Design and Construction of Concrete Formwork

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Forms are extremely important in concrete construction. They mold the concrete to the required size and shape while controlling its position and alignment (Figure 7.1). Forms are self-supporting structures that are also sufficient to hold the dead load of the reinforcement and fresh concrete and the live load of equipment, workers, and miscellaneous materials (Figure 7.2). In building and designing formwork, three major objectives must be considered:

1. **Quality**—Forms must be designed and built with sufficient stiffness and accuracy so the size, shape, position, and finish of the cast concrete are attained within the required tolerances.
2. **Safety**—Forms must be built with sufficient strength and factors of safety so they are capable of supporting all dead and live loads without collapse or danger to workers and to the concrete structure.
3. **Economy**—Forms must be built efficiently, minimizing time and cost in the construction process and schedule for the benefit of both the contractor and the owner.
Economy is important because the costs of formwork often range from 35 to 60% or more of the total cost of the concrete structure. Considering the impact of formwork on total cost, it is critical that the structural engineer of the facility also design the facility structure for economy of forming, not just for economy of the materials in the finished structure.

Ideally, the builder will achieve maximum economy with no cost to either safety or specified quality. In designing formwork, the construction engineer can reduce costs by carefully considering the materials and equipment to be used; the fabrication, erection, and stripping procedures; and the reuse of forms. However, economy measures that result in either formwork failure or poor-quality products that require (often expensive) modification are self-defeating.

Correctly designed formwork will ensure that the concrete maintains the desired size and shape by having the proper dimensions and being rigid enough to hold its shape under the stresses of the concrete. It must be stable and strong to keep large sections of concrete in alignment (Figure 7.3). Finally, formwork must be substantially constructed so it can be reused and frequently handled while maintaining its shape. Formwork must remain in place until the concrete is strong enough to carry its own weight. In addition, the surface finish of the concrete is dependent on the contact material of the form.

The quality of the formwork itself has a direct impact on safety, accidents, and failures. A floor formwork system filled with wet concrete has its weight at the top and is not inherently stable. As a result, one of the most frequent causes of failure is from effects that induce lateral forces or displacement of supporting elements; therefore, inadequate cross-bracing or horizontal bracing is one of the most frequently involved factors in formwork failure. Poor bracing can make a minor failure turn into a major disaster, in what might be thought of as a domino effect or a **progressive failure**: A failure at one point in the formwork that can become an extensive collapse through chain reaction. Vibration is one factor that can trigger failure through inadequate bracing. Two other formwork problems are unstable soil under mudsills and shoring that is not plumb. Formwork is stable if adequately braced and built so all loads are carried to solid ground through vertical and bracing members.
Regardless of the quality of the formwork, premature removal of the forms or shores, often out of a wish for economy, can result in collapse or sagging. Sagging, while not an immediate problem, can lead to hairline cracks and extensive maintenance problems. Careless reshoring, often involving inadequate size, spacing, or attachment, can also cause damage or collapse. Specific related standards (e.g., OSHA, ACI, ASCE) for formwork are discussed in Section 7.3.

In addition to optimizing material in the form design process, there are three major factors to be considered when planning forms that are cost effective:

- Designing and planning for maximum reuse
- Economical form assembly
- Efficient setting and stripping

Each factor must be balanced with the other two to determine the most efficient form design.

In planning for maximum reuse, the specifications, rate of concrete strength gain, and local code requirements regarding stripping must be taken into account. The sooner a form can be stripped safely, the more practical it is to reschedule many reuses. In addition, for a minimum of cost, the least number of forms required for a smooth work schedule should be built. For example, the formwork on the outside of a spandrel beam can be stripped sooner than the formwork on the bottom; hence, fewer side forms than bottom forms must be built because they can be reused more frequently. It is also important to consider the labor involved in reuse; for example, does the form need to be disassembled and rebuilt? To create a plan for reuse, the construction engineer needs to make a detailed study of the work flow and construction sequence to determine the practical number of reuses that will result in smooth and efficient construction with the lowest total cost (Figure 7.4). When comparing designs, the construction engineer should calculate the ratio of total contact area of the formed concrete structure to the first-use form contact area for various alternatives, which is a general indication of overall reuse efficiency.
Several considerations are involved in determining an economical form construction, such as:

- Cost and feasibility of adapting materials on hand vs. cost of buying or renting new materials
- Cost of a higher grade of material vs. cost of lower grade of material plus labor to improve for required quality and use
- Selection of more expensive materials that provide greater durability and capability for reuse vs. less expensive materials that have a shorter use-life
- Building on-site vs. building in a central shop and shipping to site (this depends on the site itself and space available, the size of project, the distance of shipping, etc.)

An estimate of the cost of constructing a particular job-built form can be obtained by determining the quantity of wood materials required to make 1 square foot of contact area, while allowing for waste and rejection of some wood, and multiplying by the unit prices of lumber involved. This provides the contractor with an estimate of lumber costs for 1 square foot of contact area. In addition to lumber, the costs for labor, hardware, miscellaneous materials, handling, and clean-up must also be evaluated. The construction engineer should also consider the possibility of using prefabricated forms, either rented or purchased. The advantages of using rented prefabricated forms are reduced risk, no investment cost, and transfer of some management responsibility to the formwork supplier or subcontractor.

The last major factor, efficient setting and stripping practices, has a direct impact on the two already discussed. Reuse of a form is only fully efficient if the form can be stripped and rebuilt without too much labor time or damage to the form. The estimate for constructing a form must take into account the worker hours required to erect and dismantle the form during each reuse. When calculating time and cost for setting and stripping forms, the contractor should allow for delays from weather, equipment problems, etc., as well as cleaning and other miscellaneous expenses.

In addition to the above elements of cost, planning of formwork operations should consider the overall flow of operations, including the following:

- Crew efficiency—Providing a reasonable schedule creates a smooth daily repetition of the same operation so the workers can be familiar with their tasks and thus perform efficiently.
- Concreting—The ease and speed of pouring the concrete are directly related to the choice of design and the construction schedule.
- Bar setting and other trades (mechanical, electrical, piping)—Schedules of these activities must be coordinated with the concreting schedule so that all groups can work efficiently.
- Cranes and hoists—Plan to use appropriate cranes at times when they are not needed for other functions and reduce idle time.

### 7.2 Types of Formwork

Formwork components can be assembled in a wide variety of systems for casting many structural shapes. The terms formwork and falsework are often used in combination. Formwork is the total system of support for freshly placed concrete and includes the sheathing that is in contact with the concrete as well as all supporting members, hardware, and necessary bracing. Falsework is a temporary structure erected to support work in the process of construction. Falsework may be the temporary support for steel bridge girders, for precast concrete elements to be post-tensioned together, or for many other applications. When this term is used in relation to formwork, the forms are often considered to be the horizontal system of elements directly under heavier concrete placements, such as cast-in-place bridges (Figure 7.2), and the falsework includes the temporary girders, shores or vertical posts, and lateral bracing. Forms can either be job built or prefabricated. Job-built forms, frequently of wood (Figure 7.5), are most frequently selected where the shape does not conform to the constraints of commercial systems or where the economics are viable. Prefabricated forms can be purchased, rented, or rented with an option to buy. They are usually constructed substantially for the purpose of frequent reuse. These forms can either be ready made or custom made (Figure 7.6). The latter is designed for specialized use, usually on a single job, but is often reused multiple times on that project.
7.2.1 Contact Surface Materials

The material serving as the contact face of forms is known as sheathing and sometimes is referred to as lagging or sheeting in specialty applications. Plywood is frequently used for sheathing, but some forms use steel sheet metal, steel plate, fiber-reinforced plastic, paperboard, wood boards, or other materials.

FIGURE 7.5 Wood column form.

FIGURE 7.6 Custom-made steel form with integrated access platforms.
A primary characteristic in selection of the sheathing type and grade is the quality of surface required by the specification. For some applications, steel or high-density overlay plywood may be needed. In other cases where a decorative surface is required, the sheathing may be specially treated (wire brushing to expose wood grain, addition of rustication strips, etc.) or fitted with a commercial plastic liner imprinted or shaped to provide a specified design (Figure 7.7). Permanent forms are any form that remains in place after the concrete has developed its design strength. The form may or may not become an integral part of the structure. Metal deck forms, the most prevalent permanent form, are made of a ribbed or corrugated steel sheet, usually galvanized to reduce the potential for future rust staining, and are secured to the structure with clips or by welding. The diaphragm created by the attached deck may also contribute to the lateral stability of the supporting members during concrete placement. Metal deck forms are used in floor and roof slabs cast over steel joists or beams, in bridge decks (Figure 7.8), and over pipe trenches and other inaccessible locations where removing wood forms is impractical. Precast concrete deck forms are often used in combination with prestressed concrete bridge girders. Sometimes
the deck forms must be temporarily shored at intermediate points to support the loads applied during construction; however, deck forms of adequate section profile can often be selected to span between the permanent structural members and safely support the weights of reinforcement, fresh concrete, and construction live loads. In the latter case, the added cost of the stronger section is often offset by savings in shoring materials and labor.

### 7.2.2 Floor-Forming Systems

Floor-forming systems vary somewhat with the configuration of the concrete floor system being cast. Figure 7.9 illustrates the basic wood member arrangements for flat-plate floors, most areas of flat slabs, and the slab areas of slab and beam floors. The terms used to describe the members are the same in systems assembled from steel or aluminum members. The joist is a horizontal structural member supporting the deck-form sheathing and usually rests on stringers or ledgers. Stringers are horizontal structural members usually (in slab-forming) supporting joists and resting on vertical supports such as shores.

![Figure 7.9](image-url)
In one-way (pan joist) and two-way (waffle) joist construction, a similar layout is usually adopted. **Pans** and **domes** (Figure 7.10) are used in concrete joist construction, which is a cast-in-place floor system with a thin slab integral with regularly spaced joists that span to beams and columns. In some cases, the pans or domes are placed on the plywood sheathing; in other cases, the pan or dome edges are supported on wide joists, and the sheathing is omitted. Pans are prefabricated form units, usually steel or fiberglass, used to form single-direction joists. Domes, also usually made of steel or fiberglass, are square pan forms used in two-way, or waffle, concrete joist construction.

Steel and aluminum joists and stringers are usually flanged shapes. Some aluminum extrusions have special configurations allowing easy connection and incorporating a top channel for a wood nailer. Commercial steel sections fabricated specifically for formwork systems also incorporate connection features. **Horizontal shoring** is formed by adjustable beams, trusses, or combinations of the two that support formwork over clear spans and eliminate numerous vertical supports. Metal or timber support beams are used for small spans. Telescoping shores, steel lattice, plate, or box members are used to support forms in spans of 6 to 30 feet. Heavy-duty horizontal shores (for example, trusses supporting flying-form panels) can span up to 80 feet. The disadvantage of using horizontal shoring is the potential need for special bearing plates to support the high end load on each individual shore.

**Flying forms**, or table forms, are large crane-handled sections of floor formwork (frequently including supporting truss, beam, or scaffolding frames) that are completely unitized. Such forms can be lowered for clearance under joists or beams, rolled out the face of the building bay, picked by a crane, and reset at the next floor level. By having a large movable unit, the costs of stripping and reassembly are reduced, particularly when a crane is available on site.

