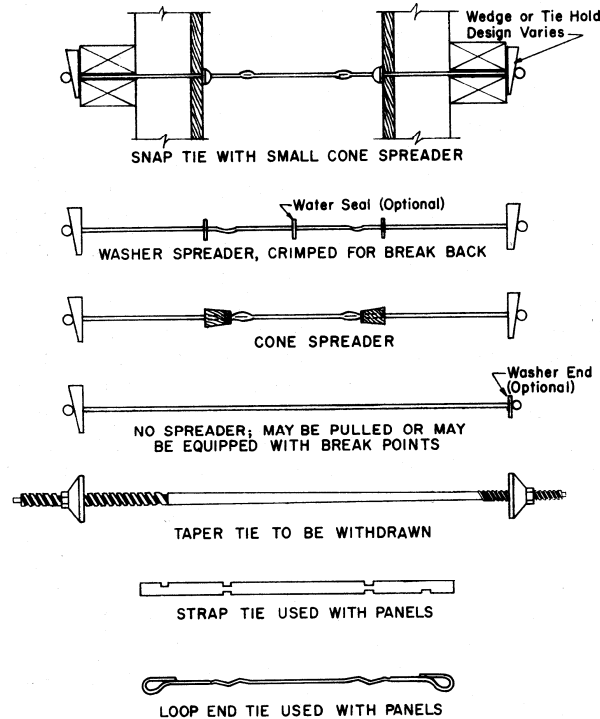
Figure 7 – A typical single member ties (from M.K. Hurd)

Ties are manufactured in two basic types:

▪ Continuous single member ties; in which the tensile unit is a single piece, have a special holding device added for engaging the tensile unit against the exterior of the form (Figure 8). Some single member ties may be pulled as an entire unit from the concrete; others are broken back a predetermined distance. Some are cut flush with the concrete surface. It is generally used for lighter loads, ranging up to about 5,000 lb safe load
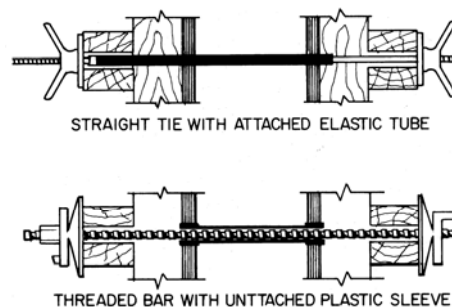
Figure 8 – A continuous single member tie (working loads from 4500 to 50,000 lbs)

▪ Internal disconnecting type ties, in which the tensile unit has an inner part with threaded connections to removable external members generally remain in the concrete (Figure 9). It is available for light or medium loads, but finds its greatest application under heavier construction loads up to about 70,000 lb.
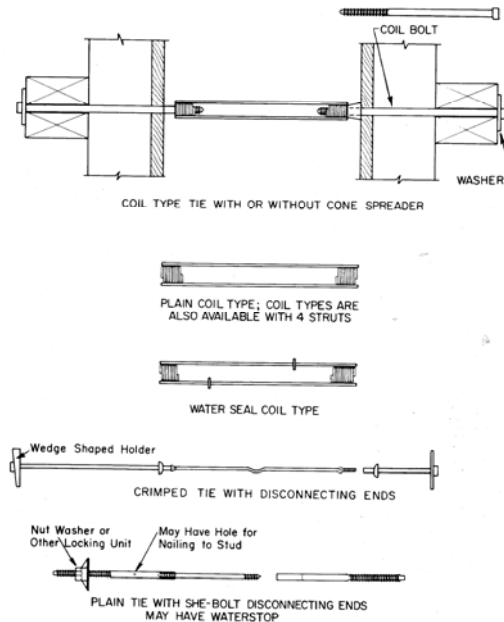

Figure 9 - An internal disconnecting ties

Lumber Finish and Sizes

"Dressed" lumber is referred to a lumber which has been surfaced in a planing machine to achieve surface smoothness and uniformity of size. The lumber may be surfaced on one side (*S1S*), one edge (*S1E*), two sides (*S2S*), two edges (*S2E*), or combination of sides and edges (*S1S1E*, *S1S2E*, *S2S1E*) or on all four sides (*S4S*).

Dressed lumber is generally used for formwork, because it is easier to handle and work, but rough sawn boards and timbers may be used in bracing and shoring, or as a form surfacing material to secure a special texture effect in the finished concrete. Minimum sizes of both rough and dressed lumber are specified by the American Softwood Lumber Standards, PS 20-70.

Lumber is commonly referred to by its nominal size (Figure 10). Minimum sizes for green lumber are selected so that as moisture is lost, it becomes the same size as dry lumber.
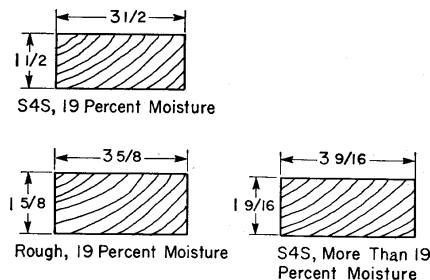

Figure 10 - Specified actual size of a 2×4 for different moisture contents and finishes

Table 4-1B on page 4-4 of the text shows actual dimensions and cross section properties of American Standard lumber at 19 percent moisture content. <u>Actual</u>, not nominal, sizes must always be used for design. The values in Table 4-1B can be safely used with either dry or green lumber. Design for formwork are based on the allowable or working stresses. Allowable stress depends on many factors, including the species of wood and its grade, cross section, moisture content, and load duration.

Adjustment for Load Duration

For form work materials with limited reuse, ACI Committee 347 (http://aci-int.org/committees/CommitteeHome.asp?Committee_Code=0000347-00)   permits design using allowable stresses for temporary structures or for temporary loads on permanent structures. In case of lumber, this is interpreted to mean the 25 percent working stress increase (adjustment factor of 1.25) shown in Table 4-2 (page 4-6, text) for 7 days or less duration of load.

Adjustment Factors for Size and Flat Use

Size Factor – except for Southern Pine, the No. 1 and No. 2 lumber frequently used for formwork is subject to stress adjustment based on member size (use Table 4-2B).

Flat use factor – when dimension lumber 2 to 4 inches thick is loaded on the wide face, the base value of bending stress can be multiplied by adjustment factors shown in Table 4-2B (page 4-7, text).

Horizontal Shear Adjustment

The shear stress factor can be applied to the base design value to increase the allowable horizontal shear stress when the length of splits or size of shakes and checks is known, as shown in Table 6-3 (page 6-9, text).

Designers may estimate an appropriate adjustment factor when they have general knowledge of the lumber quality available. Conservative practice would suggest use of the factor 1.00 whenever there is absolutely no information on splits, checks and shakes.

Engineered Wood Products

Plywood

Plywood is widely used for job built forms and prefabricated form panel systems. Plywood is a flat panel made of a number of thin sheets of wood. A single sheet in the panel may be referred to as a ply, or layer. A layer may consist of a single ply or it may be two or more plies laminated together with their grain direction parallel. Plywood is pieces of wood made of three or more layers of veneer joined with glue, and usually laid with the grain of adjoining plies at right angles. Almost always an odd number of plies are used to provide balanced construction. Some of the major structural uses for plywood are:

- roof, floor, and wall sheathing;
- horizontal and vertical diaphragms (shearwall);
- structural components (Laminated veneer Lumber);
- gusset plates; and
- concrete formwork.

