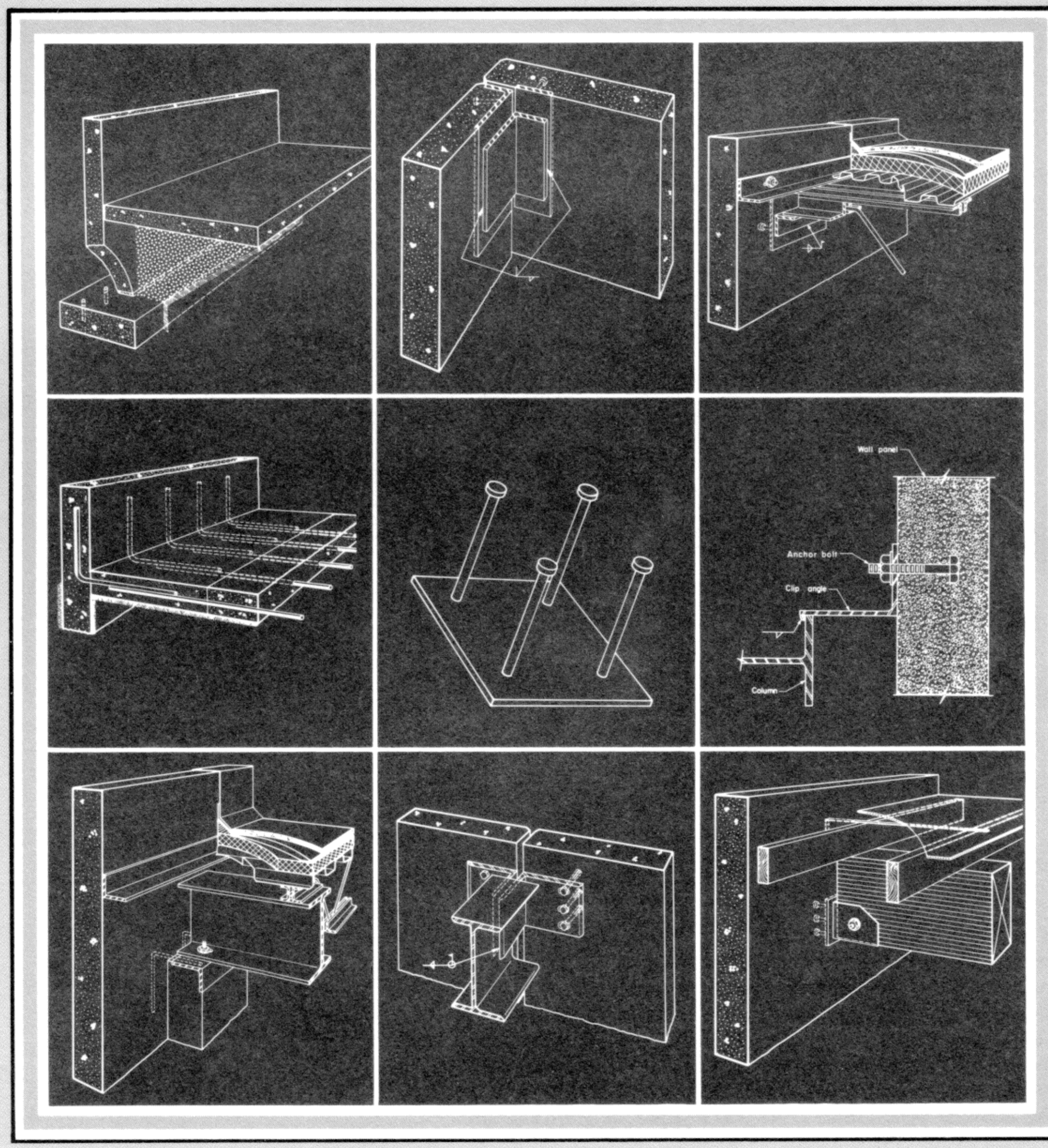
PORTLAND CEMENT
ASSOCIATION

Connections for Tilt-Up Wall Construction



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A large portion of the material found in chapters concerning materials, fabrication, and design was gleaned from References 1, 2, and 3. Details shown in the section of this document entitled "Connection Details" were, in large part, obtained from engineers and contractors experienced in tilt-up construction and design. The designers and contractors that provided details are listed below:

Al Shankle Construction Company, Anaheim, California

American Buildings Company, Eufala, Alabama

Armco Building Systems, Cincinnati, Ohio

The Burke Company, Sacramento, California

Dayton Superior, Miamisburg, Ohio

Dominion Construction Company, Vancouver, British Columbia, Canada

The Haskell Company, Jacksonville, Florida

K. M. Kripanarayanan, El Monte, California

Lockwood, Jones, and Beal, Inc., Dayton, Ohio

Richmond Screw Anchor Company, Fort Worth, Texas

William M. Simpson, Newport Beach, California

Steinbecker and Associates, Dayton, Ohio

Notation

A_b = Cross-sectional area of steel stud shank, sq in.
A_f = Tensile failure surface area of the flat bottom of the base, sq in.
A_s = Tensile failure surface area of sloping sides, sq in.
A_{vf} = Area of shear-friction reinforcement, sq in.
A_1 = Loaded bearing area in concrete, sq in.
A_2 = The area of the lower base of the largest frustum of a pyramid cone or tapered wedge contained wholly within the support and having for its upper base the loaded area and having side slopes of 1 vertical to 2 horizontal, sq in.
b = Length of concrete bearing area, in.
B_n = Nominal bearing strength of plain concrete
C_{es} = Strength reduction factor for single-headed studs located near a free edge equals

