4

Design and Construction of Concrete Formwork

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4.1 Introduction

Concrete formwork serves as a mold to produce concrete elements having a desired size and configuration. It is usually erected for this purpose and then removed after the concrete has cured to a satisfactory strength. In some cases, concrete forms may be left in place to become part of the permanent structure. For satisfactory performance, formwork must be adequately strong and stiff to carry the loads produced by the concrete, the workers placing and finishing the concrete, and any equipment or materials supported by the forms.

For many concrete structures, the largest single component of the cost is the formwork. To control this cost, it is important to select and use concrete forms that are well suited for the job. In addition to being economical, formwork must also be constructed with sufficient quality to produce a finished concrete element that meets job specifications for size, position, and finish. The forms must also be designed, constructed, and used so that all safety regulations are met.

Formwork costs can exceed 50% of the total cost of the concrete structure, and formwork cost savings should ideally begin with the architect and engineer. They should choose the sizes and shapes of the elements of the structure, after considering the forming requirements and formwork costs, in addition to the usual design requirements of appearance and strength. Keeping constant dimensions from floor to floor, using dimensions that match standard material sizes, and avoiding complex shapes for elements in order to save concrete are some examples of how the architect and structural engineer can reduce forming costs.

The designer of concrete formwork must choose appropriate materials and utilize them so that the goals of safety, economy, and quality are met. The formwork should be easily built and stripped so that it saves time for the contractor. It should have sufficient strength and stability to safely carry all live and dead loads encountered before, during, and after the placing of the concrete. And, it should be sufficiently resistant to deformations such as sagging or bulging in order to produce concrete that satisfies requirements for straightness and flatness.

Concrete forms that do not produce satisfactory concrete elements are not economical. Forms not carefully designed, constructed, and used will not provide the surface finish or the dimensional tolerance required by the specifications for the finished concrete work. To correct concrete defects due to improperly designed and constructed forms may require patching, rubbing, grinding, or in extreme cases, demolition and rebuilding.

To produce concrete forms that meet all job requirements, the construction engineer must understand the characteristics, properties, and behaviors of the materials used; be able to estimate the loads applied to the forms; and be familiar with the advantages and shortcomings of various forming systems. Form economy is achieved by considering four important factors:

- Cost of form materials
- Ease of form fabrication
- Efficient use of forms — erecting and stripping
- Planning for maximum reuse to lower per use cost

Design methods for concrete formwork generally must follow the same codes, specifications, and regulations that apply to permanent structures. Some codes may allow increased allowable loads and stresses because temporary structures are used for a shorter period of time. The Occupational Safety and Health Act (OSHA) of the U.S. government contains criteria that the designer of concrete formwork must follow. State and local safety codes may also exist that regulate form design and construction as it pertains to job site safety.

For the materials ordinarily used in the construction of concrete forms, building codes commonly follow and incorporate by reference the basic technical codes published by national organizations. These national organizations include the American Concrete Institute (ACI), the American Institute of Steel Construction (AISC), the American Society for Testing of Materials (ASTM), the Aluminum Association (AA), the Engineered Wood Association (APA — formerly the American Plywood Association), and the American Forest and Paper Association (AF&PA). These organizations developed specifications and standards for concrete, steel, aluminum, plywood and similar engineered panels, lumber, and so on. They are as follows:

- ACI Standard 318 Building Code Requirements for Reinforced Concrete
- AISC Specification for Design, Fabrication, and Erection of Structural Steel for Buildings
- AISC Code of Standard Practice
- AA Specifications for Aluminum Structures
- APA Plywood Design Specification
- AF&PA National Design Specification (NDS) for Wood Construction
- Design Values for Wood Construction, supplement to the National Design Specifications for Wood Construction
- ASTM Annual Book of ASTM Standards

Many technical manuals and publications are used to assist in the design of temporary and permanent structures. Those most commonly encountered include *Formwork for Concrete* published by ACI, *Concrete Forming* published by APA, *Manual of Steel Construction* published by AISC, *Manual of Concrete Practice* published by ACI, *Timber Construction Manual* published by the American Institute of Timber Construction (AITC), *Concrete Manual* published by the U.S. Department of the Interior Bureau of Reclamation, *Recommended Practice for Concrete Formwork* by ACI, *Wood Handbook: Wood as an Engineering Material* published by the U.S. Department of Agriculture, *Standard Specifications and Load Tables for Open Web Steel Joists* published by the Steel Joist Institute, *Light Gage Cold Formed Steel Design Manual* published by the American Iron and Steel Institute, *Minimum Design Loads for Buildings and Other Structures* by the American National Standards Institute (ANSI), and *Formwork, Report of the Joint Committee* published by the Concrete Society as Technical Report No.13 (Great Britain).

4.2 Concrete Formwork

Two major categories of formwork are job built and prefabricated. Job-built forms are often designed, built, and used with the particular requirements of a single project in mind. They are most often constructed using plywood sheathing and lumber framing. They may also incorporate proprietary hardware in their assembly. Job-built forms are often the economical choice when complicated forming is required that would be difficult or more expensive if using commercial form systems.

Prefabricated or commercial forms are usually constructed with materials that can be reused many times. Their higher initial cost is offset by the potential for more reuse cycles than job-built forms of lumber and plywood or possible cost savings from increased productivity in erecting and stripping the forms. Commercial concrete forms may be of standard design or custom built for a particular application. Some types of commercial forms are designed to span relatively long distances without intermediate supports. Some girder forms of this type are constructed so that the sides of the forms behave like a plate girder to carry the dead and live loads. This type of form would be a viable choice for elements constructed high off the ground, over water, or over difficult terrain, where it would be difficult to use intermediate supports.

4.3 Materials

Most concrete forms are constructed using basic materials such as lumber, plywood, and steel, or are prefabricated panels sold or leased to contractors by the panel manufacturers. Use of the prefabricated panels may save labor costs on jobs where forms are reused many times. Panel manufacturers will provide layout drawings, and they sometimes provide supervision of the construction where prefabricated forms are used. Even when prefabricated forms are chosen, there are often parts of the concrete structure that must be formed using lumber and plywood job-built forms.

Lumber

Lumber suitable for constructing concrete forms is available in a variety of sizes, grades, and species groups. The form designer should determine what is economically available before specifying a particular grade or species group of lumber for constructing the forms.

Some of the most widely available species groups of lumber include Douglas fir-larch, southern pine, ponderosa pine, and spruce-pine-fir. Douglas fir and southern pine are among the strongest woods available and are often chosen for use in formwork. The strength and stiffness of lumber varies widely with different species groups and grades. Choice of species groups and grade will greatly affect size and spacing of formwork components.

Most lumber has been planed on all four sides to produce a uniform surface and consistent dimensions and is referred to as S4S (surfaced on four sides) lumber. The sizes produced have minimum dimensions specified in the *American Softwood Lumber Standard*, PS 20–94. Nominal dimensions are used to specify standard lumber sizes (e.g., 2 × 4, 2 × 6, 4 × 6, etc.). The actual dimensions are somewhat smaller for finished and rough-sawn lumber. Rough-sawn lumber will have dimensions about 1/8-in. larger than finished S4S lumber. Lumber sizes commonly used for formwork along with their section properties are given in Table 4.1.

Lumber used in forming concrete must have a predictable strength. Predictable strength is influenced by many factors. Lumber that has been inspected and sorted during manufacturing will carry a grade

TABLE 4.1 Properties of Dressed Lumber

Standard Size Width × Depth	S4S Dressed Size Width × Depth	Cross-Sectional Area A (in.^{2})	Moment of Inertia I (in.^{4})	Section Modulus S (in.^{3})	Weight in Pounds per Lineal Foot[a]
1 × 4	¾ × 3½	2.63	2.68	1.53	0.64
1 × 6	¾ × 5¼	4.13	10.40	3.78	1.00
1 × 8	¾ × 7¼	5.44	23.82	6.57	1.32
1 × 12	¾ × 11¼	8.44	88.99	15.82	2.01
2 × 4	1½ × 3½	5.25	5.36	3.06	1.28
2 × 6	1½ × 5½	8.25	20.80	7.56	2.01
2 × 8	1½ × 7¼	10.88	47.64	13.14	2.64
2 × 10	1½ × 9¼	13.88	98.93	21.39	3.37
2 × 12	1½ × 11¼	16.88	177.98	31.64	4.10
4 × 2	3½ × 1½	5.25	.98	1.31	1.28
4 × 4	3½ × 3½	12.25	12.51	7.15	2.98
4 × 6	3½ × 5½	19.25	48.53	17.65	4.68
4 × 8	3½ × 7¼	25.38	111.15	30.66	6.17
6 × 2	5½ × 1½	8.25	1.55	2.06	2.01
6 × 4	5½ × 3½	19.25	19.65	11.23	4.68
6 × 6	5½ × 5½	30.25	76.26	27.73	7.35
6 × 8	5½ × 7¼	41.25	193.36	51.53	10.03
8 × 2	7¼ × 1½	10.88	2.04	2.72	2.64
8 × 4	7¼ × 3½	25.38	25.90	14.80	6.17
8 × 6	7¼ × 5½	41.25	103.98	37.81	10.03
8 × 8	7¼ × 7¼	56.25	263.67	70.31	13.67

[a] Weights are for wood with a density of 35 pounds per cubic foot.

stamp indicating the species, grade, moisture condition when surfaced, and perhaps other information. Grading is accomplished by following rules established by recognized grading agencies and are published in the *American Softwood Lumber Standard.* Lumber can be graded visually by a trained technician or by a machine. Visually graded lumber has its design values based on provisions of ASTM-D245, *Methods for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber.* Machine stress-rated (MSR) lumber has design values based on nondestructive stiffness testing of individual pieces. Some visual grade requirements also apply to MSR lumber. Lumber with a grade established by a recognized agency should always be used for formwork where strength is important.

