PDHonline Course S197 (4 PDH)

Calculating Conventional Wood Frame Connections for Residential Structures (Nails, Bolts, Screws, etc.)

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Course Content

1.1 General

The objectives of connection design are

- to transfer loads resisted by structural members and systems to other parts of the structure to form a "continuous load path";
- to secure nonstructural components and equipment to the building; and
- to fasten members in place during construction to resist temporary loads during installation (i.e., finishes, sheathing, etc.).

Adequate connection of the framing members and structural systems within a residual structure is a critical design and construction consideration. Regardless of the type of materials used, structures are only as strong as their connections. Structural systems can behave as a unit only with proper interconnection of the components and assemblies; therefore, this course is dedicated to connections. A connection transfers loads from one framing member to another (i.e., a stud to a top or bottom plate) or from one assembly to another (i.e., a roof to a wall, a wall to a floor, and a floor to a foundation). Connections generally consist of two or more framing members and a mechanical connection device such as a fastener or specialty connection hardware. Adhesives are also used to supplement mechanical attachment of wall finishes or floor sheathing to wood.

This course focuses on conventional wood connections that typically use nails, bolts, screws, and some specialty hardware. The course also addresses relevant concrete and masonry connections in accordance with the applicable provisions of *Building Code Requirements for Structural Concrete* (ACI-318) and *Building Code Requirements for Masonry Structures* (ACI-530)(ACI, 1999a; ACI 1999b). When referring to the NDS, ACI-318, or ACI-530, the course identifies particular sections as NDS 12.1, ACI-318 22.5, or ACI-530 5.12.

For most connections in typical residential construction, the connection design may be based on prescriptive tables found in the applicable residential building code (ICC, 2003). Table 1.1 depicts a commonly recommended nailing schedule for wood-framed homes.

TABLE 1.1 Recommended Nailing Schedule for a Wood-Framed Home

Application	Nailing Method	Number of Nails	Size of Nails	Notes
Header to joist	End-nail	3	16d	
Joist to sill or girder	Toenail Toenail	2 3	10d 8d	
Header and stringer (band) joists to sill	Toenail		8d	16 inches on center
Board sheathing	Face-nail	2 or 3	8d	To each joist
Stud to sole plate or top plate	End-nail Toenail	2 4	16d 8d	At each stud
Sole plate to joist or blocking	Face-nail		16d	16 inches on center
Doubled studs	Face-nail, stagger		10d	16 inches on center
End stud of interior wall to exterior wall stud	Face-nail		16d	16 inches on center
Upper top plate to lower top plate	Face-nail		10d	16 inches on center
Double top plate, laps and intersections	Face-nail	4	10d	
Continuous header, two pieces, each edge	Face-nail		10d	12 inches on center
Ceiling joist to top wall plates	Toenail	3	8d	
Ceiling joist laps at partition	Face-nail	4	16d	
Rafter to top plate	Toenail	3	8d	
Rafter to ceiling joist	Face-nail	4	16d	
Rafter to valley or hip rafter	Toenail	4	10d	
Rafter to ridge board	End-nail Toenail	3 4	16d 8d	
Collar beam to rafter, 2-inch member	Face-nail	2	12d	
Collar beam to rafter, 1-inch member	Face-nail	3	8d	
Diagonal let-in brace to each stud and plate, 1-inch member	Face-nail	2	8d	
Intersecting studs at corners	Face-nail		16d	12 inches on center
Build-up girder and beams, three or more members, each edge	Face-nail		10d	12 inches on center each ply
Maximum ½-inch thick (or less) wood structural panel wall edge	Face-nail	6d at 6 inches in center at panel edges: 12 inches on center at intermediate framing		
Minimum ½-inch thick (or greater) wood structural panel wall/roof/floor sheathing	Face-nail	8d at 6 inches in center at panel edges: 12 inches on center at intermediate framing		
Wood sill plate to concrete or masonry		½-inch diameter anchor bolt at 6 feet on center and within 1 foot from ends of sill members		

Source: *Based on current industry practice and other sources (ICC,1998, NAHB, 1994; NAHB, 1982).*
Note:
In practice, types of nails include common, sinker, box, or pneumatic; refer to Section 1.2 for descriptions of these fasteners. Some recent codes have specified that common nails are to be used in all cases. However, certain connections may not necessarily require such a nail or may actually be weakened by use of a nail that has too large a diameter (i.e., causing splitting of wood members). Other codes allow box nails to be used in most or all cases. NER-272 guidelines for pneumatic fasteners should be consulted (NES, Inc., 1997). However, the NER-272 guidelines are based on simple, conservative conversions of various code nail schedules, such as above, using the assumption that the required performance is defined by a common nail in all applications. In short, there is a general state of confusion regarding appropriate nailing requirements for the multitude of connections and related purposes in conventional residential construction.

The NDS recognizes in NDS 7.1.1.4 that “extensive experience” constitutes a reasonable basis for design; therefore, the engineer may use Table 1.1 for many, if not all, connections. However, the engineer should consider carefully the footnote to Table 1.1 and verify that the connection complies with local requirements, practice, and design conditions for residential construction. A connection design based on the NDS or other sources may be necessary for special conditions such as high-hazard seismic or wind areas and when unique structural details or materials are used.

In addition to the conventional fasteners mentioned above, many specialty connectors and fasteners are available on today's market. The reader is encouraged to gather, study, and scrutinize manufacturer literature regarding specialty fasteners, connectors, and tools that meet a wide range of connection needs.

1.2 Types of Mechanical Fasteners

Mechanical fasteners that are generally used for wood-framed house design and construction include the following:

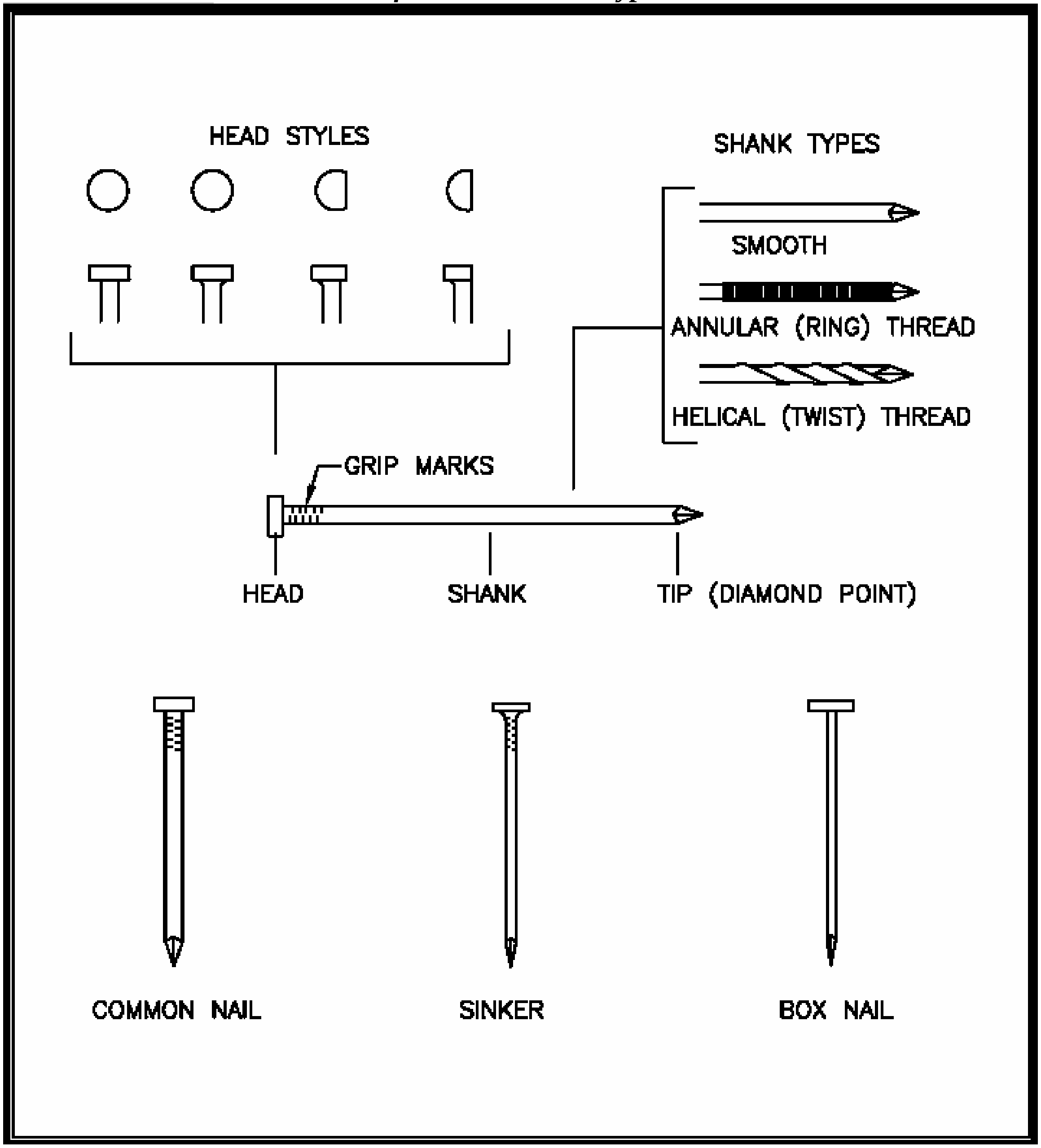
- nails and spikes;
- bolts;
- lag bolts (lag screws); and
- specialty connection hardware.

This section presents some basic descriptions and technical information on the above fasteners. Sections 1.3 and 1.4 provide design values and related guidance. Design examples are provided in Section 1.5 for various typical conditions in residential wood framing and other construction.

1.2.1 Nails

Several characteristics distinguish one nail from another. Figure 1.1 depicts some of the features for a few types of nails that are essential to wood-framed design and construction. This section discusses some of a nail's characteristics relative to structural design; the reader is referred to *Standard Terminology o Nails for Use with Wood and Wood-Base Materials* (ASTM F547) and *Standard Specification for Driven Fasteners: Nails, Spikes, and Staples* (ASTM F 1667) for additional information (ASTM, 1990; ASTM, 1995).

FIGURE 1.1 Elements of a Nail and Nail Types



The most common nail types used in residential wood construction follow:

- *Common nails* are bright, plain-shank nails with a flat head and diamond point. The diameter of a common nail is larger than that of sinkers and box nails of the same length. Common nails are used primarily for rough framing.
- *Sinker nails* are bright or coated slender nails with a sinker head and diamond point. The diameter of the head is smaller than that of a common nail with the same designation. Sinker nails are used primarily for rough framing and applications where lumber splitting may be a concern.
- *Box nails* are bright, coated, or galvanized nails with a flat head and diamond point. They are made of lighter-gauge wire than common nails and sinkers and are commonly used for toenailing and many other light framing connections where splitting of lumber is a concern.
- *Cooler nails* are generally similar to the nails above, but with slightly thinner shanks. They are commonly supplied with ring shanks (i.e., annular threads) as a drywall nail.
- *Power-driven nails* (and staples) are produced by a variety of manufacturers for several types of power-driven fasteners. Pneumatic driven nails and staples are the most popular power-driven fasteners in residential construction. Nails are available in a variety of diameters, lengths, and head styles. The shanks are generally cement-coated and are available with deformed shanks for added capacity. Staples are also available in a variety of wire diameters, crown widths, and leg lengths, however staples should not be used in residual construction. Refer to NER-272 for additional information and design data (NES, Inc., 1997).

Nail lengths and weights are denoted by the *penny weight*, which is indicated by *d*. Given the standardization of common nails, sinkers, and cooler nails, the penny weight also denotes a nail's head and shank diameter. For other nail types, sizes are based on the nail's length and diameter. Table 1.2 arrays dimensions for the nails discussed above. The nail length and diameter are key factors in determining the strength of nailed connections in wood framing. The steel yield strength of the nail may also be important for certain shear connections, yet such information is rarely available for a "standard" lot of nails.

TABLE 1.2 Nail Types, Sizes, and Dimensions

Type of Nail	Nominal Size (penny weight, d)	Length (inches)	Diameter (inches)
Common	6d	2	0.113
	8d	2 ½	0.131
	10d	3	0.148
	12d	3 ¼	0.148
	16d	3 ½	0.162
	20d	4	0.192
Box	6d	2	0.099
	8d	2 ½	0.113
	10d	3	0.120
	12d	3 ¼	0.128
	16d	3 ½	0.135
Sinker	6d	1 7/8	0.092
	8d	2 3/8	0.113
	10d	2 7/8	0.120
	12d	3 1/8	0.135
	16d	3 ¼	0.148
Pneumatic	6d	1 7/8 to 2	0.092 to 0.113
	8d	2 3/8 to 2 ½	0.092 to 0.131
	10d	3	0.120 to 0.148
	12d	3 ¼	0.120 to 0.131
	16d	3 ½	0.131 to 0.162
	20d	4	0.131
Cooler	4d	1 3/8	0.067
	5d	1 5/8	0.080
	6d	1 7/8	0.092

Notes
Based on ASTM F 1667 (ASTM, 1995).
Based on a survey of pneumatic fastener manufacturer data and NER-272 (NES, Inc., 1997).

There are many types of *nail heads*, although three types are most commonly used in residential wood framing.

- The *flat nail head* is the most common head. It is flat and circular, and its top and bearing surfaces are parallel but with slightly rounded edges.
- The *sinker nail head* is slightly smaller in diameter than the flat nail head. It also has a flat top surface; however, the bearing surface of the nail head is angled, allowing the head to be slightly countersunk.
- *Pneumatic nail heads* are available in the above types; however, other head types such as a half-round or D-shaped heads are also common.