$$\frac{d_e}{\ell_e} \leq 1$$

d_b = Diameter of stud, in.
d_e = Distance from centroid of the embedded steel to the concrete free edge, in.
d_h = Stud head diameter, in.
f = Unfactored compressive stress, psi
f'_c = Specified compressive strength of concrete, psi
f_s = Maximum tensile strength of the stud material, psi
f_{tf} = Tensile stress level on base of failure surface, psi
f_{ts} = Tensile stress level on the sloping sides of failure surface, psi
f_y = Specified yield strength of steel, psi
G = Shear modulus, psi
G_t = Long-term shear modulus = 0.5G, psi
ℓ_e = Embedment length of stud, in.
N = Unfactored axial tension
P_{nc} = Nominal tensile strength of headed stud governed by failure in concrete
P_{ns} = Nominal tensile strength of headed stud governed by failure in steel
P_u = Factored axial load on concrete members at given eccentricity
t = Thickness of single-layer bearing pad or thickness of each lamination in laminated pads, in.
t_t = Total thickness of pad assembly, in.
V = Unfactored vertical reaction
V_n = Nominal shear strength of reinforced concrete
V_{nc} = Nominal shear strength of stud governed by failure in concrete
V_{ns} = Nominal shear strength of stud governed by failure in steel
V_u = Factored shear force at a concrete section
w = Width of concrete bearing area, in.
x, y = Surface dimensions of assumed failure plane around stud group, in.
Δ = Shear deformation, in.
λ = Correction factor related to unit weight of concrete
μ = Coefficient of friction
Φ = Strength-reduction factor

Introduction

Tilt-up concrete walls have been successfully used for many years in low- and mid-rise structures of all types. Tilt-up panels offer an efficient, economical alternative to preengineered buildings. Since they are sitecast, transportation is eliminated and handling is greatly reduced. These low-maintenance, fire-resistant walls offer high thermal energy savings. Architectural treatments are almost unlimited.

Among the important design features of tilt-up construction are the structural connections. The engineer must design connections based on strength, ductility, durability, and economy. Connections for plant-cast concrete are addressed in several publications.[1,2,3] However, connections used in tilt-up are more varied due to the use of steel and wood as well as concrete in the floor and roof designs.

This book's primary function is as a reference for connections used in tilt-up construction. It includes a compilation of connection details presently used for tilt-up construction in the United States and Canada.

For those unfamiliar with tilt-up development, a brief history is included in this section. Other sections discuss specific aspects of design, portions of which were gleaned from Reference 1. Guidelines for design are based on the provisions in Reference 4 and experimental research.

The concluding section contains 28 architectural perspectives of connections between wall panels and roofs, floors, adjacent walls, and foundations. The details were obtained from designers of tilt-up and through a review of literature.

HISTORICAL REVIEW

Tilt-up construction was introduced in North America around the turn of the twentieth century. However, it did not become popular until after World War II.[5] An article published by Robert Aiken[6] in 1909 describes early tilt-up walls for single-story military buildings.

From the start, tilt-up proved to be an efficient construction method. However, it entered a lull that lasted until after World War II. In 1946 more tilt-up buildings were constructed than in any other preceding decade.[5] Tilt-up became most popular in the commercial and industrial sector.

Tilt-up's increased popularity in the 1950's led to a demand for construction procedures and details. Collins recognized the information deficiency and wrote a set of three "Know How" booklets.[7,8,9] Collins later developed a single manual that combined his three earlier publications.[10] Expansion of tilt-up continued through the 1970's and improvements broadened in scope. In 1979, ACI Committee 551 on Tilt-up Concrete Construction was organized to study and report on current practices and develop standards.

From this brief history, it is evident that tilt-up is a growing industry. This book is intended to serve as a design guide and encompasses concrete, steel, and wood structural systems.

TILT-UP WALL CONNECTIONS

Early literature includes many panel-to-panel connection details; however, little is found concerning methods of connecting tilt-up walls to roofs, floors, or foundations.

In the early 1970's, the Prestressed Concrete Institute (PCI) became involved in providing recommendations for connections of precast members. A PCI committee wrote a manual on the design of connections for precast members that is currently being revised.[1] The tilt-up industry embraced applicable PCI recommended guidelines. The following sections provide the first guidelines that encompass a wide variety of connections for tilt-up construction.

Structural and Material Considerations

When designing connections, strength and serviceability criteria must be met. Details that are not properly considered in design may result in costly construction delays or unsafe structures. The following is an overview of important design criteria and materials that should be considered in connections for tilt-up wall construction.

DESIGN LOADS

Some design loads are obvious such as vertical live and dead loads and lateral loads due to wind, soil pressure, and seismic events. In connection design less apparent loads such as temporary erection loads and volume changes must also be considered. These considerations are addressed in References 11 and 23.

Overly strengthened connections can introduce unwanted restraints. The amount of fixity of a connection influences the load paths, which in turn affect other elements of the structural system. Therefore, an approach that considers connections as an integral part of the structure must be used in design.[12]

Connections are often designed with the intent of resisting only one type of loading. For instance, a connection that has a large tensile capacity but has little shear capacity to accommodate movement due to volume changes fits in this category.

DUCTILITY OF CONNECTIONS

Ductile connections are those that exhibit an ability to withstand deformation and load beyond the initial yield. It is desirable to design connections to behave in a ductile manner so they can support loads if unexpected forces occur and large deformations develop.