Allowable Stresses for Lumber

The National Design Specification for Wood Construction (NDS) (AF&PA, 1997) makes comprehensive recommendations for engineered uses of stress-graded lumber. Stress values for all commercially available species groups and grades of lumber produced in the U.S. are tabulated in the NDS. The moduli of elasticity for all species groups and grades are also included in these tables. These tabulated values of stresses and moduli of elasticity are called base design values. They are modified by applying adjustment factors to give allowable stresses for the graded lumber.

The adjustment factors reduce (or in some cases increase) the base design stress values to account for specific conditions of use that affect the behavior of the lumber. A list of these adjustment factors and a discussion of their use follows.

Load Duration — C_D

The stress level that wood will safely sustain is inversely proportional to the duration that the stress is applied. That is, stress applied for a very short time (e.g., an impact load) can have a higher value than stress applied for a longer duration and still be safely carried by a wood member. This characteristic of wood is accounted for in determining allowable stresses by using a load duration factor, C_D. The load duration factor varies from 20 for an impact load (duration equal to one second) to 0.9 for a permanent load (duration longer than 10 years). ACI Committee 347 recommends that for concrete formwork, a load duration factor appropriate for a load of 7 days should be used. This corresponds to a value for C_D of 1.25. ACI Committee 347 says this load duration factor should only be applied to concrete forms intended for limited reuse. No precise definition of limited reuse is given by the ACI committee, but the no increase for duration of load should be used for concrete forms designed to be reused a high number of cycles.

Moisture — C_M

Wood is affected by moisture content higher than about 19%. Higher moisture content significantly softens the wood fibers and makes it less stiff and less able to carry stresses. The reduction in allowable strength depends on the type of stress (e.g., shear stress is affected less than perpendicular to grain compressive stress) and the grade of the lumber.

Size — C_F

Research on lumber allowable stresses has shown that as cross-sectional size increases, allowable stresses are reduced. A size factor, C_F, is used to increase base design values for different sizes of lumber.

Repetitive Members — C_r

The NDS allows bending stresses to be increased for beams that share their loads with other beams. The increased allowable stress is referred to as a repetitive member stress. For a beam to qualify as a repetitive member, it must be one of at least three members spaced no further apart than two feet and joined by a load-distributing element such as plywood sheathing. When these three requirements are met, the allowable bending stress can be increased by 15%. This corresponds to a value for C_r of 1.15. Repetitive member stresses may be appropriate for some formwork components. Because the intent of allowing increased stress for repetitive flexural members is to take advantage of the load sharing provided by continuity, gang panels assembled securely by bolting or nailing and intended for multiple reuse would seem to qualify for this increase. ACI Committee 347 specifies that they should not be used where the bending stresses have already been increased by 25% for short duration loads.

Perpendicular to Grain Compression — C_b

Allowable perpendicular to grain bearing stress at the ends of a beam may be adjusted for length of bearing according to:

$$C_b = \left(l_b + .375\right)/l_b$$

l_b is the length of bearing parallel to grain.

Horizontal Shear Constant — C_H

Shear stress in lumber beams used as components of concrete forms is usually highest at the ends of the members. For beams having limited end defects (e.g., splits, checks, cracks), the values of allowable shear stress can be increased. This is done by using a shear constant C_H that depends on the size of end defects and varies from 1 to 2.

Temperature — C_T

Sustained high temperatures adversely affect some properties of wood. It is unusual for concrete forms to be exposed to temperatures high enough to require the use of a temperature adjustment factor. For temperatures in excess of 100°F, the stresses and moduli should be adjusted using C_T.

Stablity — C_P

Like all columns, wood shores will safely carry axial loads in inverse proportion to their effective slenderness. The more slender a wood shore is, the less load it will support because of the increased influence of buckling. Prior to the 1997 edition of the NDS, wood columns were divided into three categories (short, intermediate, and long) according to their slenderness. Allowable stresses and loads were then found using three different formulas — one for each category. Beginning with the 1997 NDS, allowable loads for all wood columns are found using a stability adjustment factor, C_P, that reduces the base stress to account for the buckling tendency of the column. It is no longer necessary to divide wood shores into three categories to find allowable loads.

Finding Allowable Lumber Stresses

The first step in finding an allowable stress for lumber (or determining the value for modulus of elasticity) is to look up the base design value. These base design values are given in the NDS supplement, and values for a few commonly available species groups are shown in Table 4.2.

Next, the adjustment factors appropriate for the conditions of use are found. These may be looked up in Tables 4.2A through 4.2C. The number of adjustment factors used for a particular situation can vary from none to several.

The allowable stress value, F′ is then found by multiplying the base design value F by all the adjustment factors for the conditions of use.

$$F' = F\, C_i$$

Example 1

Allowable Stresses and Modulus of Elasticity

#1 S4S Douglas fir-larch 2 × 4s are used as studs in a wall form panel. The forms assembled from these panels will be used twice in building a reinforced concrete wall. If the 2 × 4 studs are spaced 12 in. apart, and upon examination the ends have no splits or checks in them, what value for allowable bending stress, allowable shear stress, and modulus of elasticity should be used to design the form panel?

From Table 4.2, the base design values for bending stress, shear stress, and modulus of elasticity are 1000 psi, 95 psi, and 1,700,000 psi.

Adjustments to base design values:

Since the forms will be used a limited number of times (two), it is appropriate to use the load duration factor that increases the base design values for stress by 25%.

$$C_D = 1.25$$

Unless lumber and form panels are stored inside or otherwise protected from rain on a job site, it is logical to assume that the moisture content will exceed 19%, and a moisture adjustment should be applied to the base design values. From Table 4.2C, the values of C_M to use are as follows:

$$C_M = .85\ (\text{bending})$$

$$C_M = .97\ (\text{shear})$$

$$C_M = .9\ (\text{modulus of elasticity})$$

While the 2 × 4 studs meet the three requirements for repetitive members, the 15% increase is not applied. This is because the 25% load duration increase will be used, and the ACI Committee 347 in *Formwork for Concrete* recommends not using both adjustments.

The ends of the studs have no splits or checks, so from Table 4.2B, it is seen that the stress adjustment factor C_H is 2.

For a 2 × 4, the size factor adjustment from Table 4.2A for bending stress CF is 1.5.

Applying adjustments for duration of load, size, and moisture to the base design value for bending stress:

$$F_b' = F_b(C_i) = F_b(C_D)(C_M)(C_F) = 1000(1.25)(.85)(1.5) = 1594 \text{ psi}$$

TABLE 4.2 Base Design Values for Selected Species Groups of Lumber

Species and Grade	Size Classification	Stress Values in Pounds per Square Inch (psi)				
		Bending	Horizontal Shear	Compression Perpendicular	Compression Parallel	Modulus of Elasticity
Douglas Fir — Larch (Surfaced Dry or Green, Used at 19% Maximum Moisture)						
No. 1	2″ to 4″	1000	95	625	1500	1,700,000
No. 2	Thick	900	95	625	1350	1,600,000
No. 3	2″ and wider	525	95	625	775	1,400,000
Stud	2″ to 4″	70	95	625	850	1,400,000
Construction	—	1000	95	625	1150	1,500,000
Standard	2″ to 4″	575	95	625	925	1,400,000
Utility	—	275	95	625	600	1,300,000
Spruce-Pine-Fir (Surfaced Green, Used Any Condition)						
No. 1/no. 2	2″ to 4″	875	70	425	825	1,400,000
No. 3	2″ to 4″	500	70	425	400	1,200,000
Stud	Wide	675	70	425	400	1,200,000
Construction	—	1000	70	425	725	1,300,000
Standard	—	550	70	425	600	1,200,000
Utility	—	275	70	425	400	1,100,000

Source: Design Values for Wood Construction, A Supplement to the 1997 Edition National Design Specification, American Forest and Paper Association, Washington, DC.

Similarly, applying the adjustments for duration of load, shear stress, and moisture to the base design value for shear stress gives:

$$Fv' = 230 \text{ Psi}$$

The only adjustment factor for the modulus of elasticity is for moisture. Applying this factor gives:

$$E' = E(C_M) = 1{,}530{,}000 \text{ psi}$$

Plywood

Plywood is used extensively for concrete forms and provides the following advantages:

- It is economical in large panels.
- It is available in various thicknesses.
- It creates smooth, finished surfaces on concrete.
- It has predictable strength.
- It is manufactured in more than 40 surface textures that can provide various architectural finishes.

Plywood is available in two types: exterior and interior. The exterior type is made with waterproof glue and has all plies made with C grade or better veneers. While many exterior plywood panels could be used, the plywood industry produces a special product intended for concrete forming called Plyform. This panel has two smooth sides (usually B grade veneer on front and back) and is available in three classes — Class I, Class II, and Structural I. Class I is stronger than Class II because of the higher grade of veneers used in the panel. Structural I is the strongest of the three classes and is intended for applications where high strength and stiffness or maximum reuse are desired. Plyform is also available with a surface treatment of thermosetting, resin-impregnated material that is bonded to the panel surfaces. This abrasion-resistant surface, which gives an extremely smooth finish to concrete and allows more reuses of gang forms, is called a high-density overlay (HDO). Table 4.3 summarizes plywood grades and uses for concrete forms.