### 7.2.3 Column Forms

Column-form materials tend to vary with the column shape. Wood or steel is often used with square or rectangular columns (Figure 7.11). Round column forms (Figure 7.12), more typically premanufactured in a range of standard diameters, are available in steel, paperboard, and fiber-reinforced plastic. Square and rectangular forms are composed of short-span bending elements contained by external ties or clamps. Round column forms are more structurally efficient because the internal concrete pressures can be resisted by a hoop membrane tension in the form skin with little or no bending induced. Round, single-piece glass-fiber-reinforced plastic tubes with a single joint can be removed from the column without cutting. They are held together with either bolts or clamps. Round paperboard tubes are single-use forms that are stripped by unwrapping and then discarded. They can be cut to the exact length needed, and sections of the tube can be adapted to making partial column sections (e.g., half-round, quarter-round). Steel column forms have built-in bracing for short heights so the only external bracing required serves to keep the column plumb and for taller columns. Both half-round and rectangular panel units are available in various section heights that can be connected vertically to form tall columns. Round steel forms are generally used for larger columns and bridge piers and come in diameters ranging from about 14 inches to 10 feet.
7.2.4 Wall Forms

Wall forms principally resist the lateral pressures generated by fresh concrete as a liquid or semi-liquid material. The pressures can be quite large, certainly many times the magnitude of live loads on permanent floors. Thus, wall form design often involves closely spaced and well-supported members, as shown in Figure 7.13. As mentioned, the contact surface of the wall form is referred to as sheathing. Studs are
vertical supporting members to which sheathing is attached. **Wales** are long horizontal members (usually double) used to support the studs. The studs and wales are often wood, steel, or aluminum beam-like elements. Commercial form suppliers are innovative in devising elements as well as hardware for connections. The wall form members are sometimes oriented with the stud members placed horizontal rather than vertical, and the wales are run vertical.

The wales are in turn supported on washer plates or other bearing devices attached to form ties. A concrete form tie is a tensile unit connecting opposite sides of the form and providing a link for equilibrium. Form ties are usually steel, although some fiber-reinforced plastic ties are also available. The ties come in a wide range of types (Figure 7.14) and tension working capacities rated by the manufacturer. Snap ties, loop ties, and flat ties are single-use ties, usually of relatively low capacity (1500 to 3200 lb) that are twisted and snapped off a specified distance back from the concrete surface. Coil ties, she bolts, and he bolts are examples of ties where some parts are left embedded within the cast wall and some parts can be reused. The taper tie, a tapered rod threaded on each end, is completely removed and reused. The tension capacity of heavy ties can range upward to over 60,000 lb. Some of the ties have built-in provisions for spacing the forms a definite distance apart; this is particularly true of single-use ties if this feature is ordered. An alternative means of maintaining the correct inside distance is by means of a spreader, a strut (usually of wood) inserted inside the forms that can be retrieved with an attached rope or wire when the concrete placement reaches that level.

**Strong-backs**, frames attached to the back of a form or additional vertical wales placed outside horizontal wales, are sometimes added for strength, to improve alignment, or to assemble a ganged form. **Gang forms** (Figure 7.15) are prefabricated panels joined to make a much larger unit for convenience in erection, stripping, and reuse; they are usually braced with wales, strong-backs, or special lifting hardware. Such units require the use of a crane for stripping and resetting.

**Panel forms**, sections of form sheathing constructed from boards, plywood, metal, etc. that can be erected and stripped as units, are primarily used in wall construction. They may be adapted for use as slab or column forms. The four basic types of panel forms are unframed plywood, plywood in a metal frame (Figure 7.16), all-metal, and heavier steel frame. The first two are most frequently used for general light
Construction and erecting walls with heights ranging from 2 to 24 feet. Both are sometimes backed by steel braces. All-metal panel forms can be made of either steel or aluminum. The heavier steel frame panel forms are faced with either wood or plywood and are used for projects involving large pressures or loads.

**Slip forms** are forms that move, usually continuously, during placing of the concrete. Movement may be either horizontal or vertical. Slip-forming is like an extrusion process, with the forms acting as moving dies to shape the concrete. In wall forming, the slip form is usually moved vertically at a rate of 6 to 12 inches per hour. This method can be economical when constructing concrete cores of tall buildings, tall concrete stacks, and concrete towers.

**FIGURE 7.14** Examples of form ties. (From Hurd, M.K., *Formwork for Concrete*, 7th ed., SP-4, American Concrete Institute, Farmington Hills, MI, 2005.)
Climbing forms, or jump forms, are forms that are raised vertically for succeeding lifts of concrete in a given structure, usually supported by anchors embedded in the previous lift. The form is moved only after an entire lift is placed and (partially) hardened; this should not be confused with a slip form that moves during placement of the concrete. Support of the climbing form is usually provided by anchors cast in the previous placements. It is critical that the concrete strength gain at the anchors be sufficient at each stage of the operation to resist the imposed loads.
Although the use of proper support chairs for reinforcement is routinely required in horizontal construction, similar elements are sometimes neglected in wall and column forms. Side form spacers are devices similar to chairs that can be used to advantage in wall and column forms to attain correct cover for the reinforcement.

7.2.5 Shoring

Shores are vertical or inclined column-like compression supports for forms. Shoring systems may be made of wood or metal posts, scaffold-type frames, or various patented members. Scaffolding is an elevated platform to support workmen, tools, and materials. In concrete work, heavy-duty scaffolding is often adapted to double as shoring. The simplest type of vertical shore is a 4 × 4 or 6 × 6 piece of lumber with special hardware attached to the top to facilitate joining to the stringers with a minimum of nailing. Metal shore-jack fittings may be placed at the lower end of the shore to allow some adjustment for exact height. Various manufacturers sell all-metal adjustable shores, also known as jack shores or simply as jacks, in a number of designs. They are usually available in adjustable heights from 6 to 16 ft and can carry working loads ranging from 2500 to 9000 lb with a safety factor of 2.5, depending on the type and the length of the shore. A third type of vertical support is a device that attaches to a column or bearing wall of the structure. Components of this kind of shore include friction collars and shore brackets that are attached to the support with through bolts or heavy embedded anchors. These attached supports are particularly useful in supporting slab-forming systems.

Basic scaffold-type shoring is made from tubular steel frames. End frames are assembled with diagonal bracing, locking connections, and adjustable bases to create a shoring tower. These may have flat top plates, U-heads, or other upper members for attaching to supported forms. Most scaffold-type shoring has a safe working load between 4000 and 25,000 lb per leg, depending on the height, bracing, and construction of the tower (Figure 7.17). Ultra-high-load shoring frames can support up to 100,000 lb of load per leg. Scaffold-type shoring can also be made of tubular aluminum, which has the advantage of being more lightweight than a similar steel frame.

In multistory concrete building construction (Figure 7.4), a process called shoring and reshoring is used (ACI Committee 347, 2005). The weight of the fresh concrete, reinforcing, forms, and construction live load for an individual floor usually exceeds the design live-load capacity of the floor below. Furthermore, that floor has not gained full 28-day strength, as floors are often cast at intervals of 4 to 14 days. By interconnecting several floors with shores and reshores, the loads at the top can be distributed over several floors. When this construction process is engineered and controlled properly, the time-varying loads on all elements (floors, forms, shores, and reshores) are safely within their time-varying strengths. One or more sets of floor forms with shores may be used. After the forms and shores are stripped completely from the lowest form-supported floor, that floor and the cast floors above deflect and must share the equivalent of the support that has been removed. Reshores are then placed under the stripped floor. Reshores are shores placed snugly under a concrete floor so future loads imposed from construction at the highest level can be shared over sufficient floors to carry the dead and live loads safely.

In some multistory construction, this process is varied slightly. Backshores are shores placed snugly under a concrete slab or structural member after the original forms and shores have been removed from a small area without allowing the slab or member to deflect or support its own weight or existing construction loads from above. Preshores are added shores placed snugly under selected panels of a deck-forming system before any primary (original) shores are removed. Preshores and the panels they support remain in place until the remainder of the bay has been stripped and backshored, a small area at a time.

Shoring systems produced by some manufacturers include a drophead mechanism (Figure 7.18). The systems include panels, beams, and a shore with a top plate that is in direct contact with the concrete slab underside. When the drophead mechanism is released, the beams and panels can be lowered and removed while the shore stays in place. This allows rapid reuse of the panels and beams while the shore continues to support the early-age, low-strength slab. Overall, this approach allows rapid reuse of components, increasing speed of construction and reducing forming materials needed. Keeping the shores
in place longer under the most recently placed slab also increases safety. After appropriate slab strength gain to support the slab self-weight and construction loads, the slab is activated by releasing the shores and then resnuggling them to act as reshores.
Centering is a specialized temporary vertical support used in construction of arches, shells, and space structures where the entire temporary support is lowered (struck or decentered) as a unit to avoid introduction of injurious stresses in any part of the structure. Shores that are supported on the ground must have a temporary footing (termed, in formwork, a mudsill) that is of adequate strength and size (Figure 7.19). The mudsill may be a plank, wood grillage, or precast pad, depending on the loads and ground conditions.

7.2.6 Bracing and Lacing

A brace is any structural member used to support another, always designed for compression loads and sometimes for tension under special load conditions. In formwork, diagonal bracing is a supplementary (not horizontal or vertical) member designed to resist lateral load. Form braces are frequently made of wood or steel. Commercial steel pipe braces in various diameters and wall thicknesses and load-rated for adjustable lengths are popular. Buckling strength of braces is always a primary design consideration. Horizontal lacing, horizontal members attached to shores or braces to reduce their unsupported length, can thus increase the available load capacity. Both bracing and lacing must be adequately connected at each end. This can be accomplished with bolts, nails, and a variety of commercial devices, depending on the materials involved. When attaching braces to the ground, a buried or above-ground concrete mass known as a deadman is sometimes used (Figure 7.20).

7.2.7 Other Forms and Components

The above represents only a brief summary of some of the most typical form elements. Many other configurations must be considered in many jobs. Beam forms are somewhat like short wall forms in that lateral pressures must be resisted; however, they also involve concentrations of vertical load, requiring strong bottom forms and more shoring. The casting of footings sometimes requires forming where the concrete cannot be cast against vertical earth sides. This forming often must be braced from the outside to hold the lateral pressure of the earth if the width of the foundation is large. Supplementary forming elements such as templates are often incorporated in foundation forms to precisely locate anchor bolts and dowels (Figure 7.21). Where concrete elements must be subdivided into two or more placement sections, a form bulkhead is usually placed, either as a construction joint or as an end closure. The bulkhead, although involving short spans, must be carefully designed and connected as it must resist the same pressure magnitudes as the faces of the form.
Forms may also incorporate a host of other features. **Chamfer strips** are triangular or curved inserts placed in the inside corner of a form to produce a rounded or beveled corner. These are often specified in rectangular columns and at outside corners of walls. **Cleanouts** are openings sometimes provided at the bottom of wall or column forms for removal of refuse before the concrete is placed to ensure a good construction joint. There must be a means of closing and supporting the cleanout door to resist concrete pressure. **Wrecking strips** are small pieces or panels fitted into a formwork assembly in such a way that they can be easily removed ahead of the main panels or sections, making it easier to strip the major form components. Various references, such as Hurd (2005) and form-supplier catalogs, provide numerous illustrations of formwork details.

### 7.3 Formwork Standards and Recommended Practices

Ultimately, formwork safety is dependent on the system in place on individual projects to ensure proper and safe design, fabrication, handling, erection, inspection, monitoring, and stripping of the forms and supports (Figure 7.22). As noted earlier, formwork materials and labor are roughly equal in cost to the concrete and reinforcing materials and placing labor. The loads supported by formwork
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Concrete loads (say, for an individual floor) are usually similar in magnitude to the loads supported by the finished structure and are sometimes much greater. Thus, it is justifiable for the design and planning of formwork to require the significant time of a professional, just as is required for design of the structure of the facility being built. Savings can accrue from a well planned and designed system. This section summarizes some of the resources available to the construction engineer to guide the planning and design of formwork. Most of the resources are in the form of guides or recommended practices. Except for some provisions of OSHA, there has previously been no uniformly mandated standard in the United States for design of temporary structures such as formwork; however, this is in the process of change, as is noted in the following sections. As in all engineering work, the construction engineer should seek the most recent edition of the following resources. Most standards are updated or reconfirmed on a cycle of 6 years or less.