The *shank*, as illustrated in Figure 1.1, is the main body of a nail. It extends from the head of the nail to the point. It may be plain or deformed. A plain shank is considered a “smooth” shank, but it may have “grip marks” from the manufacturing process. A deformed shank is most often either threaded or fluted to provide additional withdrawal or pullout resistance. Threads are annular (i.e., ring shank), helical, or longitudinal deformations rolled onto the shank, creating ridges and depressions. Flutes are helical or vertical deformations rolled onto the shank. Threaded nails are most often used to connect wood to wood while fluted nails are used to connect wood to concrete (i.e., sill plate to concrete slab or furring strip to concrete or masonry). Shank diameter and surface condition both affect a nail's capacity.

The *nail tip*, as illustrated in Figure 1.1, is the end of the shank–usually tapered–that is formed during manufacturing to expedite nail riving into a given material. Among the many types of nail points, the *diamond point* is most commonly used in residential wood construction. The diamond point is a symmetrical point with four approximately equal beveled sides that form a pyramid shape. A *cut point* used for concrete cut nails describes a blunt point. The point type can affect nail drivability, lumber splitting, and strength characteristics.

The *material* used to manufacture nails may be steel, stainless steel, heat-treated steel, aluminum, or copper, although the most commonly used materials are steel, stainless steel, and heat-treated steel. *Steel* nails are typically formed from basic steel wire. *Stainless steel* nails are often recommended in exposed construction near the coast or for certain applications such as cedar siding to prevent staining. Stainless steel nails are also recommended for permanent wood foundations. *Heat-treated* steel includes annealed, case-hardened, or hardened nails that can be driven into particularly hard materials such as extremely dense wood or concrete.

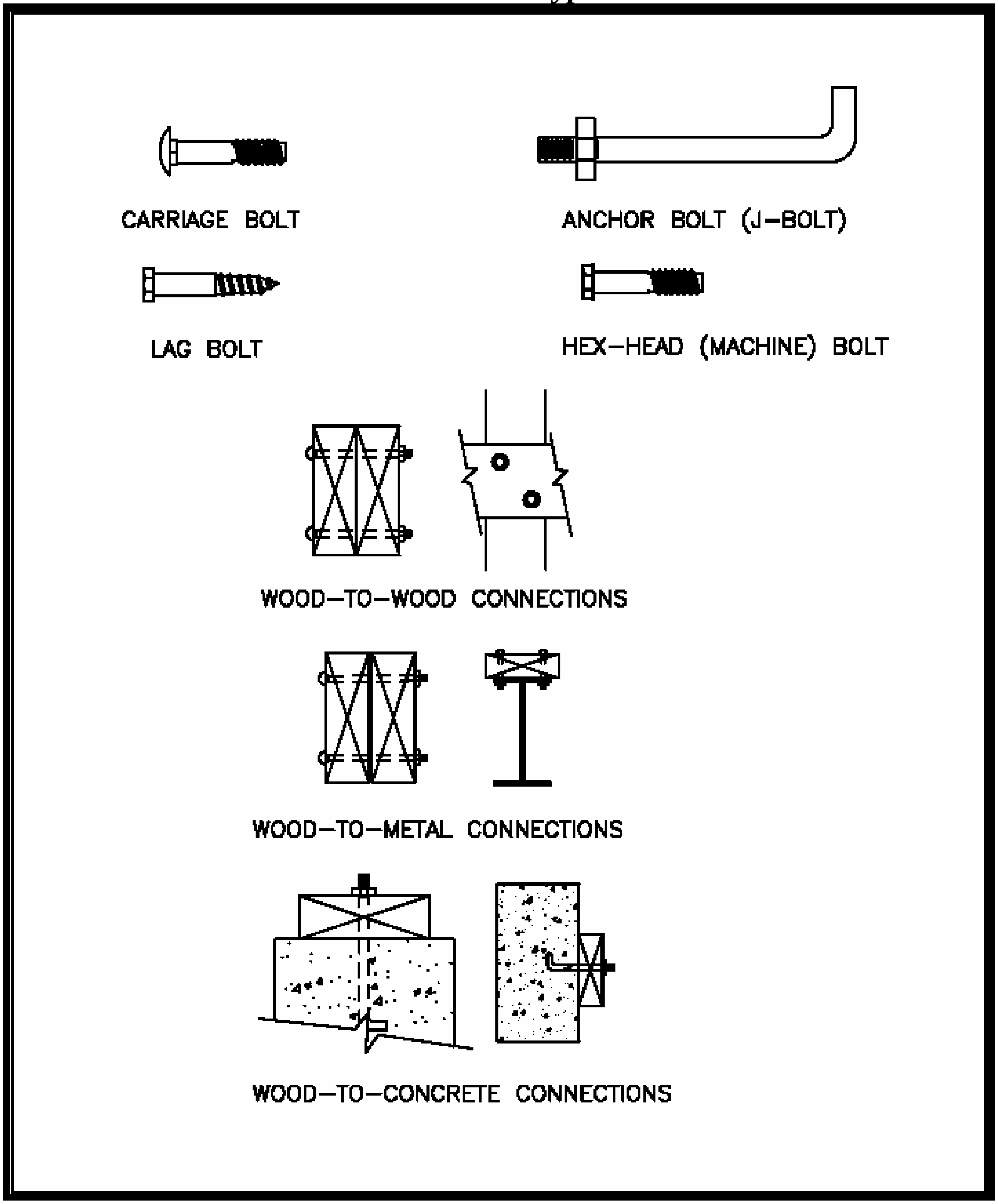
Various nail *coatings* provide corrosion resistance, increased pullout resistance, or ease of driving. Some of the more common coatings in residential wood construction are described below.

- *Bright*. Uncoated and clean nail surface.
- *Cement-coated*. Coated with a heat-sensitive cement that prevents corrosion during storage and improves withdrawal strength depending on the moisture and density of the lumber and other factors.
- *Galvanized*. Coated with zinc by barrel-tumbling, dipping, electroplating, flaking, or hot-dipping to provide a corrosion resistant coating during storage and after installation for either performance or appearance. The coating thickness increases the diameter of the nail and improves withdrawal and shear strength.

1.2.2 Bolts

Bolts are often used for "heavy" connections and to secure wood to other materials such as steel or concrete. In many construction applications, however, special power-driven fasteners are used in place of bolts. Refer to Figure 1.2 for an illustration of some typical bolt types and connections for residential use.

FIGURE 1.2 Bolt and Connection Types



In residential wood construction, bolted connections are typically limited to wood-to-concrete connections unless a home is constructed in a high-hazard wind or seismic area and hold-down brackets are required to transfer shear wall overturning forces. Foundation bolts, typically embedded in concrete or grouted masonry, are commonly referred to as *anchor bolts, J-bolts*,or *mud-sill anchors.* Another type of bolt sometimes used in residential construction is the *structural bolt*, which connects wood to steel or wood to wood. Low-strength ASTM A307 bolts are commonly used in residential construction as opposed to high-strength ASTM A325 bolts, which are more common in commercial applications. Bolt diameters in residential construction generally range from 1/4- to 3/4-inch, although 1/2- to 5/8-inch-diameter bolts are most common, particularly for connecting a wood sill to grouted masonry or concrete.

Bolts are most often installed in predrilled holes. If holes are too small, the possibility of splitting the wood member increases during installation of the bolt. If bored too large, the bolt holes encourage non-uniform dowel (bolt) bearing stresses and slippage of the joint when loaded. Bolt holes should range from 1/32- to 1/16-inch larger than the bolt diameter to prevent splitting and to ensure reasonably uniform dowel bearing stresses.

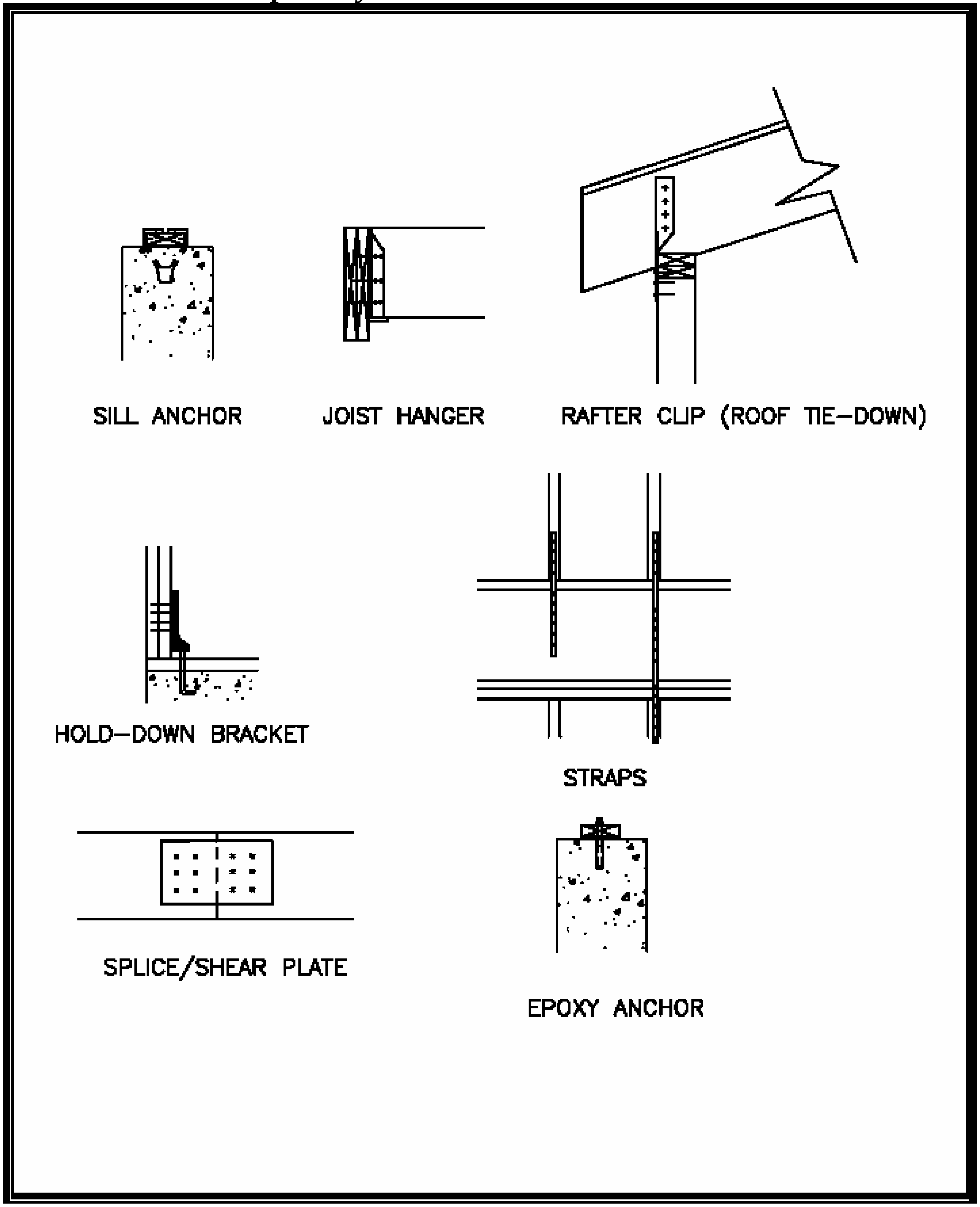
1.2.3 Specialty Connection Hardware

Many manufacturers fabricate specialty connection hardware. The load capacity of a specialty connector is usually obtained through testing to determine the required structural design values. The manufacturer's product catalogue typically provides the required values. Thus, the engineer can select a standard connector based on the design load determined for a particular joint or connection. However, the engineer should carefully consider the type of fastener to be used with the connector; sometimes a manufacturer requires or offers proprietary nails, screws, or other devices. It is also recommended that the engineer verify the safety factor and strength adjustments used by the manufacturer, including the basis of the design value. In some cases, as with nailed and bolted connections, the basis is a serviceability limit state (i.e., slip or deformation) and not ultimate capacity.

A few examples of specialty connection hardware are illustrated in Figure 1.3 and discussed below.

- *Sill anchors* are used in lieu of foundation anchor bolts. Many configurations are available in addition to the one shown in Figure 1.3.
- *Joist hangers* are used to attach single or multiple joists to the side of girders or header joists.
- *Rafter clips* and *roof tie-downs* are straps or brackets that connect roof framing members to wall framing to resist roof uplift loads associated with high-wind conditions.
- *Hold-down brackets* are brackets that are bolted, nailed, or screwed to wall studs or posts and anchored to the construction below (i.e., concrete, masonry, or wood) to "hold down" the end of a member or assembly (i.e., shear wall).
- *Strap ties* are pre-punched straps or coils of strapping that are used for a variety of connections to transfer tension loads.
- *Splice plates* or *shear plates* are flat plates with pre-punched holes for fasteners to transfer shear or tension forces across a joint.
- *Epoxy-set anchors* are anchor bolts that are drilled and installed with epoxy adhesives into concrete after the concrete has cured and sometimes after the framing is complete so that the required anchor location is obvious.

FIGURE 1.3 Specialty Connector Hardware



1.2.4 Lag Screws

Lag screws are available in the same diameter range as bolts; the principal difference between the two types of connectors is that a lag screw has screw threads that taper to a point. The threaded portion of the lag screw anchors itself in the main member that receives the tip. Lag screws (often called lag bolts) function as bolts in joints where the main member is too thick to be economically penetrated by regular bolts. They are also used when one face of the member is not accessible for a "through-bolt." Holes for lag screws must be carefully drilled to one diameter and depth for the shank of the lag screw and to a smaller diameter for the threaded portion. Lag screws in residential applications are generally small in diameter and may be used to attach garage door tracks to wood framing, steel angles to wood framing supporting brick veneer over wall openings, various brackets or steel members to wood, and wood ledgers to wall framing.