TABLE 4.2A

Size Factors, C_F					
		F_b			
		Thickness (Breadth)			
Grades	Width (Depth)	2” and 3”	4”	F_t	F_c
Select	2″, 3″, and 4″	1.5	1.5	1.5	1.15
Structural,	5″	1.4	1.4	1.4	1.1
No. 1 and Btr.	6″	1.3	1.3	1.3	1.1
No. 1, No. 2, No. 3	8″	1.2	1.3	1.2	1.05
	10″	1.1	1.2	1.1	1.0
	12″	1.0	1.1	1.0	1.0
	14″ and wider	0.9	1.0	0.9	0.9
Stud	2″, 3″, and 4″	1.1	1.1	1.1	1.05
	5″ and 6″	1.0	1.0	1.0	1.0
	8″ and wider	Use No. 3 Grade tabulated design values and size factors			
Construction and Standard	2″, 3″, and 4″	1.0	1.0	1.0	1.0
Utility	4″, 2″, and 3″	1.0	1.0	1.0	1.0
		0.4	—	0.4	0.6

Source: Wood Construction, 1997 Edition, National Design Specification, p. 62. American Forest and Paper Association, Washington, DC.

TABLE 4.2B Shear Stress Factors, C_H

Length of Split on Wide Face of 2″ (Nominal) Lumber	C_H	Length of Split on Wide Face of 3″ (Nominal) and Thicker Lumber	C_H	Size of Shake[a] in 2″ (Nominal) and Thicker Lumber	C_H
No split	2	No split	2	No shake	2
½ × wide face	1.67	½ × narrow face	1.67	⅙ × narrow face	1.67
¾ × wide face	1.5	¾ × narrow face	1.5	¼ × narrow face	1.5
1 × wide face	1.33	1 × narrow face	1.33	⅓ × narrow face	1.33
1½ × wide face or more	1	1½ × narrow face or more	1	½ × narrow face or more	1

[a] Shake is measured at the end between lines enclosing the shake and perpendicular to the loaded face.
Source: Wood Construction, 1997 Edition, National Design Specification, p. 62. American Forest and Paper Association, Washington, DC.

TABLE 4.2C Wet Service Adjustments, C_M

F_b	F_t	F_v	$F_{c\perp}$	F_c	E
0.85[a]	1	0.97	0.67	0.8[b]	0.9

[a] When $(F_b)(C_F) \leq 1100$ psi, $C_M = 1$.
[b] When (F_C) $(C_F) \leq 750$ psi, $C_M = 1$.
Source: Wood Construction, 1997 Edition, National Design Specification, p. 62. American Forest and Paper Association, Washington, DC.

Engineering Properties of Plywood

Plywood is manufactured by peeling veneers from a log in thin layers, then gluing these veneers together to form plywood panels. Depending on the panel thickness, different numbers of veneer layers are used. To produce a panel that has desirable properties in both directions, the grain direction in different layers of veneer is oriented perpendicular to adjacent layers. Laying panels with veneer grain in perpendicular

TABLE 4.3 Plywood Grade — Use Guide for Concrete Forms

Terms for Specifying	Description	Typical Trademarks	Veneer Grade		
			Faces	Inner Plies	Backs
APA B-B Plyform Classes I and II	Specifically manufactured for concrete forms; many reuses; smooth, solid surfaces		B	C	B
APA High-Density Overlaid Plyform Classes I and II	Hard, semi-opaque resin-fiber overlay, heat fused to panel faces; smooth surface resists abrasion; up to 200 reuses		B	C-Plugged	B
APA Structural I Plyform	Especially designed for engineered applications; all group 1 species; stronger and stiffer than Plyform Classes I and II; recommended for high pressures; also available in HDO faces		B	C or C-plugged	B
APA B-C EXT	Sanded panel often used for concrete forming where only one smooth side is required		B	C	C

Source: Concrete Forming, APA Design/Construction Guide, p. 6, Engineered Wood Association, Tacoma, WA.

directions in alternate layers is called cross-banding. Because of cross-banding, the mechanical properties of adjacent veneers are not the same.

The section properties of plywood, such as moment of inertia and section modulus, cannot be calculated using the same formulas used for other common materials. The calculations of these section properties involve using a transformed area approach with the veneers. The form designer does not have to make these calculations. For plywood that conforms to the *U.S. Product Standard*, the engineering data are published in the *Plywood Design Specification (PDS)* from the Engineered Wood Association. Section properties for Plyform Class I, Class II, and Structural I are shown in Table 4.4. These section properties have been adjusted to account for cross-banded veneers. These values have been calculated by transforming all plies to the properties of the face ply. In using these values, the designer only needs the allowable stresses for the face ply and does not have to be concerned with the actual construction of the panel. The tabulated section properties are used for calculating flexural stresses, shear stresses, and deflections. For types of plywood other than Plyform panels, section properties can be found in the *PDS*.

The section properties in Table 4.4 are for a 12-in. wide strip of plywood. The values are given for both possible orientations of the face grain of the panel with the direction of stress. These two orientations are sometimes called the "strong direction" and the "weak direction." When the panel is supported so that the stresses are in a direction parallel to the face grain, it is said to be oriented in the strong direction. This is sometimes described as having the supports perpendicular to the grain or having the span parallel to the grain. All of these describe the strong direction orientation.

The three different section properties found in Table 4.4 are moment of inertia (I), effective section modulus (KS), and rolling shear constant (Ib/Q).

The moment of inertia is used in calculating deflections in plywood. Deflections due to flexure and shear are calculated using I. Standard formulas can be used to find bending and shear deflections.

The effective section modules (KS) is used to calculate bending stress (f_b) in the plywood:

$$f_b = \frac{M}{KS} \tag{4.1}$$

It should be noted that because of the cross-banded veneers and the fact that veneers of different strengths may be used in different plies, bending stress cannot be correctly calculated using the moment of inertia. That is, KS is not equal to I divided by half the panel thickness:

$$KS \neq \frac{I}{c} \tag{4.2}$$

TABLE 4.4 Section Properties for Plywood

Thickness (in.)	Approximate Weight (psf)	Properties for Stress Applied Parallel with Face Grain: Moment of Inertia I (in.^{4})	Effective Section Modulus KS (in.^{3})	Rolling Shear Constant lb/Q (in.^{2})	Properties for Stress Applied Perpendicular to Face Grain: Moment of Inertia I (in.^{4})	Effective Section Modulus KS (in.^{3})	Rolling Shear Constant lb/Q (in.^{2})
			Class I				
15/32	1.4	0.066	0.244	4.743	0.018	0.107	2.419
1/2	1.5	0.077	0.268	5.153	0.024	0.130	2.739
19/32	1.7	0.115	0.335	5.438	0.029	0.146	2.834
5/8	1.8	0.130	0.358	5.717	0.038	0.175	3.094
23/32	2.1	0.180	0.430	7.009	0.072	0.247	3.798
3/4	2.2	0.199	0.455	7.187	0.092	0.306	4.063
7/8	2.6	0.296	0.584	8.555	0.151	0.422	6.028
1	3.0	0.427	0.737	9.374	0.270	0.634	7.014
1⅛	3.3	0.554	0.849	10.430	0.398	0.799	8.419
			Class II				
15/32	1.4	0.063	0.243	4.499	0.015	0.138	2.434
1/2	1.5	0.075	0.267	4.891	0.020	0.167	2.727
19/32	1.7	0.115	0.334	5.326	0.025	0.188	2.812
5/8	1.8	0.130	0.357	5.593	0.032	0.225	3.074
25/32	2.1	0.180	0.430	6.504	0.060	0.317	3.781
3/4	2.2	0.198	0.454	6.631	0.075	0.392	4.049
7/8	2.6	0.300	0.591	7.990	0.123	0.542	5.997
1	3.0	0.421	0.754	8.614	0.220	0.812	6.987
1⅛	3.3	0.566	0.869	9.571	0.323	1.023	8.388
			Structural I				
15/32	1.4	0.067	0.246	4.503	0.021	0.147	2.405
1/2	1.5	0.078	0.271	4.908	0.029	0.178	2.725
19/32	1.7	0.116	0.338	5.018	0.034	0.199	2.811
23/32	2.1	0.183	0.439	6.109	0.085	0.338	3.780
3/4	2.2	0.202	0.464	6.189	0.108	0.418	4.047
7/8	2.6	0.317	0.626	7.539	0.179	0.579	5.991
1	3.0	0.479	0.827	7.978	0.321	0.870	6.981
1⅛	3.3	0.623	0.955	8.841	0.474	1.098	8.377

Source: Concrete Forming, APA Design/Construction Guide, p. 14, Engineered Wood Association, Tacoma, WA.

Therefore,

$$f_b \neq \frac{Mc}{I} \tag{4.3}$$

The rolling shear constant is used to calculate the rolling shear stress in plywood having loads applied perpendicular to the panel. The name *rolling shear stress* comes from the tendency of the wood fibers in the transverse veneer plies to roll over one another when subjected to a shear stress in the veneer plane. Horizontal shear stress in a beam is:

$$f_v = \frac{VQ}{Ib} = \frac{V}{Ib/Q} \tag{4.4}$$

where V is the shear force in the beam at the section and Ib/Q is the section property depending on size and shape of the cross section.

TABLE 4.5 Allowable Stresses and Pressures for Plyform Plywood

Allowable Stresses	Plyform Class I	Plyform Class II	Plyform Structural I
Modulus of elasticity, E^a (psi)	1,500,000	1,300,000	1,500,000
Bending stress, F_b (psi)	1930	1330	1930
Rolling shear stress, F_s (psi)	72	72	102

Recommended Maximum Pressures on Plyform Class I (Pounds per Square Foot, psf)
Face Grain Parallel to Supports, Plywood Continuous Across Two or More Spans
Plywood Thickness (in.)