7.3.1 American Concrete Institute Recommendations

Prior to about 1958, formwork was designed based on only limited data and guidance for loads. At that time, recommendations for loads and pressures for the design of formwork became available from the American Concrete Institute (ACI). These recommendations have evolved over the years and are available in three often-consulted and periodically updated ACI publications:

- Guide to Formwork for Concrete, ACI 347-04 (ACI Committee 347, 2004)
- Formwork for Concrete, 7th ed., ACI SP-4 (Hurd, 2005)
- Guide for Shoring and Reshoring of Multistory Concrete Buildings, ACI 347.2R (ACI Committee 347, 2005)

The first provides the recommended practice for design and construction of formwork, including recommendations for loads and pressures. The second is a manual that extensively describes systems and provides design procedures, design aids, and examples. The third provides methods for determining shore, reshore, and early-age slab loads during multistory building construction. ACI also publishes, through its journals and other monographs, numerous articles on concrete formwork as well as guides for craftspersons involved in formwork fabrication and erection. ACI Committee 347 recommendations for loads and pressures to be applied in the design of formwork are discussed more thoroughly in Section 7.4 of this chapter.
7.3.2 OSHA Standards

By the early 1970s, legislation to establish the Occupational Health and Safety Administration (OSHA) began to have an impact on formwork. OSHA expectations not only include an adequate temporary structure but also emphasize components for worker fall protection (Figure 7.23 and Figure 7.24) and procedures for safe handling and erection of formwork. Some states have their own requirements that are different from, but cannot be less restrictive than, the federal requirements. Sections of the federal OSHA Occupational Safety and Health Standards for the Construction Industry (29 CFR, Part 1926) particularly affecting formwork planning, design, and execution includes the following requirements:

- Subpart L—Scaffolding
- Subpart M—Fall Protection
- Subpart Q—Concrete and Masonry Construction
Subpart Q contains sections on scope, application, and definitions; general requirements; equipment and tools; cast-in-place concrete; precast concrete; lift-slab operations; and masonry construction. Of primary interest here is the following section on cast-in-place concrete:

§ 1926.703 Requirements for cast-in-place concrete.

(a) General requirements for formwork.

(1) Formwork shall be designed, fabricated, erected, supported, braced, and maintained so that it will be capable of supporting without failure all vertical loads that may reasonably be anticipated to be applied to the formwork. Formwork which is designed, fabricated, erected, supported, braced, and maintained in conformance with the Appendix to this section will be deemed to meet the requirements of this paragraph.

(2) Drawings or plans, including all revisions, for the jack layout, formwork (including shoring equipment), working decks, and scaffolds, shall be available at the jobsite.

(b) Shoring and reshoring.

(1) All shoring equipment (including equipment used in reshoring operations) shall be inspected prior to erection to determine that the equipment meets the requirements specified in the formwork drawings.

(2) Shoring equipment found to be damaged such that its strength is reduced to less than that required by § 1926.703(a)(1) shall not be used for shoring.

(3) Erected shoring equipment shall be inspected immediately prior to, during, and immediately after concrete placement.

(4) Shoring equipment that is found to be damaged or weakened after erection, such that its strength is reduced to less than that required by § 1926.703(a)(1), shall be immediately reinforced.

(5) The sills for shoring shall be sound, rigid, and capable of carrying the maximum intended load.

(6) All base plates, shore heads, extension devices, and adjustment screws shall be in firm contact, and secured when necessary, with the foundation and the form.

(7) Eccentric loads on shore heads and similar members shall be prohibited unless these members have been designed for such loading.

(8) Whenever single post shores are used one on top of another (tiered), the employer shall comply with the following specific requirements in addition to the general requirements for formwork:

(i) The design of the shoring shall be prepared by a qualified designer and the erected shoring shall be inspected by an engineer qualified in structural design.

(ii) The single post shores shall be vertically aligned.

(iii) The single post shores shall be spliced to prevent misalignment.

(iv) The single post shores shall be adequately braced in two mutually perpendicular directions at the splice level. Each tier shall also be diagonally braced in the same two directions.

(9) Adjustment of single post shores to raise formwork shall not be made after the placement of concrete.

(10) Reshoring shall be erected, as the original forms and shores are removed, whenever the concrete is required to support loads in excess of its capacity.

(c) Vertical slip forms.

(1) The steel rods or pipes on which jacks climb or by which the forms are lifted shall be

(i) Specifically designed for that purpose; and

(ii) Adequately braced where not encased in concrete.
(2) Forms shall be designed to prevent excessive distortion of the structure during the jacking operation.

(3) All vertical slip forms shall be provided with scaffolds or work platforms where employees are required to work or pass.

(4) Jacks and vertical supports shall be positioned in such a manner that the loads do not exceed the rated capacity of the jacks.

(5) The jacks or other lifting devices shall be provided with mechanical dogs or other automatic holding devices to support the slip forms whenever failure of the power supply or lifting mechanism occurs.

(6) The form structure shall be maintained within all design tolerances specified for plumbness during the jacking operation.

(7) The predetermined safe rate of lift shall not be exceeded.

(d) Reinforcing steel.

(1) Reinforcing steel for walls, piers, columns, and similar vertical structures shall be adequately supported to prevent overturning and to prevent collapse.

(2) Employers shall take measures to prevent unrolled wire mesh from recoiling. Such measures may include, but are not limited to, securing each end of the roll or turning over the roll.

(e) Removal of formwork.

(1) Forms and shores (except those used for slabs on grade and slip forms) shall not be removed until the employer determines that the concrete has gained sufficient strength to support its weight and superimposed loads. Such determination shall be based on compliance with one of the following:

(i) The plans and specifications stipulate conditions for removal of forms and shores, and such conditions have been followed, or

(ii) The concrete has been properly tested with an appropriate ASTM standard test method designed to indicate the concrete compressive strength, and the test results indicate that the concrete has gained sufficient strength to support its weight and superimposed loads.

(2) Reshoring shall not be removed until the concrete being supported has attained adequate strength to support its weight and all loads in place upon it.

APPENDIX TO § 1926.703(a)(1) General requirements for formwork. (This Appendix is non-mandatory.)

This appendix serves as a non-mandatory guideline to assist employers in complying with the formwork requirements in §1926.703(a)(1). Formwork which has been designed, fabricated, erected, braced, supported, and maintained in accordance with Sections 6 and 7 of the American National Standard for Construction and Demolition Operations—Concrete and Masonry Work, ANSI A10.9-1983, shall be deemed to be in compliance with the provision of § 1926.703(a)(1).

7.3.3 American National Standards Institute

The standard suggested in the above OSHA provisions as a means for achieving compliance is the American National Standard for Construction and Demolition Operations—Concrete and Masonry Work—Safety Requirements, ANSI A10.9-1997(R2004). Section 6, on vertical shoring, has provisions for loads and design, field practices, removal, tubular welded frame shoring, tube and coupler tower shoring, and single-post shores. Section 7, on formwork, has provisions for loads, formwork design, placing and removal of forms, vertical slip forms, flying-deck forms, and horizontal shoring beams. Most of the load and design provisions refer to the ACI Committee 347 Guide to Formwork for Concrete as the procedure to follow.

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7.3.4 Scaffolding, Shoring, and Forming Institute Guides and Rules

This industry-supported institute publishes many relevant guides, rules, and training aids related to formwork and its support systems. Examples include the following:

- Scaffold Code of Safe Practice
- Safety Procedures for Vertical Formwork
- Flying and Handling Concrete Formwork
- The Difference Between Scaffolding and Shoring
- Wind Loads and Shoring
- Safe Practices for Erecting and Dismantling Frame Shoring
- Horizontal Shoring Beam Safety Rules
- Single Post Shore Safety Rules
- Rolling Shore Bracket Safety Rules
- Recommended Frame Shoring Erection Procedures
- Flying Deck Form Safety Rules

7.3.5 American Society of Civil Engineers Standards

A committee of the American Society of Civil Engineers has developed ASCE 37-02 Design Loads on Structures during Construction, a standard for loads imposed on structures during construction, including temporary structures. The standard covers many topics, including loads for formwork, falsework, and shoring. The loads and pressures imposed by concrete, construction interval winds, and other sources are defined. For wind loadings, reference for analysis method is made to ASCE 7, but ASCE 37 defines wind velocity reductions reflecting the short period of exposure during construction. As with any voluntary consensus standard, it is available for groups to specify in individual contracts, or it could be cited as the standard for use by government agencies in rule making. Although various guides and industry practices have been available for many years for the design of formwork, this standard makes available for the first time in the United States a document that defines loads in mandatory language that can be incorporated in contracts, codes, and safety laws.

7.3.6 Information from Formwork Suppliers

Formwork suppliers, of course, publish literature defining their products and limitations for their use. Data sheets indicate the load ratings of products such as form ties, shores, braces, wales, etc. for reference by form designers of job-built systems. In addition, many form manufacturers supply preengineered formwork systems (Figure 7.25) under rental or purchase arrangements for individual projects or general use. Such systems will come with rules and limitations whose implementation is required by the contractor as part of the rental or sales agreement. It is critical that these requirements be implemented as the normal procedures for use and for training workers involved with the formwork.

7.3.7 General Material Design Standards

Although they do not specifically address formwork, the same criteria used for the design of permanent structures in any particular material should also be applied to the design of formwork once the loads and safety factors appropriate for formwork have been determined. Most forms are made of wood, steel, or aluminum or some combination of these materials. The usual reference design standards for these materials are as follows:

7.4 Loads and Pressures on Formwork

The possible loads acting on formwork are many. Vertical loads are usually associated with the dead load of the placed concrete and formwork and the live load of workers and their equipment. Internal pressures on vertical formwork result from the liquid or semi-liquid behavior of the fresh concrete. External forces such as wind exert horizontal loads on the forms, requiring bracing systems for lateral form stability.

7.4.1 Vertical Loads

Vertical loads acting on formwork (Figure 7.26) include the self-weight of the forms, the placed reinforcement, the weight of fresh concrete, the weight of the workers, and the weight of placing equipment and tools. The dead load of the concrete is usually estimated at 145 to 150 lb/ft³, including an allowance for normal amounts of reinforcement. In cases where the reinforcement appears to be heavy, the materials should be calculated separately to determine the actual unit weight. Adjustments are also made for lightweight concrete densities. An 8-in. normal weight slab thus imposes a dead load of 150 lb/ft³ × (8 in./12 in./ft), or 100 psf, on horizontal formwork. ACI Committee 347 (2004) recommends that horizontal formwork be designed for a minimum vertical live load of 50 psf to allow for workers and their incidental...
placing tools such as screeds, vibrators, and hoses. When motorized carts or buggies are used, a minimum live load of 75 psf is recommended. Furthermore, it is recommended that the minimum combined total design dead and live load should be no less than 100 psf, or 125 psf if motorized buggies are used. Formwork self-weight can be calculated from the unit weights and sizes of the components. The weight of the formwork is often much less than the dead load of the concrete and the construction live loads; thus, during design, an allowance is frequently made for the form components as a superimposed load per square foot. Because forms often weigh 5 to 15 psf, an initial estimate is made in this range based on experience and then is verified after the members are sized.

7.4.2 Lateral Pressures of Concrete

Vertical formwork, such as that for walls and columns, is subjected to internal lateral pressure from the accumulated depth of concrete placed. In a placement, fresh concrete, at least near the top and sometimes at greater depths, behaves as a liquid during vibration and generates lateral pressures equal to the vertical liquid head. Although concrete is a granular material with internal friction, the fluidization of the concrete resulting from internal vibration at least temporarily creates a liquid state; however, many factors appear to contribute to the lateral pressures being less than a liquid head at depths below the controlled depth of vibration. If the vertical placement rate is slow, the concrete mass below may have time to begin stiffening. If the concrete is warm, this stiffening may begin earlier. Internal concrete granular friction, form friction, migration of pore water, and other factors may also reduce the resulting lateral pressures. Admixtures, different types of cements, cement substitutes, and construction practices also affect the level of lateral pressure.