1.3 Wood Connection Design

1.3.1 General

This section covers the design procedures for nails, bolts, and lag screws. The procedures are intended for allowable stress design (ASD) such that loads should be determined accordingly. While wood connections are generally responsible for the complex, nonlinear behavior of wood structural systems, the design procedures are straightforward. The connection values are generally conservative from a structural safety standpoint. Further, the basic or tabulated design values are associated with tests of single fasteners in standardized conditions. As a result, several adjustments to account for various factors that alter the performance of a connection; in particular, the performance of wood connections is highly dependent on the species (i.e., density or specific gravity) of wood. Table 1.3 provides the specific gravity values of various wood species typically used in house construction.

TABLE 1.3
Common Framing Lumber Species and Specific Gravity Values

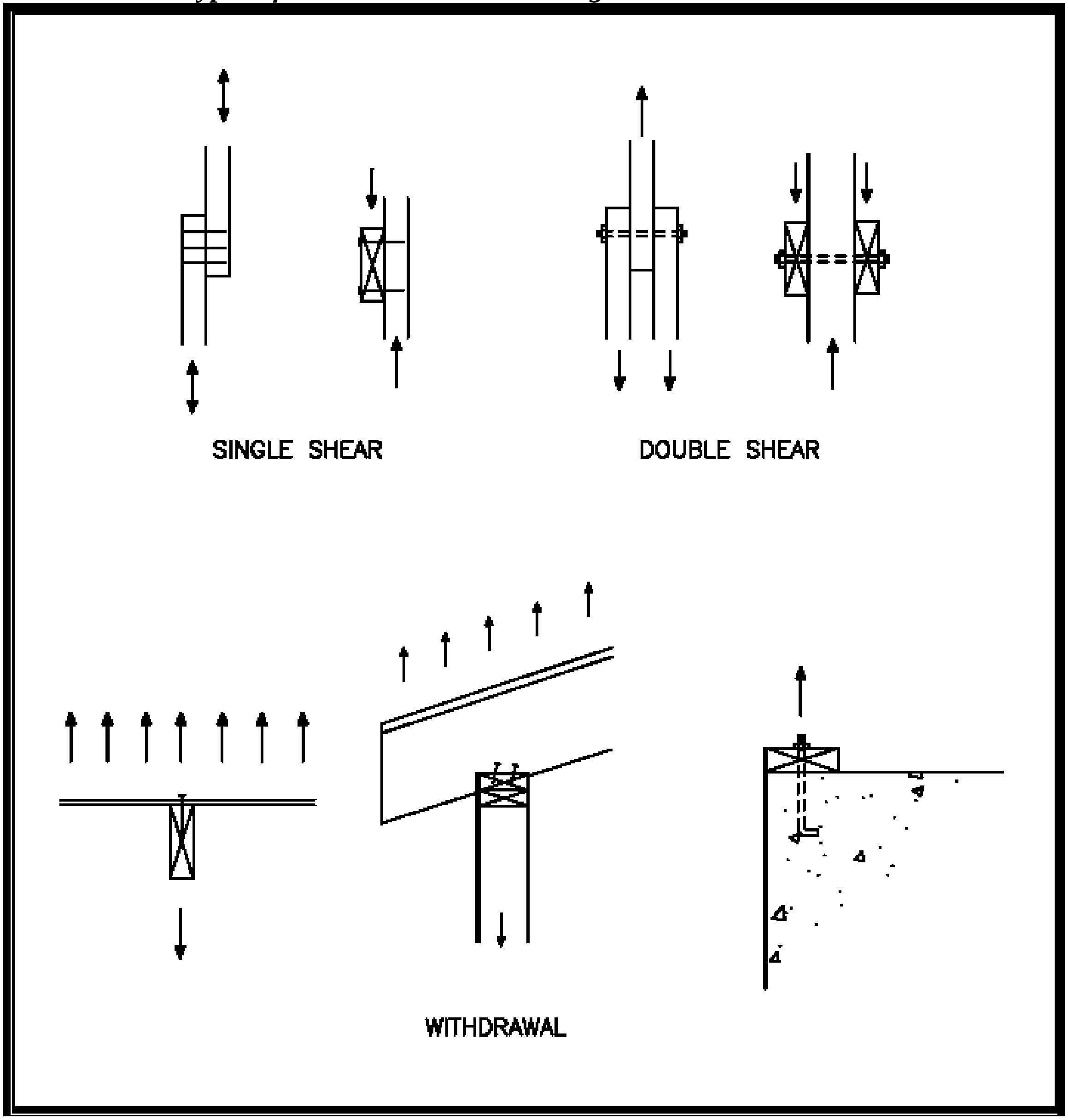
Lumber Species	Specific Gravity, G
Southern Pine (SP)	0.55
Douglas Fir-Larch (DF-L)	0.50
Hem-Fir (HF)	0.43
Spruce/Pine/Fir (SPF)	0.42
Spruce/Pine/Fir (South)	0.36

The moisture condition of the wood is also critical to long-term connection performance, particularly for nails in withdrawal. In some cases, the withdrawal value of fasteners installed in moist lumber can decrease by as much as 50 percent over time as the lumber dries to its equilibrium moisture content. At the same time, a nail may develop a layer of rust that increases withdrawal capacity. In contrast, deformed shank nails tend to hold their withdrawal capacity much more reliably under varying moisture and use conditions. For this and other reasons, the design nail withdrawal capacities for smooth shank nails are based on a fairly conservative reduction factor, resulting in about one-fifth of the average ultimate tested withdrawal capacity. The reduction includes a safety factor as well as a load duration adjustment (i.e., decreased by a factor of 1.6 to adjust from short-term tests to normal duration load). Design values for nails and bolts in shear are based on a deformation (i.e., slip) limit state and not their ultimate capacity, resulting in a safety factor that may range from 3 to 5 based on ultimate tested capacities. One argument for retaining a high safety factor in shear connections is that the joint may creep under long-term load. While creep is not a concern for many joints, slip of joints in a trussed assembly (i.e., rafter-ceiling joist roof framing) is critical and, in key joints, can result in a magnified deflection of the assembly over time (i.e., creep).

In view of the above discussion, there are a number of uncertainties in the design of connections that can lead to conservative or unconservative designs relative to the intent and practical experience. The engineer is advised to follow procedures carefully, but should be prepared to make practical adjustments as dictated by sound judgment and experience and what is allowed by NDS.

Withdrawal design values for nails and lag screws in the NDS are based on the fastener being oriented perpendicular to the grain of the wood. Shear design values in wood connections are also based on the fastener being oriented perpendicular to the grain of wood. However, the lateral (shear) design values are dependent on the direction of loading relative to the wood grain direction in each of the connected members. Refer to Figure 1.4 for an illustration of various connection types and loading conditions.

FIGURE 1.4 Types of Connections and Loading Conditions



The NDS provides tabulated connection design values that use the following symbols for the three basic types of loading:

- W–withdrawal (or tension loading);
- $Z_{\perp}$–shear perpendicular to wood grain; and
- $Z_{\parallel}$–shear parallel to wood grain.

In addition to the already tabulated design values for the above structural resistance properties of connections, the NDS provides calculation methods to address conditions that may not be covered by the tables and that give more flexibility to the design of connections. The methods are appropriate for use in hand calculations or with computer spreadsheets.

For withdrawal, the design equations are relatively simple empirical relationships (based on test data) that explain the effect of fastener size (diameter), penetration into the wood, and density of the wood. For shear, the equations are somewhat more complex because of the multiple failure modes that may result from fastener characteristics, wood density, and size of the wood members. Six shear-yielding modes (and a design equation for each) address various yielding conditions in either the wood members or the fasteners that join the members. The critical yield mode is used to determine the design shear value for the connection. Refer to NDS Appendix I for a description of the yield modes.

The yield equations in the NDS are based on general dowel equations that use principles of engineering mechanics to predict the shear capacity of a doweled joint. The general dowel equations can be used with joints that have a gap between the members and they can also be used to predict ultimate capacity of a joint made of wood, wood and metal, or wood and concrete. However, the equations do not account for friction between members or the anchoring/cinching effect of the fastener head as the joint deforms and the fastener rotates or develops tensile forces. These effects are important to the ultimate capacity of wood connections in shear and, therefore, the general dowel equations may be considered to be conservative; refer to Section 1.3.6. For additional guidance and background on the use of the general dowel equations, refer to the NDS *Commentary* and other useful design resources available through the American Forest & Paper Association (AF&PA, 1999; Showalter, Line, and Douglas, 1999).

1.3.2 Adjusted Allowable Design Values

Design values for wood connections are subject to adjustments in a manner similar to that required for wood members themselves. The calculated or tabulated design values for W and Z are multiplied by the applicable adjustment factors to determine adjusted allowable design values, Z' and W', as shown below for the various connection methods (i.e., nails, bolts, and lag screws).

[NDS 12.3 & 1.3]

$Z' = ZC_DC_MC_tC_gC_\Delta$ for bolts

$Z' = ZC_DC_MC_tC_gC_\Delta\ C_dC_{eg}$ for lag screws

$Z' = ZC_DC_MC_tC_dC_{eg}\ C_{di}C_{tn}$ for nails and spikes

[NDS 12.2&1.3]

$W' = WC_DC_MC_tC_{tn}$ for nails and spikes

$W' = WC_DC_MC_tC_{eg}$ for lag screws

The adjustment factors and their applicability to wood connection design are briefly described as follows:

- C_D*–Load Duration Factor* (NDS 2.3.2)–applies to W and Z values for all fasteners based on design load duration but shall not exceed 1.6 (i.e., wind and earthquake load duration factor).
- C_M*–Wet Service Factor* (NDS 7.3.3)–applies to W and Z values for all connections based on moisture conditions at the time of fabrication and during service; not applicable to residential framing.
- C_t*–Temperature Factor* (NDS 7.3.4)–applies to the W and Z values for all connections exposed to sustained temperatures of greater than 100$_o$F; not typically used in residential framing.
- C_g*–Group Action Factor* (NDS 7.3.6)–applies to Z values of two or more bolts or lag screws loaded in single or multiple shear and aligned in the direction of the load (i.e., rows).
- C_Δ*–Geometry Factor* (NDS 8.5.2, 9.4.)–applies to the Z values for bolts and lag screws when the end distance or spacing of the bolts is less than assumed in the unadjusted design values.
- C_d*–Penetration Depth Factor* (NDS 9.3.3, 12.3.4)–applies to the Z values of lag screws and nails when the penetration into the main member is less than 8D for lag screws or 12D for nails (where D = shank diameter); sometimes applicable to residential nailed connections.
- C_{eg}*–End Grain Factor* (NDS 9.2.2, 9.3.4, 12.3.5)–applies to W and Z values for lag screws and to Z values for nails to account for reduced capacity when the fastener is inserted into the end grain (C_{eg}=0.67).
- C_{di}*–Diaphragm Factor* (NDS 12.3.6)–applies to the Z values of nails only to account for system effects from multiple nails used in sheathed diaphragm construction (C_{di} = 1.1).
- C_{tn}*–Toenail Factor* (NDS 12.3.7)–applies to the W and Z values of toenailed connections (C_{tn} = 0.67 for withdrawal and = 0.83 for shear). It does not apply to slant nailing in withdrawal or shear; refer to Section 1.3.6.

The total allowable design value for a connection (as adjusted by the appropriate factors above) must meet or exceed the design load determined for the connection. The values for W and Z are based on single fastener connections. In instances of connections involving multiple fasteners, the values for the individual or single fastener can be summed to determine the total connection design value only when C_g is applied (to bolts and lag screws only) and fasteners are the same type and similar size. However, this approach may overlook certain system effects that can improve the actual performance of the joint in a constructed system or assembly (see Section 1.3.6). Conditions that may decrease estimated performance, such as prying action induced by the joint configuration and/or eccentric loads and other factors should also be considered.

In addition, the NDS does not provide values for nail withdrawal or shear when wood structural panel members (i.e., plywood or oriented strand board) are used as a part of the joint. This type of joint–wood member to structural wood panel–occurs frequently in residential construction. Z values can be estimated by using the yield equations for nails in NDS 12.3.1 and assuming a reasonable specific gravity (density) value for the wood structural panels, such as G = 0.5. W values for nails in wood structural panels can be estimated in a similar fashion by using the withdrawal equation presented in the next section. The tabulated W and Z values in NDS 12 may also be used, but with some caution as to the selected table parameters.

1.3.3 Nailed Connections

The procedures in NDS 12 provide for the design of nailed connections to resist shear and withdrawal loads in wood-to-wood and metal-to-wood connections. Many specialty "nail-type" fasteners are available for wood-to-concrete and even wood-to-steel connections. The engineer should consult manufacturer data for connection designs that use proprietary fastening systems.

The withdrawal strength of a smooth nail (driven into the side grain of lumber) is determined in accordance with either the empirical design equation below or NDS Table 12.2A.

[NDS 12.2.1]

$W = 1380(G)^{5/2} DL_p$ unadjusted withdrawal design value (lb) for a smooth shank nail

where,

G = specific gravity of the lumber member receiving the nail tip
D = the diameter of the nail shank (in)
L_p = the depth of penetration (in) of the nail into the member receiving the nail tip

The design strength of nails is greater when a nail is driven into the side rather than the end grain of a member. Withdrawal information is available for nails driven into the side grain; however, the withdrawal capacity of a nail driven into the end grain is assumed to be zero because of its unreliability. Furthermore, the NDS does not provide a method for determining withdrawal values for deformed shank nails. These nails significantly enhance withdrawal capacity and are frequently used to attach roof sheathing in high-wind areas. They are also used to attach floor sheathing and some siding materials to prevent nail "back-out." The use of deformed shank nails is usually based on experience or preference.