	15/32		1/2		19/32		5/8		23/32		3/4		1⅛	
Deflection limit	L/360	l/270	L/360	l/270	L/360	l/270	L/360	l/270	L/360	l/270	L/360	l/270	L/360	l/270
Support Spacing														
4	2715	2715	2945	2945	3110	3110	3270	3270	4010	4010	4110	4110	5965	5965
8	885	885	970	970	1195	1195	1260	1260	1540	1540	1580	1580	2295	2295
12	335	395	405	430	540	540	575	575	695	695	730	730	1370	1370
16	150	200	175	230	245	305	265	325	345	390	370	410	740	770
20	—	115	100	135	145	190	160	210	210	270	225	285	485	535
24	—	—	—	—	—	100	—	110	110	145	120	160	275	340
32	—	—	—	—	—	—	—	—	—	—	—	—	130	170

Recommended Maximum Pressures on Plyform Class I (Pounds per Square Foot, psf)
Face Grain Across Supports, Plywood Continuous across Two or More Spans
Plywood Thickness (in.)

	15/32		1/2		19/32		5/8		23/32		3/4		1⅛	
Deflection limit	L/360	l/270	L/360	l/270	L/360	l/270	L/360	l/270	L/360	l/270	L/360	l/270	L/360	l/270
Support Spacing														
4	1385	1385	1565	1565	1620	1620	1770	1770	2170	2170	2325	2325	4815	4815
8	390	390	470	470	530	530	635	635	835	835	895	895	1850	1850
12	110	150	145	195	165	225	210	280	375	400	460	490	1145	1145
16	—	—	—	—	—	—	—	120	160	215	200	270	710	725
20	—	—	—	—	—	—	—	—	115	125	145	155	400	400
24	—	—	—	—	—	—	—	—	—	—	—	100	255	255

[a] Use when shear deflection is not computed separately.
Note: All stresses have been increased by 25% for short-term loading.
Source: Concrete Forming, APA Design/Construction Guide, p. 14, Engineered Wood Association, Tacoma, WA.

Rolling shear stress, f_s, in a plywood beam is:

$$f_s = \frac{V}{Ib/Q} \tag{4.5}$$

where Ib/Q is the rolling shear constant. The rolling shear constant for Plyform is shown in Table 4.4.

Allowable Stresses for Plywood

Table 4.5 shows the values of allowable stresses and moduli of elasticity for the three classes of Plyform. Table 4.5 also has load tables showing allowable pressures on Class I Plyform. Like sawn lumber, plywood stresses are adjusted for conditions of use. A load duration factor may be applied to increase the allowable stresses if the plywood loads have a duration of not more than 7 days and if the forms are not for multiple reuse. Stresses in Table 4.5 have been reduced for “wet use” because fresh concrete will be in contact with the plywood form sheathing. The stresses in Table 4.5 also have been increased by 25% for load duration. ACI Committee 347 recommends that the allowable stresses shown in Table 4.5 should be reduced by 25% if the plywood is used in formwork intended for multiple reuse.

These allowable stresses can be used without regard to the direction of the grain. Grain orientation is accounted for in the calculations of the section properties in Table 4.4 that are used to calculate actual stresses.

Ties

Ties are devices used to hold the sides of concrete forms together against the fluid pressure of fresh concrete. Ties are loaded in tension and have an end connector that attaches them to the sides of the form. In order to maintain the correct form width, some ties are designed to spread the forms and hold them at a set spacing before the concrete is placed. Some ties are designed to be removed from the concrete after it sets and after the forms have been removed. These ties take the form of tapered steel rods that are oiled or greased so they can be extracted from one side of the wall. They usually have high strength and are used in heavier panel systems, where it is desirable to minimize the total number of ties.

The removal of ties allows the concrete to be patched. Patching allows for a smoother concrete surface and helps to eliminate the potential for staining from rusting of steel tie ends. Another type of tie is designed to be partially removed by either unscrewing the tie ends from a threaded connector that stays in the concrete or by breaking the tie ends back to a point weakened by crimping. Some ties have waterstops attached. These would be used if ordinary grout patching will not provide a watertight seal. Figure 4.1 shows some of these types of ties.

Nonmetallic ties have recently been introduced. They are produced of materials such as a resin-fiber composite and are intended to reduce tie-removal and concrete-patching costs. Because they are made of nonmetallic materials, they do not rust or stain concrete that is exposed to the elements.

The rated strength of ties should include a factor of safety of 2.0. ACI Committee 347 revised previous recommendations that called for a factor of safety of 1.5 for ties. The new rating matches the factors of safety applied to the design of other form components. When specifying ties, the manufacturer's data should be carefully checked to ensure that the required factor of safety is incorporated within the rated capacity. If older ratings are found that use a 1.5 factor of safety, then use the ties at only 75% of their rated capacity.

Anchors

Form anchors are devices embedded in previously placed concrete, or occasionally in rock, that may be used to attach or support concrete formwork. There are two basic parts to the anchors. One part is the embedded device, which stays in the concrete and receives and holds the second part. The second part is the external fastener, which is removed after use. The external fastener may be a bolt or other type of threaded device, or it may have an expanding section that wedges into the embedded part. Figure 4.2 shows some typical anchors.

ACI Committee 347 recommends two factors of safety for form anchors. For anchors supporting only concrete and dead loads of the forms, a factor of safety of 2 is used. When the anchor also supports construction live loads and impact loads, a factor of safety of 3 should be used.

The rated capacity of various anchors is often given by the manufacturers. Their holding power depends not only on the anchor strength but on the strength of the concrete in which they are embedded. The depth of embedment and the area of contact between the anchor and concrete are also important in determining capacity. It is necessary to use the data provided by the manufacturers, which are based on actual load tests for various concrete strengths, to determine the safe anchor working load for job conditions. This will require an accurate prediction of the concrete strength at the time the anchor is loaded. Estimated concrete strengths at the age when anchor loads will be applied should be used to select the type and size of anchor required.

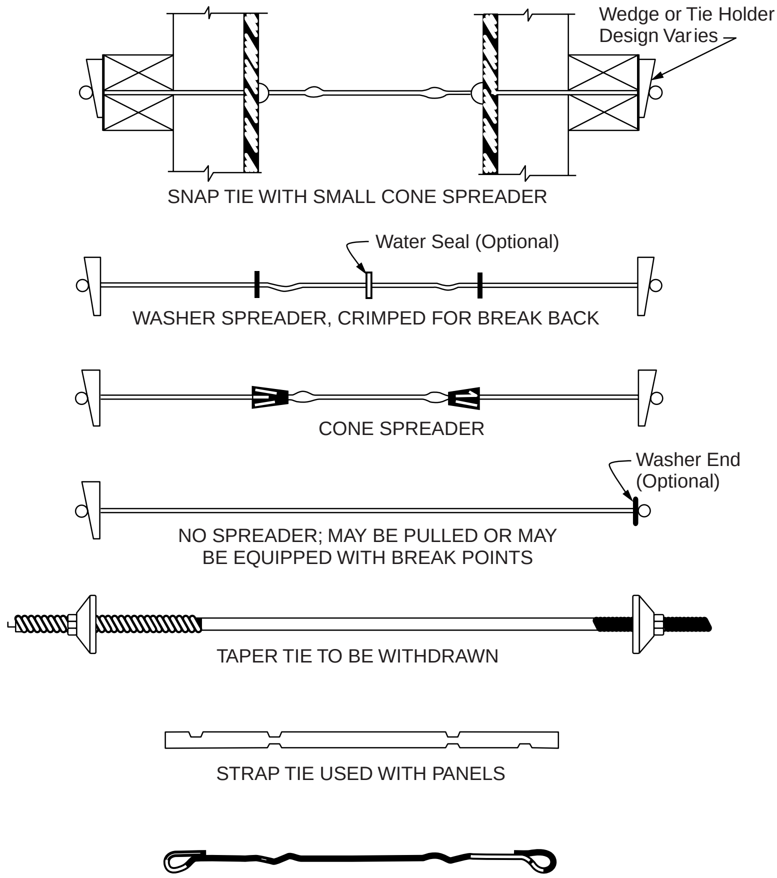


FIGURE 4.1 A drawing showing common types of form ties. (From Hurd, M.K., *Formwork for Concrete*, 6th ed., American Concrete Institute, Detroit, MI, 1989.)

Hangers

Hangers are used to support concrete formwork by attaching the formwork to structural steel or precast concrete structural framing members. Various designs are available, and each manufacturer's safe load rating should be used when designing hanging assemblies. For hangers having more than one leg, the form designer should carefully check to see if the rated safe load is for the entire hanger or each leg. ACI Committee 347 recommends a safety factor of 2 for hangers. Figure 4.3 shows some types of hangers.

Column Clamps

Devices that surround a column form and support the lateral pressure of the fresh concrete are called column clamps. They may be loaded in tension or flexure or in a combination of both. Several commercial types of column clamps designed to fit a range of sizes of column forms are available. Care should be taken to follow the manufacturer's instructions for using their clamps. Rate of placement of the concrete and maximum height of the form may be restricted. Deflection limits for the forms may be exceeded if only the strength of the clamps is considered. Where deflection tolerances are important, either previous satisfactory experience with the form system or additional analysis of the clamp and form sheathing is suggested.