A thin sheet of wood obtained from the peeler log is called veneer or ply. The cross-laminated pieces of wood in a plywood panel are known as layers. A layer is often an individual ply, but it can consist of more than one ply. The face and back plies have the grain running parallel to the 8-ft dimension of the panel. (Note: wood is stronger parallel to the grain than perpendicular to the grain).

Typical plywood sheathing applications use the plywood continuously over two or more spans. The required thickness of the plywood is determined by sheathing load and joist spacings. The standard size of plywood is 4×8 ft. Varying moisture conditions change dimensions. Therefore, edge and end spacing of 1/8 inch is recommended in installation of plywood.

When the sheathing is nailed onto green supporting beams, the nails "pop" upward through the sheathing as the lumber supports dry. This can cause problems with finish flooring (the nails should be set below the surface of the sheathing and the nail holes should not be filled).

Table 4-3 (page 4-10, text) shows the effective section properties for plywood.

Plywood at the bottom − face grain parallel to span − is used the strong way. In Figure 11, with face grain perpendicular to the span direction, the specimen at the top is used the weak way.

Plywood Used The Weak Way

Plywood Used The Strong Way
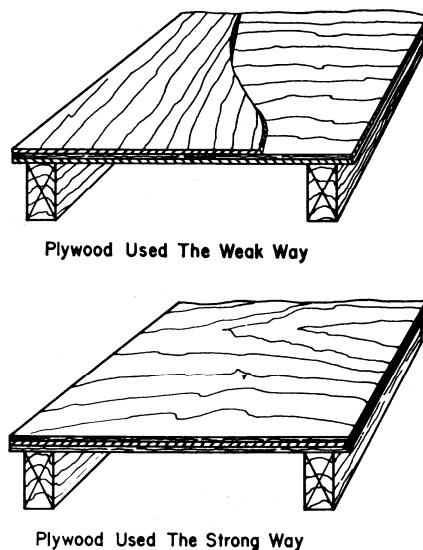
Figure 11 – Plywood used in weak and strong way

Bearing or Crushing

Bearing Stresses (Compression Perpendicular to the Grain)

A. Allowable stresses for compression perpendicular to the grain are available from tables providing wood properties for various species and grades of lumber.

These allowable stresses may be modified (increased) if both of the following criteria are satisfied:

1. Bearing is applied 3 inches or more from the end of the member being stressed.

2. Bearing length is less than 6 inches.

When criteria are met, the allowable stresses are modified by the following factor:

$$\frac{l + 0.375}{l}$$

Where l is the bearing strength in inches measured along the grain of the wood. For round washers, assume l is equal to the diameter of washer.

B. To check for a bearing failure, such as crushing of wood fibers, divide the imposed load by the area of contact and compare this determined actual bearing stress to the allowable bearing stress. If the actual bearing stress exceeds the allowable bearing stress, a failure results.

The multiplying factors for indicated lengths of bearing on such small area plates and washers are shown in the table below:

| Length of bearing, inches	1/2	1	1 1/2	2	3	4	6 or more
Factor….	1.79	1.37	1.25	1.19	1.13	1.09	1.00

EXAMPLE

Design of Column/Braces

Wood members are subjected to axial compression (compression parallel to the grain). The capacity of a wood column is dependent on the following properties:

1. Cross-sectional area.
2. Slenderness Ratio.
3. Allowable compressive stress parallel to the grain (the basic allowable stress depends on the wood species and grade. This allowable stress may be modified depending on the slenderness ratio).

Buckling is the major mode of failure in wood columns. The slenderness ratio is the ratio of the unsupported length (l) of a member to the width (d) of the face of the member under consideration. Two values of the slenderness ratio (l/d) must be calculated for wood members used in construction because buckling can occur about either axis of the cross-section. If a column is unbraced, the controlling slenderness ratio (the larger one) will be the one determined by using the dimension of the narrower face. For wood members, l/d cannot exceed 50. The following examples will illustrate the calculation of the slenderness ratios:

1. Unbraced Column:
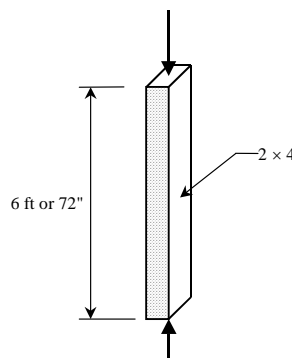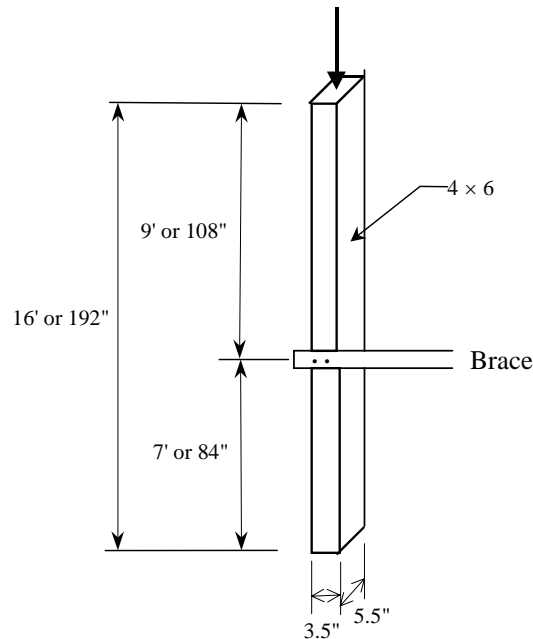
a. Slenderness ratio parallel to narrow face:

$$\frac{l}{d} = \frac{72}{1.5} = 48.0$$

b. Slenderness ratio parallel to wide face:

$$\frac{l}{d} = \frac{72}{3.5} = 20.57$$

The larger, thus controlling, slenderness ratio is 48. Since the column is unbraced in both dimensions, it is intuitive that the slenderness ratio on the narrow face would control. Note that if the column were unbraced and 7 foot long, the controlling slenderness ratio would be 56 (over 50) and column would not be permitted without modification (larger member section or additional lateral bracing).

2. Braced Column:

a. Slenderness ratio parallel to narrow face:

$$\frac{l}{d} = \frac{108}{3.5} = 30.86$$

Note: Use longest unbraced length - 9 feet

b. Slenderness ratio parallel to wide face:

$$\frac{l}{d} = \frac{192}{5.5} = 34.91$$

The controlling slenderness ratio is 34.91.

Lateral Pressure of Fresh Concrete

Loads imposed by fresh concrete against wall or column forms differ from the gravity load on a horizontal slab form. The freshly placed concrete behaves temporarily like a fluid, producing a hydrostatic pressure that acts laterally on the vertical forms. This lateral pressure is comparable to full liquid head when concrete is placed full height within the period required for its initial set.

With slower rate of placing, concrete at the bottom of the form begins to harden and lateral pressure is reduced to less than full fluid pressure by the time concreting is completed in the upper parts of the form. The effective lateral pressure −a modified hydrostatic pressure− has been found to be influenced by the weight, rate of placement, temperature of concrete mix, use of retardant admixtures, and vibration.

Factors affecting lateral pressure on forms are:

- Weight of concrete
- Rate of placing (the average rate of rise in the form)
- Vibration
- Temperature (affecting the set time)
- Other variables
  - Consistency of concrete
  - Ambient temperature
  - Amount and location of reinforcement
  - Maximum aggregate size (MSA)
  - Cement type, etc.

Table 5-4 (page 5-12, text) shows the maximum pressure to be used for design of wall forms with placement rates up to 10 feet per hour.