The design shear value, Z, for a nail is typically determined by using the following tables from NDS 12:

- Tables 12.3A and B. Nailed wood-to-wood, single-shear (two member) connections with the same species of lumber using box or common nails, respectively.
- Tables 12.3E and F. Nailed metal plate-to-wood connections using box or common nails, respectively.

The yield equations in NDS 12.3 may be used for conditions not represented in the design value tables for Z. Regardless of the method used to determine the Z value for a single nail, the value must be adjusted as described in Section 1.3.2. As noted in the NDS, the single nail value is used to determine the design value.

It is also worth mentioning that the NDS provides an equation for determining allowable design value for shear when a nailed connection is loaded in combined withdrawal and shear (see NDS 12.3.8, Equation 12.3-6). The equation appears to be most applicable to a gable-end truss connection to the roof sheathing under conditions of roof sheathing uplift and wall lateral load owing to wind. The engineer might contemplate other applications but should take care in considering the combination of loads that would be necessary to create simultaneous uplift and shear worthy of a special calculation.

1.3.4 Bolted Connections

Bolts may be designed in accordance with NDS 8 to resist shear loads in wood-to-wood, wood-to-metal, and wood-to-concrete connections. As mentioned, many specialty “bolt-type” fasteners can be used to connect wood to other materials, particularly concrete and masonry. One common example is an epoxyset anchor. Manufacturer data should be consulted for connection designs that use proprietary fastening systems.

The design shear value Z for a bolted connection is typically determined by using the following tables from NDS 8:

- Table 8.2A. Bolted wood-to-wood, single-shear (two-member) connections with the same species of lumber.
- Table 8.2B. Bolted metal plate-to-wood, single-shear (two member) connections; metal plate thickness of 1/4-inch minimum.
- Table 8.2D. Bolted single-shear wood-to-concrete connections; based on minimum 6-inch bolt embedment in minimum f_c = 2,000 psi concrete.

The yield equations of NDS 8.2 (single-shear joints) and NDS 8.3 (double-shear joints) may be used for conditions not represented in the design value tables. Regardless of the method used to determine the Z value for a single bolted connection, the value must be adjusted as described in Section 1.3.2.

It should be noted that the NDS does not provide W values for bolts. The tension value of a bolt connection in wood framing is usually limited by the bearing capacity of the wood as determined by the surface area of a washer used underneath the bolt head or nut. When calculating the bearing capacity of the wood based on the tension in a bolted joint, the engineer should use the small bearing area value C_b to adjust the allowable compressive stress perpendicular to grain $F_{c\perp}$ (see NDS 2.3.10). It should also be remembered that the allowable compressive stress of lumber is based on a deformation limit state, not capacity. In addition, the engineer should verify the tension capacity of the bolt and its connection to other materials (i.e., concrete or masonry as covered in Section 1.4). The bending capacity of the washer should also be considered. For example, a wide but thin washer will not evenly distribute the bearing force to the surrounding wood.

The arrangement of bolts and drilling of holes are extremely important to the performance of a bolted connection. The engineer should carefully follow the minimum edge, end, and spacing requirements of NDS 8.5. When necessary, the engineer should adjust the design value for the bolts in a connection by using the geometry factor C_Δ and the group action factor C_g discussed in Section 1.3.2.

Any possible tensional load on a bolted connection (or any connection for that manner) should also be considered in accordance with the NDS. In such conditions, the pattern of the fasteners in the connection can become critical to performance in resisting both a direct shear load and the loads created by a tensional moment on the connection. Fortunately, this condition is not often applicable to typical light-frame construction. However, cantilevered members that rely on connections to “anchor” the cantilevered member to other members will experience this effect, and the fasteners closest to the cantilever span will experience greater shear load. One example of this condition sometimes occurs with balcony construction in residential buildings; failure to consider the effect discussed above has been associated with some notable balcony collapses.

For wood members bolted to concrete, the design lateral values are provided in NDS Table8.2E. The yield equations (or general dowel equations) may also be used to conservatively determine the joint capacity. A recent study has made recommendations regarding reasonable assumptions that must be made in applying the yield equations to bolted wood-to-concrete connections (Stieda, et al., 1999). Using symbols defined in the NDS, the study recommends an R_e value of 5 and an R_t value of 3. These assumptions are reported as being conservative because fastener head effects and joint friction are ignored in the general dowel equations.

1.3.5 Lag Screws

Lag screws (or lag bolts) may be designed to resist shear and withdrawal loads in wood-to-wood and metal-to-wood connections in accordance with NDS 9. As mentioned, many specialty “screw-type” fasteners can be installed in wood. Some tap their own holes and do not require predrilling. Manufacturer data should be consulted for connection designs that use proprietary fastening systems.

The withdrawal strength of a lag screw (inserted into the side grain of lumber) is determined in accordance with either the empirical design equation below or NDS Table 9.2A. It should be noted that the equation below is based on single lag screw connection tests and is associated with a reduction factor of 0.2 applied to average ultimate withdrawal capacity to adjust for load duration and safety. Also, the penetration length of the lag screw L_p into the main member does not include the tapered portion at the point. NDS Appendix L contains dimensions for lag screws.

[NDS 9.2.1]

$$W = 1800\,(G)^{3/2}\,D^{3/4}\,L_p \quad \text{unadjusted withdrawal design value (lb) for a lag screw}$$

where,

G = specific gravity of the lumber receiving the lag screw tip
D = the diameter of the lag screw shank (in)
L_p = the depth of penetration (in) of the lag screw into the member receiving the tip, less the tapered length of the tip

The allowable withdrawal design strength of a lag screw is greater when the screw is installed in the side rather than the end grain of a member. However, unlike the treatment of nails, the withdrawal strength of lag screws installed in the end grain may be calculated by using the C_{eg} adjustment factor with the equation above.

The design shear value Z for a lag screw is typically determined by using the following tables from NDS 9:

- Table 9.3A. Lag screw, single-shear (two-member) connections with the same species of lumber for both members.
- Table 9.3B. Lag screw and metal plate-to-wood connections.

The yield equations in NDS 9.3 may be used for conditions not represented in the design value tables for Z. Regardless of the method used to determine the Z value for a single lag screw, the value must be adjusted as described in Section 1.3.2.

The NDS provides an equation for determining the allowable shear design value when a lag screw connection is loaded in combined withdrawal and shear (see NDS 9.3.5, Equation 9.3-6). The equation does not appear to apply to typical uses of lag screws in residential construction.

1.3.6 System Design Considerations

As with any building code or design specification, the NDS provisions may or may not address various conditions encountered in the field.

First, as a general design consideration, "crowded" connections should be avoided. If too many fasteners are used (particularly nails), they may cause splitting during installation. When connections become "crowded," an alternative fastener or connection detail should be considered. Basically, the connection detail should be practical and efficient.

Second, system effects within a particular joint (i.e., element) that uses multiple bolts or lag screws (i.e. the group action factor C_g), doe not include provisions regarding the system effects of multiple joints in an assembly or system of components. Therefore, some consideration of system effects is given below based on several relevant studies related to key connections in a home that allow the dwelling to perform effectively as a structural unit.

Sheathing Withdrawal Connections

Several recent studies have focused on roof sheathing attachment and nail withdrawal, primarily as a result of Hurricane Andrew (HUD, 1999a; McClain, 1997; Cunningham, 1993; Mizzell and Schiff, 1994; and Murphy, Pye, and Rosowsky, 1995). The studies identify problems related to predicting the pull-off capacity of sheathing based on single nail withdrawal values and determining the tributary withdrawal load (i.e., wind suction pressure) on a particular sheathing fastener. One clear finding, however, is that the nails on the interior of the roof sheathing panels are the critical fasteners (i.e., initiate panel withdrawal failure) because of the generally larger tributary area served by these fasteners. The studies also identified benefits to the use of screws and deformed shank nails. However, use of a standard geometric tributary area of the sheathing fastener and the wind loads, along with the NDS withdrawal values (Section 1.3.3), will generally result in a reasonable design using nails. A wind load duration factor should be applied to adjust the withdrawal values since a commensurate reduction is implicit in the design withdrawal values relative to the short-term, tested, ultimate withdrawal capacities (see Section 1.3).

One of the above stated studies found that the lower-bound (i.e., 5th percentile) sheathing pull-off resistance was considerably higher than that predicted by use of single-nail test values (Murphy, Pye, and Rosowsky, 1995). The difference was as large as a factor of 1.39 greater than the single nail values. This suggests a withdrawal system factor of at least 1.3 for sheathing nails, it should be subject to additional considerations. For example, sheathing nails are placed by people using tools in somewhat adverse conditions (i.e., on a roof), not in a laboratory. Therefore, this system effect may be best considered as a reasonable "construction tolerance" on actual nail spacing variation relative to that intended by design. Thus, an 8 to 9 inch nail spacing on roof sheathing nails in the panel's field could be "tolerated" when a 6 inch spacing is "targeted" by design.

Roof-to-Wall Connections

A couple of studies (Reed, et al., 1996; Conner, et al., 1987) have investigated the capacity of roof-to-wall (i.e., sloped rafter to top plate) connections using conventional toenailing and other enhancements (i.e., strapping, brackets, gluing, etc.). The primary concern is related to high wind conditions, such as experienced during Hurricanes and other extreme wind events.

First, as a matter of clarification, the toenail reduction factor C_{tn} does not apply to s*lant-nailing* such as those used for rafter-to-wall connections and floor-to-wall connections in conventional residential construction. Toenailing occurs when a nail is driven at an angle in a direction parallel-to-grain at the end of a member (i.e., a wall stud toenail connection to the top or bottom plate that may be used instead of end nailing). Slant nailing occurs when a nail is driven at an angle, but in a direction perpendicular-to-grain through the side of the member and into the face grain of the other (i.e., from a roof rafter or floor band joist to a wall top plate). Though a generally reliable connection in most homes and similar structures built in the United States, even a well-designed slant-nail connection used to attach roofs to walls will become impractical in hurricane-prone regions or similar high-wind areas. In these conditions, a metal strap or bracket is preferred.

Based on the studies of roof-to-wall connections, five key findings are summarized as follows:

1. In general, it was found that slant-nails (not to be confused with toenails) in combination with metal straps or brackets do not provide directly additive uplift resistance.
2. A basic metal twist strap placed on the interior side of the walls (i.e., gypsum board side) resulted in top plate tear-out and premature failure. However, a

strap placed on the outside of the wall (i.e., structural sheathing side) was able to develop its full capacity without additional enhancement of the conventional stud-to-top plate connection (see Table 1.1).

3. The withdrawal capacity for single joints with slant nails was reasonably predicted by NDS with a safety factor of about 2 to 3.5. However, with multiple joints tested simultaneously, a system factor on withdrawal capacity of greater than 1.3 was found for the slant nailed rafter-to-wall connection. A similar system effect was not found on strap connections, although the strap capacity was substantially higher. The ultimate capacity of the simple strap connection (using five 8d nails on either side of the strap–five in the spruce rafter and five in the southern yellow pine top plate) was found to be about 1,900 pounds per connection. The capacity of three 8d common slant nails used in the same joint configuration was found to be 420 pounds on average, and with higher variation. When the three 8d common toenail connection was tested in an assembly of eight such joints, the average ultimate withdrawal capacity per joint was found to be 670 pounds with a somewhat lower variation. Similar "system" increases were not found for the strap connection. The 670 pounds capacity was similar to that realized for a rafter-to-wall joint using three 16d box nails in Douglas fir framing.
4. It was found that the strap manufacturer's published value had an excessive safety margin of greater than 5 relative to average ultimate capacity. Adjusted to an appropriate safety factor in the range of 2 to 3 (as calculated by applying NDS nail shear equations by using a metal side plate), the strap (a simple 18g twist strap) would cover a multitude of high wind conditions with a simple, economical connection detail.
5. The use of deformed shank (i.e., annular ring) nails was found to increase dramatically the uplift capacity of the roof-to-wall connections using the slant nailing method.

Heel Joint in Rafter-to-Ceiling Joist Connections

The heel joint connections at the intersection of rafters and ceiling joists have long been considered one of the weaker connections in conventional wood roof framing. This highly stressed joint is one of the accolades of using a wood truss rather than conventional rafter framing (particularly in high-wind or snow-load conditions). However, the engineer must understand the performance of a conventional rafter-ceiling joist heel joint connection since they are frequently encountered in residential construction.

First, conventional rafter and ceiling joist (cross-tie) framing is simply a "site-built" truss. Therefore, the joint loads can be analyzed by using methods that are applicable to trusses (i.e., pinned joint analysis). However, the performance of the system should be considered. A system factor of 1.1 is applicable to tension members and connections. Therefore, the calculated shear capacity of the nails in the heel joint (and in ceiling joist splices) may be multiplied by a system factor of 1.1, which is considered conservative. Second, it must be remembered that the nail shear values are based on a

deformation limit and generally have a conservative safety factor of three to five relative to the ultimate capacity.
Finally, the nail values should be adjusted for duration of load (i.e., snow load duration factor of 1.15 to 1.25). With these considerations and with the use of rafter support braces at or near mid-span (as is common), reasonable heel joint designs should be possible for most typical design conditions in residential construction.

Wall-to-Floor Connections

When wood sole plates are connected to wood floors, many nails are often used, particularly along the total length of the sole plate or wall bottom plate. When connected to a concrete slab or foundation wall, there are usually several bolts along the length of the bottom plate. This points toward the question of possible system effects in estimating the shear capacity (and uplift capacity) of these connections for design purposes.