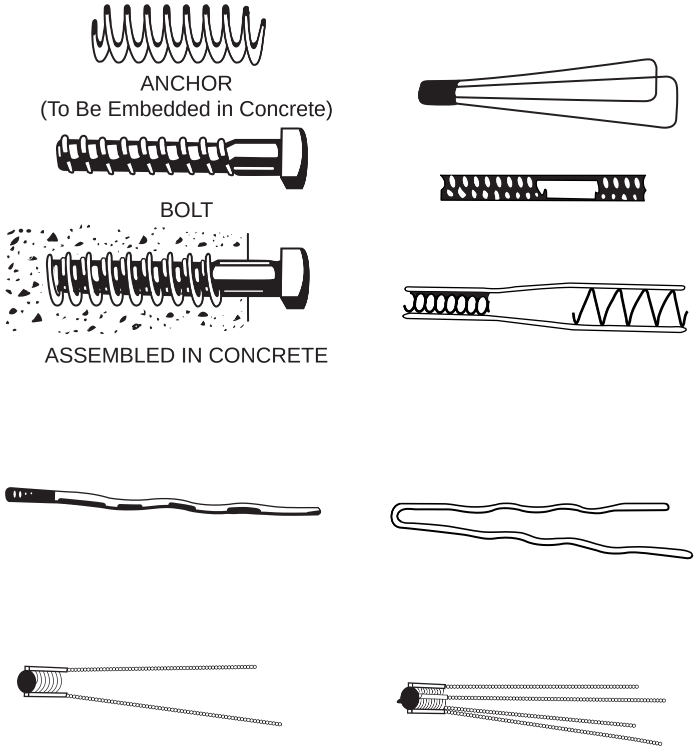


FIGURE 4.2 Common types of anchors used in concrete. (From Hurd, M.K., *Formwork for Concrete,* 6th ed., American Concrete Institute, Detroit, MI, 1989.)

4.4 Loads on Concrete Formwork

Concrete forms must be designed and built so that they will safely carry all live and dead loads applied to them. These loads include the weight and pressure of concrete, the weight of reinforcing, the weight of the form materials and any stored construction materials, the construction live loads imposed by workers and machinery applied to the forms, and loads from wind or other natural forces.

Lateral Pressure of Concrete

Fresh concrete still in the plastic state behaves somewhat like a fluid. The pressure produced by a true fluid is called hydrostatic pressure and depends on the fluid density and on the depth below the surface of the fluid. The hydrostatic pressure formula is:

$$p = \gamma h \tag{4.6}$$

where p is the fluid pressure, γ is the fluid density, and h is the depth below the free surface of the fluid. This pressure, which is due to the weight of the fluid above it, always pushes against the container in a direction perpendicular to the surface of the container.

If concrete behaved like a true fluid, the pressure it would produce on the forms would have a maximum value of

$$p = 150h \tag{4.7}$$

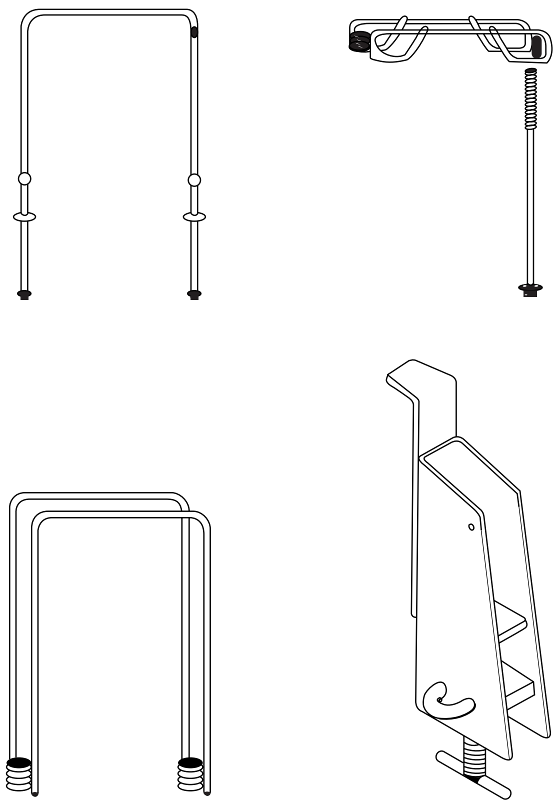


FIGURE 4.3 Examples of types of form hangers used to support formwork from existing structural members.

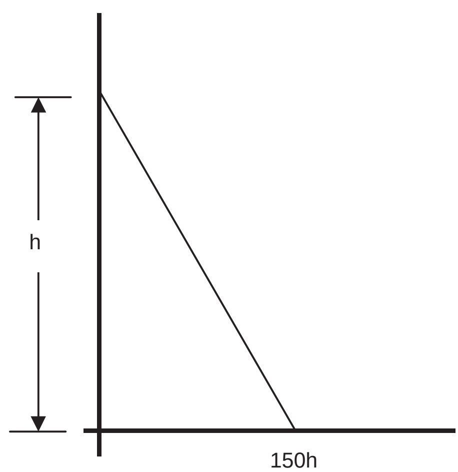


FIGURE 4.4 A diagram showing the hydrostatic pressure distribution in fresh concrete.

for concrete of normal density. Normal-density concrete is usually assumed to weigh 150 pounds for a cubic foot, and h is the depth of concrete in the form. The pressure of the concrete would vary from the maximum pressure of 150h at the bottom of the form to zero at the surface of the concrete. This pressure distribution is shown in Fig. 4.4. For placing conditions where the form is filled rapidly and the concrete behaves as a fluid, the form should be designed to resist this maximum hydrostatic pressure.

There are several factors that affect the degree to which concrete behaves like a true fluid:

- Fresh concrete is a mixture of aggregate, water, cement, and air, and can only approximate fluid behavior.
- The internal friction of the particles of solids in the mixture against each other and against the forms and reinforcing tends to reduce actual pressure to below hydrostatic levels. This internal friction tends to be reduced in a concrete mixture that is wetter, that is, has a high slump; therefore, the pressure will more closely approach the hydrostatic level in high slump mixes.
- The bridging of the aggregate from one side of the form to the opposite in narrow forms also tends to limit pressure. The weight of the concrete tends to be partly supported by the bridging, which prevents the pressure from increasing to true fluid levels.
- In addition to the interaction of the solids in the mixture, the stiffening of the concrete due to the setting of the cement has a marked influence on the pressure. As the concrete stiffens, it tends to become self-supporting, and the increase in pressure on the forms is reduced as the filling continues. Because the setting of the cement can begin in as little as 30 min or in as long as several hours after mixing, the lateral pressure may or may not be changed from the hydrostatic condition for any given filling of a form. The conditions directly affecting the stiffening of the concrete include the concrete temperature, the use of admixtures such as retarders, the amount and type of cement, and the amount and types of cement substitutes, such as fly ash or other pozzolans.

Variables that most directly influence the effective lateral pressure in the concrete are the density of the mixture, the temperature of the mixture, the rate of placement of the concrete in the forms, the use of admixtures or cement replacements, and the effect of vibration or other consolidation methods.

Other factors considered as influences on pressure include aggregate size and shape, size of form cross section, consistency of the concrete, amount and location of the reinforcement, and the smoothness of the surface of the forms. Studies have shown that the effect of these other factors is usually small when conventional placement practices are used, and that the influence of these other factors is generally ignored.

The temperature of the concrete has an important effect on pressure, because it affects the setting time of the cement. The sooner the setting occurs, the sooner the concrete will become self-supporting, and the sooner the pressure in the form will cease to increase with increasing depth of concrete.

The rate of placement (usually measured in feet of rise of concrete in the form per hour) is important, because the slower the form is filled, the slower the hydrostatic pressure will rise. When the concrete stiffens and becomes self-supporting, the pressure in the concrete tends to level off. At low rates of placement, the hydrostatic pressure will have reached a much lower value before setting starts, and the maximum pressure reached at any time during the filling of the forms will be reduced.

Internal vibration is the most common method of consolidating concrete in formwork. When the probe of the vibrator is lowered into the concrete, it liquefies the surrounding mixture and produces full fluid pressure to the depth of vibration. This is why proper vibration techniques are important in avoiding excessive pressure on concrete forms. Vibrating the concrete below the level necessary to eliminate voids between lifts could reliquefy concrete that has started to stiffen and increase pressure beyond expected design levels. The pressures in concrete placed using proper internal vibration usually exceed by 10 to 20% those pressures from placement where consolidation is by other means. When vibration is to be used, the forms should be constructed with additional care to avoid leaking.

In some cases, it is acceptable to consolidate concrete using vibrators attached to the exterior of the forms or to revibrate the concrete to the full depth of the form. These techniques produce greater pressures than those from normal internal vibration and usually require specially designed forms.

Recommended Design Values for Form Pressure

ACI Committee 347, after reviewing data from field and laboratory investigations of formwork pressure, published recommendations for calculating design pressure values. The basic lateral pressure value for freshly placed concrete is as follows:

$$p = wh \tag{4.8}$$

No maximum or minimum controlling values apply to the use of this formula, which represents the equivalent hydrostatic pressure in the fresh concrete. The weight of the concrete, *w*, is the weight of the concrete in pounds per cubic foot, and *h* is the depth of plastic concrete in feet. For forms of small cross sections that may be filled before initial stiffening occurs, *h* should be taken as the full form height.

For concrete made with Type I cement, without pozzolans or admixtures, with a maximum slump of 4 in., with density of 150 pcf, and with placement using proper internal vibration, ACI Committee 347 recommends the following formulas for calculating design pressure values.

Wall Forms

For wall forms that are filled at a rate of less than 7 ft/hr, the maximum pressure is:

$$p = 150 + 9000\frac{R}{T} \tag{4.9}$$

where *p* is the maximum lateral pressure of concrete in the form, *R* is the rate of placement of the concrete in ft/hr, and *T* is the temperature of the concrete in the form in degrees Fahrenheit. For wall forms filled at a rate of between 7 ft/hr and 10 ft/hr, the maximum pressure is

$$p = 150 + \frac{43,400}{T} + 2800\frac{R}{T} \tag{4.10}$$

Design values from Eqs. (4.9) and (4.10) should not exceed 150*h* and 2000 psf. These formulas predict the maximum value of pressure in the wall form during placing. From these maximum values, no prediction should be made about what the pressure distribution is at any given time. An envelope of maximum pressure can be found by considering the concrete to be fully fluid to the depth in the form where the maximum pressure is reached. This depth, where the maximum pressure is reached, is the depth at which the concrete has become self-supporting and, the pressure has ceased to increase.