RESTRAINT TO VOLUME CHANGE

Shrinkage from drying, changes in temperature and creep all cause movements in wall panels. Where possible, it is advisable to design connections that will accommodate all volume changes.

Shrinkage occurs due to drying of the concrete. After drying, if immersed in water, it absorbs water and expands. However, it does not return to its original volume. Concrete also expands or contracts as the ambient temperature increases or decreases. The coefficient of thermal expansion ranges from 3.2 to 7 millionths per °F (5.7 to 12.8 millionths per °C), with 5.5 millionths per °F (10 millionths per °C) the accepted average.[13] Steel has a comparable expansion. Creep of concrete is a time-dependent volume change related to deformation under sustained load.

DURABILITY

Durability refers to a material's ability to maintain its strength and serviceability throughout its service life. Exposure of connections to weather may foster deterioration of the components and subsequent reduction in strength; therefore, proper protection is essential. In climates where freeze-thaw cycles occur, concrete should have sufficient air entrainment. Connections incorporating wood must use treated wood. Exposed steel components must be given protective coatings.

FIRE RESISTANCE

Codes dictating fire-protection requirements for structural members address connections. The PCI manual on fire resistance of concrete structures[14] suggests usage of fire retardants such as intumescent mastic, mineral fibers, and vermiculite materials. Intumescent mastic is a paint-on liquid that, when dry, foams under elevated temperatures. Mineral fibers are mixed with bonding agents and sprayed or troweled to provide a fire barrier. Vermiculite and cement pastes are mixed together and applied by troweling or spraying. These methods of protection are all acceptable. Specific application is left to the engineer's discretion based on architectural or other considerations.

Design Considerations to Ensure Efficiency

Efficient connection designs consider fabrication methods as well as other criteria. Connections must be designed to optimize construction time. Designs that do not evaluate the influence of connection details on the overall erection plans can result in costly construction delays.

DESIGN SIMPLICITY

It is often said that the best designs are the simplest ones. A straightforward approach requiring simple fabrication and erection methods is essential. By reducing the number of components for a connection, construction economy and efficiency can be enhanced.

REPETITION OF DETAILS

Fast, efficient, economical wall-panel installation requires optimizing the number of details for connections. Plates, angles, and reinforcing bars should be standardized. Also, the number of different-size components should be minimized. For instance, if a plate is sized to a 9½-in. (240-mm) width, if possible, use a 10-in. (250-mm) plate. If a No. 3 reinforcing bar is required for one connection and a No. 4 bar required for another, consider using all No. 4 bars.

Some connections that are detailed similarly may be subject to slightly different service conditions. For instance, one may be designed for a 10-kip load and a similar connection designed for a 20-kip load. It may be prudent to design both connections for a 20-kip load. It should be noted, however, that overly strengthened connections may introduce undue restraints.

REINFORCEMENT

Connections with reinforcement should be evaluated prior to construction to ensure feasibility of fabrication. Choosing the smallest bars allowable may help alleviate congestion. Smaller bars also require shorter development lengths. To ensure adequate clearances and proper dimensioning, scale drawings should be provided.

EMBEDDED STEEL SHAPES AND THREADED INSERTS

Misalignment of embedded structural steel and threaded inserts generally results in erection problems. Plates and angles with predrilled holes should be securely fastened to the forms. Threaded inserts must be firmly anchored or tied in place to prevent movement during concreting. Anchoring or tying in place prior to concreting, rather than inserting the devices during concreting, ensures quality control with little supervision.

Where concrete must be placed under a horizontal portion of an embedded structural-steel component, holes should be provided in the component to avoid trapping air under the embedment. In the case of angles, the horizontal leg should be clearly marked so that holes are made in the proper leg.

DIMENSIONS

Dimensions of all components of the connections should be to the nearest half inch to simplify production. Plate dimensions should be standard widths. Clearances between reinforcing bars and other components should be at least equal to 1⅓ times the maximum aggregate size.

FORMWORK

Wall-panel forms should rest on a flat and level surface and be square and vertical. If an embedded angle is attached to a skewed form, allowable tolerances may be insufficient to accommodate the misplaced angle. Edge forms must be accurately positioned and firmly anchored to prevent movement during concreting.

Erection Considerations

All phases of tilt-up design and construction are important for overall efficiency and economy of the project. Connection design is especially significant because of the time demands on skilled labor and equipment during erection. Connection locations are also important. Connections that can be made at ground level are generally more economical than assembly of connections while working from a ladder. The following is a brief overview of the items a designer should consider.

CLEARANCES AND TOLERANCES

Inadequate clearances can impede construction and failure to adhere to specified tolerances may effect the strength of the connections. Although clearances and tolerances are important economic considerations, there currently are no published values specifically for tilt-up. The following suggestions are offered based on present practices and the provisions noted in Reference 1, the PCI manual. Architectural and structural drawings should specify clearances and tolerances.

As a rule, small clearances should be avoided. If a larger clearance is architecturally and structurally acceptable, it should be used. The type of connection often governs the clearance needed. Using splice plates or clip angles to bridge the space between embedded components can accommodate large clearances. Current practice suggests that a minimum of ½ in. (12 mm) and preferably ¾ in. (19 mm) be allowed between panels. Larger clearance of 1 in. (25 mm) minimum and 2 in. (50 mm) preferred is recommended between structural support members and panels.