In recent shear wall tests, walls connected with pneumatic nails (0.131-inch diameter by 3 inches long) spaced in pairs at 16 inches on center along the bottom plate were found to resist over 600 pounds in shear per nail. The bottom plate was Spruce-Pine-Fir lumber and the base beam was Southern Yellow Pine. This value is about 4.5 times the adjusted allowable design shear capacity predicted by use of the NDS equations. Connections using 5/8-inch-diameter anchor bolts at 6 feet on center (all other conditions equal) were tested in full shear wall assemblies; the ultimate shear capacity per bolt was found to be 4,400 pounds. This value is about 3.5 times the adjusted allowable design shear capacity per the NDS equations. These safety margins appear excessive and should be considered by the engineer when evaluating similar connections from a practical "system" standpoint.

1.4 Design of Concrete and Masonry Connections

1.4.1 General

In typical residential construction, the interconnection of concrete and masonry elements or systems is generally related to the foundation and usually handled in accordance with standard or accepted practice. The bolted wood member connections to concrete as in Section 1.3.4 are suitable for bolted wood connections to properly grouted masonry. Moreover, numerous specialty fasteners or connectors (including power driven and cast-in-place) can be used to fasten wood materials to masonry or concrete. The engineer should consult the manufacturer's literature for available connectors, fasteners, and design values.

This section discusses some typical concrete and masonry connection designs in accordance with the ACI 318 concrete design specification and ACI 530 masonry design specification.

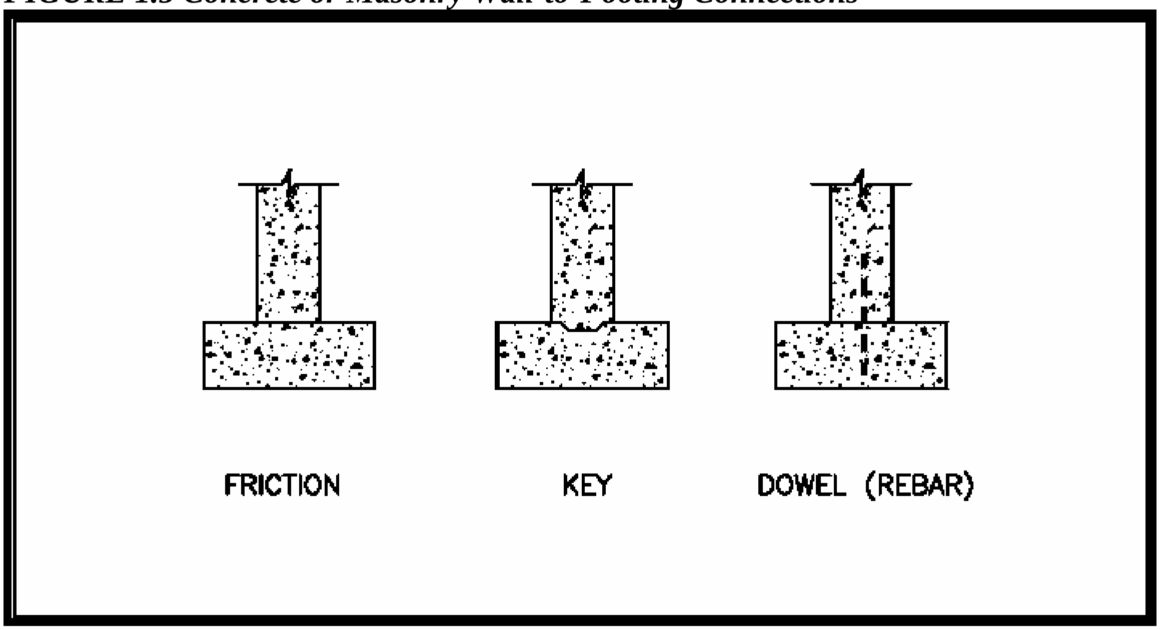
1.4.2 Concrete or Masonry Foundation Wall to Footing

Footing connections, if any, are intended to transfer shear loads from the wall to the footing below. The shear loads are generally produced by lateral soil pressure acting on the foundation. Footing-to-wall connections for residential construction are constructed in any one of the following three ways (refer to Figure 1.5 for illustrations of the connections):

- no vertical reinforcement or key;
- key only; or
- dowel only.

Generally, no special connection is needed in nonhurricane-prone or low- to moderate-hazard seismic areas. Instead, friction is sufficient for low, unbalanced backfill heights while the basement slab can resist slippage for higher backfill heights on basement walls. The basement slab abuts the basement wall near its base and thus provides lateral support. If gravel footings are used, the unbalanced backfill height needs to be sufficiently low (i.e., less than 3 feet), or means must be provided to prevent the foundation wall from slipping sideways from lateral soil loads. Again, a basement slab can provide the needed support. Alternatively, a footing key or doweled connection can be used.

FIGURE 1.5 Concrete or Masonry Wall-to-Footing Connections



Friction Used to Provide Shear Transfer

To verify the amount of shear resistance provided by friction alone, assume a coefficient of friction between two concrete surfaces of $\mu = 0.6$. Using dead loads only, determine the static friction force, $F = \mu NA$, where F is the friction force (lb), N is the dead load (psf), and A is the bearing surface area (sf) between the wall and the footing.

Key Used to Provide Shear Transfer

A concrete key is commonly used to “interlock” foundation walls to footings. If foundation walls are constructed of masonry, the first course of masonry must be grouted solid when a key is used.

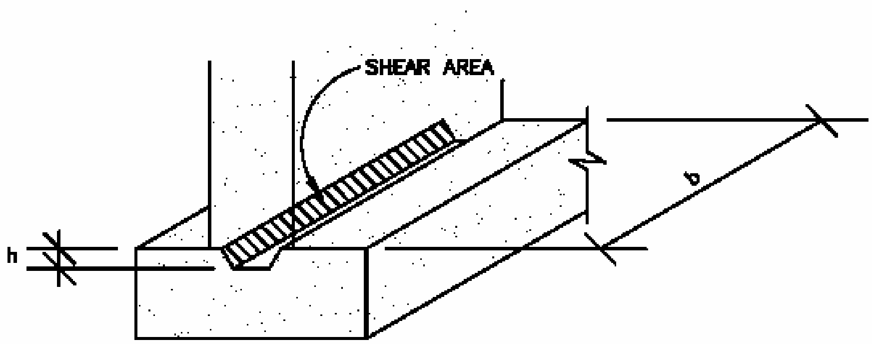
In residential construction, a key is often formed by using a 2x4 wood board with chamfered edges that is placed into the surface of the footing immediately after the concrete pour. Figure 1.6 illustrates a footing with a key. Shear resistance developed by the key is computed in accordance with the equation below.

[ACI-318 22.5]

$$V_u \leq \phi V_n$$

$$V_n = \frac{4}{3}\sqrt{f'_c}\,bh$$

FIGURE 1.6 Key in Concrete Footings



Dowels Used to Provide Adequate Shear Transfer

Shear forces at the base of exterior foundation walls may require a dowel to transfer the forces from the wall to the footing. The equations below described by ACI-318 as the Shear-Friction Method used to develop shear resistance with vertical reinforcement (dowels) across the wall-footing interface.
[ACI-318 11.7]

Masonry Walls

$$l_{be} \geq 12d_b$$

$$B_v = \text{minimum of} \begin{Bmatrix} 350\sqrt[4]{f'_m A_v} \\ 0.12 A_v f_y \end{Bmatrix}$$

Concrete Walls

$$V_u \leq \phi V_n$$

$$V_n = A_{vf} f_y \mu \leq \begin{Bmatrix} 0.2 f'_c A_c \\ 800 A_c \end{Bmatrix}$$

$$A_{vf} = \frac{V_u}{\phi f_y \mu}$$

$$\phi = 0.85$$

If dowels are used to transfer shear forces from the base of the wall to the footing, use the equations below to determine the minimum development length required (refer to Figure 1.7 for typical dowel placement). If development length exceeds the footing thickness, the dowel must be in the form of a hook, which is rarely required in residential construction.

[ACI-318 12.2, 12.5]

Concrete Walls

Standard Hooks

$$l_{hb} = \frac{1200 d_b}{\sqrt{f'_c}} \quad \text{where } f_y = 60{,}000 \text{ psi}$$

$$\xi = \frac{f_y}{60{,}000}$$

$$\omega = \frac{A_{s,required}}{A_{s,provided}}$$

Deformed Bars

$$l_{db} = \left(\frac{3 f_y}{40\sqrt{f'_c}}\right)\left(\frac{\alpha\beta\gamma\lambda}{\frac{c + K_{TR}}{d_b}}\right)$$

$$\frac{c + K_{TR}}{d_b} \leq 2.5$$

$$l_d = l_{db}\left(\frac{A_{s,required}}{A_{s,provided}}\right) \geq 12''$$

[ACI-530 1.12.3,2.1.8]

Masonry Walls

Standard Hooks

$$l_d = 0.0015 d_b F_s \geq 12 \text{ in.}$$

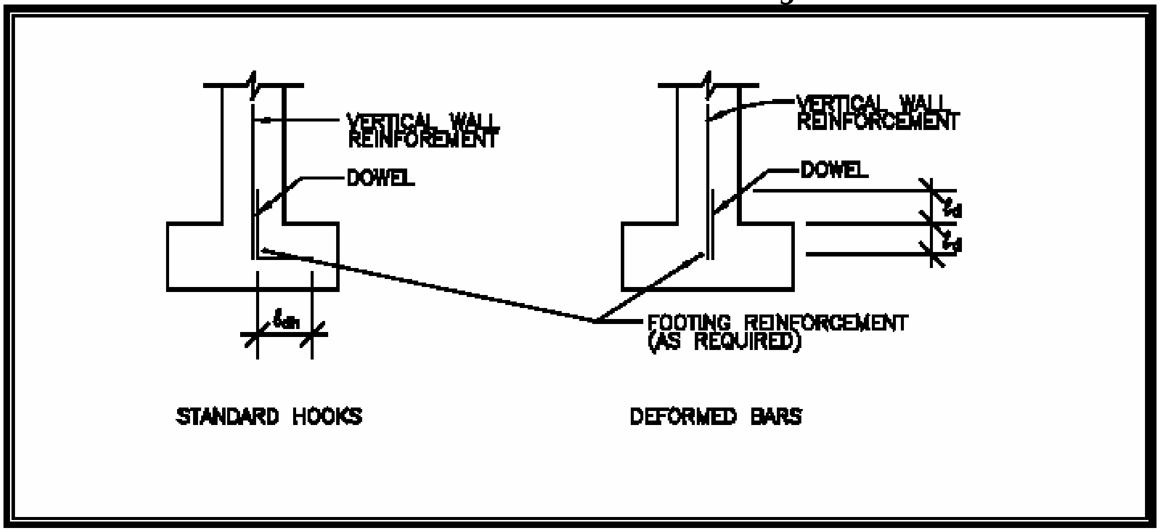
$$l_e = 11.25 d_b$$

Deformed Bars

$$l_d = \text{maximum } \{12 d_b\}$$

$$l_d = \text{maximum } \{12 d_b\}$$

FIGURE 1.7 Dowel Placement in Concrete Footings



The minimum embedment length is a limit specified in ACI-318 that is not necessarily compatible with residential construction conditions and practice. Therefore, it is suggested that a minimum embedment length of 6 to 8 inches for footing dowels, when necessary, in residential construction applications. In addition, dowels are sometimes used in residential construction to connect other concrete elements, such as porch slabs or stairs, to the house foundation to control differential movement. However, exterior concrete "flat work" adjacent to a home should be founded on adequate soil bearing or reasonably compacted backfill. Finally, connecting exterior concrete work to the house foundation requires caution, particularly in colder climates and soil conditions where frost heave may be a concern.

1.4.3 Anchorage and Bearing on Foundation Walls

Anchorage Tension (Uplift) Capacity

The equations below determine whether the concrete or masonry shear area of each bolt is sufficient to resist pull-out from the wall as a result of uplift forces and shear friction in the concrete.

[ACI-318 11.3, ACI-530 2.1.2]

Concrete Foundation Wall

$$V_u \leq \phi V_c$$

$$V_c = 4A_v\sqrt{f'_c}$$

$$A_v = \text{minimum of}\begin{Bmatrix}\pi l_b^{\,2}\\ \pi h^2\end{Bmatrix}$$

Masonry Foundation Wall

$$b_a \leq B_a$$

$$B_a = \text{minimum of}\begin{Bmatrix}0.5A_p\sqrt{f'_m}\\ 0.2A_b f_y\end{Bmatrix}$$

$$A_p = \text{minimum of}\begin{Bmatrix}\pi l_b^{\,2}\\ \pi l_{be}^{\,2}\end{Bmatrix}$$

Bearing Strength

Determining the adequacy of the bearing strength of a foundation wall follows ACI-318 10.17 for concrete or ACI-530 2.1.7 for masonry. The bearing strength of the foundation wall is typically adequate for the loads encountered in residential construction.

[ACI-318 10.17 and ACI-530 2.1.7]

Concrete Foundation Wall	Masonry Foundation Wall
B_c = factored bearing load	
$B_c \le \phi 0.85 f'_c A_1$	$f_a \le F_a$
	$f_a = \dfrac{P}{A_1}$
$\phi = 0.7$	$F_a \le 0.25 f'_m$

When the foundation wall's supporting surface is wider on all sides than the loaded area, the engineer is permitted to determine the design bearing strength on the loaded area by using the equations below.