Tests have indicated that the pressures often have a distribution, as indicated in Figure 7.27, starting as a liquid pressure near the top and reaching a maximum at some lower level. For simplicity, design practice usually assumes that the maximum pressure is uniform at a conservative value. ACI Committee 347 (2004) recommends that, unless certain conditions are met, calculation of the pressure magnitude vs. depth should be as a full liquid head:

\[ p = wh \]  

(7.1)

where \( p \) is the lateral pressure of concrete (lb/ft\(^2\)), \( w \) is the unit weight of fresh concrete (lb/ft\(^3\)), and \( h \) is the depth of fluid or plastic concrete from the top of the placement to the point of consideration in the form (ft).
ACI 347-04 further states, however, that for concrete having a slump no greater than 7 in. and placed with normal internal vibration to a depth of 4 ft or less, the formwork may be designed for a lateral pressure as follows, where $p_{\text{max}}$ is the maximum lateral pressure (psf), $R$ is the rate of vertical placement (ft/hr), $T$ is the temperature of concrete (°F), $C_W$ is the unit weight coefficient per Table 7.1, and $C_C$ is the unit weight coefficient per Table 7.2:

1. For columns:

$$p_{\text{max}} = C_W C_C [150 + 9000R / T]$$

(7.2)

with a minimum of $600C_W$ psf, but in no case greater than $wh$.

2. For walls with rate of placement ($R$) less than 7 ft/hr and a placement height not exceeding 14 ft:

$$p_{\text{max}} = C_W C_C [150 + 9000R / T]$$

(7.3)

with a minimum of $600C_W$ psf, but in no case greater than $wh$. 

---

**TABLE 7.1** Unit Weight Coefficient ($C_W$)

<table>
<thead>
<tr>
<th>Unit Weight of Concrete</th>
<th>$C_W$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than 140 lb/ft$^3$</td>
<td>$C_W = 0.5[1 + (w/145 \text{ lb/ft}^3)]$</td>
</tr>
<tr>
<td>140 to 150 lb/ft$^3$</td>
<td>1.0</td>
</tr>
<tr>
<td>More than 150 lb/ft$^3$</td>
<td>$C_W = w/145 \text{ lb/ft}^3$</td>
</tr>
</tbody>
</table>

**TABLE 7.2** Chemistry Coefficient ($C_C$)

<table>
<thead>
<tr>
<th>Cement Type or Blend</th>
<th>$C_C$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Types I, II, and III without retarders$^*$</td>
<td>1.0</td>
</tr>
<tr>
<td>Types I, II, and III with a retarder$^*$</td>
<td>1.2</td>
</tr>
<tr>
<td>Other types or blends containing less than 70% slag or 40% fly ash without retarders$^*$</td>
<td>1.2</td>
</tr>
<tr>
<td>Other types or blends containing less than 70% slag or 40% fly ash with a retarder$^*$</td>
<td>1.4</td>
</tr>
<tr>
<td>Blends containing more than 70% slag or 40% fly ash</td>
<td>1.4</td>
</tr>
</tbody>
</table>

$^*$ Retarders include any admixture, such as a retarder, retarding water reducer, retarding midrange water-reducing admixture, or high-range water-reducing admixture (superplasticizer), that delays setting of concrete.
3. For walls with rate of placement \( R \) less than 7 ft/hr where placement height exceeds 14 ft and for walls with a placement rate of 7 to 15 ft/hr:

\[
p_{\text{max}} = C_{w}C_{c}[150 + 43,400 / T + 2800R / T]
\]

(7.4)

with a minimum of \( 600C_{w} \) psf but in no case greater than \( wh \).

For the purpose of the pressure formulas, columns are defined as vertical elements with no plan dimension exceeding 6.5 ft, and walls are vertical elements with at least one plan dimension exceeding 6.5 ft.

When working with concrete mixtures using newly introduced admixtures that increase set time or increase slump characteristics, such as self-consolidating concrete, Equation 7.1 should be used until the effect on formwork pressure is understood by measurement. Caution should be used if a concrete form is filled by pumping upward from the base of the form. Not only can a liquid head be developed, but also a portion of the pump surcharge pressure is likely to be developed due to friction and drag resistance of the concrete moving upward through the form and reinforcing.

Internal concrete lateral pressures are usually resisted in wall forms by transferring the pressures through beamlike members (plywood, studs, and wales) to tension ties linking the two wall form sides (Figure 7.28). Because tension elements are the most efficient structural members, this is also the most cost-effective solution. The internal pressures in most column forms are transferred to external tie elements on adjacent faces of the form that serve as links between the opposite sides of a square or rectangular column form. Circular columns have a great forming advantage in that the column-form skin can act as a hoop, resisting the pressures in tension. Although these are effective methods of resisting internal pressures of the concrete, separate resisting elements, such as external braces, must be provided to resist external horizontal loads that tend to overturn wall, column, and slab forms.
7.4.3 Horizontal Loads

Horizontal loads from such forces as wind, seismic activity, cables, inclined supports, inclined dumping of concrete, and starting or stopping of equipment must be resisted by braces and shores. ACI Committee 347 (2004) recommends the following minimum loads for design to prevent lateral collapse of the formwork. For elevated floor formwork in building construction, the horizontal load \( w \) for design in any direction at each floor line should not be less than 100 lb per linear foot of floor edge or 2% of the total dead load on the form distributed as a uniform load per foot of slab edge, whichever is greater (see Figure 7.29). For wall forms, bracing should be designed to meet the minimum wind load requirements of ASCE 7-05 with wind velocity adjustment for shorter recurrence interval as provided in ASCE 37-02. If exposed to elements, the minimum wind design pressure \( q \) should not be less than 15 psf, and bracing should be designed for at least \( w = 100 \) lb per linear foot of wall, applied at the top. In Figure 7.30, the minimum lateral load \( w \) for design of the bracing system would be the greater of \( q \times h/2 \) or 100 lb/ft.

The formwork designer must also be alert for other conditions such as post-tensioning operations, unusual geometry, or sequence of construction of operations that may create special loads on the forming system.

7.5 Formwork Design Criteria

Formwork components can be designed and constructed in many materials, such as plywood, wood, steel, aluminum, and fiber composites. Frequently, a mixture of materials is used (Figure 7.31). Steel, aluminum, and fiber composites are more likely to be parts of manufactured components or systems that are rated or designed by the producer and may be supplied predesigned on a rental basis for the project needs. Forms intended to be job built are often made of wood and require design by the construction engineer associated with the project or by a consultant to the contractor. The examples in this chapter illustrate the latter case for wood components designed by allowable stress methods. To understand the examples, it is necessary to provide some of the essentials of wood design. Readers undertaking the design of formwork in wood are advised to obtain and follow the more comprehensive specifications in the National Design Specification for Wood Construction (AFPA, 2005) and the Plywood Design Specification (APA, 1997).

Most of the lumber used in formwork is surfaced on four sides (S4S) to achieve its final dimensions as shown in Table 7.3. The S4S dimensions are smaller than the nominal sizes referred to in the table. Except for classification purposes, it is the actual dimensions and actual section properties that are used.
in design. A second set of sizes known as rough lumber (not shown in Table 7.3) has slightly larger dimensions but is still not the full nominal size. Rough lumber sizes are sometimes used in heavy falsework-supporting forms.

Plywood is frequently used as the surface layer of the formwork in contact with the fresh concrete. Plywood has different strengths and stiffness depending on the direction of its span relative to the direction of the grain in the outer layers. The equivalent section, considering the varying elastic modulus and strength between parallel-to-grain loading and side-grain loading, is illustrated by equivalent sections in Figure 7.32. When the grain of the outer layers is parallel to the span direction, the strength and stiffness are greatest (Figure 7.33). Many types of plywood are available. Section properties for B-B Plyform, Class I, plywood, one of the most frequently selected types for moderate reuse in formwork, are given in Table 7.4. Note that, due to the alternating grain directions in the plywood veneer layers, conventional methods for calculating section properties of homogeneous, isotropic sections do not apply. The section properties given in Table 7.4 have been determined by considering the varying properties in the different layers as well as the complications of weakness induced by the tendency of fibers to roll over each other in shear lateral to the grain, or rolling shear (Figure 7.34). For these reasons, use the listed value of $S$ only in bending calculations, use $I$ only for deflection calculations, and use $Ib/Q$, the rolling-shear constant, for shear calculations.

The basic design values for wood and for plywood of the species, grades, sizes, and types frequently used in formwork are listed in Table 7.5. The species and grades readily available in the area of the project should always be verified. Contractors also often have stocks of form lumber for reuse from previous projects. Such lumber should always be inspected for defects as the material is assembled, and unsuitable pieces must be rejected.

### 7.5.1 ASD Adjustment Factors for Lumber Stresses

The AFPA-NDS (AFPA, 2005) provides for adjustment of the lumber reference design values ($F$), such as those given in Table 7.5, by a series of multipliers yielding the allowable design values ($F'$) for stress as follows:

**Bending:**

$$F'_b = F_b \times C_D \times C_M \times C_2 \times C_F \times C_{fu} \times C_3 \times \left[ C_i \times C_r \right]$$

(7.5)
### TABLE 7.3 Example Section Properties of Standard Dressed (S4S) Sawn Lumber

<table>
<thead>
<tr>
<th>Nominal Size</th>
<th>Standard Dressed Size (S4S)</th>
<th>Area of Section (A) (in.²)</th>
<th>x-axis Axis</th>
<th>y-axis Axis</th>
<th>Approximate Weight (lb/ft) When Wood Density Equals the Following Weights</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 x 4</td>
<td>1-1/2 x 3-1/2</td>
<td>5.250</td>
<td>3.063</td>
<td>5.359</td>
<td>1.313 0.984 0.911 1.094 1.276</td>
</tr>
<tr>
<td>2 x 6</td>
<td>1-1/2 x 5-1/2</td>
<td>8.250</td>
<td>7.563</td>
<td>20.80</td>
<td>2.063 1.547 1.432 1.719 2.005</td>
</tr>
<tr>
<td>2 x 8</td>
<td>1-1/2 x 7-1/4</td>
<td>10.88</td>
<td>13.14</td>
<td>47.63</td>
<td>2.719 2.039 1.888 2.266 2.643</td>
</tr>
<tr>
<td>2 x 10</td>
<td>1-1/2 x 9-1/4</td>
<td>13.88</td>
<td>21.39</td>
<td>98.93</td>
<td>3.469 2.602 2.409 2.891 3.372</td>
</tr>
<tr>
<td>4 x 4</td>
<td>3-1/2 x 3-1/2</td>
<td>12.25</td>
<td>7.146</td>
<td>12.51</td>
<td>7.146 12.51 2.127 2.522 2.977</td>
</tr>
<tr>
<td>4 x 6</td>
<td>3-1/2 x 5-1/2</td>
<td>19.25</td>
<td>17.65</td>
<td>48.53</td>
<td>11.23 19.65 3.342 4.010 4.679</td>
</tr>
<tr>
<td>4 x 8</td>
<td>3-1/2 x 7-1/4</td>
<td>25.38</td>
<td>30.66</td>
<td>111.1</td>
<td>14.80 25.90 4.405 5.286 6.168</td>
</tr>
</tbody>
</table>

Note: Table is an abbreviated list of properties of selected sizes; see AFPA-NDS (2005) for additional data.
FIGURE 7.32 Plywood equivalent sections recognizing the weakness of lateral modulus.

FIGURE 7.33 Plywood load capacity and stiffness varies with direction of face grain span.

Shear:

\[ F'_s = F \times C_D \times C_M \times \left[ C_s \times C_i \right] \]  

(7.6)

Bearing:

\[ F'_{\parallel} = F_{\parallel} \times C_M \times C_i \times \left[ C_s \times C_i \right] \]  

(7.7)

Compression:

\[ F'_s = F_s \times C_M \times C_F \times C_P \times \left[ C_s \times C_i \right] \]  

(7.8)

Elastic modulus:

\[ E' = E \times C_M \times \left[ C_s \times C_i \right] \]  

(7.9a)

\[ E'_{\min} = E_{\min} \times C_M \times \left[ C_s \times C_i \times C_T \right] \]  

(7.9b)
Some of the adjustment factors (in brackets) only apply to truss members (buckling stiffness factor, $C_T$), when the member is incised (incising factor, $C_I$) or when the temperature is $>100\degree\text{F}$ (temperature factor, $C_t$), and thus have only rare uses in formwork. The remaining factors are discussed below.