Form Design

When the material for formwork has been chosen, and the anticipated loading estimated, a form should be designed strong enough to carry the anticipated loads safely, and stiff enough to hold its shape under full load. At the same time the builder or contractor wants to keep costs down by not overbuilding the form.

> Before formwork design can properly begin, a thorough evaluation should be made concerning the variables surrounding the design. Some of these include the following:
>
> A. Review the Job Conditions
> 1. Type of placement procedure (trucks, crane, buggy, concrete pump)
> 2. Expected rate of placement
> 3. Limitations of batch plant or ready-mix supplier
> 4. Expected temperature of concrete at the time of placement (review job schedule for time of placement)
> 5. Deflection tolerances permitted
> 6. Access to placement area

B. Review Knowledge Level of the Workers on the Job
    1. Degree of compliance with design assumptions to be expected
    2. Efficiency with reuses – skilled workers may efficiently reuse forms while an unskilled workforce may be more efficient with gang forms

C. Establish which Materials will be Utilized in the Form
    1. Plywood (sizes and allowable stresses) in inventory or readily available
    2. Lumber (sizes and allowable stresses) in inventory or readily available
    3. Hardware available (the hardware may dictate the lumber selected for the form)

Design Loads
Concrete Formwork

Vertical Loads

Design load for formwork are dead load plus live load per square foot of form contact area. The dead load is defined as the weight of the reinforced concrete plus the weight of the formwork. The live load is defined as additional loads imposed during the process of construction such as material storage, personnel and equipment. Formwork impact load is a resulting load from dumping of concrete or the starting and stopping of construction equipment on the formwork. An impact load may be several times a design load.

- Dead Load

  - Concrete and Rebar (130-160 lb/cu. ft.)
  - Embedments
  - Formworks (at least 10 lb/sf, ANSI [American National Standards Institute] A 10.9 – 1983)

- Live Load – Minimum 50 lb/sf (75 lb/sf if carts are used)

  - Personel
  - Equipment
  - Mounting of Concrete
  - Impacts

- Combined Dead and Live Load – Minimum 100 lb/sf (125 lb/sf if carts are used)

Lateral Loads

- Slab forms: The greater of:

        100 lb/lineal foot of slab edge

            or

        2% of total dead load

- Wall forms: The greater of:

15 lb/sf

or

Local Code requirements for wind load

or

100 lb/linear foot of wall (for $> 8'$ tall wall)

THE WALL FORM DESIGN

Lateral Pressures on Wall Forms

A.   The pressure imposed by concrete on a wall form is a function of the following primary factors:

1. Density of concrete

2. Temperature (T) of the concrete at the time of placing (degree Fahrenheit)

3. Rate (R) of concrete placing (feet of height per hour)

4. Height (h) of concrete placement (in feet)

B.   The pressure (P) that concrete will impose on a wall form is determined as follows:

1. If the placement rate (R) does not exceed 7 feet per hour, the pressure (P – measured in psf) is the <u>least</u> of the following; yet never less than 600 psf:

    a. $P = 150 + \dfrac{9000R}{T}$

    b. $P = 150h$

    c. $P = 2000$ psf

2. If the placement rate (R) is from 7 to 10 feet per hour, the least of the following values apply; yet never less than 600 psf:

    a. $P = 150 + \dfrac{43,400}{T} + \dfrac{2800R}{T}$

    b. $P = 150h$

    c. $P = 2000$ psf

3. If the placement rate exceeds 10 feet per hour, assume the lateral pressure is equal to 150h.

C.   The wall form pressure calculations apply only if additional assumptions are satisfied.

1. Concrete density is 150 pcf.

2. Concrete is vibrated at the time of placement and not more than 4 feet below the top of the concrete surface.

3. Concrete slump does not exceed 4 inches.

4. Concrete is made of Type I cement and contains no pozzolans or admixtures.

5. Concrete temperature is in the range of 40 to 90 degrees F.

Wall Design – Pressure Determined

Given or assumed values: density of Concrete = 150 pcf
Height of Wall (h) = 12'-8"
Rate of Placement (R) = 5 ft/hr
Concrete Temperature (T) = 80°F

Determine maximum wall pressure (Per ACI 347)

Since $R < 7$ ft/hr, the maximum pressure is the least of the following:

$$P = 150 + \frac{9000R}{T} = 150 + \frac{9000(5)}{80} = 712.5 \text{ psf}$$

$$P = 150h = 150\,(12.67') = 1900 \text{ psf}$$

$$P = 2000 \text{ psf}$$

Using the smallest value: Use $P = 712.5$ psf
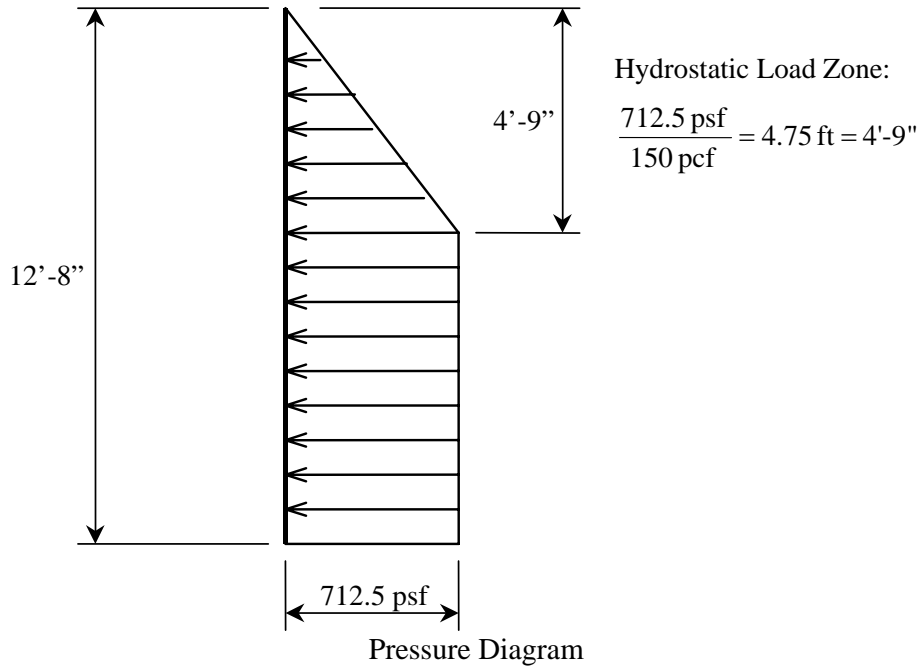
Verify with Table 5-4
P = 712 psf

TABLE 5-4: MAXIMUM LATERAL PRESSURE FOR DESIGN OF WALL FORMS

Applies only for normal weight concrete made with Type I cement, no admixtures or pozzolans, slump no more than 4 in., and vibration depth limited to 4 ft or less.

Rate of placement, R, ft per hr	p, maximum lateral pressure, psf, for temperature indicated					
	90F	80F	70F	60F	50F	40F
1	600 psf minimum governs					
2						
3					690	825
4			664	750	870	1050
5	650	712	793	900	1050	1275
6	750	825	921	1050	1230	1500
7	850	938	1050	1200	1410	1725
8	881	973	1090	1246	1466	1795
9	912	1008	1130	1293	1522	1865
10	943	1043	1170	1340	1578	1935

NOTE: Do not use design pressures in excess of 150 x height of fresh concrete in forms.