Example 2

A wall form 12 ft high is filled with normal-weight concrete having a temperature of 70°. The concrete rises during placement at a rate of 5 ft/hr.

Using Eq. (4.9), the maximum pressure is

$$p = 150 + 9000\frac{R}{T}$$

$$p = 150 + 9000\frac{5}{70} = 793 \text{ psf}$$

The hydrostatic pressure for the 12-ft depth is 150(12) = 1800 psf.

Comparing the pressure from the formula to the fully hydrostatic pressure, or the limiting value of 2000, shows that the maximum pressure expected in the wall form at any time during placement is 793 psf.

The envelope of maximum pressure will show a hydrostatic pressure to a depth below the top of the form of 793/150 = 5.29 ft. Figure 4.5 shows the envelope of maximum pressure.

Column Forms

Column forms usually have a smaller total volume than wall forms and will fill faster for a given volume delivery rate of concrete. When the column form fills in a relatively short period of time, it is likely that the concrete will act as a fluid, and the pressure will be fully hydrostatic. Equation (4.8) gives the pressure for this condition.

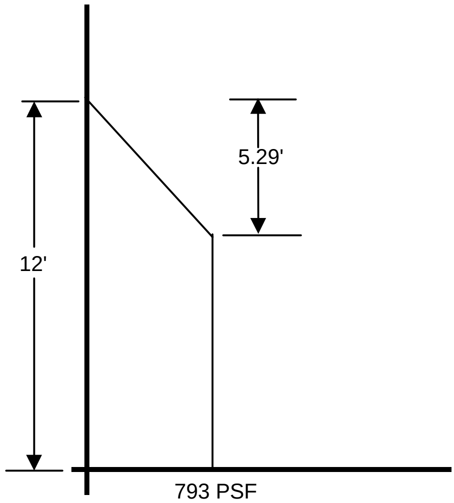


FIGURE 4.5 A diagram showing the envelope of maximum pressure in Example 2.

According to ACI Committee 347 recommendations, when the concrete is made with Type I cement weighing no more than 150 pcf, having a slump of not more than 4 in, having no pozzolans or admixtures, and having been consolidated using internal vibration to a depth of not more than 4 ft below the surface, Eq. (4.9) may be used to calculate the maximum lateral pressure in the form.

$$p = 150 + 9000\frac{R}{T} \tag{4.11}$$

where p is the pressure in psf, R is the rate of placement in feet per hour, and T is the temperature of the concrete in degrees Fahrenheit. This formula is limited to columns where lifts do not exceed 18 ft. The pressure from the formula has a minimum recommended value of 600 psf and a maximum value of 3000 psf.

The pressure distribution in a column form is hydrostatic until the concrete begins to stiffen. For normal-weight concrete, pressure is assumed to increase by 150 psf per foot of depth until the maximum value given by Eq. (4.9) is reached. The pressure then remains constant to the bottom of the form.

Example 3

A 16-ft high concrete column form is filled at a rate of 10 ft/hr with 80° concrete. The maximum pressure, using Eq. (4.11), is

$$p = 150 + 9000\frac{10}{80} = 1275 \text{ psf}$$

and will occur at a depth of

$$1275/150 = 8.5 \text{ feet}$$

below the top of the form.

The envelope of maximum pressure to be used to design the column form is shown in Fig. 4.6.

The formulas recommended by ACI Committee 347 report for pressure in wall and column forms apply when the conditions stated above exist. In some cases, conditions may vary from these standard conditions. Adjustments to the design pressures can be made to account for conditions other than those specified.

These adjustments are explained below.

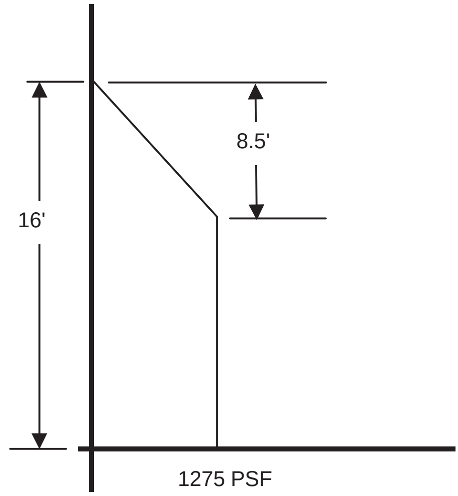


FIGURE 4.6 A diagram showing the envelope of maximum pressure in Example 3.

Consolidation by Spading

Lower pressures result when concrete is spaded rather than consolidated using internal vibration. The pressures from the formulas may be reduced 10% in this situation.

Mixtures Containing Retarders, Fly Ash or Superplasticizers

Concrete that uses retarders, fly ash or other pozzolans, or superplasticizers will have its initial stiffening delayed. Because this will increase the pressure of the concrete on the forms, the forms should be designed using the assumption of a full liquid head [Eq. (4.8)].

Concrete Density

For concrete having densities ranging between 100 pcf and 200 pcf, the pressure can be found by multiplying the pressure from normal-weight concrete (150 pcf) by the ratio of the densities. If the concrete has a density of 200 pcf, its pressure would be 200/150 or 1.33 times the pressure for normal-weight concrete.

Gravity Loads on Formwork

Gravity loads are from all live and dead loads applied to and supported by the formwork. These include the weight of the concrete and reinforcing steel, the weight of the forms, and any construction loads from workers, equipment, or stored materials. Loads from upper floors may also be transferred to lower-level forms in multistory construction. The largest loads are generally due to the weight of the concrete being formed and to the construction live load from workers and equipment. Because the majority of concrete used has a weight of around 140 to 145 pounds per cubic foot (lb/ft^3), it is common to use a value for design of 150 lb/ft^3, which includes an allowance for the reinforcing steel. Where lightweight or heavyweight concrete is used, the density of that particular mix should be used in calculating formwork loads.

Trying to predict what value should be used for construction loads to account for the weight of workers and equipment is difficult. The weights of the workers and equipment would have to be estimated for each situation and their locations taken into account when trying to determine worst-case loadings. As a guide to the designer in ordinary conditions, ACI Committee 347 recommends using a minimum construction live load of 50 psf of horizontal projection when no motorized buggies are used for placing concrete. When motorized buggies are used, a minimum construction live load of 75 psf should be used. The dead load of the concrete and forms should be added to the value of the live load.

The dead load of the concrete depends on the thickness of the concrete element. For every inch of thickness of the concrete, 150/12 or 12.5 psf should be used. The dead load of the forms can vary from a value of 4 or 5 psf to as much as 15 to 18 psf. In some cases, the weight of the forms is small when compared to the other loads and can be safely neglected. When the design of the form is complete and form component sizes are known, the form weight should be calculated and compared to the assumed loads. ACI Committee 347 recommends that a minimum total load for design of 100 psf be used (regardless of concrete thickness) without use of motorized buggies, or 125 psf when motorized buggies are used.

Example 4

A reinforced concrete slab with a thickness of 9 in. is placed with a concrete pump. For normal-weight concrete, find the gravity loads the slab forms should be designed to support.

Dead load of concrete slab = 150(9/12) = 112.5 psf
Dead load of forms (estimate) = 10 psf (check this after design is completed and after member sizes and weights are known)
Construction live load = 50 psf (ACI Committee 347 recommendations)
Total design load for slab form = 173 psf

Lateral Loads

In addition to fluid pressures, formwork must also resist lateral loads caused by wind, guy cable tensions, starting and stopping of buggies, bumping by equipment, and uneven dumping of concrete. Because many formwork collapses can be attributed to inadequate bracing for handling lateral loads, it is important that these loads be properly resisted by an adequate bracing system.

The first step in choosing what bracing is required is to determine the magnitude of the lateral loads created by the effects listed above. When lateral loads cannot be easily or precisely determined, minimum lateral loads recommended by ACI Committee 347 may be used.

Slab Forms

ACI Committee 347 recommends that bracing of the slab forms should be provided to resist the greater of 100 pounds per lineal foot of slab edge, or 2% of the total dead load on the form distributed as a uniform load on the slab edge. When considering dead load, use only the area of the slab formed in a single pour. If slab forms are enclosed, as might be the case in cold-weather operations, the lateral load produced by the wind acting on the forms, enclosure, and any other windbreaks attached to the forms, should be considered. Local building codes should be consulted to estimate the applicable wind loads.

Wall Forms

Bracing for wall forms should resist a minimum load of 100 lb/ft of wall applied at the top of the form, or should resist the wind load prescribed by the local code. The wind load used should be at least 15 lb/ft^2. If the wall forms are located below grade and are not subjected to wind loads, bracing should be designed to be adequate to hold the panels in alignment during concrete placement.

Considerations for Multistory Construction

Formwork loads in multistory construction are sometimes transferred by shores and reshores to the floors below. The shores directly support the slab forms and carry the loads to the level below. When the shores are removed from the slab and the slab forms are stripped, new shores are placed under that slab. These new shores are called reshores. The reshores transfer additional loads applied to the floor slab directly to the level below. The loads in the shores and reshores from the slab may be carried all the way to the ground when the building is only a few stories high or when work on a taller building is still only a few levels above the ground. Otherwise, the loads have to be supported by the lower floors of the building. Depending on the speed of the construction, the loads may be imposed on floors that have not yet reached their full design strength. This understrength, due to lack of curing time combined with the possibility that loads from several floors above may be applied to the floor, can create dangerous overloads or even failures. Most of the failures of concrete buildings occur during the construction phase (Chen and Mosallam, 1991). To avoid a disastrous accident during the construction of a multistory building, careful analysis should be made of the loads in the forms, the shores, the reshores, and the concrete floor systems for each step in the building process. According to ACI Committee 347, the structure's capacity to carry these construction loads should be reviewed or approved by the structural engineer. The plan for the forms and shoring remain the responsibility of the contractor.