Tolerances in the placement of embedded plates, angles, and inserts are also governed by the type of connections provided. In general, inserts to receive bolts must have lower tolerances than welded connections. The following values are considered attainable:

Item	Recommended tolerances, in.
Field-placed anchor bolts	±¼
Elevation of footings or piers	+½, −2
Position of bearing plates	±½
Position of embedded plates	±1
Position of inserts	±½
Specified clearance space	±⅜

Metric equivalent: 1 in. = 25.4 mm

FIELD WELDING

Field welding should not be used indiscriminately. Welded connections generally are quite rigid and may fail when subjected to large unpredicted forces in excess of design limits. Large forces can result from volume changes discussed earlier. Consideration should be given to employing a combination of bolting and welding where more control of movement is possible. When only a few connections are to be welded, alternate methods should be explored for more economical solutions.

Applicable specifications and procedures for welding structural and reinforcing steel should be in compliance with AWS Designation D1.1, Structural Welding Code—Steel,[15] and AWS D1.4, Structural Welding Code—Reinforcing Steel,[16] respectively.

Welders should adhere to erection drawings and provide only specified amounts of weld to avoid causing excess fixity. Connections should be designed to allow sufficient working space for welders. Avoid cramped and congested areas. Locate weld joints to permit them to be done in the down-hand position wherever possible.

Structural steel exposed to cold temperatures and reinforcing steel may require preheating prior to making the weld. Care should be taken to avoid any damage to the surrounding concrete as a result of high temperatures.

TEMPORARY CONNECTIONS

During erection of tilt-up panels, temporary bracing, guywires, or other means of support may be required. Wherever possible, utilize the permanent connection devices rather than temporary built-in connections to attach the bracing. If this is not feasible, then temporary connections must be provided for the bracing. They should be removed after final connections are made in order to avoid unforeseen distress in the structure.

Conceptual Design

The following conceptual design considerations, although general in nature, are important to the overall treatment of connection design.

LOAD PATHS

Each structure with all its elements and connections should be considered as an interdependent structural system. Each connection is not an isolated element but rather part of an integrated system. An applied external load is distributed through the structural system to the foundation and supports through load paths. Load paths induce internal forces between elements of the system. An efficient design considers all possible load paths. This is done to optimize the number and magnitude of internal forces within a structural system in an effort to simplify the connections.

FAILURE MODES

The engineer should be aware of the potential modes of failure in each connection. Sufficient redundancy should be provided to eliminate the potential for a progressive collapse. Failure mechanisms are often obvious and easy to define. Failure modes that are difficult to identify should be isolated by testing.[1]

Connections that subject concrete to tensile forces can result in brittle failure modes. Unlike a ductile failure, a brittle failure is usually sudden and without warning. If a nonrigid connection cannot be provided, the engineer should account for this by increasing the safety factor of the connection.

PLIANT CONNECTIONS

Rigid connections can be subject to unanticipated stresses due to volume changes. As a result, they may fail. An alternate to a rigid connection is one that relieves stress by allowing movement to occur. Flexibility can be attained in various ways. Bearing pads supporting structural members can offer stress relief. Low friction materials allow a member to slip, thus accommodating movement.

Connections can be "softened" through the use of slotted holes in bolted connections. The bolt is tightened sufficiently to hold the member in place; however, the slot allows the member to move with little restraint. But if the connection is bolted tight against the end of the slot, movement is restricted. This should be avoided.

Connection Elements

Connection elements and materials commonly used in tilt-up construction are discussed in the following sections. These include standard bolts, threaded rods, headed studs, threaded inserts, expansion inserts, structural-steel shapes, deformed bar anchors, sitecast concrete, welding, dowels, grout, and epoxies. A description is given and applications, design considerations, and design data are discussed.

STANDARD BOLTS AND THREADED RODS

Description

Standard bolts and threaded rods are medium-strength materials. They are ductile and conform with the "stretch before breaking" philosophy of design. Standard bolts conform to the American Society for Testing and Materials (ASTM) Designation A 307, Standard Specification for Carbon Steel Externally Threaded Fasteners.[17] For threaded rods, the most common material conforms with ASTM A 36, Standard Specification for Structural Steel.[18] Threading of bolts and rods conforms to the American Standards Institute (ANSI) B1.1, Unified Inch Screw Threads.[19]

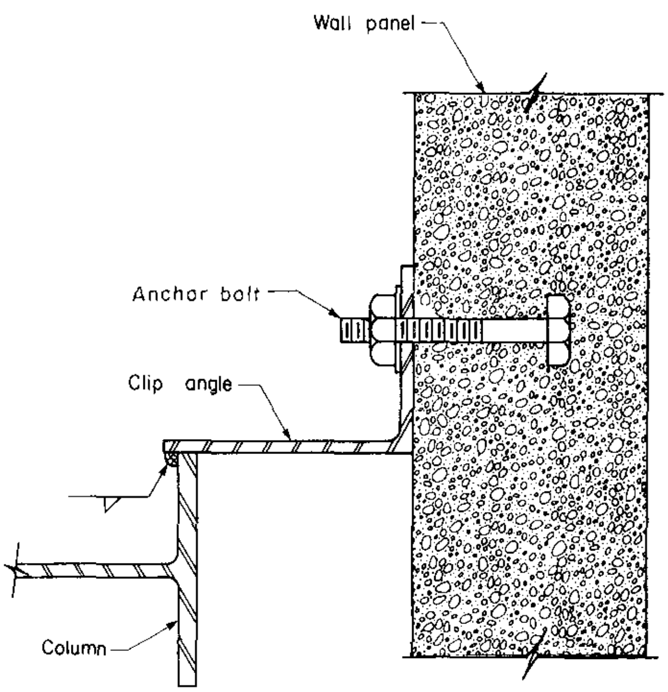


Fig. 1. Typical application of embedded bolts.