Diagram of Lateral Pressure on Wall Form

Hydrostatic Load Zone:

$$\frac{712.5 \text{ psf}}{150 \text{ pcf}} = 4.75 \text{ ft} = 4\text{'-}9\text{''}$$

4'-9"

12'-8"

712.5 psf

Pressure Diagram

Formwork Design

The formwork design aims at designing a form that is strong enough to handle the calculated loads safely and stiff enough to maintain its shape under full load. When designing concrete formwork, the following design simplifications and assumptions are made:

1. All loads are assumed to be uniformly distributed.

2. Beams that are supported over three or more spans are considered to be continuous;

3. The design values for simple spans can safely be used for beams that are supported over two spans;

4. When determining size of main form members, the strength of nailed connections is neglected.

Typical Design Formulas

Typically, the components of formwork are sheathing, studs, joists, wales, stringers, shores, and tie rods. Sheathing retains the concrete and is supported by studs in vertical forms and joists in horizontal forms. Studs are supported by wales and joists by stringers. The wales are held in place by tension members such as tie rods and stringers are supported by shores or posts. Other than tie rods and shores, the other components of the formwork structurally behave like beams, whether being horizontal or vertical. Beam formulas are used to analyze the formwork components. Below the formulas for bending, deflection and shear are introduced. From these formulas the quantities of l, the safe span is calculated. In formwork design, the smallest value of l calculated for each category of bending, deflection and shear is used as the safe span that satisfies all conditions.
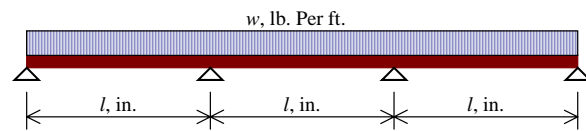
Bending:

The maximum bending stress, maximum deflection and shear for beams supported over two spans and three or more spans are shown below:
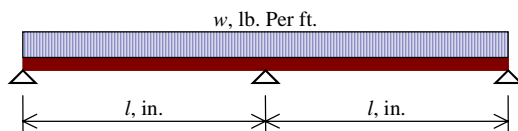
$$M_{Max} = \frac{wl^2}{96} \text{ in. - lb}$$

$$M_{Max} = \frac{wl^2}{120} \text{ in. - lb}$$

$$\Delta_{max} = \frac{5}{384} \times \frac{w}{12} \times \frac{l^4}{EI} \text{ in.}$$

$$\Delta_{max} = \frac{1}{145} \times \frac{w}{12} \times \frac{l^4}{EI} \text{ in.}$$

$$V = \frac{5w}{8}\left(L - \frac{2d}{12}\right) \text{ lb}$$

$$V = 0.6w\left(L - \frac{2d}{12}\right) \text{ lb}$$

1. For simple beams:

In the above formula, the resisting moment, M_r, is the product of allowable stress in the extreme fiber of the beam in bending and the section modulus, S. Therefore,

$$M_{Max} = fS = \frac{wl^2}{96}$$

In formwork design, the safe span, l, is used for analysis. Simplifying the above equation:

$$wl^2 = 96fs \implies l^2 = \frac{96fs}{w} \quad \text{or}$$

$$l = 9.80\sqrt{\frac{fS}{w}}$$

2. For continuous beams (more than three supports):

Similar to the analysis above, the safe span for a continuous beam can be derived as:

$$l = 10.95\sqrt{\frac{fS}{w}}$$

Deflection:

The amount of deflection allowed will be addressed in the specifications. If no deflection criterion is specified, it is common to use $l/360$ for structural concrete, where l is the span of the formwork member. Other common allowable deflection quantities are: $l/180$, or maximum allowable deflection of 1/16-in. for sheathing or 1/8-in. for other components. For long members (5 ft. or greater), ¼-in. deflection is normally acceptable.

1. For simple beams:

A. Deflection limited to $l/360$ span:

The deflection for a simple beam is:

$$\Delta_{max} = \frac{5}{384} \times \frac{w}{12} \times \frac{l^4}{EI} \text{ in.} = \frac{l}{360} \implies l^3 = \frac{384 \times 12 \times EI}{5 \times 360 w} = \frac{2.56EI}{w}$$

$$l = 1.37\sqrt[3]{\frac{EI}{w}}$$

B. Deflection limited to $l/180$ span:

$$l = 1.72\sqrt[3]{\frac{EI}{w}}$$

2. For continuous beams:

A. Deflection limited to $l/360$ span:

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

B. Deflection limited to $l/180$ span:

$$l = 2.13\sqrt[3]{\frac{EI}{w}}$$

C. Deflection limited to 1/8" span:

$$l = 3.84\sqrt[4]{\frac{EI}{w}}$$

D. Deflection limited to 1/16" span:

$$l = 3.23\sqrt[4]{\frac{EI}{w}}$$

Shear:

1. For plywood (continuous):

The following formula is used to check the rolling shear in plywood (f_s is the actual rolling shear stress in plywood, psi):

$$f_s = \frac{VQ}{bI}$$

For simply supported beams, the shear is $wL/2$ and foe a continuous beam $0.6wL$. The difference between L and l is that L represents the clear span in feet, where as l represents the distance between center-to-center supports in inches. The rolling shear constant, Ib/Q is listed in Table 4-3 in the text.

For a continuous support condition (F_S is the allowable rolling shear stress in plywood, psi):

$$F_S = \frac{V_{Max}Q}{bI} \text{ and } V_{Max} = 0.6wL \Rightarrow F_S = 0.6wL\frac{Q}{bI} \text{ or}$$

$$L = \frac{F_S}{0.6w} \times \frac{Ib}{Q}$$

$$\text{Note: } \frac{Ib}{Q} = \text{rolling shear constant}$$

Similar calculations can be performed for the beams:

2. For simple beams:

$$l = \frac{16F_v'bd}{w} + 2d$$

3. For continuous beams:

$$l = 13.33\frac{F_v'bd}{w} + 2d$$

where: d = depth of the beam, in.; b = width of the beam, in.; w = uniform load, lb/lf

WALL FORM DESIGN EXAMPLE:

Design forms for 12′-10″ (12.8 ft.) high wall to be concreted at the rate of 4 ft per hour, internally vibrated. Assume the mix is made with Type I cement, with no pozzolans or admixtures, and that the temperature of concrete at placing is 70°F. The unit weight of concrete is 150 pcf with a slump of 3½″. The forms will have continuing reuse. Assume that deflection is limited to *l*/360 of the span. All lumbers are *S4S*.

Form grade plywood sheathing ¾-in. thick is available in 4×8-ft sheets, and 4300-lb coil ties are on hand. Framing lumber of No. 2 Douglas Fir-Larch is to be purchased as required.

STEP 1: FIND THE LATERAL PRESSURE
The concrete used for this project satisfied the conditions of Table 5-4 (it is a normal weight concrete with a unit weight of 150 pcf, made with Type I cement, no admixtures or pozzolans are used and the slump of 3½ inches).

Using Table 5-4, for $R = 4$ ft/hr, and $T = 70$°F, the minimum pressure for design is:

$P = 664$ psf

**TABLE 5-4: MAXIMUM LATERAL PRESSURE
FOR DESIGN OF WALL FORMS**

Applies only for normal weight concrete made with Type I
cement, no admixtures or pozzolans, slump no more than 4
in., and vibration depth limited to 4 ft or less.