### 7.5.2 Load Duration Factor ($C_D$)

The adjustment for load duration ($C_D$) reflects the ability of wood to exhibit increased strength under shorter periods of loading. The following values may be applied for the indicated cumulative maximum load durations:

- $C_D = 0.9$  Load duration $> 10$ year
- $C_D = 1.0$  2 months $< \text{load duration} \leq 10$ year
- $C_D = 1.15$  7 days $< \text{load duration} \leq 2$ months
- $C_D = 1.25$  Load duration $\leq 7$ days
- $C_D = 1.6$  Wind/earthquake
- $C_D = 2.0$  Impact

For most formwork, an adjustment of $C_D = 1.25$ is applied; however, when the components are reused for longer cumulative durations at maximum level, $C_D$ should be appropriately reduced.
### TABLE 7.5 Example Reference Design Values for Visually Graded Dimension Lumber at 19% Maximum Moisture and Plywood Used Wet

| Species, Specific Gravity, Grade, and Size Category | Extreme Fiber Bending \( (F_b) \) (psi) | Compression \( \perp \) to Grain \( (F_{b\perp}) \) (psi) | Compression \( || \) to Grain, \( F_c \) (psi) | Shear \( || \) to Grain \( (F_v) \) (psi) | Modulus of Elasticity \( E \) (psi) | \( E_{\text{min}} \) (psi) |
|---------------------------------------------------|---------------------------------------------|---------------------------------------------|---------------------------------------------|---------------------------------------------|-------------------------------|-------------------------------|
| Douglas Fir–Larch, \( G = 0.50 \)                  |                                             |                                             |                                             |                                             |                               |                               |
| No. 2, 2–4 in. thick, 2 in. and wider              | 900                                         | 625                                         | 1350                                        | 180                                         | 1,600,000                     | 580,000                       |
| Construction, 2–4 in. thick, 2–4 in. wide         | 1000                                        | 625                                         | 1650                                        | 180                                         | 1,500,000                     | 550,000                       |
| Douglas Fir–South, \( G = 0.49 \)                 |                                             |                                             |                                             |                                             |                               |                               |
| No. 2, 2–4 in. thick, 2 in. and wider              | 850                                         | 520                                         | 1350                                        | 180                                         | 1,200,000                     | 440,000                       |
| Construction, 2–4 in. thick, 2–4 in. wide         | 975                                         | 520                                         | 1650                                        | 180                                         | 1,200,000                     | 440,000                       |
| Southern Pine, \( G = 0.55 \) (size-adjusted values) |                                             |                                             |                                             |                                             |                               |                               |
| No. 2, 2–4 in. thick, 2–4 in. wide                 | 1500                                        | 565                                         | 1650                                        | 175                                         | 1,600,000                     | 580,000                       |
| No. 2, 2–4 in. thick, 5–6 in. wide                 | 1250                                        | 565                                         | 1600                                        | 175                                         | 1,600,000                     | 580,000                       |
| No. 2, 2–4 in. thick, 8 in. wide                   | 1200                                        | 565                                         | 1550                                        | 175                                         | 1,600,000                     | 580,000                       |
| Construction, 2–4 in. thick, 4 in. wide            | 1100                                        | 565                                         | 1800                                        | 175                                         | 1,500,000                     | 550,000                       |
| Spruce-Pine-Fir, \( G = 0.42 \)                   |                                             |                                             |                                             |                                             |                               |                               |
| No. 2, 2–4 in. thick, 2 in. and wider              | 875                                         | 425                                         | 1150                                        | 135                                         | 1,400,000                     | 510,000                       |
| Construction, 2–4 in. thick, 4 in. wide            | 1000                                        | 425                                         | 1400                                        | 135                                         | 1,300,000                     | 470,000                       |
| Hem-Fir, \( G = 0.43 \)                           |                                             |                                             |                                             |                                             |                               |                               |
| No. 2, 2–4 in. thick, 2 in. and wider              | 850                                         | 405                                         | 1300                                        | 150                                         | 1,300,000                     | 470,000                       |
| Construction, 2–4 in. thick, 2–4 in. wide          | 975                                         | 405                                         | 1550                                        | 150                                         | 1,300,000                     | 470,000                       |
| Adjustment factor \( (C_{\text{sf}}) \) for moisture content > 19% (lumber) | 0.85\(^c\) | 0.67                                       | 0.8\(^d\)                                   | 0.97                                        | 0.9                           | —                             |
| Adjustment factor \( (C_{\text{sl}}) \) for maximum load duration 7 days or less (lumber and plywood) | 1.25                                        | —                                           | 1.25                                        | 1.25                                        | —                             | —                             |
| Other applicable adjustment factors for lumber      | \( C_{b}, C_{b\perp}, C_{b\parallel}, C_{c}, C_{v}, C_{p} \) | \( C_{b}, C_{b\perp}, C_{b\parallel} \) | \( C_{c}, C_{p} \) | \( C_{v}, C_{p} \) | \( C_{c} \) | \( C_{p} \) |
| Plywood sheathing used wet: B-B Plyform, Class I    | 1545\(^b\)                                  | 210                                         | —                                           | 57\(^h\)                                    | 1,500,000                     | —                             |

\(^a\) Size adjustments apply to all lumber basic bending and compression parallel to the grain design values, except for Southern Pine. The size adjustments are already included in the Southern Pine basic design values. Consult Table 7.4 for details of size adjustments.

\(^b\) Plywood stresses include an experience factor of 1.3 recommended by the American Plywood Association.

\(^c\) When \( F_{b\perp} \leq 1150 \) psi, \( C_{\text{sl}} = 1.0 \).

\(^d\) When \( F_c \) \leq 750 psi, \( C_{\text{sl}} = 1.0 \).

Note: Data are based on recommendations of the American Forest and Paper Association and American Plywood Association.
7.5.3 Moisture Factor \((C_M)\)

Wood gains in strength as it loses moisture in a range below the fiber saturation point (about 30% moisture content). The basic design values are established for lumber that has a moisture content of 19% or less, typical of air-dried lumber. When the exposure is such that the wood moisture content will exceed 19% for an extended period of time, the design values should be multiplied by the \(C_M\) values indicated in Table 7.5.

7.5.4 Size Factor \((C_F)\)

Tests indicate that member overall size affects the failure stress. To account for these variations, the size factor \((C_F)\) as shown in Table 7.6 is applied to the bending and compression basic design values. Note that the size factor does not apply to the basic design values of Southern Pine, whose basic design values in Table 7.5 are preadjusted to reflect most of the size effect.

7.5.5 Flat-Use Factor \((C_{fu})\)

Lumber loaded on its wide face and bending about its weak axis \((y-y)\) exhibits a slightly higher failure stress. To reflect these variations, the flat-use factor \((C_{fu})\) adjustments in Table 7.6 may be applied to the basic design values for bending stress.

7.5.6 Beam-Stability Factor \((C_L)\)

The AFPA-NDS (AFPA, 2005) provides equations for determining the beam-stability factor \((C_L)\), an adjustment less than 1.0, when the compression edge of a beam may become unstable. For sawn lumber, however, the AFPA-NDS also provides prescriptive \(d/b\) ratios, based on nominal dimensions and lateral support conditions where the member may be assumed to be stable and no reduction for \(C_L\) is needed, as follows:

- \(d/b = 2\) to 1 or less, no lateral support is necessary.
- \(d/b = 3\) to 1 or 4 to 1, ends shall be held in position against lateral rotation or displacement by blocking or connection to other members.
- \(d/b = 5\) to 1, one edge shall be held in line for the entire length.
- \(d/b = 6\) to 1, bridging, blocking, or cross-bracing shall be installed at intervals not exceeding 8 ft unless both edges are held in line.
- \(d/b = 7\) to 1, both edges shall be held in line for the entire length.

<table>
<thead>
<tr>
<th>Grade</th>
<th>Width (in.)</th>
<th>Size Factor (All Species Except Southern Pine)</th>
<th>Flat-Use Factor (Bending Stress (F_b))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Bending Stress (F_b)</td>
<td>Compression (F_c)</td>
</tr>
<tr>
<td>No. 1 and No. 2</td>
<td>2 and 3</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>1.4</td>
<td>1.4</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>1.2</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>1.1</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>1.0</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>14 and wider</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td>Construction</td>
<td>2 and 3</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>
7.5.7 Column-Stability Factor \((C_p)\)

The column-stability factor \((C_p)\) is an adjustment less than 1.0 to reduce the allowable compression stress parallel to the grain when longer column-like members such as shores or braces may fail in a buckling mode rather than by crushing. The factor is given by:

\[
C_p = \frac{1 + \left( \frac{F_{cd}}{F'_c} \right)}{2c} - \frac{\sqrt{1 + \left( \frac{F_{cd}}{F'_c} \right)^2}}{2c} - \frac{F_{cd}}{F'_c}
\]

(7.10)

where:

\(F'_c = \) tabulated compression design value multiplied by all applicable adjustment factor except \(C_p\),
\(c = 0.8\) (for sawn lumber),
\(F_{cd} = 0.822F'_c \frac{l_e}{(l/d)^2}\), where \(l_e\) is the effective length, and \(l/d\) is the larger of the slenderness ratios about the possible buckling axes; the value of \(l/d\) shall not normally exceed 50, except for short-duration loadings during construction when it shall not exceed 75.

7.5.8 Bearing-Area Factor \((C_b)\)

The bearing-area factor \((C_b)\) is for the case of bearing perpendicular to the grain of the wood—that is, bearing on the side grain. The bearing factor is normally taken as 1.0; however, if the bearing area is more than 3 in. from the end of the member and less than 6 in. in length as measured along the grain, then the following increase factor may be applied:

\[
C_b = \frac{l_b + 0.375}{l_b}
\]

(7.11)

where \(l_b\) is the length of bearing (in inches) measured parallel to the grain. Note that in formwork design, unless there is certainty that the bearing area will not be within 3 in. of the end of the member, it is usually best to assume \(C_b = 1.0\).

7.5.9 Repetitive-Use Factor \((C_r)\)

The AFPA-NDS (AFPA, 2005) includes a repetitive-use factor \((C_r)\) that may be used to increase the bending design value when there are at least three members spaced not more than 24 in. on-center, such as joists and studs, and they are joined by a load-distributing member, such as sheathing. The increase is allowed because it is unlikely that normal defects would occur in the repetitive members at the same critical location and the load could be shared if one had a defect. In formwork design, however, ACI SP-4 (Hurd, 2005) suggests that this factor should only be applied to carefully constructed panels whose components are securely nailed or bolted together. The factor values > 1.0 are listed in the AFPA-NDS.

7.5.10 Adjustment Factors for Plywood Stresses

Relative to formwork, the allowable stresses given by the APA—The Engineered Wood Association (APA, 2004) are subject to three primary adjustments: load duration, wet use, and experience factors. A load duration factor similar to the value of \(C_D\) for wood is normally applied as listed in Table 7.5. The wet use and experience factors appropriate to formwork applications have been incorporated in the allowable stresses listed for B-B Plyform, Class I, plywood in Table 7.3. The calculation of plywood deflection can be refined to include both the bending deflection and the shear deflection. In this presentation, for simplicity, only the bending deflection is considered, but the lower value of the plywood elastic modulus will be used to partially compensate for this. Consult APA (2004) for procedures for calculating the shear deflection component.
7.5.11 Manufactured Wood Products

Various manufactured wood products are sometimes used in formwork applications. These include not only sheathing but also structural composite lumber. Laminated veneer lumber (LVL) is made of wood-veneer sheet elements 1/4-in. thick or less, bonded with an exterior adhesive, with wood fibers oriented along the length of the member. Parallel strand lumber (PSL) is made of wood strands having a least dimension of 1/4 in. or less and an average length of at least 150 times the least dimension, bonded with exterior adhesive, and with strands oriented along the length of the member. Laminated strand lumber (LSL) is made of wood strands with a least dimension of about 1/32 in., approximately 12 in. long, parallel to the length of the member, and bonded with an exterior adhesive. The suitability of these materials for exposed formwork should be evaluated for each application. Design information can be obtained from the manufacturers or the APA.