The basic method for ensuring that loads applied to any floor level in the building do not exceed that floor's capacity is to use as many levels of shores and reshores as necessary to distribute the loads. With this system, the loads can either be carried through all levels to the ground below, or when the building has too many levels to make that arrangement practical, the latest-applied loads can be distributed through the shores and reshores to several floors simultaneously, so that no one level has a load that exceeds its capacity. The ability of any floor to support construction loads depends on the service capacity the floor was designed to carry as well as the age of the concrete when the construction loads are applied. Because the service capacity is based on the specified minimum 28-day strength, the floor capacity prior to the concrete reaching that strength will be less than this service capacity.

When preparing a forming system for multistory buildings, the sizes and specifications for the forming elements must be selected and the schedule for when the forms will be erected and stripped must be carefully prepared. In planning this kind of system and its schedule of use, consider the following:

- The dead load of the concrete and reinforcing, and the dead load of the forms when significant
- The construction live loads (workers, equipment, storage of materials)
- Design strength of concrete specified
- Cycle time for placing of floors of building
- Structural design load for the floor element supporting construction loads (slab, beam, girder, etc.) — include all loads the engineer designed the slab to carry
- The rate at which the concrete will gain strength under job conditions — use to find the strength of the concrete when loads are applied to it
- The way the loads applied at the different levels are distributed to the floors at the different levels by the elements of the form system

Because of the complex ways the elements of forms for multistory buildings interact with the building elements, and because these interactions may change from one building to another (due to cycle times, concrete properties, weather, different ratios of dead-to-live loads, and many other variables), no single method or procedure for forming and shoring will be satisfactory for all projects.

4.5 Analysis and Design for Formwork

The objective for the formwork designer is to choose a system that will allow concrete elements that meet the requirements of the job to be cast in a safe and economical way. The designer has to choose the materials for constructing the forms and determine the size of the form components, the spacing of all supporting members, and the best way to properly assemble the forms to produce a stable and usable structure. After the materials for constructing the form have been selected and the loads that the form must withstand are determined, the designer must determine how to make the form strong enough to carry the stresses from these loads. The forms must also be proportioned so that deformations are less than those allowable under the specifications and conditions of the job. Many times, the form designer can rely on past experience with other jobs with similar requirements and simply use the same forming system as used before. However, the current job requirements, the materials to be used, or the loading conditions are sometimes different enough to make it necessary for the forms to be designed for the new situation. Form design can be approached in two ways: by relying on tables that give allowable loads for various materials such as lumber and plywood or by following a rational design procedure similar to those used in designing permanent structures. In the rational design procedure, known elastic properties of the materials are used to determine the required member sizes, spacings, and other details of the form system, and established engineering principles are followed.

Simplifying Assumptions for Design

Because of the many assumptions that can be made about loads, job conditions, workmanship, and quality of materials for formwork, too much refinement in design calculations may be unwarranted. The designer's effort to attain a high degree of precision in calculations is usually negated by the accuracy of the field construction and by uncertainties about loads and materials. A simplified approach for the rational design of formwork members is usually justified. The need to make the forms convenient to construct makes choosing modular spacing desirable, even when other dimensions are indicated by calculations. For example, the strength of the plywood sheathing for a wall form may be sufficient to allow a stud spacing of 14 in. For ease of construction and to ensure that the panel edges are supported, a spacing of 12 in. for the studs would probably be chosen, even though it is conservative and requires more supports.

For concrete formwork that supports unusually heavy loads, requires a high degree of control of critical dimensions, or presents unusual danger to life or property, a complete and precise structural design may be essential. In this situation, a qualified structural engineer experienced in this kind of design should be engaged.

The following simplifying assumptions are commonly used to allow more straightforward design calculations:

- Assume that all loads are uniformly distributed. Loads on sheathing and other members directly supporting the sheathing are, in fact, distributed, though not necessarily uniformly distributed. Loads on other form members may be point loads but can usually be approximated by equivalent distributed loads. In cases where the spacing of the point loads is large compared to the member span, bending stresses and deflections should be checked for actual load conditions.
- Beams continuous over three or more spans have values for moment, shear, and deflection approximated by the formulas shown in Table 4.6 for continuous beams.

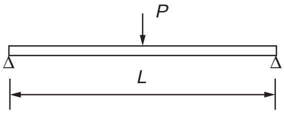
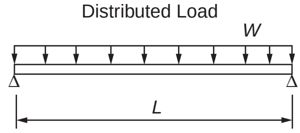
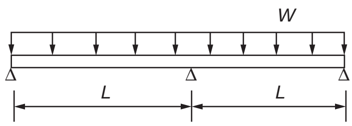
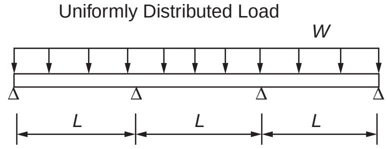
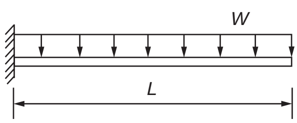
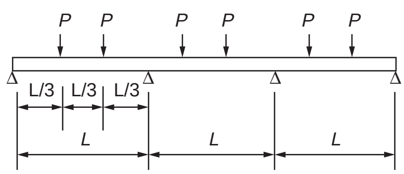
Beam Formulas for Analysis

The components that make up a typical wall form include the sheathing to contain the concrete, studs to support the sheathing, and wales to support the studs. The wales are usually supported by wall ties. For a slab form, the components are similar and are commonly called sheathing, joists, and stringers. The stringers are supported by shores or scaffolding. Other types of concrete forms have similar components. All of the above components, except the wall ties and shores, may be assumed to behave as beams when the forms are loaded. They may be oriented vertically or horizontally and have different kinds of supports, but they all behave similarly and can be designed using beam design principles. Values for bending moment, shear, and deflection for these beams can usually be found by using standard formulas such as those given in Table 4.6. Where the framing of concrete forms is more complex, a more detailed analysis may be required for design. Use of structural analysis programs and a computer will simplify this task.

Stress Calculations

The components of a concrete form should be checked to ensure that the stresses they develop are below safe levels for that material. Allowable stresses, as discussed earlier, depend on types of material and conditions of use. Because concrete forms are considered temporary structures, the form designer can often take advantage of the increased allowable stresses permitted for temporary structures. In cases

TABLE 4.6 Beam Formulas

Simply Supported Beam with Concentrated Load at Center  $M_{max} = \frac{PL}{4}$ $\Delta = \frac{PL^3}{48EI}$ $V_{max} = \frac{P}{2}$	Simply Supported Beam with Uniformly Distributed Load  $M_{max} = \frac{wL^2}{8}$ $\Delta_{max} = \frac{5wL^4}{384\ EI}$ $V_{max} = \frac{wL}{2}$
Two Span Continuous Beam with Uniformly Distributed Load  $M_{max} = \frac{wL^2}{8}$ $\Delta_{max} = \frac{wL^4}{185EI}$ $V_{max} = \frac{5wL}{8}$	Three Span Continuous Beam with Uniformly Distributed Load  $M_{max} = \frac{wL^2}{10}$ $\Delta_{max} = \frac{wL^4}{145EI}$ $V_{max} = .6wL$
Cantilever Beam with Uniformly Distributed Load  $M_{max} = \frac{wL^2}{2}$ $\Delta_{max} = \frac{wL^4}{8EI}$ $V_{max} = wL$	Three Span Continuous Beam with Concentrated Loads at Span Third Points  $M_{max} = .267PL$ $V_{max} = 1.27P$

where the forms are reused and subjected to repeated stripping, handling, and reassembly, it is recommended that the increased allowable stresses permitted for temporary structures not be used.

Bending Stress

The flexural stresses calculated for the formwork components should be compared to the allowable values of flexural stress for the material being used. The basic flexure formula is as follows:

$$f_b = \frac{M}{S} \tag{4.12}$$

where f_b is the extreme fiber bending stress, M is the bending moment in the beam, and S is the section modulus of the beam. This formula can also be applied to lumber, plywood, steel, or aluminum beams.

Shear Stress

Shear force due to the applied loads in a beam develop shear stresses. These are often called horizontal shear stresses. In plywood, this type of stress is called rolling shear stress. Shear stresses should be checked carefully in lumber beams and plywood. For short, heavily loaded spans, shear stress frequently controls the lumber beam or plywood capacity. For steel and aluminum beams, shear stress controls the design less frequently.

The formula for calculating shear stress is

$$f_v = \frac{VQ}{Ib} \tag{4.13}$$

where V is the shear force in the beam at that cross section, I is the centroidal moment of inertia, b is the beam width at the level in the cross section where f_v is desired, and Q is the moment of area of the part of the beam cross section above or below the plane, where stress is being calculated about the beam centroid. Because the maximum value of shear stress is required for formwork design, this formula can be simplified for specific materials. For lumber beams that have a rectangular cross section with width b and height d, the maximum shear stress will occur at the point in the span where the shear force, V, is maximum at the center of the cross section, and it will have the value

$$f_v = \frac{3V}{2bd} \tag{4.14}$$

The maximum shear force, V, in lumber beams may be reduced by removing the load for a distance equal to the beam depth from each beam support. This reduced value is then used to calculate the shear stress. Refer to the *NDS* for details of this reduction.

For plywood, the rolling shear stress is calculated using Eq. (4.5).