Rate of placement, R, ft per hr	p, maximum lateral pressure, psf, for temperature indicated					
	90F	80F	70F	60F	50F	40F
1	600 psf minimum governs					
2						
3					690	825
4			664	750	870	1050
5	650	712	793	900	1050	1275
6	750	825	921	1050	1230	1500
7	850	938	1050	1200	1410	1725
8	881	973	1090	1246	1466	1795
9	912	1008	1130	1293	1522	1865
10	943	1043	1170	1340	1578	1935

NOTE: Do not use design pressures in excess of 150 x height of
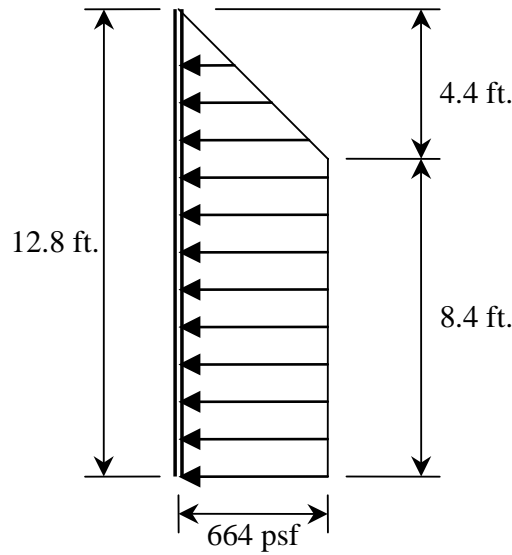fresh concrete in forms.

Or alternatively by calculation:

$$\left(P = 150 + \frac{9000R}{T} = 150 + \frac{9000 \times 4}{70} \cong 664 \, \text{psf} \right)$$

Then the depth of the hydrostatic load zone, for a concrete with a unit weight of 150 pcf is:

$$\frac{P}{150} = \frac{664 \, \text{psf}}{150 \, \text{pcf}} = 4.4 \, \text{ft.}$$

The diagram of lateral pressure on wall form is then drawn as below:

Pressure Diagram

w, loading of the beam for a 1-ft wide strip of plywood is

$$w = \frac{p}{12 \text{ in.}} = \frac{664 \text{ psf} \times 12 \text{ in./ft}}{12 \text{ in.}} = 664 \text{ lb/lf}$$

STEP 2: SHEATHING

4×8-ft. sheets of plywood will be used. Use plywood the "strong way" (face grain parallel to plywood span). Since the sheathing thickness is specified to be ¾", the maximum allowable span, which is the required spacing of studs, needs to be determined. Design for uniformly spaced supports with studs supporting the plywood sheets at the joints.

Check Bending

Consider a 12-in. wide strip of plywood. For plywood, from Table 4-2: $f = 1,545$ psi, $F_S = 57$ psi, and $E = 1,500,000$ psi.

TABLE 4-2:   REPRESENTATIVE BASE DESIGN STRESSES, PSI, NORMAL LOAD DURATION, VISUALLY GRADED DIMENSION LUMBER AT 19 PERCENT MOISTURE, AND PLYWOOD USED WET

Derived from recommendations of the American Forest & Paper Association (Reference 4-3) and from recommendations of the American Plywood Association (Reference 4-8)

SPECIES AND GRADE	Extreme fiber bending stress, F_b	Compression ⊥ to grain, $F_{c\perp}$	Compression ‖ to grain, F_c	Horizontal shear, F_v (‖ to grain)	Modulus of elasticity, E
PLYWOOD SHEATHING USED WET: Plyform B-B, Class 1 (Grade stress level S2)	1545**	Bearing on face: 210	---	57***	1,500,000***

*NOTE: Size adjustments apply to all base bending stresses and compression parallel to the grain except Southern Pine. The size adjustments are already included in Southern Pine bending stresses and compression parallel to grain (in accordance with Reference 4-3). This makes Southern Pine seem relatively stronger in bending and compression ‖. Consult Table 4-2B and Chapter 6 for details of size adjustments.

** Plywood stresses include an experience factor of 1.3 recommended by the American Plywood Association.
***Value for rolling shear in the plane of the plies; see page 6-9 for design formula. Modulus of elasticity may be increased 10 percent if shear deflection is computed separately from bending deflection.
† When size adjusted bending stress is less than or equal to 1150 psi, no moisture adjustment is required.
‡ When size adjusted compression ‖ is less than or equal to 750 psi, no moisture adjustment is required.

From Table 4-3, $S = 0.412$ in.^{3}, $\dfrac{Ib}{Q} = 6.762$ in.^{2}, and $I = 0.197$ in.^{4}.

TABLE 4-3: EFFECTIVE SECTION PROPERTIES FOR PLYWOOD (12-IN. WIDTHS)* FACE PLIES OF DIFFERENT SPECIES FROM INNER PLIES

Sanded plywood, net thickness, in.	Minimum number of layers	Effective thickness for shear, all grades using exterior glue	12-in. width, used with face grain parallel to span				12-in. width, used with face grain perpendicular to span				Approximate weight, lb	
			Area for tension and compression, in.^{2}	Moment of inertia I in.^{4}	Effective section modulus S in.^{3}	Rolling shear constant Ib/Q in.^{2}	Area for tension and compression, in.^{2}	Moment of inertia I in.^{4}	Effective section modulus S in.^{3}	Rolling shear constant Ib/Q in.^{2}	4x8-ft sheet	per sq ft
¼	3	0.267	0.996	0.008	0.059	2.010	0.348	0.001	0.009	2.019	26	0.8
⅜	3	0.288	1.307	0.027	0.125	3.088	0.626	0.002	0.023	3.510	35	1.1
½	3	0.425	1.947	0.077	0.236	4.466	1.240	0.009	0.087	2.752	48	1.5
⅝	5	0.550	2.475	0.129	0.339	5.824	1.528	0.027	0.164	3.119	58	1.8
¾	5	0.568	2.884	0.197	0.412	6.762	2.081	0.063	0.285	4.079	70	2.2
⅞	7	0.586	2.942	0.278	0.515	8.050	2.651	0.104	0.394	5.078	83	2.6
1	7	0.817	3.721	0.423	0.664	8.882	3.163	0.185	0.591	7.031	96	3.0
1⅛	7	0.836	3.854	0.548	0.820	9.883	3.180	0.271	0.744	8.428	106	3.3

*Use listed S value in bending calculations and use I only in deflection calculations. Properties taken from March 1985 edition of Reference 4-8, can be used for all sanded Group I plywoods. IF B-B PLYFORM GRADE OF PLYWOOD IS USED, SLIGHTLY HIGHER VALUES FROM REFERENCE 4-9 CAN BE USED FOR DESIGN.

For continuous beams (more than three supports):

$$l = 10.95\sqrt{\frac{fS}{w}} = 10.95\sqrt{\frac{1545 \times 0.412}{664}} = 10.95\sqrt{0.922} = 10.72 \text{ in.}$$

Check Deflection

The deflection requirement is specified to be $l/360$ of the span. From Tables 4-2 and 4-3: $E = 1,500,000$ psi and $I = 0.197$ in.^{4}.

For $\Delta = l/360 \;\Rightarrow\; l = 1.69\sqrt[3]{\dfrac{EI}{w}} = 1.69\sqrt[3]{\dfrac{1500000 \times 0.197}{664}} = 1.69 \times 7.635 = 12.90 \text{ in.}$

Check Rolling Shear

Calculate the maximum span which satisfies shear stress requirements. Use the equation for maximum shear for a continuous plyform and solve for L:

$$L = \frac{F_s}{0.6w} \times \frac{Ib}{Q} = \frac{57}{0.6 \times 664} \times 6.762 = 0.97 \text{ ft.} = 11.61 \text{ in.}$$

SPACING OF THE STUDS

From the above calculations, the smallest value obtained for l is 10.72 in. (bending governs).