7.5.12 Safety Factors for Formwork Accessories

Various accessories are used in formwork such as form ties, anchors, and hangers. Most of these devices are made of steel and are rated by the manufacturers. The ratings may be listed as either the allowable or ultimate strengths of the devices under certain types of loading. The type of rating (allowable or ultimate) should be carefully determined from the information provided. If ultimate strengths are listed, the allowable strength should be calculated by dividing by an appropriate factor of safety. ACI Committee 347 (2004) has recommended the safety factors listed in Table 7.7 for such accessories.

7.6 Formwork Design

Most components of a form system can be subdivided into members that are primarily bending elements (sheathing, joists, studs, stringers, and wales) and members that are primarily tension or compression elements (shores, braces, etc.). In addition, there are numerous details to design, such as connections (Figure 7.35), hangers, and footings or mudsills (Figure 7.36). The following sections provide example designs, in wood, of the main members of an elevated floor slab form and a vertical wall form to convey a sense of the procedures involved. After the design is complete, the formwork material specifications, member layout, member sizes, connection details, erection procedures, and use limitations should be conveyed by means of drawings with appropriate notes to the field workers who will fabricate and erect the form.

7.6.1 Determination of Resultants from Loads

The bending members of a wood form system are either single-span or continuous multiple-span elements, usually with bearing supports as illustrated in Figure 7.37. Although the members may sometimes have more than three spans, the benefits from considering more than three span conditions are very limited. Many of the member loads are uniform. In other cases, the loadings may be a series of closely spaced concentrated loads that can often be approximated as a uniform load if there is a sufficient number in a span. Figure 7.37 also provides the formulas for the maximum moments, shears, and
deflections for the uniformly loaded one-, two-, and three-span cases. It should be noted that the formulas for calculating the maximum shear force are modified to calculate the shear at a distance $d$ from the face of the supporting member, where $d$ is the depth of the member being designed and $l_b$ is the length of bearing at the supporting member. In wood design, the AFPA-NDS provides that loads within a distance $d$ of the face of the support can be neglected when designing for shear if the member is loaded on one face and supported on the opposite face or edge. However, in cases where the member is notched or connection is made in the web, as by bolting, other AFPA-NDS special provisions should be consulted which, in effect, magnify the shear force used for design.

### 7.6.2 Fundamental Relations between Resultants and Stresses

From mechanics of solids, the following relationships apply to the elastic design of wood elements:

**Bending of beams or plywood:**

$$ f_b = \frac{M}{S} \quad (7.12) $$

**Shear of solid rectangular beams:**

$$ f_s = \frac{3}{2} \frac{V}{bd} \quad (7.13) $$
Design Notes: 1. If \( l_b \) is unknown, use \( l_b = 0 \) for shear calculations.
2. If \( d \) is unknown when calculating shear force, either:
   a) Assume \( d = 0 \) in calc. and reevaluate with \( d \) determined if shear controls.
   b) Assume a likely value of \( d \) and check with an additional iteration when \( d \) is determined.

FIGURE 7.37 Beam formulas for one-, two-, and three-span conditions.

Shear of plywood:

\[
f_r = \frac{V}{I_b Q} = \frac{V}{I_b / Q}
\]

(7.14)

where \( I_b/Q \) is the rolling shear constant.

Bearing and axial compression:

\[
f_{c\perp} \text{ or } f_c = \frac{P}{A}
\]

(7.15)

When \( f \) is used in the above equations, the actual stress is sought in the calculation from the actual resultant. When \( F \) is used, the maximum allowable stress is implied and the maximum resultants are sought. Load–deflection relationships are given in Figure 7.37 as a function of the number of spans.
7.6.3 Basis of Examples
When undertaking a form design, some parameters are frequently known, and others must be calculated to meet the required strength needs and deflection limitations. For example, the spacings of members may be dictated by the geometry of the area to be formed or the modular needs of the system or plywood facing. For this case, the unknowns become the required member sizes. In another case, the contractor may have material of a given size that is desired to be used. For the latter case, the unknowns are the maximum spacings of the members required to limit the loads on each to its allowable value. In the following elevated slab form example, the spacings are assumed to be set by job conditions and the members are the unknowns. In the wall form design example, the member sizes are assumed to be predetermined by available material and the spacings are the unknowns. For both examples, the ASD procedures of the AF&PA NDS-2005 will be used.

7.7 Slab-Form Design Example
Assume that an elevated flat-plate floor slab of 8-in. thickness is to be formed with a top elevation 10 ft above the supporting surface below. The general layout of the plywood sheathing, joists, stringers, and shores is shown in Figure 7.9 and Figure 7.38. Because of the floor layout, the contractor desires to space the joists at 16 in. on-center, the stringers at 5 ft on-center under the joists, and the shores at 5 ft on-center under the stringers. The materials are B-B Plyform, Class I, plywood and No. 2 Spruce–Pine–Fir (SPF) joists, stringers, and shores. It has been determined that the strength gain of the concrete will allow the forms to be stripped in 4 days, followed by installation of reshores. The plywood will be assumed to be used wet, as it is frequently exposed for a lengthy period during rebar placement and in contact with the fresh concrete; however, the lumber elements will be assumed to be relatively sheltered and not exposed to significant moisture for lengthy periods. The plywood will be oriented with its face grain parallel to the span direction—that is, the strong way. From the earlier section on vertical loads, the distributed pressure \( q \) for design on a working basis is:

<table>
<thead>
<tr>
<th>Description</th>
<th>Value (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load of 8-in. concrete slab</td>
<td>100</td>
</tr>
<tr>
<td>Construction live load</td>
<td>50</td>
</tr>
<tr>
<td>Formwork estimated dead load</td>
<td>5</td>
</tr>
<tr>
<td>Total</td>
<td>155</td>
</tr>
</tbody>
</table>

7.7.1 Sheathing Design
The plywood span is the spacing of the joists, and 8-ft-long plywood panels can typically have three or more spans. The plywood allowable stresses adjusted for load duration (see Table 7.5) are:

\[
F'_y = F_y \times C_D = 1545 \text{ psi} \times 1.25 = 1931 \text{ psi}
\]

\[
F'_c = F_c \times C_D = 57 \text{ psi} \times 1.25 = 71 \text{ psi}
\]

\[
E' = E = 1,500,000 \text{ psi}
\]

Considering the loading to be a uniform load \( w \) of 1-ft width, corresponding to the section properties of the plywood based on 12-in. width:

\[
w = q \times \text{unit width} = 155 \text{ psf} \times 1.0 \text{ ft} = 155 \text{ lb/ft}
\]

For the limitation of flexural bending, the maximum moment \( M \) is:

\[
M = \frac{wL^2}{10} = \frac{155 \text{ lb/ft} \times (1.33 \text{ ft})^2}{10} = 27.5 \text{ ft-lb} = 330 \text{ in.-lb}
\]
and the required section modulus \((S)\) is given by:

\[
S = \frac{M}{F_b'} = \frac{330 \text{ in.-lb}}{1931 \text{ psi}} = 0.171 \text{ in.}^2
\]

For the shear limitation, the maximum shear force for design is determined at the center of the support as the support width is not yet known:

\[
V = 0.62wl = 0.62(155 \text{ lb/ft})(16 \text{ in./12 in./ft}) = 124 \text{ lb}
\]

\[
\frac{lb}{F_b'} = \frac{V}{F_b'} = \frac{124 \text{ lb}}{71 \text{ psi}} = 1.74 \text{ in.}^2
\]

The deflection limit is determined from:

\[
\frac{l}{360} = \frac{1}{145} \frac{wl^4}{EI}
\]

\[
l = \frac{360 \text{ 155 lb/ft}}{145 \text{ 12 in./ft}} \frac{(16 \text{ in.})^3}{1,500,000 \text{ psi}} = 0.0876 \text{ in.}^4
\]

From these requirements, 1/2-in. plywood would be adequate for bending and shear, but 5/8-in. is required for deflection. Select the plywood thickness to be 5/8 in. Because deflection controls, there is no benefit to further refining the shear calculation after the joist width is known.

### 7.7.2 Joist Design

The joist span is the 5-ft spacing of the stringers, and the joists can be constructed with a three-span continuous arrangement (Figure 7.9 and Figure 7.38). Lateral buckling of the joists will be restrained by nailing the sheathing to the joists. For selection of the joist reference design values (Table 7.5) and adjustment factors, assume initially that a 2 × 6 joist might work. Allowable stresses for the No. 2 SPF joist are as follows:

\[
F_b' = F_b \times C_D \times C_M \times C_L \times C_t \times C_r \times C_e = 875 \text{ psi} \times 1.25 \times 1.0 \times 1.0 \times 1.3 \times 1.0 = 1421 \text{ psi}
\]

\[
F_v' = F_v \times C_D \times C_M = 135 \text{ psi} \times 1.25 \times 1.0 = 169 \text{ psi}
\]

\[
E' = E \times C_M = 1,400,000 \text{ psi} \times 1.0 = 1,400,000 \text{ psi}
\]
Considering the loading to be a uniform load \( w \) of 16-in. tributary width corresponding to the spacing of the joists:

\[
w = q \times \text{tributary width} = 155 \text{ psf} \times 1.33 \text{ ft} = 207 \text{ lb/ft}
\]

For the limitation of flexural bending, the maximum moment \( M \) is:

\[
M = \frac{w l^2}{10} = \frac{207 \text{ lb/ft} \times (5 \text{ ft})^2}{10} = 517 \text{ ft-lb} = 6210 \text{ in.-lb}
\]
and the required section modulus \( S \) is given by:

\[
S = \frac{M}{F_b''} = \frac{6210 \text{ in.-lb}}{1421 \text{ psi}} = 4.37 \text{ in.}^3
\]

For the shear limitation, the maximum shear force for design is determined at \( d \) from the center of the support, as the supporting stringer width is not yet known. At the interior support,

\[
V = 0.6wl - wd = 0.6(207 \text{ lb/ft}) - 207 \text{ lb/ft} \left( \frac{5.5}{12} \text{ ft} \right) = 526 \text{ lb}
\]

\[
bd = (3/2)(V / F_v') = 3 \times 526 \text{ lb} / (2 \times 169 \text{ psi}) = 4.67 \text{ in.}^2
\]

The deflection limit is determined from:

\[
\frac{l}{360} = \frac{1}{145} \frac{w l^4}{E' I}
\]

\[
I = \frac{360 \times 207 \text{ lb/ft}}{145 \times 12 \text{ in./ft}} = \frac{(60 \text{ in.})^3}{1,400,000 \text{ psi}} = 6.61 \text{ in.}^4
\]

From these combined requirements, a \( 2 \times 6 \) joist is the optimum size having the least area. The result also agrees with the size assumption for selecting the reference design values and size factor. If the size did not agree, the calculations would be repeated with an improved size assumption. In this case, there is no benefit to further refining the shear calculation after the joist width is known because shear alone is not controlling the size.