Applications

The most common applications for standard bolts is to connect steel components. They are also used to connect steel shapes to concrete tilt-up walls. They are either embedded in the concrete or threaded into inserts anchored in the concrete. Fig. 1 illustrates a typical use of a bolt embedded in concrete. It should be noted, however, that holding the bolt in place during concrete placement can be difficult. An alternate to the standard fastener is a threaded steel rod and nuts. Bolt or rod assemblies are an excellent solution for low-cost connections. They provide excellent anchorage for light loads.

Design Considerations

Standard bolts and threaded rods are not adequate for friction-type connections. Consider using high-strength bolts where friction connections are needed. Note too that the capacity of embedded bolts and threaded rods may be limited by failure of the concrete as well as in the strength of embedded steel.

The mating elements of bolts—such as threaded inserts—must be accessible for easy placement and proper tightening. With threaded inserts, proper tolerances must be provided.

Design Data

Table 1 provides tensile and shear capacities of bolts and threaded fasteners. Note the footnotes of the tables concerning the allowable tension and shear capacities. The American Institute of Steel Construction, Inc., (AISC) *Manual of Steel Construction*[20] contains additional information concerning design.

Table 1. Allowable Working Stresses and Loads on Standard Bolts (ASTM A307) and Threaded Rods (ASTM A36) [Adapted from Reference 20]

Tension							
ASTM Designation	Allowable working stress, ksi	Allowable load, kips*					
		Nominal bolt diameter, in.					
		½	⅝	¾	⅞	1	1¼
A 36	19.1	3.8	5.9	8.4	11.5	15.0	23.4
A 307	20.0	3.9	6.1	8.8	12.0	15.7	24.5

*Based on tensile stress on the nominal (gross) area of the bolt.

Single shear							
ASTM Designation	Allowable working stress, ksi	Allowable load, kips*					
		Nominal bolt diameter, in.					
		½	⅝	¾	⅞	1	1¼
A 36	9.9	1.9	3.0	4.4	6.0	7.8	12.1
A 307	10.0	2.0	3.1	4.4	6.0	7.9	12.3

*Threads not included in the shear plane. Steel-to-steel connections.
Metric equivalents: 1 in. = 25.4 mm, 1 kip = 4.45 kN, 1 ksi = 6.89 MPa

HIGH-STRENGTH BOLTS

Description

High-strength bolts are seldom used for connections of tilt-up panels. They are normally used only where steel components are fastened together. They are reserved for loading conditions with high tensile and shear stress requirements. High-strength bolts conform to ASTM A 325, Standard Specification for High-Strength Bolts for Structural Steel Joints, Including Suitable Nuts and Plain Hardened Washers,[21] or ASTM A 490, Standard Specification for Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints.[22] Threading conforms to ANSI B1.1.[19]

Application

The common application for high-strength bolts is to connect steel components firmly enough to prevent separation or sliding. Components are those that are subject to large tensile and shear forces. The high strength of the bolts provides sufficient compression between the components to make a friction-type connection.

High-strength bolts are not embedded in concrete, since the pullout strength of the concrete controls the capacity of the connection and high-strength steel is not efficiently used.

Design Considerations

Placement and torquing of friction-type connections require strict quality control. Bolts must be accessible for each placement. Sufficient clearances are necessary to torque the nuts and develop the tension in the bolts.

In designing, it is important to know that the allowable shear on the bolts depends upon whether or not the threads are in the shear plane. Refer to design data to determine the thread/shear plane effects.

Design Data

Design data, including standard bolt dimensions and allowable stresses and loads can be found in Reference 20. Both bearing-type and friction-type connections are included. Both cases of inclusion and exclusion of threads in the shear plane are considered.

For friction-type connections, all mill scale must be removed from the surfaces of connected materials. The hole size is considered standard and is ¹⁄₁₆ in. (1.6 mm) larger than the bolt. Oversize holes for friction connections should not be permitted.

COIL BOLTS AND THREADED COIL RODS

Description

Coil bolts and threaded coil rods are coarse-threaded fasteners for use with helically coiled inserts. The inserts are discussed in the section "Threaded Inserts." The bolts and rods are available in standard diameters ranging from ½ to 1½ in. (13 to 38 mm). Lengths up to 10 ft (3.1 m) are available. A typical coil bolt and coil rod are illustrated in Fig. 2.

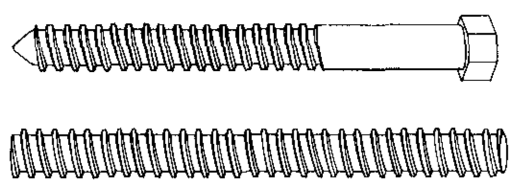


Fig. 2. Coil bolt and continuously threaded coil rod.

Applications

Coil bolts and coil rods are used primarily for lifting and temporary connections. Coil bolts and coil rods are not recommended for permanent connections in areas of high-risk seismic zones.

Design Considerations

The mating elements of coil bolts and coil rods—such as threaded inserts—must be accessible for easy placement and proper tightening. Sufficient tolerances must be provided.

Where these threaded elements are used for lifting and temporary connections they are generally reused many times. As a result, the threads should be regularly examined for wear.