We are using 8-ft.-wide plywood sheets. The sheets should have stud support at the joints. Therefore an equal-spacing of studs at 10.67-inches satisfies all conditions $\left(\dfrac{8' \times 12\,\text{in./ft}}{9} = 10.67''\right)$.

$\therefore$ *USE STUDS WITH THE SPACING OF 10.67 in.*

STEP 3:

STUD SIZE and SPACING OF THEIR SUPPORTS (WALE SPACING)

Lets select stud size of 2×4 *S4S*. The allowable span of studs will determine wale spacing. Find the maximum span that can support a lateral pressure of 664 psf.

Equivalent uniform load, *w*, is the maximum lateral pressure times the stud spacing. Hence:

$$w\left(\text{stud}\right) = \frac{664\,\text{psf} \times 10.67\,\text{in.}}{12\,\text{in./ft.}} = 590\ \text{lb/lf}$$
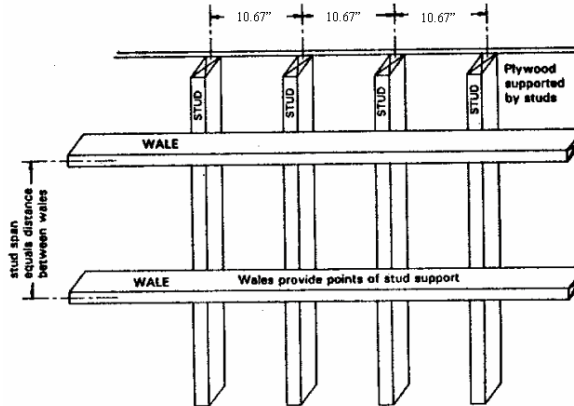
Check Bending

Since the stud size is selected, calculate its maximum allowable span, which is the required spacing of wales. This is a 12'-10" high wall, therefore, studs are continuous over three or more spans. Assume using No. 2 Douglas Fir-Larch studs. From Table 4-2, the extreme fiber bending stress, F_b, is 875 psi. However, this value should be adjusted by a factor called the "*size factor*" obtained from Table 4-2B, which is 1.5. Therefore: $F_b' = 875\,\text{psi} \times 1.5 = 1312.5\ \text{psi}$

TABLE 4-2:   REPRESENTATIVE BASE DESIGN STRESSES, PSI, NORMAL LOAD DURATION, VISUALLY GRADED DIMENSION LUMBER AT 19 PERCENT MOISTURE, AND PLYWOOD USED WET

Derived from recommendations of the American Forest & Paper Association (Reference 4-3) and from recommendations of the American Plywood Association (Reference 4-8)

SPECIES AND GRADE	Extreme fiber bending stress, F_b	Compression ⊥ to grain, $F_{c\perp}$	Compression ∥ to grain, F_c	Horizontal shear, F_v (∥ to grain)	Modulus of elasticity, E
DOUGLAS FIR-LARCH					
No. 2, 2-4 in. thick, 2 in. and wider	875	625	1300	95	1,600,000
Construction, 2-4 in. thick, 2-4 in. wide	1000	625	1600	95	1,500,000

TABLE 4-2B: SIZE AND FLAT USE ADJUSTMENT FACTORS FOR BENDING STRESS AND COMPRESSION PARALLEL TO THE GRAIN FOR NO. 1 AND NO.2 DIMENSION LUMBER
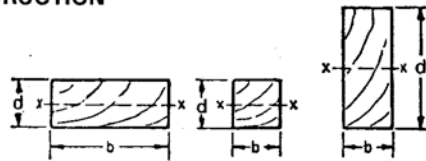
WIDTH OF LUMBER	BENDING STRESS ADJUSTMENT MULTIPLIER				ADJUSTMENT FACTOR FOR COMPRESSION PARALLEL TO GRAIN
	Size Factor		Flat Use Factor		
	2 and 3 in. thick	4 in. thick	2 and 3 in. thick	4 in. thick	
2 and 3 in.	1.5	1.5	1.00	---	1.15
4 in.	1.5	1.5	1.10	1.00	1.15
5 in.	1.4	1.4	1.10	1.05	1.10
6 in.	1.3	1.3	1.15	1.05	1.10
8 in.	1.2	1.3	1.15	1.05	1.05
10 in	1.1	1.2	1.20	1.10	1.00
12 in.	1.0	1.1	1.20	1.10	1.00
14 in.and wider	0.9	1.0	1.20	1.10	0.90

From Table 4-1B: $S = 3.06$ in.^{3} for 2×4 *S4S* No. 2 Douglas Fir-Larch.

TABLE 4-1B: PROPERTIES OF AMERICAN STANDARD BOARD, PLANK, DIMENSION, AND TIMBER SIZES COMMONLY USED FOR FORM CONSTRUCTION

Based on data supplied by the National Forest & Paper Association

X – X is the neutral axis

Nominal size, inches, bxd	American Standard size, inches, bxd, S4S, at 19% maximum moisture	Area of section sq in., $A = bd$		Moment of inertia, in.^{4} $I = bd^3/12$		Section modulus, in.^{3} $S = bd^2/6$		Board feet per linear foot of piece	Approximate weight, lb per linear ft, for use in form design**
		Rough	S4S	Rough	S4S	Rough	S4S		
2x4	1½x3½	5.89	5.25	6.45	5.36	3.56	3.06	0.67	1.3
2x6	1½x5½	9.14	8.25	24.10	20.80	8.57	7.56	1.00	2.0
2x8	1½x7¼	11.98	10.87	54.32	47.63	14.73	13.14	1.33	2.6
2x10	1½x9¼	15.23	13.87	111.58	98.93	23.80	21.39	1.67	3.4
2x12	1½x11¼	18.48	16.87	199.31	177.97	35.04	31.64	2.00	4.1

The allowable stud span as a continuous beam is:

$$l = 10.95\sqrt{\frac{F_b'S}{w}} = 10.95\sqrt{\frac{1312.5 \times 3.06}{590}} = 28.6 \text{ in.}$$

Check Deflection

Calculate the maximum allowable span which meets the deflection requirement of $l/360$ of the span. From Table 4-2, for 2×4 *S4S* No. 2 Douglas Fir-Larch: $E = 1,600,000$ psi. Table 4-1B: $I = 5.36$ in.^{4}.

For $\Delta = l/360 \Rightarrow \quad l = 1.69\sqrt[3]{\frac{EI}{w}} = 1.69\sqrt[3]{\frac{1600000 \times 5.36}{590}} = 1.69 \times 24.4 = 41.2 \text{ in.}$

Check Shear

From Table 4-2, allowable F_V (rolling shear stress) can be found to be $F_s = 95$ psi, which should be multiplied by a factor of 2.0 for horizontal shear adjustment (from Table 6-3, page 6-9, text).