### 7.7.3 Stringer Design

The stringer span is the 5-ft spacing of the shores, and the stringers can be constructed with a three-span continuous arrangement. For selection of the stringer basic design values (Table 7.5), assume initially that a \( 4 \times 6 \) stringer might work. Lateral buckling of a stringer of this size would not be a problem because \( d/b \leq 2 \) to 1. Allowable stresses for the No. 2 SPF stringers are as follows:

\[
F_b'' = F_b \times C_D \times C_M \times C_L \times C_r \times C_i = 875 \text{ psi} \times 1.25 \times 1.0 \times 1.0 \times 1.3 \times 1.0 = 1421 \text{ psi}
\]

\[
F_v' = F_v \times C_D \times C_M = 135 \text{ psi} \times 1.25 \times 1.0 = 169 \text{ psi}
\]

\[
E' = E \times C_M = 1,400,000 \text{ psi} \times 1.0 = 1,400,000 \text{ psi}
\]

The stringer loading is actually a series of concentrated loads from the joists at 16 in. on-center; however, the starting position of the loads can vary in each span. Due to the complications involved in considering many possible starting positions and recognizing that loads within distance \( d \) of the support can be neglected for shear calculations, it is often the practice in formwork to assume a uniform load as being adequately similar (Hurd, 2005). This assumption works reasonably well when three or more equally
spaced concentrated loads of equal magnitude are in the span. Considering the loading to be a uniform load \( w \) of 5-ft tributary width corresponding to the spacing of the stringers:

\[
w = q \times \text{tributary width} = 155 \text{ psf} \times 5 \text{ ft} = 775 \text{ lb/ft}
\]

For the limitation of flexural bending, the maximum moment \( M \) is:

\[
M = \frac{wL^2}{10} = \frac{775 \text{ lb/ft} \times (5 \text{ ft})^2}{10} = 1938 \text{ ft-lb} = 23,250 \text{ in.-lb}
\]

and the required section modulus \( S \) is given by:

\[
S = \frac{M}{F_b} = \frac{23,250 \text{ in.-lb}}{1421 \text{ psi}} = 16.36 \text{ in.}^3
\]

For the shear limitation, the maximum shear force for design is determined at distance \( d \) from the face of the support, assuming the shore will be at least \( 4 \times 4 \) or have a head of equal or larger size if it is a metal shore. At the interior support where shear is greatest:

\[
V = 0.6wl - w\left(d + \frac{h}{2}\right) = 0.6(775 \text{ lb/ft})(5 \text{ ft}) - 775 \text{ lb/ft} \left[\left(\frac{5.5}{12}\right) + \left(\frac{3.5}{12}\right)\right] = 1857 \text{ lb}
\]

\[
bd = \left(\frac{3}{2}\right)\left(V/F_s'\right) = \left(\frac{3}{2}\right) \times (1857 \text{ lb}/169 \text{ psi}) = 16.5 \text{ in.}^2
\]

The deflection limit is determined from:

\[
\frac{I}{360} = \frac{1}{145} \frac{wL^4}{E'f}
\]

\[
I = \frac{360 \times 775 \text{ lb/ft} \times (60 \text{ in.})^3}{145 \times 12 \text{ in./ft} \times 1,400,000 \text{ psi}} = 24.73 \text{ in.}^4
\]

From these requirements, a \( 4 \times 6 \) stringer is the optimum size meeting all the requirements and having the least area. The result also agrees with the size assumption for selecting the reference design values and size factor. If the size did not agree, the calculations would be repeated with an improved size assumption. A check is also necessary to make sure that there is adequate bearing area for the joist on the stringer.

The tributary load is:

\[
P = 155 \text{ psf} \times 1.33 \text{ ft} \times 5 \text{ ft} = 1031 \text{ lb}
\]

The allowable bearing stress is given by:

\[
F_{cc} = F_{cc} \times C_{amb} \times C_b = 425 \text{ psi} \times 1.0 \times 1.0 = 425 \text{ psi}
\]

and the actual stress is:

\[
f_{cc} = \frac{P}{A} = \frac{1031 \text{ lb}}{1.5 \text{ in.} \times 3.5 \text{ in.}} = 196 \text{ psi} < 425 \text{ psi}
\]

### 7.7.4 Shore Design

To determine the shore unbraced height, the slab thickness and depth of forming elements are subtracted from the floor-to-floor height:

\[
l_e = 120 - 8 - 0.625 - 5.5 - 5.5 \text{ in.} = 100.4 \text{ in.}
\]
For design purposes, the shore is assumed to be concentrically loaded, pinned at the top and the bottom and prevented from translation by an adequate overall form diagonal bracing system. The load to be supported based on the tributary area is:

\[
P = 155 \text{ psf} \times 5 \text{ ft} \times 5 \text{ ft} = 3875 \text{ lb}
\]

Two stress checks are necessary to size the shore: (1) the bearing perpendicular to the grain of the stringer at the upper end of the shore, and (2) the compression parallel to the grain of the shore (including the possibility of buckling). Evaluation of bearing first will establish a first trial size of the shore area. The allowable bearing stress is:

\[
F'_{C_b} = F_{C_b} M_b C_b = 425 \text{ psi} \times 1.0 \times 1.0 = 425 \text{ psi}
\]

\(C_b\) is assumed to be 1.0, because the bearing length is not yet known and the shore may sometimes be located at the end of the stringer. The required bearing area is given by:

\[
A = \frac{P}{F'_{C_b}} = \frac{3875 \text{ lb}}{425 \text{ psi}} = 9.12 \text{ in.}^2
\]

A 4 \times 4 shore is adequate for bearing perpendicular to the grain of the stringer and matches the width of the 4 \times 6 stringer. For this 4 \times 4 shore size, the allowable compression stress parallel to grain is:

\[
F'_{C_p} = F_{C_p} M_p C_p = 1150 \text{ psi} \times 1.25 \times 1.0 \times 1.15 \times C_p = 1653 \text{ psi} \times C_p = F'_{C_p} C_p
\]

\[
E'_{\text{min}} = E_{\text{min}} C_M = 510,000 \text{ psi} \times 1.0 = 510,000 \text{ psi}
\]

Because the trial shore is square and braced only at the ends to resist translation, the \(l/d\) ratios for each buckling axis are equal; thus,

\[
\frac{l}{d} = \frac{100.4 \text{ in.}}{3.5 \text{ in.}} = 28.68 < 50
\]

and

\[
F_{cd} = 0.822 E'_{\text{min}} \left( \frac{l}{d} \right)^2 = 0.822 \times 510,000 \text{ psi} / (28.68)^2 = 510 \text{ psi}
\]

so

\[
F_{cd} / F'_{C_p} = 510 \text{ psi} / 1653 \text{ psi} = 0.308
\]

and \(C_p\) from Equation 7.10 is:

\[
C_p = \left[ (1 + 0.308)/(2 \times 0.8) \right] - \left[ \left( (1 + 0.308)/(2 \times 0.8) \right)^2 - (0.308/0.8) \right]^{0.5} = 0.285
\]

Thus, the allowable stress in compression parallel to the grain for the unbraced length is:

\[
F'_{C} = F_{C} = 1653 \text{ psi} \times C_p = 1653 \text{ psi} \times 0.285 = 471 \text{ psi}
\]

Because \(F'_{C} > F'_{C_b}\), compression perpendicular to the grain in the bearing controls the design, and the 4 \times 4 shore is adequate.

In summary, the design requires 5/8-in. Plyform spanning the strong way, 2 \times 6 joists at 16 in. on-center, 4 \times 6 stringers at 5 ft on-center, and 4 \times 4 shores at 5 ft on-center each way.
7.7.5 Bracing Design Considerations

The slab form must be braced, as a minimum, to resist the horizontal loads recommended by ACI 347-04. When the concrete columns have been placed prior to erection of the floor forms, the column can contribute to lateral stability if the form has an adequate horizontal diaphragm and is tied to the columns. Horizontal cross-bracing can be added to improve the diaphragm. Vertical cross-bracing in two directions at right angles, in combination with an adequate diaphragm, can also provide a workable system. The braces need not be located in the opening between every pair of shores (Figure 7.29). Often, they are located between alternate pairs of shores (and sometimes farther apart) in a well-dispersed, symmetrical pattern in each direction. The brace size is usually controlled by the buckling resistance in the compression direction of force, and the analysis and design proceed in a manner similar to that for the shore or wall form brace design. It is critical to provide a connection at the top and bottom ends of the brace that is adequate to resist the forces imposed.

7.8 Wall-Form Design Example

Assume that a form is needed for the placement of a 12-ft high wall. The general layout of the plywood sheathing, studs, and wales is shown in Figure 7.39. Because of the availability of the material from previous projects, the contractor wishes to use 3/4-in. B-B Plyform, Class I, plywood for the sheathing and No. 2 Southern Pine 2 × 4 studs and double 2 × 4 wales. Ties can be ordered to meet project needs. The project specification requires that the formwork element deflections be no greater than l/240. The plywood will be assumed to be used wet, as it is in contact with the fresh concrete, and the lumber elements will be assumed to be exposed to sufficient rain moisture on the job such that the wood moisture content might rise to above 19%. The normal weight 145 pcf concrete, made with Type II cement, has a slump of <7 in. and contains a set-retarding admixture. Internal vibration during placement will be controlled and limited to an immersion depth <4 ft. The rate of vertical placement (R) will be limited to 4 ft per hour, and the concrete temperature (T) is expected to be above a minimum of 80°F.

From the earlier section on lateral pressures, the maximum distributed pressure (p) for design on a working basis is given by Equation 7.2:
The pressure is assumed to gradually increase with depth at a rate of 150 lb/ft³/ft until reaching the maximum uniform pressure at a depth of 720 psf/150 lb/ft³ or 4.8 ft. For purposes of illustrating the form design procedure, first consider the spacing of members in the uniform pressure region. Because the sizes of the members are predetermined, the unknowns are the maximum allowable spacings of their supports.

### 7.8.1 Sheathing Design

The plywood span is the spacing of the studs, and the 8-ft-long plywood panels can typically have three or more spans. The plywood allowable stresses adjusted for load duration (see Table 7.5) are:

\[ F_b' = F_b \times C_D = 1545 \text{ psi} \times 1.25 = 1931 \text{ psi} \]
\[ F_v' = F_v \times C_D = 57 \text{ psi} \times 1.25 = 71 \text{ psi} \]
\[ E' = E = 1,500,000 \text{ psi} \]

Considering the loading to be a uniform load \( w \) of 1-ft unit width corresponding to the section properties of the plywood based on 12-in. width,

\[ w = p_{\text{max}} \times \text{unit width} = 720 \text{ psf} \times 1.0 \text{ ft} = 720 \text{ lb/ft} \]

For the limitation of flexural bending, the maximum allowable moment \( M \) is:

\[ M = f'_b S = 1931 \text{ psi} \times 0.455 \text{ in.}^3 = 878 \text{ in.-lb} = 73.2 \text{ ft-lb} \]

and the maximum span \( l \) is given by:

\[ M = wL^2/10 \]
\[ 73.2 \text{ ft-lb} = 720 \text{ lb/ft} \times l/10 \]
\[ l = 1.01 \text{ ft} = 12.1 \text{ in.} \]

For the shear limitation, the support width is known to be 1.5 in., so the maximum shear force for design can be determined at the face of the support:

\[ V = F_v'(L/2) = 71 \text{ psi} \times 7.187 \text{ in.}^2 = 510 \text{ lb} \]
\[ V = 0.6wl - w(l_s/2) \]
\[ 510 \text{ lb} = 0.6(720 \text{ lb/ft})(l) - 720[(1.5 \text{ in./2})/12 \text{ in./ft}] \]
\[ l = 1.26 \text{ ft} = 15.4 \text{ in.} \]

For the deflection limit, the maximum span is determined from:

\[ l/240 = (1/145)(wL^3/E'I) \]
\[ l/240 = (1/145)[720 \text{ lb/ft} \times l^3/(12 \text{ in./ft} \times 1,500,000 \text{ psi} \times 0.199 \text{ in.}^4)] \]
\[ l = 14.4 \text{ in.} \]

From these requirements, 12.1 in. (based on bending) is the maximum span of the plywood. Considering the 8-ft modular length of the plywood panel, provide 8 equal spans with the studs spaced at 12 in. on-center.