[ACI-318 10.7 and ACI-530 2.1.7]

Concrete Foundation Wall

$$B_c = \phi 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}} \quad \text{where} \quad \sqrt{\frac{A_2}{A_1}} \le 2$$

Masonry Foundation Wall

$$f_a = \frac{P}{A_1 \sqrt{A_2/A_1}} \quad \text{where} \quad \sqrt{\frac{A_2}{A_1}} \le 2$$

1.5 Design Examples

EXAMPLE 1.1 Roof Sheathing Connections

Given

- Design wind speed is 130 mph gust with an open (coastal) exposure
- Two-story home with a gable roof
- Roof framing lumber is Southern Yellow Pine (G=0.55)
- Roof framing is spaced at 24 inches on center
- Roof sheathing is 7/16-inch-thick structural wood panel

Find 1. Wind load (suction) on roof sheathing.
2. Nail type/size and maximum spacing.

Solution

1. Determine the wind load on roof sheathing

Step 1: Basic velocity pressure = 24.6 psf *(From basic wind velocity tables for two-story home)*
Step 2: Adjust for open exposure = 1.4(24.6 psf) = 34.4 psf
Step 3: Skip
Step 4: Roof sheathing G_{cp} = -2.2 *(From wind pressure coefficient tables)*
Step 5: Design load = (-2.2)(34.4 psf) = 76 psf

2. Select a trial nail type and size, determine withdrawal capacity, and calculated required spacing

Use an 8d pneumatic nail (0.113 inch diameter) with a length of 2 3/8 inches. The unadjusted design withdrawal capacity is determined using the equation in Section 1.3.3.

$W = 1380(G)^{2..5}DL_p$

G = 0.55
D = 0.113 in
L_p = (2 3/8 in) – (7/16 in) = 1.9 in

$W = 1380(0.55)^{2..5}(0.113 \text{ in})(1.9 \text{ in}) = 66.5 \text{ lb}$

Determine the adjusted design withdrawal capacity using the applicable adjustment factors discussed in Section 1.3.2.

$W' = WC_D = (66.5 \text{ lb})(1.6) = 106 \text{ lb}$

Determine the required nail spacing in the roof sheathing panel interior.

Tributary sheathing area = (roof framing spacing)(nail spacing)
= (2 ft)(s)
Withdrawal load per nail = (wind uplift pressure)(2 ft)(s)
= (76 psf)(2 ft)(s)

$$W' \geq \text{design withdrawal load}$$
$$106 \text{ lb} \geq (76 \text{ psf})(2 \text{ ft})(s)$$
$$s \leq 0.69 \text{ ft}$$

Use a maximum nail spacing of 8 inches in the roof sheathing panel interior.

Notes:
1. If Spruce-Pine-Fir (G=0.42) roof framing lumber is substituted, W' would be 54 lb and the required nail spacing would reduce to 4 inches on center in the roof sheathing panel interior. Thus, it is extremely important to carefully consider and verify the species of framing lumber when determining fastening requirements for roof sheathing.
2. The above analysis is based on a smooth shank nail. A ring shank nail may be used to provide greater withdrawal capacity that is also less susceptible to lumber moisture conditions at installation and related long-term effects on withdrawal capacity.
3. With the smaller tributary area, the roof sheathing edges that are supported on framing members may be fastened at the standard 6 inch on center fastener spacing. For simplicity, it may be easier to specify a 6 inch on center spacing for all roof sheathing fasteners, but give an allowance of 2 to 3 inches for a reasonable construction tolerance; refer to Section 1.3.6.
4. As an added measure given the extreme wind environment, the sheathing nail spacing along the gable end truss/framing should be specified at a closer spacing, say 4 inches on center. These fasteners are critical to the performance of light-frame gable roofs in extreme wind events. NDS 12.3.8 provides an equation to determine nail lateral strength when subjected to a combined lateral and withdrawal load. This equation may be used to verify the 4 inch nail spacing recommendation at the gable end.

Conclusion

This example problem demonstrates a simple application of the nail withdrawal equation in the NDS. The withdrawal forces on connections in residential construction are usually of greatest concern in the roof sheathing attachment. In hurricane prone regions, it is common practice to use a 6-inch nail spacing on the interior of roof sheathing panels. In lower wind regions of the United States, a standard nail spacing applies (i.e., 6 inches on panel edges and 12 inches in the panel field); refer to Table 1.1.

EXAMPLE 1.2 Roof-to-Wall Connections

Given

- Design wind speed is 120 mph gust with an open coastal exposure
- One-story home with a hip roof (28 ft clear span trusses with 2 ft overhangs)
- Roof slope is 6:12
- Trusses are spaced at 24 in on center

Find 1. Uplift and transverse shear load at the roof-to-wall connection
2. Connection detail to resist the design loads

Solution

1. Determine the design loads on the connection

Dead load

Roof dead load = 15 psf *(From basic load tables)*
Dead load on wall = (15 psf)[0.5(28 ft) + 2 ft] = 240 plf

Wind load

Step 1: Basic velocity pressure = 18.8 psf *(From basic wind velocity pressure tables)*
Step 2: Adjust for open exposure = 1.4(18.8 psf) = 26.3 psf
Step 3: Skip
Step 4: Roof uplift G_{cp} = -0.8
Overhang G_{cp} = +0.8
Step 5: Roof uplift pressure = -0.8(26.3 psf) = -21 psf
Overhang pressure = 0.8 (26.3 psf) = 21 psf

Determine the wind uplift load on the wall.

Design load on wall = $0.6D + W_u$ *(From basic wind load tables)*
= 0.6 (240 plf) + {(-21 psf)[0.5(28 ft) + 2 ft] – (21 psf)(2 ft)}
= - 234 plf (upward)

Design load per wall-to-truss connection = (2 ft)(-234 plf) = -468 lb (upward)

Determine the transverse shear (lateral) load on the roof-to-wall connection. The transverse load is associated with the role of the roof diaphragm in supporting and transferring lateral loads from direct wind pressure on the walls.

Design lateral load on the wall-to-truss connection
= 1/2 (wall height)(wall pressure)(truss spacing)

Adjusted velocity pressure = 26.3 psf
Wall GC_p = -1.2,+1.1*
Wind pressure = 1.1(26.3 psf) = 29 psf

*The 1.1 coefficient is used since the maximum uplift on the roof and roof overhang occurs on a windward side of the building (i.e., positive wall pressure).

= 1/2 (8 ft)(29 psf)(2 ft)
= 232 lb

Thus, roof-to-wall connection combined design loads are:

468 lb (uplift)
232 lb (lateral, perpendicular to wall)*

*The lateral load parallel to a wall is not considered to be significant in this example problem, although it may be checked to verify the transfer of lateral wind loads on the roof to shear walls; refer to Chapter 6.

2. Determine a roof-to-wall connection detail to resist the combined design load. Generally, manufacturers publish loading data for metal connectors for multiple loading directions. The engineer should verify that these values are for simultaneous multidirectional loading or make reasonable adjustments as needed. In this example problem, the NDS will be used to design a simple roof tie-down strap and slant nail connection. A tie down strap will be used to resist the uplift load and typical slant nailing will be used to resist the lateral load. The slant nailing, however, does not contribute appreciably to the uplift capacity when a strap or metal connector is used; refer to Section 1.3.6.

Uplift load resistance

Assuming an 18g (minimum 0.043 inches thick) metal strap is used, determine the number of 6d common nails required to connect the strap to the truss and to the wall top plate to resist the design uplift load.

The nail shear capacity is determined as follows:

$Z = 60$ lb (NDS Table 12.3F)
$Z' = ZC_D$ (Section 1.3.2)
$= (60 \text{ lb})(1.6)$
$= 96$ lb

The number of nails required in each end of the strap is

(486 lb)/(96 lb/nail) = 5 nails

The above Z value for metal side-plates implicitly addresses failure modes that may be associated with strap/nail head tear-through. However, the width of the strap must be calculated. Assuming a minimum 33 ksi steel yield strength and a standard 0.6 safety factor, the width of the strap is determined as follows:

$0.6(33{,}000 \text{ psi})(0.043 \text{ in})(w) = 468 \text{ lb}$

$w = 0.55$ in

Therefore, use a minimum 1-inch wide strap to allow for the width of nail holes and a staggered nail pattern. Alternatively, a thinner strap may be used (i.e., 20g or 0.033 inches thick) which may create less of a problem with installing finishes over the connection.

Lateral load resistance

Assuming that a 16d pneumatic nail will be used (0.131 in diameter by 3.5 inches long), determine the number of slant-driven nails required to transfer the lateral load from the wall to the roof sheathing diaphragm through the roof trusses. Assume that the wall framing is Spruce-Pine-Fir (G = 0.42).

$Z = 88$ lb (NDS Table 12.3A)*

*A 1-1/4- inch side member thickness is used to account for the slant nail penetration through the truss at an angle.

$Z' = ZC_D$** **The C_{tn} value of 0.83 is not used because the nail is slant driven and is not a toe-nail; refer to Section 1.3.6.

$Z' = (88\ \text{lb})(1.6) = 141\ \text{lb}$

Therefore, the number of nails required to transfer the transverse shear load is determined as follows:

(232 lb)/(141 lb/nail) = 2 nails

Conclusion

The beginning of the uplift load path is on the roof sheathing which is transferred to the roof framing through the sheathing nails; refer to Example 1.1. The uplift load is then passed through the roof-to-wall connections as demonstrated in this example problem. It should be noted that the load path for wind uplift cannot overlook any joint in the framing.

One common error is to attach small roof tie-straps or clips to only the top member of the wall top plate. Thus, the uplift load must be transferred between the two members of the double top plate which are usually only face nailed together for the purpose of assembly, not to transfer large uplift loads. This would not normally be a problem if the wall sheathing were attached to the top member of the double top plate, but walls are usually built to an 8 ft – 1 in height to allow for assembly of interior finishes and to result in a full 8 ft ceiling height after floor and ceiling finishes. Since sheathing is a nominal 8 ft in length, it cannot span the full wall height and may not be attached to the top member of the top plate. Also, the strap should be placed on the structural sheathing side of the wall unless framing joints within the wall (i.e., stud-to-plates) are adequately reinforced.

Longer sheathing can be special ordered and is often used to transfer uplift and shear loads across floor levels by lapping the sheathing over the floor framing to the wall below. The sheathing may also be laced at the floor band joist to transfer uplift load, but the cross grain tension of the band joist should not exceed a suitably low stress value (i.e., $1/3F_v$).

EXAMPLE 1.3 Rafter-to-Ceiling Joist Connection (Heel Joint)

Given

- Rafter and ceiling joist roof construction (without intermediate rafter braces)
- Roof horizontal span is 28 ft and rafter slope is 6:12 (26 degrees)
- Roof framing is Hem-Fir (G=0.43) with a spacing of 16 inches on-center
- Roof snow load is 25 psf
- Rafter & roofing dead load is 10 psf
- Ceiling dead load is 5 psf

Find
1. The tension load on the heel joint connection
2. Nailing requirements

Solution

1. Determine the tensile load on the heel joint connection

Using basic principles of mechanics and pinned-joint analysis of the rafter and ceiling joist "truss" system, the forces on the heel joint can be determined. First, the rafter bearing reaction is determined as follows:

B = (snow + dead load)(1/2 span)(rafter spacing)
= (25 psf + 10 psf)(14 ft)(1.33 ft)
= 652 lb

Summing forces in the y-direction (vertical) for equilibrium of the heel joint connection, the compression (axial) force in the rafter is determined as follows:

$C = (652 \text{ lb})/\sin(26^{o}) = 1{,}487 \text{ lb}$

Now, summing the forces in the x-direction (horizontal) for equilibrium of the heel joint connection, the tension (axial) force in the ceiling joist is determined as follows:

$T = (1{,}487 \text{ lb})\cos(26^{o}) = 1{,}337 \text{ lb}$

2. Determine the required nailing for the connection

Try a 12d box nail. Using NDS Table 12.3A, the following Z value is obtained:

Z = 80 lb
$Z' = ZC_DC_d$ (Section 1.3.2)

$C_D = 1.25$* (From typical snow load duration tables)
*NDS uses a factor of 1.15
$C_d = p/(12D)$ (NDS 12.3.4)
p = penetration into main member = 1.5 inches
D = nail diameter = 0.128 inches
$C_d = 1.5/[12(0.128)] = 0.98$

Z' = (80 lb)(1.25)(0.98) = 98 lb

A system factor of 1.1 for tension members and connections in trussed, light-frame roofing systems for repetitive member applications (i.e., framing spaced no greater than 24 inches on center) is applicable. Therefore, the Z' value may be adjusted as follows:

Z' = (98 lb)(1.1) = 108 lb

The total number of 12d box nails required is determined as follows:

(1,337 lb)/(108 lb/nail) = 12.3

If a 16d common nail is substituted, the number of nails may be reduced to about 8. If, in addition, the species of framing lumber was changed to Southern Yellow Pine (G = 0.55), the number of nails could be reduced to 6.

Conclusion

This example problem demonstrates the design of one of the most critical roof framing connections for site-built rafter and ceiling joist framing. In some cases, the ceiling joist or cross-tie may be located at a higher point on the rafter than the wall bearing location which will increase the load on the joint. In most designs, a simple pinned-joint analysis of the roof framing is used to determine the connection forces for various roof framing configurations.

The snow load duration factor of 1.25 was used in lieu of the 1.15 factor recommended by the NDS. In addition, a system factor for repetitive member, light-frame roof systems was used. The 1.1 factor is considered to be conservative which may explain the difference between the design solution in this example and the nailing required in Table 1.1 by conventional practice (i.e., four 16d common nails). If the slant nailing of the rafter to the wall top plate and wall top plate to the ceiling joist are considered in transferring the tension load, then the number of nails may be reduced relative to that calculated above. If a larger system factor than 1.1 is considered (say 1.3), then the analysis will become more closely aligned with conventional practice. It should also be noted that the NDS safety factor on nail lateral capacity is generally in the range of 3 to 5. However, in more heavily loaded conditions (i.e., lower roof slope, higher snow load, etc.) the connection design should be expected to depart somewhat from conventional practice that is intended for "typical" conditions of use.