TABLE 6-3: HORIZONTAL SHEAR ADJUSTMENT FACTORS FOR DIMENSION LUMBER

Length of split on wide face of 2-in, (nominal) lumber	Adjustment factor	Length of split on wide face of 3-in. (nominal) and thicker lumber	Adjustment factor	Size of shake in 2-in. (nominal) and thicker lumber	Adjustment factor
no split	2.00	no split	2.00	no shake	2.00
½ × wide face	1.67	½ × narrow face	1.67	1/6 × narrow face	1.67
¾ × wide face	1.50	¾ × narrow face	1.50	¼ × narrow face	1.50
1 × wide face	1.33	1 × narrow face	1.33	1/3 × narrow face	1.33
1½ or more × wide face	1.00	1½ or more × narrow face	1.00	½ or more × narrow face	1.00

Hence, the allowable shear stress is:

$$F'_V = 95 \times 2 = 190 \text{ psi}$$

A 2×4 *S4S* has an actual *b* = 1 ½ in. and *d* = 3 ½ in., which is obtained from Table 4-1B. Use the equation for maximum shear for a continuous beam and solve for *l*:

$$l = 13.33 \frac{F'_V bd}{w} + 2d = 13.33 \frac{190 \times 1\frac{1}{2} \times 3\frac{1}{2}}{590} + 2 \times 3\frac{1}{2} = 22.5 + 7 = 29.5 \text{ in.}$$

SPACING OF THE WALES

From the stud spans calculated above, the shortest span is based on bending which is 28.6 inches. This means the wales, which are the stud supports CANNOT be spaced more than 28.6 inches apart. The top and bottom wales are often set about 1 ft from top and bottom of wall forms.

12'-10" – 2' = 10'-10" or 130" remains for spacing the other wales, which can be no more than 28.6 in. apart. Set them at 26 in.
(130"/26' = 5).
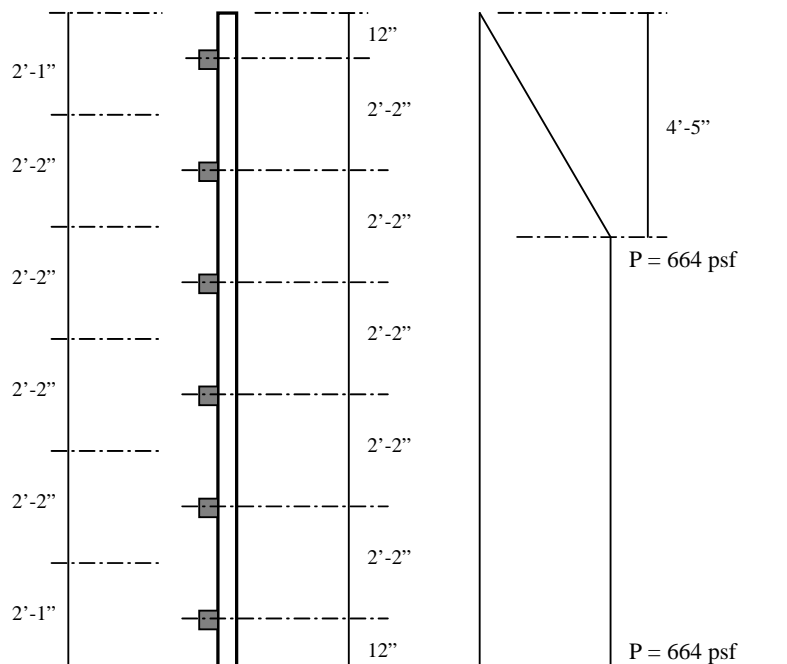
STEPS 4 & 5: TIE DESIGN, WALE SIZE and TIE SPACING

The required spacing of supports for wales determines tie location. From the pressure diagram, the equivalent uniform load per lineal foot of wale is determined to be:

664 psf $\times 2.167$ ft $\cong 1,440$ lb/lf

The problem statement indicates that 4300-lb coil ties are available and will be used. With the maximum load per lineal foot of wale being 1500 lbs, then the maximum tie spacing is:

COIL TYPE TIE WITH OR WITHOUT CONE SPREADER
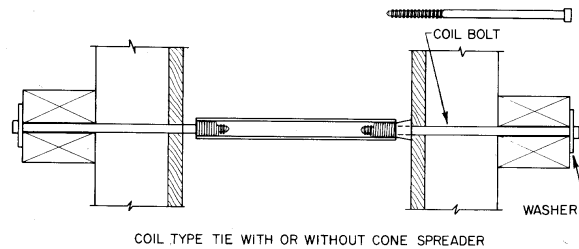
$$\frac{\text{Tie capacity}}{\text{Wale load}} = \frac{4300\,\text{lb}}{1440\,\text{lb/ft}} \approx 3\,\text{ft.}$$

Check Bending

Since tie spacing is selected (3 feet), solve the equation for section modulus, S, and select wales with this calculated required section modulus. Using the maximum bending moment for a uniformly loaded continuous beam (more than 3 supports) equation:

$$M_{Max} = \frac{wl^2}{120}\ \text{in.-lb.}\ \ \&\ \ M_{Max} = F_b'S \qquad \text{Therefore:} \qquad F_b'S = \frac{wl^2}{120} \qquad \text{or}$$

$$S = \frac{wl^2}{120F_b'}$$

F_b' is the allowable stress in the extreme fiber and was calculated to be 1312.5 psi. The span, l, is 3 ft. or 36 inches, and $w = 1440$ lb/lf. Therefor the required section modulus, S:

$$S = \frac{wl^2}{120F_b'} = \frac{1440 \times 36^2}{120 \times 1312.5} = 11.8\ \text{in.}^3$$

In order to avoid drilling of timbers, they commonly use double-member wales. So the required section modulus of 11.8 in.^{3} is for two members. Referring to Table 4-1B, double 3×4s will yield a section modulus of 2×5.10 or 10.20 in.^{3}, which is less than 11.8 in.^{3}, and therefore not acceptable. Checking the next larger size, 4×4, will result in: $S = 2\times 7.15 = 14.30$ in.^{3} > 11.8 in.^{3}, which satisfies the section modulus requirements. $\Rightarrow$ Use double 4×4 wales.
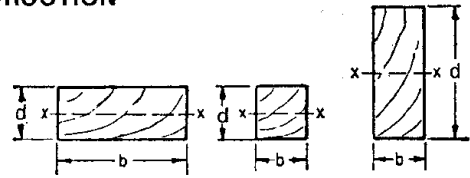
Check Shear

To check the horizontal shear for the double 4×4 wales, use equation 6-12 in the text. From Table 4-1B, the value of bd for a 4×4 member can be obtained as: 12.25 in.^{2}.

TABLE 4-1B: PROPERTIES OF AMERICAN STANDARD BOARD, PLANK, DIMENSION, AND TIMBER SIZES COMMONLY USED FOR FORM CONSTRUCTION

Based on data supplied by the National Forest & Paper Association

X — X is the neutral axis

Nominal size, inches, bxd	American Standard size, inches, bxd, S4S, at 19% maximum moisture	Area of section sq in., $A = bd$		Moment of inertia, in.⁴ $I = bd^3/12$		Section modulus, in.³ $S = bd^2/6$		Board feet per linear foot of piece	Approximate weight, lb per linear ft, for use in form design**
		Rough	S4S	Rough	S4S	Rough	S4S		
4x4	3½x3½	13.14	12.25	14.39	12.50	7.94	7.15	1.33	3.0
4x6	3½x5½	20.39	19.25	53.76	48.53	19.12	17.65	2.00	4.7
4x8	3½x7¼	26.73	25.38	121.17	111.15	32.86	30.66	2.67	6.2
4x10	3½x9¼	33.98	32.38	248.91	230.84	53.10	49.91	3.50	7.9

$$f_V = \frac{0.9w}{bd}\left(L - \frac{2d}{12}\right) = \frac{0.9 \times 1440}{2 \times 12.25} \times \left(3 - \frac{2 \times 3.5}{12}\right) = 52.9 \times 2.42 \cong 128 \text{ psi} \langle 190 \text{ psi} \Rightarrow \text{O.K.}$$

Therefore the stress in the double 4×4 members meets the requirements.