### 7.8.2 Stud Design

The stud span is the spacing of the double wales, and the studs can be constructed with a three-span continuous arrangement. Lateral buckling of the studs will be restrained by nailing the sheathing to the studs. Allowable stresses for the No. 2 Southern Pine 2 × 4 stud from Table 7.5 are as follows:
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\[ F_b' = F_b \times C_D \times C_M \times C_t \times C_r = 1500 \text{ psi} \times 1.25 \times 0.85 \times 1.0 \times 1.0 = 1593 \text{ psi} \]
\[ F_v' = F_v \times C_D = 175 \text{ psi} \times 1.25 = 212 \text{ psi} \]
\[ E' = E \times C_M = 1,600,000 \text{ psi} \times 0.9 = 1,440,000 \text{ psi} \]

Considering the loading as a uniform load \((w)\) of 12-in. tributary width corresponding to the spacing of the studs,
\[ w = q \times \text{tributary width} = 720 \text{ psf} \times 1.0 \text{ ft} = 720 \text{ lb/ft} \]

For the limitation of flexural bending, the maximum allowable moment \((M)\) is:
\[ M = F_b'S = 1593 \text{ psi} \times 3.063 \text{ in.}^3 = 4879 \text{ in.-lb} = 406.6 \text{ ft-lb} \]

and the maximum allowable span is given by:
\[ M = \frac{wl^2}{10} \]
\[ 406.6 \text{ ft-lb} = \frac{720 \text{ lb/ft} \times l}{10} \]
\[ l = 2.37 \text{ ft} = 28.5 \text{ in.} \]

For the shear limitation, the maximum shear force for design is determined at distance \(d\) from the face of the support because the supporting wale width \((2 \times 1.5 \text{ in.} = 3 \text{ in.})\) is known. At the interior support where shear is greatest:
\[ V = (2/3)F_b'd = (2/3)\times212 \text{ psi} \times 1.5 \text{ in.} \times 3.5 \text{ in} = 742 \text{ lb} \]
\[ V = 0.6wl - w(d + h_b/2) \]
\[ 742 \text{ lb} = 0.6(720 \text{ lb/ft})(l) - 720 \text{ lb/ft} \left[ \frac{3.5}{12} + \frac{3.0}{12} \right] = 1857 \text{ lb} \]
\[ l = 2.41 \text{ ft} = 28.9 \text{ in.} \]

The deflection limit is determined from:
\[ l/240 = (1/145)(wd^4/E'I) \]
\[ l/240 = (1/145) \left[ \frac{720 \text{ lb/ft} \times l^4}{(12 \text{ in./ft} \times 1,440,000 \text{ psi} \times 5.359 \text{ in.}^4)} \right] \]
\[ l = 42.6 \text{ in.} \]

From these requirements, the spans for bending and shear control the maximum span, which is 28.5 in. Use 28 in. for layout convenience.

7.8.3 Wale Design

The wale span is the spacing of the form ties, and the wales can be constructed with a three-span continuous arrangement. Lateral buckling of the wale is not a problem because \(d/b \leq 2\) to 1. Allowable stresses for the No. 2 Southern Pine double \(2 \times 4\) wales are the same as determined above for the studs. The wale loading is actually a series of concentrated loads from the studs at 12 in. on-center; however, the starting position of the loads can vary in each span. Due to the complications involved in considering the many possible starting positions and recognizing that loads within distance \(d\) of the support can be neglected for shear calculations, it is often the practice in formwork to assume a uniform load to be adequately similar to many closely spaced equally concentrated loads (Hurd, 2005). This assumption works reasonably well when three or more equally spaced concentrated loads of equal magnitude are in the span. Considering the loading to be a uniform load \((w)\) of 2.33-ft tributary width corresponding to the spacing of the wales,
\[ w = q \times \text{tributary width} = 720 \text{ psf} \times 2.33 \text{ ft} = 1680 \text{ lb/ft} \]
For the limitation of flexural bending, the maximum allowable moment \((M)\) is:

\[
M = F'_{b}S = 1593 \text{ psi} \times 2 \times 3.063 \text{ in.}^3 = 9758 \text{ in.-lb} = 813 \text{ ft-lb}
\]

and the maximum allowable span is given by:

\[
M = wL/10
\]

\[
813 \text{ ft-lb} = 1680 \text{ lb/ft} \times F/10
\]

\[
l = 2.2 \text{ ft} = 26.4 \text{ in.}
\]

For the shear limitation, the maximum shear force for design is determined at distance \(d\) from the face of the support assuming that the tie-washer plate will have at least a 3-in. length of bearing. The wale interior support is examined where shear is greatest. At the interior support:

\[
V = (2/3)F'_{b}d = (2/3) \times 212 \text{ psi} \times 2 \times 1.5 \text{ in.} \times 3.5 \text{ in.} = 1484 \text{ lb}
\]

\[
V = 0.6wL - w(d + l_{b}/2)
\]

\[
1484 \text{ lb} = 0.6(1680 \text{ lb/ft})(l) - 1680 \text{ lb/ft} \left[\left(\frac{3.5}{12} + \frac{3.0}{2}\right)\text{ ft}\right]
\]

\[
l = 2.17 \text{ ft} = 26.0 \text{ in.}
\]

The deflection limit is determined from:

\[
l/240 = (1/145)(wL^4/EI)
\]

\[
l/240 = (1/145)\left[1680 \text{ lb/ft} \times l^4/\left(12 \text{ in.} / \text{ft} \times 1,440,000 \text{ psi} \times 5.359 \text{ in.}^4\right)\right]
\]

\[
l = 40.5 \text{ in.}
\]

From these requirements, the maximum span is 26.0 in. and is controlled by shear. Select a wale span of 24 in. so the tie spacing can be coordinated easily with the stud spacing.

A check is also needed to make sure that there is adequate bearing area for the stud on the double wale. The tributary load is:

\[
P = 720 \text{ psf} \times 1.0 \text{ ft} \times 2.33 \text{ ft} = 1678 \text{ lb}
\]

The allowable bearing stress is given by:

\[
F'_{c} = F_{c}C_{M}C_{b} = 565 \text{ psi} \times 0.67 \times 1.0 = 378 \text{ psi}
\]

and the actual stress is:

\[
f_{c} = P/A = 1678 \text{ lb} / (1.5 \text{ in.} \times 3.0 \text{ in.}) = 373 \text{ psi} < 378 \text{ psi}
\]

### 7.8.4 Tie Design

Select the tie capacity to exceed the tie force \((T)\) based on the tributary area supported:

\[
T = p_{\text{max}} \times \text{wale spacing} \times \text{tie spacing} = 720 \text{ psf} \times 2.33 \text{ ft} \times 2.0 \text{ ft} = 3350 \text{ lb}
\]

Select a heavy snap tie with a 3350-lb working capacity, and check the bearing of the 3-in. square tie washer on the double wale (Figure 7.40):

\[
f_{c} = P/A = 3350 \text{ lb} / (2 \times 1.5 \text{ in.} \times 3.0 \text{ in.}) = 372 \text{ psi} < 378 \text{ psi}
\]

### 7.8.5 Bracing Design

Braces are required to keep the form from overturning due to wind or incidental construction loads such as ladders leaning on the form or workers climbing on the form. The effects of an elevated work platform may also have to be considered in some designs. If designing for the minimum recommendations of ACI
The loadings would include a wind pressure of 15 psf or a line load of 100 lb/linear foot at the top, whichever is greater. The contractor desires to lay out the braces as shown in Figure 7.41, with the braces at 4 ft on-center, the brace attached at an elevation of 10 ft on the 12-ft form, and the brace anchored to the ground 7.5 ft from the form. This results in a brace length of 12.5 ft, including the adjustable connectors at each end. For design purposes, the brace is assumed to be concentrically loaded, pinned at the top and the bottom, and prevented from translation by the longitudinal rigidity of the form. The horizontal brace load \( H \) from the wind is:

\[
H = 15 \text{ psf} \times 4 \text{ ft} \times 12 \text{ ft} \times 6 \text{ ft} / 10 \text{ ft} = 432 \text{ lb}
\]

Alternatively, the horizontal brace load from the 100 lb/ft line load is:

\[
H = 100 \text{ lb/ft} \times 4 \text{ ft} \times 12 \text{ ft} / 10 \text{ ft} = 480 \text{ lb}
\]
The brace should be designed to resist a horizontal component of 480 lb, which results in an axial compression force ($P$) of:

$$P = 480 \text{ lb} \times 12.5 \text{ ft} / 7.5 \text{ ft} = 800 \text{ lb}$$

Try a No. 2 Southern Pine $2 \times 4$ brace with a bracing strut at mid-length in the weak buckling direction. For this $2 \times 4$ size, the allowable compression stress is:

$$F'_{c} = F_{c} \times C_{D} \times C_{M} \times C_{P} = 1650 \text{ psi} \times 1.25 \times 0.8 \times 1.0 \times C_{P} = 1650 \text{ psi} \times C_{P} = F'_{c} \times C_{P}$$

$$F'_{min} = E_{min} \times C_{M} = 580,000 \text{ psi} \times 0.8 = 464,000 \text{ psi}$$

Because the trial brace is rectangular, check the $l/d$ ratios for each buckling axis:

$$\frac{l_{1}}{d_{1}} = \frac{150 \text{ in.}}{3.5 \text{ in.}} = 42.8 \leq 50$$

$$\frac{l_{2}}{d_{2}} = \frac{75 \text{ in.}}{1.5 \text{ in.}} = 50.0 \leq 50$$

For this short-term construction period loading, a $l/d$ limit of 75 might also be considered if necessary. Thus, the weak axis controls, and $l/d = 50$. Substituting to determine $C_{P}$:

$$F_{cE} = 0.822 F'_{c} / (l_{1}/d_{1})^2 = 0.822 \times 464,000 \text{ psi} / (50.0)^2 = 152 \text{ psi}$$

so

$$F_{cE}/F'_{c} = 152 \text{ psi} / 1650 \text{ psi} = 0.092$$

and $C_{P}$ from Equation 7.10 is:

$$C_{P} = \left[ \left( 1+0.092 \right) / (2 \times 0.8) \right] - \left[ \left( 1+0.092 \right) / (2 \times 0.8) \right]^{0.5} = 0.090$$

Thus, the allowable stress in compression parallel to the grain for the unbraced length is:

$$F'_{c} = F_{c} \times C_{P} = 1650 \text{ psi} \times 0.090 = 149 \text{ psi}$$

$$f_{c} = P/A = 800 \text{ lb} / 5.25 \text{ in.}^2 = 153 \text{ psi} > 149 \text{ psi} \text{ (not adequate)}$$

The $2 \times 4$ brace with the mid-length strut is slightly less than the needed section. Increase to a $2 \times 6$ brace with a mid-length strut, which will be adequate based on the increased area. Alternatively, a steel-pipe brace rated for the load at the extended length of 12.5 ft could be used. Appropriate connection, adequate for a tension or compression of 800 lb, must be made at the top and bottom ends of the brace by bolting, nailing, or use of a rated commercial hardware device, without inducing significant eccentricities.

A possible layout for the wall form is shown in Figure 7.41. The top and bottom extensions of the studs beyond the wales are usually about 1/3 of the adjacent span up to perhaps 12 in. At the top of the form, where the pressures are lower, the span of the studs can be investigated with ramp loading, and the wale spacing can often be increased, sometimes eliminating a level of wales and ties in comparison to the number required for a constant spacing. Careful consideration of all design details, communicating these requirements to the form craftspersons, and providing knowledgeable verification of execution can lead to safe and economical support of concrete during construction.
References

ACI Committee 347, 2005. Guide to Shoring and Reshoring of Multistory Concrete Buildings, ACI 347.2R-05, 18 pp. American Concrete Institute, Farmington Hills, MI.

Further Information

American Concrete Institute, P.O. Box 9094, Farmington Hills, MI 48333-9094, www.concrete.org.
Scaffolding, Shoring and Forming Institute, Inc., 1300 Sumner Avenue, Cleveland, OH 44115-2851, www.ssfi.org.
(a) Construction loading in high-rise buildings. (Photograph courtesy of Portland Cement Association, Skokie, IL.)

(b) Ronald McDonald House under construction in New York. (Photograph courtesy of New York Construction News and Edward G. Nawy, Rutgers University.)