In any event, 12 nails per rafter-ceiling joist joint may be considered unacceptable by some builders and engineers since the connection is marginally "over-crowed" with fasteners. Therefore, alternative analysis methods and fastener solutions should be considered with some regard to extensive experience in conventional practice; refer to NDS 1.1.1.4 and the discussion above.

EXAMPLE 1.4 Wall Sole Plate to Floor Connection

Given

- A 2x4 wall bottom (sole) plate of Spruce-Pine-Fir is fastened to a wood floor deck
- Floor framing lumber is Hem-Fir
- A 3/4-inch-thick wood structural panel subfloor is used
- The bottom plate is subject to the following design loads due to wind and/or earthquake lateral loads:
  250 plf shear parallel-to-grain (shear wall slip resistance)
  120 plf shear perpendicular-to-grain (transverse load on wall)
- The uplift load on the wall, if any, is assumed to be resisted by other connections (i.e., uplift straps, shear wall hold-downs, etc.)

Find A suitable nailing schedule for the wall sole plate connection using 16d pneumatic nails (0.131inch diameter by 3.5 inches long).

Solution

It is assumed that the nails will penetrate the sub-flooring and the floor framing members. It will also be conservatively assumed that the density of the sub-floor sheathing and the floor framing is the same as the wall bottom plate (lowest density of the connected materials). These assumptions allow for the use of NDS Table 12.3A. Alternatively, a more accurate nail design lateral capacity may be calculated using the yield equations of NDS•12.3.1.

Using NDS Table 12.3A, it is noted that the closest nail diameters in the table are 0.135 and 0.128 inches. Interpolating between these values, using a side member thickness of 1.5 inches, and assuming Spruce-Pine-Fir for all members, the following Z value is obtained:

$$Z = 79 + [(0.131-0.128)/(0.135-0.128)](88 \text{ lb} - 79 \text{ lb}) = 83 \text{ lb}^*$$

$$Z' = ZC_D = 83 \text{ lb } (1.6) = 133 \text{ lb}$$

*Using the NDS general dowel equations as presented in AF&PA Technical Report 12 (AF&PA, 1999), the calculated value is identical under the same simplifying assumptions. However, a higher design value of 90 pounds may be calculated by using only the subfloor sheathing as a side member with G = 0.5. The ultimate capacity is conservatively predicted as 261 pounds.

Assuming that both of the lateral loads act simultaneously at their full design value (conservative assumption), the resultant design load is determined as follows:

$$\text{Resultant shear load} = \text{sqrt}[(250\text{plf})^2 + (120 \text{ plf})^2] = 277 \text{ plf}$$

Using the conservative assumptions above, the number of nails per linear foot of wall plate is determined as follows:

$$(277 \text{ lb})/(133 \text{ lb/nail}) = 2.1 \text{ nails per foot}$$

Rounding this number, the design recommendation is 2 nails per foot or 3 nails per 16 inches of wall plate.

Conclusion

The number of 16d pneumatic nails (0.131 inch diameter) required is 2 nails per foot of wall bottom plate for the moderate loading condition evaluated. The number of nails may be reduced by using a larger diameter nail or by evaluating the nail lateral capacity using the yield equations of NDS 12.3.1.

As in Example 1.3, some consideration of extensive experience in conventional residential construction should also be considered in view of the conventional fastening requirements of Table 1.1 for wood sole plate to floor framing connections (i.e., one 16d nail at 16 inches on center); refer to NDS 1.1.1.4. Perhaps 2 nails per 16 inches on center is adequate for the loads assumed in this example problem. Testing has indicated that the ultimate capacity of 2-16d pneumatic nails (0.131 inch diameter) can exceed 600 lb per nail for conditions similar to those assumed in this example problem; refer to Section 1.3.6. The general dowel equations under predict the ultimate capacity by about a factor of two. Using 2 16d pneumatic nails at 16 inches on center may be expected to provide a safety factor of greater than 3 relative to the design lateral load assumed in this problem (i.e., [600 lb/nail] x [2nails/1.33 ft]/277 plf = 3.2).

The ultimate capacity of base connections for shear walls should at least exceed the ultimate capacity of the shear wall for seismic design and, for wind design, the connection should at least provide a safety factor of 2 relative to the wind load. For seismic design, the safety factor for shear walls recommended here in is 2.5. Therefore, the fastening schedule of 2-16d pneumatic nails at 16 inches on center is not quite adequate for seismic design loads of the magnitude assumed in this problem (i.e., the connection does not provide a safety factor of at least 2.5).

EXAMPLE 1.5 Side-Bearing Joist Connection

Given

- A 2x10 Douglas-Fir joist is side-bearing (shear connection) on a built-up wood girder
- The design shear load on the side-bearing joint is 400 lb due to floor live and dead loads

Find
1. The number of 16d box toenails required to transfer the side-bearing (shear) load.
2. A suitable joist hanger

Solution

1. Determine the number of 16d box toenails required

$$Z' = ZC_DC_dC_{tn}$$

$Z = 103$ lb	(NDS Table 12.3A)
$C_D = 1.0$	(normal duration load)
$C_d = 1.0$	(penetration into main member > 12D)
$C_{tn} = 0.83$	(NDS 12.3.7)

$$Z' = (103 \text{ lb})(0.83) = 85 \text{ lb}$$

The number of toenails required is determined as follows:

$$(400 \text{ lb})/(85 \text{ lb/nail}) = 4.7 \text{ nails}$$

Use 6 toenails with 3 on each side of the joist to allow for reasonable construction tolerance in assembling the connection in the field.

2. As an alternative, select a suitable manufactured joist hanger.

Data on metal joist hangers and various other connectors are available from a number of manufacturers of these products. The design process simply involves the selection of a properly rated connector of the appropriate size and configuration for the application. Rated capacities of specialty connectors are generally associated with a particular fastener and species of framing lumber. Adjustments may be necessary for use with various lumber species and fastener types.

Conclusion

The example problem details the design approach for two simple methods of transferring shear loads through a side-bearing connection. One approach uses a conventional practice of toe-nailing the joist to a wood girder. This approach is commonly used for short-span floor joists (i.e., tail joist to header joist connections at a floor stairwell framing). For more heavily loaded applications, a metal joist hanger is the preferred solution.

EXAMPLE 1.6 Wood Floor Ledger Connection to a Wood or Concrete Wall

Given

- A 3x8 wood ledger board (Douglas-Fir) is used to support a side-bearing floor system.
- The ledger is attached to 3x4 wall studs (Douglas-Fir) spaced at 16 inches on center in a balloon-framed portion of a home; as a second condition, the ledger is attached to a concrete wall.
- The design shear load on the ledger is 300 plf due to floor live and dead loads.

Find

1. The spacing of 5/8-inch-diameter lag screws required to fasten the ledger to the wood wall framing
2. The spacing of 5/8-inch-diameter anchor bolts required to fasten the ledger to a concrete wall

Solution

1. Determine connection requirements for use of a 5/8-inch-diameter lag screw

$Z' = ZC_DC_gC_\Delta C_d$ (Section 1.3.2)

$Z_{s\perp} = 630$ lb* (NDS Table 9.3A)
$C_D = 1.0$ (normal duration load)
$C_g = 0.98$ (2 bolts in a row) (NDS Table 1.3.6A)
$C_\Delta = 1.0$**
$C_d = p/(8D) = (3.09 \text{ in})/[8(5/8 \text{ in})] = 0.62$ (NDS 9.3.3)
p = (penetration into main member) – (tapered length of tip of lag screw)***
= 3.5 in – 13/32 in = 3.09 in

*The $Z_{s\perp}$ value is used for joints when the shear load is perpendicular to the grain of the side member (or ledger in this case). **A C_Δ value of 1.0 is predicated on meeting the minimum edge and end distances required for lag screws and bolts; refer to NDS 8.5.3 and NDS 9.4. The required edge distance in the side member is 4D from the top of the ledger (loaded edge) and 1.5D from the bottom of the ledger (unloaded edge), where D is the diameter of the bolt or lag screw. The edge distance of 1.5D is barely met for the nominal 3-inch-wide (2.5 inch actual) stud provided the lag screws are installed along the center line of the stud. ***A 6-inch-long lag screw will extend through the side member (2.5 inches thick) and penetrate into the main member 3.5 inches. The design penetration into the main member must be reduced by the length of the tapered tip on the lag screw (see Appendix L of NDS for lag screw dimensions).

$Z' = (630 \text{ lb})(1.0)(0.98)(1.0)(0.62) = 383$ lb

The lag bolt spacing is determined as follows:

Spacing = (383 lb/lag screw)/(300 plf) = 1.3 ft

Therefore, one lag screw per stud-ledger intersection may be used (i.e., 1.33 ft spacing). The lag screws should be staggered about 2 inches from the top and bottom of the 3x8 ledger board. Since the bolts are staggered (i.e., not two bolts in a row), the value of C_g may be revised to 1.0 in the above calculations.

2. Determine connection requirements for use of a 5/8-inch-diameter anchor bolt in a concrete wall

$Z' = ZC_DC_gC_\Delta$ (Section 1.3.2)

$Z_\perp = 650$ lb* (NDS Table 8.2E)
$C_D = 1.0$ (normal duration load)
$C_g = 1.0$**

$C_\Delta = 1.0$***

* The $Z_\perp$ value is used since the ledger is loaded perpendicular to grain
**The bolts will be spaced and staggered, not placed in a row.
***Edge and end distance requirements of NDS 8.5.3 and NDS 8.5.4 will be met for full design value.

$$Z' = (650 \text{ lb})(1.0)(1.0)(1.0) = 650 \text{ lb}$$

The required anchor bolt spacing is determined as follows:

$$\text{Spacing} = (650 \text{ lb})/(300 \text{ plf}) = 2.2 \text{ ft}$$

Therefore, the anchor bolts should be spaced at about 2 ft on center and staggered from the top and bottom edge of the ledger by a distance of about 2 inches. Note: In conditions where this connection is also required to support the wall laterally (i.e., an outward tension load due to seismic loading on a heavy concrete wall), the tension forces may dictate additional connectors to transfer the load into the floor diaphragm. In lower wind or seismic load conditions, the ledger connection to the wall and the floor sheathing connection to the ledger are usually sufficient to transfer the design tension loading, even though it may induce some cross grain tension forces in the ledger. The cross-grain tension stress may be minimized by locating every other bolt as close to the top of the ledger as practical or by using a larger plate washer on the bolts.

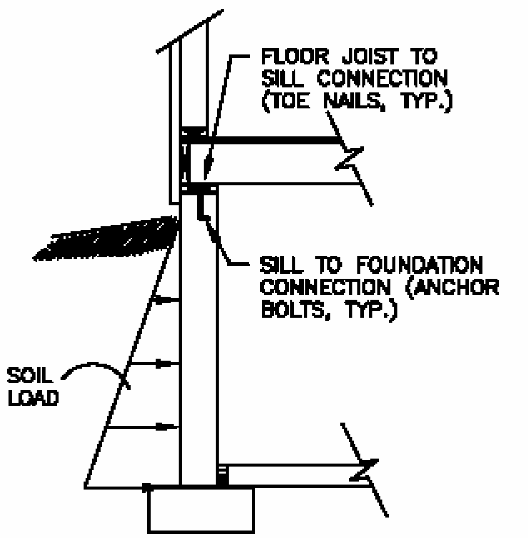
Conclusion

The design of bolted side-bearing connections was presented in this design example for two wall construction conditions. While not a common connection detail in residential framing, it is one that requires careful design consideration and installation since it must transfer the floor loads (i.e., people) through a shear connection rather than simple bearing. The example also addresses the issue of appropriate bolt location with respect to edge and end distances. Finally, the engineer was alerted to special connection detailing considerations in high wind and seismic conditions.

EXAMPLE 1.7 Wood Sill to Foundation Wall

Given

- The foundation wall is connected to a wood sill plate and laterally supported as shown in the figure below.
- Assume that the soil has a 30 pcf equivalent fluid density and that the unbalanced backfill height is 1.5 ft.
- The foundation wall unsupported height (from basement slab to top of wall) is 8 ft.
- The wood sill is preservative-treated Southern Yellow Pine.



Find
1. The lateral load on the foundation wall to sill plate connection due to the backfill lateral pressure
2. The required spacing of ½-inch-diameter anchor bolts in the sill plate

Solution

1. Determine the lateral load on the sill plate connection

The reaction at the top of the foundation wall is determined as follows:

$$R_{top} = ql^3/(6L) = (30\text{ pcf})(1.5\text{ ft})^3/[6(8\text{ ft})] = 264\text{ plf}$$

2. Determine the design lateral capacity of the anchor bolt and the required spacing

$Z' = ZC_DC_MC_tC_gC_\Delta$ (Section 1.3.2)

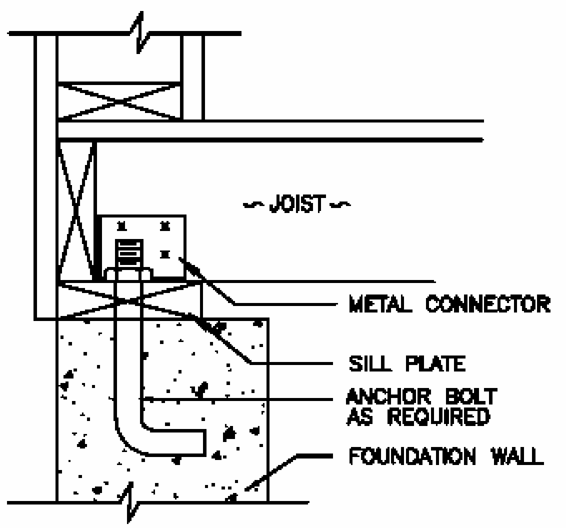
$Z_\perp = 400$ lbs* (NDS Table 8.2E)
$C_D = 0.9$ (life-time load duration tables)
$C_M = 1.0$ (MC < 19%)
$C_t = 1.0$ (temperature < 100°F)
$C_g = 1.0$ (bolts not configured in rows)

*The value is based on a recommended 6 inch standard embedment of the anchor bolt into the concrete wall. Based on conventional construction experience, this value may also be applied to masonry foundation wall construction when bolts are properly grouted into the masonry wall (i.e., by use of a bond beam).