For steel W beams, the shear stress is

$$f_v = \frac{V}{dt_w} \tag{4.15}$$

where V is the shear force, d is the depth of the steel beam, and t_w is the thickness of the web.

Bearing Stress

Wood is relatively weak when subjected to compression stresses perpendicular to the grain. Because this is the direction of bearing stresses for lumber beams, these stresses should be compared to allowable values. The bearing stress in a lumber beam is

$$f_{brg} = \frac{R}{A_{brg}} \tag{4.16}$$

where R is the beam reaction and A_{brg} is the bearing area.

Deflections

For the concrete member being constructed to have the specified dimensions, concrete forms must resist excessive deformations. The allowable amount of deformation will depend on job specifications and type of concrete elements being formed. Concrete elements that will not be concealed by other materials will have to be cast using formwork that is rigid enough to prevent perceptible bulges or waves in the finished concrete. When choosing sizes and spacing of elements of the concrete forms, deflections must be controlled and kept below the specified limits. A common way to specify limits of deflection in a formwork member is to require it to be less than some fraction of its span. A frequently used value is 1/360 of the span. A deflection limit may also be a fixed value, such as 1/16 in. for sheathing and 1/8 in. for other form members. If limits for deflections are given by job specifications, the individual members of the form system should be sized to meet these limits as well as the strength requirements. If no deflection limits are specified, form deformations should not be so large that the usefulness of the cast concrete member is compromised.

Deflections for the members of a concrete form can usually be calculated using formulas such as those in Table 4.6.

Example 5

Wall Form Design

A reinforced concrete wall 10 ft, 9 in. tall and 16 in. thick will be formed using job-built panels that are 12 ft high and 16 ft long. The wall will be poured in sections 80 ft long, and the concrete will be placed with a pump having a capacity of 18 yd³/hr. The expected temperature of the concrete is 75°F. The form panels will have sheathing that is 3/4-in.-thick B-B Plyform, Class I. The sheathing will be supported by 2 × 4 lumber studs that are, in turn, supported by horizontal double 2 × 6 lumber wales. The wales are held against spreading by wall ties that pass through the form. See Fig. 4.8. The lumber has allowable stresses (base stress values adjusted for conditions of use) as follows: bending = 1100 psi and shear = 190 psi. The modulus of elasticity for the lumber is 1,500,000 psi. Calculate lateral pressure from concrete.

The rate of placement is R = [volume placed in form (yd³/hr)]/volume of form 1 ft high. The volume of form (1 ft) = (1)(80)(16/12) = 106.7 ft³ = 3.95 yd³. R = 18/3.95 = 4.56 ft/hr. Use Eq. (4.9) to calculate pressure, p:

$$p = 150 + \frac{9000R}{T} = 150 + \frac{9000(4.56)}{75}$$

$$p = 697 \text{ psf}$$

To check full hydrostatic condition: 150h = 150 (10.75)1613 psf; therefore, use 697 psf from formula depth from top of wall to maximum pressure = 697/150 = 4.65 ft. (See Fig. 4.7.)

To find the support spacing for plywood sheathing, assume the sheets of plywood are oriented in the form panel with the face grain across the supports (strong direction). Table 4.5 shows that the plywood will carry a pressure of 730 psf with supports spacing of 12 in. This is also a convenient modular spacing for support of panel edges.

To determine the wale spacing: with a 12 in. stud spacing, the load on each stud will be 697 plf. The studs must be supported by the wales so that the bending stress, the shear stress, and the deflection in the studs do not exceed allowable levels.

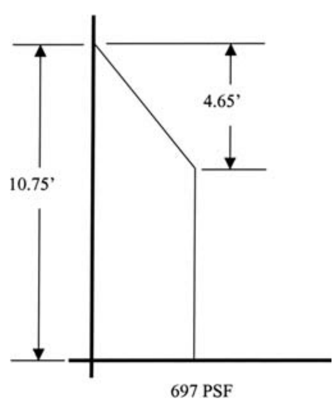


FIGURE 4.7 Sketch for Example 5.

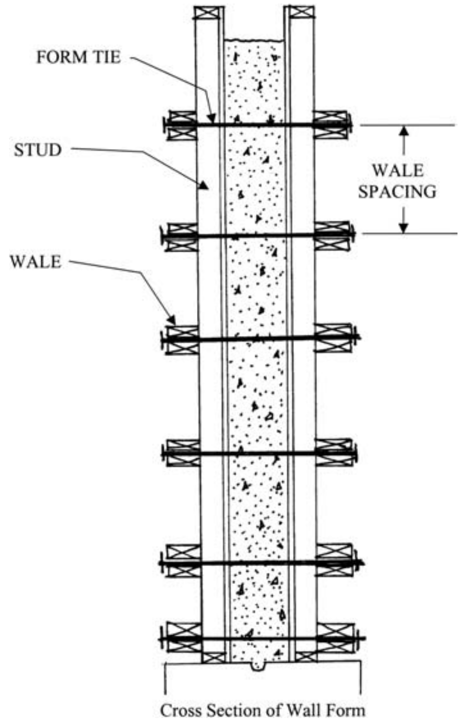


FIGURE 4.8 Cross section views of wall form for sketch in Example 5.

If the studs have three or more spans, the maximum bending moment can be approximated from Table 4.6 as

$$M = \frac{WL^2}{10}$$

Solving for L gives

$$L = \sqrt{\frac{10M}{w}}$$

The allowable bending stress, F_b, gives the allowable bending moment when multiplied by the section modulus, S.

$$S(2 \times 4) = bd^2/6 = 1.5(3.5)^2/6 = 3.06 \text{ in.}^3$$

$$M_{all} = F_b S$$

Substituting the allowable moment,

$$L = \sqrt{\frac{10F_bS}{w}} = \sqrt{\frac{10(1100)(3.06)(12 \text{ in./ft})}{697}} = 24.1 \text{ inches}$$

For convenience in layout, use a 2-ft wale spacing.

To determine deflection, assume the specifications limit deflections to l/360:

$$\Delta = \frac{wL^4}{145EI} \le \frac{L}{360} \Rightarrow \frac{697(2)^4\left(1728\text{ in.}^3/\text{ft}^3\right)}{145(1.5)(10)^6(5.36)} = \frac{2(12)}{360}$$

$$0.0165 < 0.0667 \qquad \text{deflection is OK}$$

To determine shear, from Table 4.6, obtain the maximum shear for a continuous beam:

$$V = 0.6w$$

The NDS allows reduction of shear at the ends of a wood beam by removing the load for a distance d. This gives

$$V = 0.6w\left[L - \frac{2d}{12}\right]$$

$$V = 0.6(697)\left[2 - \frac{2(3.5)}{12}\right] = 592\text{ pounds}$$

$$f_v = \frac{3V}{2bd} = \frac{2(592)}{2(1.5)(3.5)} = 169\text{ psi} < 190\text{ psi} \qquad \text{shear is OK}$$

A 2-ft wale spacing is therefore OK.

For wales, determine tie spacing and size of tie required. Loads on wale are actually point loads from studs but are often treated as distributed loads. For a 2-ft wale spacing, the equivalent distributed wale load, w, is

$$2(697) = 1394\text{ plf}$$

For 2 × 6 double wales, the section properties are

$$S = \frac{bd^2}{6} = \frac{2(1.5)(5.5)^2}{6} = 15.13\text{ in.}^3$$

$$I = \frac{bd^3}{12} = \frac{2(1.5)(5.5)}{12} = 41.6\text{ in.}^4$$

Wales are 16 ft long and act as continuous beams:

$$L^2 = \frac{10F_bS}{w} = \frac{10(1100)(15.13)(12\text{in./ft})}{1394} = 1432\text{ in.}^2$$

$$L = 37.9\text{ inches} = \text{tie spacing}$$

Use 3-ft tie spacing for convenient layout of panels.

Check deflection:

$$\Delta = \frac{wL^4}{145EI} \le \frac{L}{360} \Rightarrow \frac{1394(3)^4\left(1728\text{ in.}^3/\text{ft}^3\right)}{145(1.5)(10)^6(41.6)} = .0216\text{ in.}$$

$$\text{L}/360 = 360/360 = .100\text{ in.} > .0216\text{ in. deflection is OK}$$

Check shear:

$$V = 0.6w\left[L - \frac{2d}{12}\right] = 0.6(1394)\left[3 - \frac{2(5.5)}{12}\right] = 1734 \text{ lb}$$

$$f_v = \frac{3V}{2bd} = \frac{3(1743)}{2(2)(1.5)(5.5)} = 158 \text{ psi} < 190 \text{ psi} \qquad \text{shear is OK}$$

Find the required tie size: load on tie = tie spacing × wale load = 3(1394) = 4182 lb. The tie must have a safe working capacity of at least 4200 lb; the best choice is probably a 5K (5000 lb) tie.

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Further Information

For information about designing and using concrete forms in a safe and economical way, see *Guide to Formwork for Concrete*, reported by Committee 347 of the American Concrete Institute (ACI). This is included in the sixth edition of the comprehensive work by Hurd, *Formwork for Concrete*, published by the ACI (1989).

For recommendations and guidance in using plywood for concrete forming, see *Concrete Forming*, a design/construction guide published by the Engineered Wood Association (1988).

For information about safety requirements in concrete operations, including design and construction of formwork, see *Safety and Health Regulations for Construction* as it appears in the United States Occupational Safety and Health Act 1988 revision.

For a comprehensive discussion of techniques for analysis and design of concrete formwork, see *Concrete Buildings, Analysis for Safe Construction* by Chen and Mosallam (1991). This book is especially helpful, with its discussion of the treatment of formwork for multistory concrete buildings. It includes a matrix structural analysis procedure and a simple hand calculation procedure to estimate the distribution of loads between floor slabs during construction.