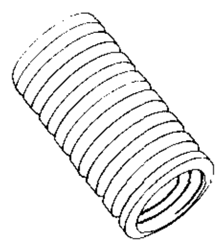
Design Data

Table 2 provides data on the tensile working load and the shear working load capacity of coil bolts and threaded coil rods, based on regular-strength material. The manufacturers of coil products can furnish capacities based on various-strength materials, including high-strength materials. Generally, the safe tensile working load is given as ⅔ of the minimum tensile strength, and the safe shear working load is ⅔ of the safe tensile working load.

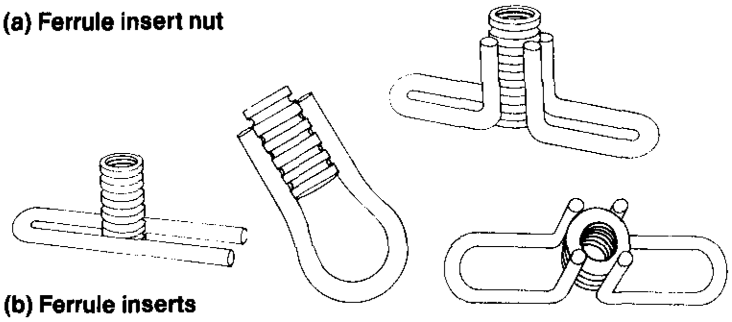
Table 2. Working-Load Capacity of Coil Bolts and Threaded Coil Rods* [Adapted from Reference 3]

Bolt diameter, in.	Tensile strength, lb	Tensile working load, lb**	Shear working load, lb†
½	13,500	9,000	6,000
¾	18,000	12,000	8,000
1	36,000	24,000	16,000
1¼	54,000	36,000	24,000
1½	72,000	48,000	32,000

*Increase factor of safety by reducing safe working loads.
**Tensile working load is ⅔ tensile capacity.
†Shear working load is ⅔ tensile working load.
Note: A minimum of two threads beyond coil is required to develop full capacity.
Metric equivalents: 1 in. = 25.4 mm, 1 lb = 4.45 N

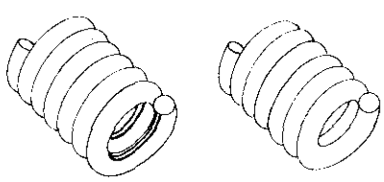


(a) Ferrule insert nut

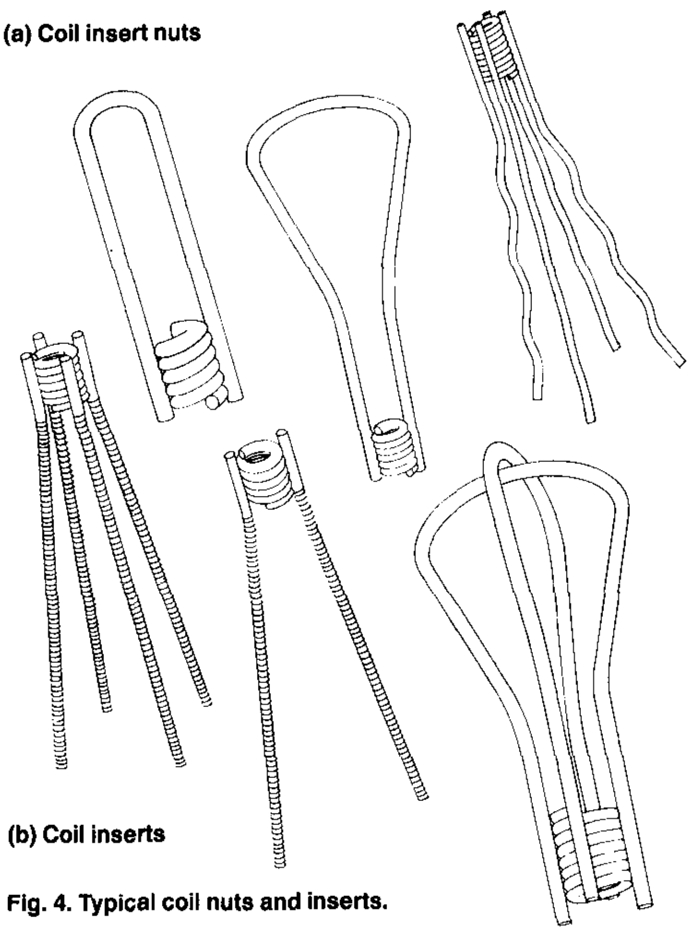


(b) Ferrule inserts

Fig. 3. Typical ferrule nut and inserts.



(a) Coil insert nuts



(b) Coil inserts

Fig. 4. Typical coil nuts and inserts.

THREADED INSERTS

Description

Ferrule Inserts. Ferrule inserts as shown in Figure 3 are nutlike anchors. They are embedded in concrete for use with standard bolts having national standard coarse threads. Steel-wire loops are welded to each nut to provide better anchorage to concrete. They are available in sizes ranging from ¼ to 1½ in. (6 to 38 mm).

Coil Inserts. Coil inserts consist of helically wound coil wire to form a nutlike anchor into which a coil bolt or rod can be threaded. Welded to the inserts are one or more wires or wire loops that provide anchorage to concrete. They are available in sizes ½ to 1½ in. (13 to 38 mm). Fig. 4 illustrates different coil-insert designs.