Since the deflection of wales is hardly ever critical, it can be ignored. However, if in doubt, it can be checked using the deflection formula introduced earlier.

STEP 6: <u>BEARING CHECK</u>
Check:

1) bearing of the studs on wales and

2) bearing between the tie washer or tie holders and wales.

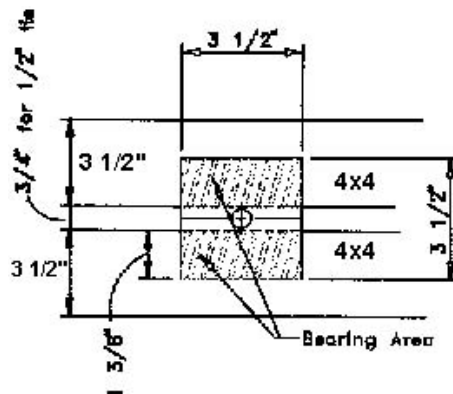From Table 4-2, the value of compression ⊥ to grain, $F_{c\perp}$, for No. 2 2×4 Douglas Fir-Larch is 625 psi.

TIES: Assume a 3½ in.-square tie washer.

Then the bearing area is:

$(3½'')^2 - ¾'' \times 3½'' = 12.25 - 2.63 = 9.63$ in.²

Since this is a short bearing length, $F_{c\perp}$ should be multiplied by a factor of 1.11 (refer to page 12 for 3½ in. bearing length):

Adjusted $F_{c\perp}' = 625\ (1.11) = 694$ psi

The actual bearing stress is then:

$$\frac{\text{Maximum tie load}}{\text{Bearing area}} = \frac{4300 \text{ lb}}{9.63 \text{ in.}^2} = 447 \text{ psi} < 694 \text{ psi} \Rightarrow \text{O.K.}$$
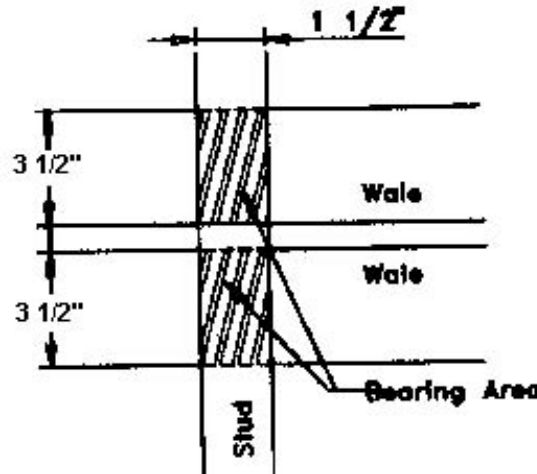
STUDS ON WALES:

The bearing area between 2×4 studs and double 4×4 wales can be calculated as:

$$2 \times (1\tfrac{1}{2}''\times3\tfrac{1}{2}'') = 2 \times 5.25 = 10.5 \text{ in.}^2$$

Load transfer to the wale = ½ the stud span above and below the wale × the lateral pressure × the stud spacing

$$\frac{26/2+26/2}{12} \times 664 \text{ psf} \times 1 \text{ ft} \approx 1439 \text{ lb}$$

$$\text{bearing stress} = \frac{1439 \text{ lb}}{10.5 \text{ in.}^2} = 137 \text{ psi} < 625 \text{ psi} \Rightarrow \text{O.K.}$$

STEP 7: LATERAL BRACING

Consider the necessary bracing for a wall form 12'-10" high, above grade, in an area where the local building code specifies a minimum 20 psf wind loading.

Table 5-7 (page 5-17, text) indicated that 128 lb per lineal foot should be used for design of bracing, since the wind force prescribed by local code gives a value larger than the 100 lb/ft minimum established by ACI Committee 347.

**TABLE 5-7: MINIMUM LATERAL FORCE
FOR DESIGN OF WALL FORM BRACING**

Minimum lateral force, *H*, lb per lineal ft, applied at top of wall form. *H* may act in either direction.
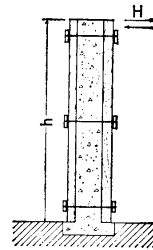
Wall height, *h*, in ft	Committee 347 minimum. 100 lb/ft or 15 psf wind	Wind force prescribed by local code *			
		10 psf	20 psf	25 psf	30 psf
(Above grade)					
4 or less	30	20	40	50	60
6	45	30	60	75	90
8	100	100	100	100	120
10	100	100	100	125	150
12	100	100	120	150	180
14	105	100	140	175	210
16	120	100	160	200	240
18	135	100	180	225	270
20	150	100	200	250	300
22 or more	7.5 *h*	5.0 *h*	10.0 *h*	12.5 *h*	15.0 *h*
Walls below grade					
8 ft or less	BRACE TO MAINTAIN ALIGNMENT				
More than 8 ft	100 psf minimum, or brace for any known lateral forces which are greater				

12.8 → (pointing to the 14 ft wall height row)

*Wind force prescribed by local code shall be used whenever it would require a lateral force for design greater than the Committee 347 minimum shown.

STRUT BRACING

If wooden strut bracing is provided, strong enough to take either a tension or compression load, then single side bracing may be used. Nailed connections at either end must be strong enough to transmit the tension load, and wales or other form members must be strong enough to transmit accumulated horizontal forces to the strut bracing.

If wooden bracing is attached any distance below the top of the wall, the bracing must carry more than the 128 lb per ft load applied at the top.
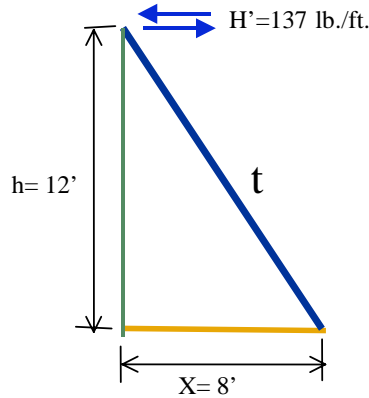
H' the horizontal bracing force 8 inches from the top of the wall would have to be

$$\left(\frac{12.8'}{12'}\right) \times 128\,\text{lb/lf} = 137\,\text{lb per ft}$$

in order to balance the 128 lb/lf design load applied at the top of the wall.
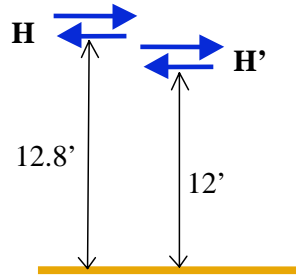
If end of the brace is put 8 feet from the wall, use the relationship between sides of the right triangle to find the length of brace and load it must carry.

$$t = \sqrt{h^2 + x^2}$$

$$t = \sqrt{12^2 + 8^2} = \sqrt{208} = 14.15 \text{ ft}$$

$$\text{(tension) compression in strut} = H' \times \frac{14.15'}{8'}$$

$$\text{(tension) compression in strut} = 137 \times \frac{14.15'}{8'}$$

$$= 242 \text{ lb per foot of wall}$$

If struts are spaced every 8 feet along the wall, then 8×242 or 1936 lb must be carried by each brace.

Many wood members strong enough to carry this load in compression will also be adequate in tension. However, the strength of connections (nails, etc.) must be made adequate for the tension load.