Z' = (400 lb)(0.9) = 360 lb

Anchor bolt spacing = (360 lb)/(264 plf) = 1.4 ft

Note: According to the above calculations, an anchor bolt spacing of about 16 inches on center is required in the sill plate. However, in conventional residential construction, extensive experience has shown that a typical anchor bolt spacing of 6 ft on center is adequate for normal conditions as represented in this design example. This conflict between analysis and experience creates a dilemma for the engineer that may only be reconciled by making judgmental use of the "extensive experience" clause in NDS 1.1.1.4. Perhaps a compromise would be to require the use of a 5/8-inch-diamter anchor bolt at a 4 ft on center spacing. This design may be further justified by consideration of friction in the connection (i.e., a 0.3 friction coefficient with a normal force due to dead load of the building). The large safety factor in wood connections may also be attributed to some of the discrepancy between practice or experience and analysis in accordance with the NDS. Finally, the load must be transferred into the floor framing through connection of the floor to the sill (see Table 1.1 for conventional toenail connection requirements). In applications where the loads are anticipated to be much greater (i.e., taller foundation wall with heavier soil loads), the joint may be reinforced with a metal bracket at shown below.



Conclusion

This example demonstrates an analytic method of determining foundation lateral loads and the required connections to support the top of the foundation wall through a wood sill plate and floor construction. It also demonstrates the discrepancy between calculated connection requirements and conventional construction experience that may be negotiated by permissible engineer judgment and use of conventional residential construction requirements.

EXAMPLE 1.8 Deck Header to Post Connection

Given

- A 2x8 preservative-treated header is attached to each side of a deck post in a bolted, double shear connection to support load from deck joists bearing on the headers.
- The deck post is a preservative treated 4x4.
- The deck framing lumber is preservative-treated Southern Yellow Pine.
- The design double shear load on the connection is 2,560 lb (1,280 lb per header).

Find Determine if two 5/8-inch-diameter bolts are sufficient to resist the design load.

Solution

Calculate the design shear capacity of the bolted joint assuming that the bolts are located approximately 2 inches from the top and bottom edge of the 2x8 headers along the centerline of the 4x4 post.

$Z' = ZC_DC_MC_tC_gC_\Delta$	(Section 1.3.2)
$Z_{s\perp} = 1{,}130$ lb*	(NDS Table 8.3A)
$C_D = 1.0$**	(Normal duration of load)
$C_M = 1.0$	(MC < 19%)
$C_t = 1.0$	(Temperature < 100°F)
$C_g = 0.98$ (2 bolts in a row)	(NDS Table 1.3.6A)
$C_\Delta = 1.0$ (for the bottom bolt only)***	(NDS 8.5.3)

*The $Z_{s\perp}$ value is used because the side members (2x8) are loaded perpendicular to grain and the main member (4x4) is loaded parallel to grain.
**A normal duration of load is assumed for the deck live load. However, load duration studies for deck live loads have not been conducted. Some recent research has indicated that a load duration factor of 1.25 is appropriate for floor live loads; refer to Table 5.3 of Chapter 5.
***The top bolt is placed 2 inches from the top (loaded) edge of the 2x8 header and does not meet the 4D (2.5 inch) edge distance requirement of NDS•8.5.3. However, neglecting the bolt entirely will under-estimate the capacity of the connection.

$$Z' = (1{,}130 \text{ lb})(0.98) = 1{,}107 \text{ lb (bottom bolt only)}$$

If the top bolt is considered to be 80 percent effective based on its edge distance relative to the required edge distance (i.e., 2 inches / 2.5 inches = 0.8), then the design shear capacity for the two bolts in double shear may be estimated as follows:

$$Z' = 1{,}107 \text{ lb} + 0.8(1{,}107 \text{ lb}) = 1{,}993 \text{ lb} \quad < 2{,}560 \text{ lb} \quad \text{NG?}$$

Conclusion

The calculation of the design shear capacity of a double shear bolted connection is demonstrated in this example. As shown in the calculations, the connection doesn't meet the required load in the manner analyzed. A larger bolt diameter or 3 bolts may be used to meet the required design load. However, as in previous examples, this connection is typical in residential deck construction (i.e., supporting deck spans of about 8 ft each way) and may be approved by the "extensive experience" clause of NDS 1.1.1.4. As additional rationale, the capacity of shear connections in the NDS is related to a yield (or deformation) limit state and not capacity. On the basis of capacity, the safety margins are fairly conservative for such applications; refer to Section 1.3.1. The use of a 1.25 load duration factor for the deck live load will also increase the joint capacity to a value nearly equivalent to the design load assumed in this example.

EXAMPLE 1.9 Wood King and Jamb Stud to Floor or Foundation Connection

Given

- From Example 1.2, the net design uplift load at the roof-to-wall connection was determined to be 234 plf for a 120 mph gust, open exposure wind condition.
- Assume that the uplift loads at the top of the wall are adequately transferred through interconnection of wall framing members (i.e. top plates, sheathing, studs, headers to king and jamb studs, etc.) to the base of the upper story wall.
- The framing lumber is Hem-Fir

Find 1. The net uplift load at the base of the king and jamb studs adjacent to a 6 ftwide wall opening
2. An adequate connection detail to transfer the uplift load

Solution

1. Determine the net design uplift load at the base of the king and jamb studs supporting the 6 ft header using ASD load combinations.

Tributary load
= (1/2 header span + 1/2 stud spacing)[uplift load – 0.6(wall dead load)]
= [0.5(6 ft) + 0.5(1.33 ft)][234 plf – 0.6(64 plf)]
= 717 lb (uplift)

2. Determine the number of 8d common nails in each end of an 18g (0.043 inch minimum thickness) steel strap

$Z' = ZC_D$ (Section 1.3.2)

$Z = 82$ lb (NDS Table 12.3F)
$C_D = 1.6$ (wind load duration)

$Z' = (82 \text{ lb})(1.6) = 131$ lb

The number of nails required in each end of the strap is determined as follows:

(717 lb)/(131 lb/nail) = 6 nails

Note: As an option to the above solution, the same strap used on the layout studs may be used on the jamb and king stud connection by using multiple straps. The uplift strap on the layout studs would be required to resist 234 plf (1.33 ft) = 311 lb. Therefore, two or three of these straps could be used at wall opening location and attached to the jamb and king studs. If the single strap is used as calculated in the example problem, the jamb and king studs should be adequately interconnected (i.e., face nailed) to transfer shear load from one to the other. For example, if the header is strapped down to the top of the jamb stud and the king stud is strapped at its base, then the two members must be adequately fastened together. To some degree, the sheathing connections and other conventional connections will assist in strengthening the overall load path and their contribution should be considered or enhanced as appropriate.

As another alternative design, the king/jamb stud uplift connection may serve a dual role as a wind uplift strap and a shear wall hold-down restraint if the wall segment adjacent to the opening is designed to be a part of the building's lateral force resisting system (i.e., shear wall segment). The uplift force due to wind would be simply added to the uplift force due to shear wall restraint to properly size a hold-down bracket or larger strap than required for wind uplift alone.

Regardless of whether or not the wall segment is intended to be a shear wall segment, the presence of wind uplift straps will provide overturning restraint to the wall such that it effectively resists shear load and creates overturning restraint forces in the uplift straps. This condition is practically unavoidable because the load paths are practically inseparable, even if the intention in the design analysis is to have separate load paths. For this reason, the opposite of the approach described in the paragraph above may be considered to be more efficient. In other words, the wind uplift strap capacity may be increased so that these multiple straps also provide multiple overturning restraints for perforated shear walls. Thus, one type of strap or bracket can be used for the entire job to simplify construction detailing and reduce the potential for error in the field. This latter approach is applicable to seismic design (i.e., no wind uplift) and wind design conditions.

Conclusion

In this example, the transfer of wind uplift loads through wall framing adjacent to a wall opening is addressed. In addition, several alternate design approaches are noted that may optimize the design and improve construction efficiency – even in severe wind or seismic design conditions.

EXAMPLE 1.10 Concrete Wall to Footing (Shear) Connection

Given

Maximum transverse shear load on bottom of wall = 1,050 plf (due to soil)
Dead load on wall = 1,704 plf
Yield strength of reinforcement = 60,000 psi
Wall thickness = 8 inches
Assume μ = 0.6 for concrete placed against hardened concrete not intentionally roughened.
f'_c = 3,000 psi

Find
- Whether a dowel or key is required to provide increased shear transfer capacity
- If a dowel or key is required, size accordingly

Solution

1. Determine factored shear load on wall due to soil load (i.e., 1.6H)

V = 1,050 plf
V_u = 1.6 (1,050 plf)= 1,680 plf

2. Check friction resistance between the concrete footing and wall

$V_{friction} = \mu N = \mu$(dead load per foot of wall)

= (0.6)(1,704 plf) = 1,022 plf < V_u = 1,680 plf

Therefore, a dowel or key is needed to secure the foundation wall to the footing.

3. Determine a required dowel size and spacing (Section 1.2 and ACI-318 5.14)

$A_{vf} = V_u / (\phi f_y \mu)$

= (1,680 plf)/[(0.85)(60,000)(0.6)] = 0.05 in^2 per foot of wall

Try a No. 4 bar (A_v = 0.20 in^2) and determine the required dowel spacing as follows:

$A_{vf} = A_v/S$
0.05 in^2/lf = (0.2 in^2)/S
S = 48 inches

Conclusion

This example problem demonstrates that for the given conditions a minimum of one No. 4 rebar at 48 inches on center is required to adequately restrict the wall from slipping. Alternatively, a key may be used or the basement slab may laterally support the base of the foundation wall.

It should be noted that the factored shear load due to the soil lateral pressure is compared to the estimated friction resistance in Step 1 without factoring the friction resistance. There is no clear guideline in this matter of engineer judgment.

EXAMPLE 1.11 Concrete Anchor

Given

- 1/2-inch diameter anchor bolt at 4 feet on center with a 6 inch embedment depth in an 8-inch thick concrete wall
- The bolt is an ASTM A36 bolt with f_y = 36 ksi and the following design properties for ASD; refer to AISC Manual of Steel Construction (AISC,1989):
  F_t = 19,100 psi (allowable tensile stress)
  F_u = 58,000 psi (ultimate tensile stress)
  F_v = 10,000 psi (allowable shear stress)
- The specified concrete has f'_c = 3,000 psi
- The nominal design (unfactored) loading conditions are as follows:
  Shear load = 116 plf
  Uplift load = 285 plf
  Dead load = 180 plf

Find Determine if the bolt and concrete are adequate for the given conditions.

Solution

1. Check shear in bolt using appropriate ASD steel design specifications (AISC, 1989) and the ASD load combinations in Chapter 3.

f_v = shear load/bolt area = 116 lb (4 ft)/(0.192 in ^{2}) = 2.367 psi

F_v = 10,000 psi

$f_v \leq F_v$ OK

2. Check tension in bolt due to uplift using appropriate ASD steel design specifications (AISC, 1989) and the appropriate ASD load combination.

T = [(285 plf) - 0.6 (180 plf)] (4 ft) = 708 lb

$f_t = T/A_{bolt}$ = 708 lb/0.196 in^2 = 3,612 psi

$f_t \leq F_t$

3,612 psi < 19,100 psf OK

3. Check tension in concrete (anchorage capacity of concrete) using ACI-318 11.3 and the appropriate LRFD load combination in Chapter 3. Note that the assumed cone shear failure surface area, A_v, is approximated as the minimum of π (bolt embedment length)^2 or π (wall thickness)^2.

$$V_u = T = [1.5\,(285 \text{ plf}) - 0.9\,(180 \text{ plf})]\,(4 \text{ ft}) = 1{,}062 \text{ lb}$$

$$A_v = \text{minimum of} \begin{cases} \pi\,(6 \text{ in})^2 = 113 \text{ in}^2 \\ \pi\,(8 \text{ in})^2 = 201 \text{ in}^2 \end{cases}$$

$$\phi V_c = \phi 4 A_v \sqrt{f'_c} = (0.85)(4)(113 \text{ in}^2)\sqrt{3{,}000 \text{ psi}} = 21{,}044 \text{ lb}$$

$$V_u \leq \phi V_c$$

1,062 lb ≤ 21,044 lb OK

Conclusion

A 1/2-inch diameter anchor bolt with a 6 inch concrete embedment and spaced 4 feet on center is adequate for the given loading conditions. In lieu of using an anchor bolt, there are many strap anchors that are also available. The strap anchor manufacturer typically lists the embedment length and concrete compressive strength required corresponding to strap gauge and shear and tension ratings. In this instance, a design is not typically required the engineer simply ensures that the design loads do not exceed the strap anchor's rated capacity.