Applications

Ferrule and coil inserts when properly imbedded in concrete provide excellent means of establishing fast bolted connections in tilt-up panels. They are especially useful for lifting and bracing connections.

Design Considerations

Ferrule and coil inserts rely on the anchorage of the wire loops to provide the load capacity. Load is transferred through the wires to the concrete. Their capacity is limited by either the pullout strength of the concrete or the tensile strength of the wires. Placement of the inserts near a free edge reduces the load-carrying capacity.

The engineer should determine the appropriate working loads to be used based on the nature and details of the specific connection being designed. Threaded inserts fail in a brittle mode when the tensile strength of the concrete is exceeded. Proper factors of safety must be provided to account for this nonductile behavior, particularly in seismic design.

Design Data

A wide variety of ferrule and coil inserts are available. Test data and recommended allowable loads are readily available from the manufacturers. Usually, test data and recommended allowable loads are for normal-weight concrete. If lightweight concrete is used, the allowable loads should be reduced to reflect reduced capacity.

EXPANSION INSERTS

Description

Expansion inserts are anchors that are inserted into holes drilled in hardened concrete. Radial expansion of the insert exerts a force on the walls of the hole, providing friction and anchorage.

Either bolts or threaded rods are used with expansion inserts. A variety of expansion inserts available are illustrated in Fig. 5.

Applications

Expansion inserts are most efficiently utilized for retrofitting misplaced or left-out cast-in-place concrete inserts. They are used mostly in temporary connections and for bracing of tilt-up panels during erection. They are not normally recommended for permanent connections.

Design Considerations

Strength of expansion inserts is developed as a result of pressure against the walls of the drilled hole. Consequently, the distance between inserts and the orientation of the insert with a free edge is critical. Embedment depths, size, and shape of the hole are also critical. Tight tolerances are required on the holes drilled in the concrete and on the torque applied to bolts during installation of most expansion anchors. Some expansion inserts require the use of calibrated torque wrenches to control the torque and to obtain proper expansion.

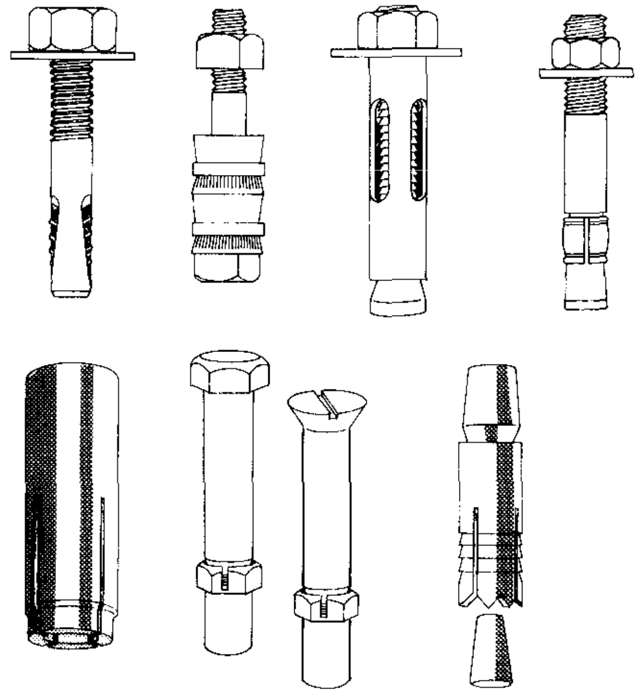


Fig. 5. Expansion inserts.

Design Data

It is recommended that test data and allowable load values be obtained from the manufacturer. However, the engineer should determine the appropriate working loads based on the nature and details of design.

STEEL EMBEDMENTS

Description

Typical steel embedments are shown in Figs. 6 and 7. They are fabricated from multiple elements including headed studs, bolts, deformed bar anchors, standard reinforcing bars, plates, and structural shapes.

Headed studs conform to ASTM A108, Standard Specification for Steel Bars, Carbon, Cold Finished, Standard Quality.[24] Reinforcing bars conform to ASTM A 615, Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement.[25]

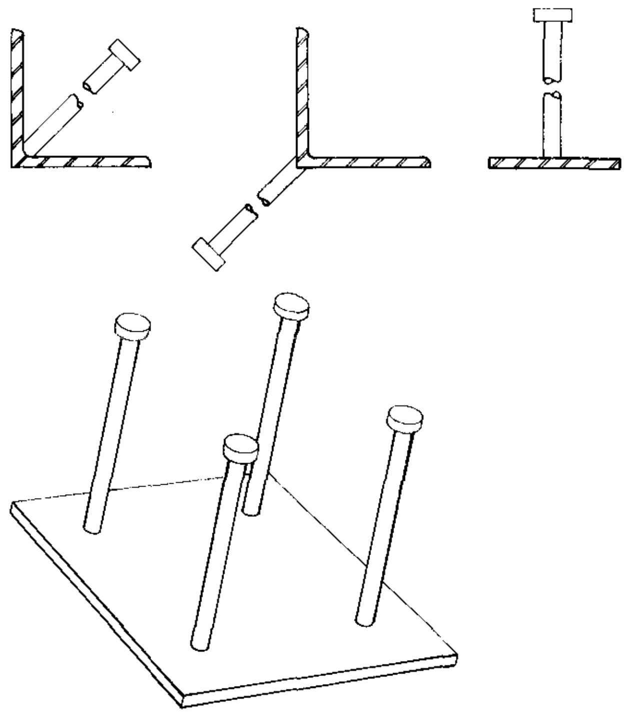


Fig. 6. Typical steel embedments with headed stud anchors.

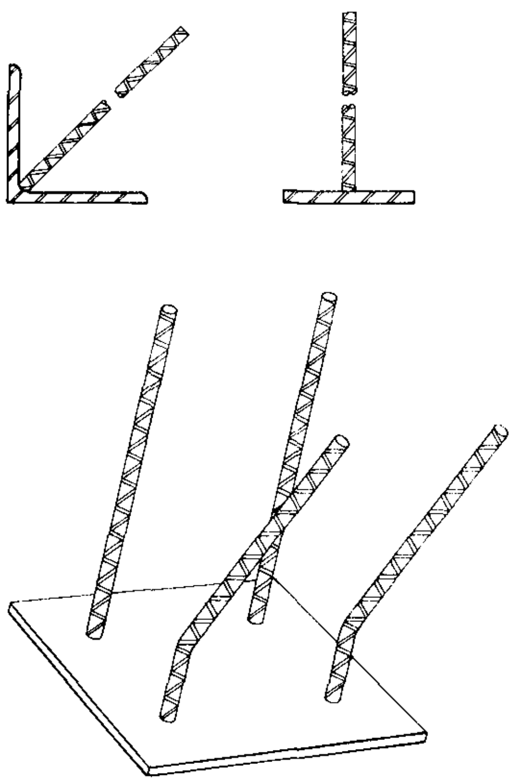


Fig. 7. Typical steel embedments with deformed-bar anchors.

Applications

Steel embedments are generally used at weldment locations to make connections of wall panels to other building components. Typically, plates and angles are cast flush with the concrete surface along a flat face or a corner. They may be used along with splice plates or angles fitted for welding or bolting to adjacent members.

Design Considerations

Loads that can be resisted by steel embedments include tension, compression, and shear, singly or in combination. When welding headed studs and deformed bars to plates, quality control is usually attained with the use of automatic stud-welding machines. Hand welding requires careful control and relies heavily on the skill of the welder.

A concrete pullout cone, as shown in Fig. 8, is the normal failure mode associated with headed studs. Specific design considerations are dependent on the details and the location of the embedments with respect to a free edge. This is discussed in more detail in the section "Connection Design."

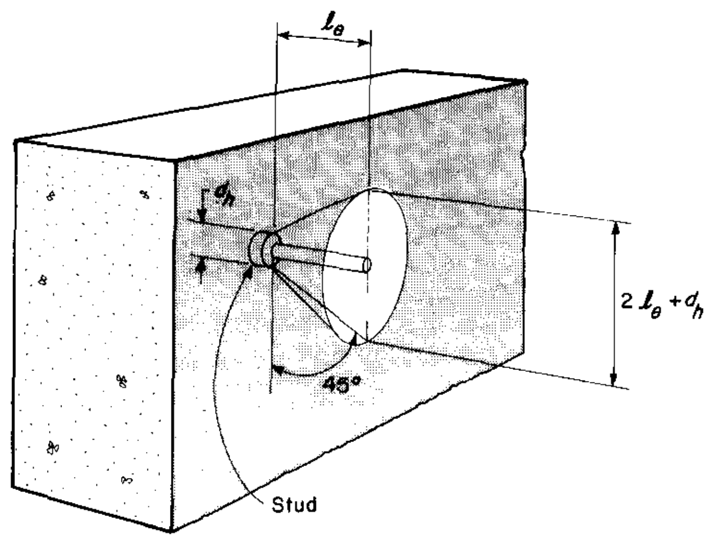


Fig. 8. Development of conrete pullout cone for a single stud subject to tension.

Design Data

Development lengths for tensile loads on deformed-bar anchors and reinforcement are given in Table 3. The lengths are based on values specified in Reference 4. Welding is discussed in the following section.

Design data for headed studs is provided in the section "Connection Design" along with a more detailed look at headed stud design.

Table 3. Development Length, in., for Reinforcing Bars and Deformed-Bar Anchors* [Adapted from Reference 4]

Concrete strength f'_c, psi	Normal-weight concrete ($\lambda = 1.0$)				Sand-lightweight concrete ($\lambda = 0.85$)			
	Bar diameter, in.				Bar diameter, in.			
	¼	⅜	½	⅝	¼	⅜	½	⅝
3000	12	12	12	15	12	12	14	18
4000	12	12	12	15	12	12	14	18
5000	12	12	12	15	12	12	14	18
6000	12	12	12	15	12	12	14	18
7000	12	12	12	15	12	12	14	18
8000	12	12	12	15	12	12	14	18

*f_y = 60,000 psi; for values above 60,000 psi multiply by $2 - \frac{60{,}000}{f_y}$

Metric equivalents: 1 in. = 25.4 mm, 1 psi = 6.89 kPa

WELDING

Description

The most common types of welds used in tilt-up are fillet and groove welds as shown in Figs. 9 and 10. The welds are generally made by the shielded metal arc-welding process applied to structural-steel shapes containing steel that conforms to ASTM A 36 specifications and reinforcing steel that conforms to ASTM A 615 specifications.

Applications

Welds are commonly used to join steel elements to form steel embedments and to make connections between structural members. Designed in proper configuration, they can be made to resist tension, compression, flexure, or torsion. Specific details for different conditions are shown in the section "Connection Details" in this book.

Design Considerations

When welded connections are used, consideration must be given to their restraint of movement between the connected parts. Welded connections are generally rigid and are not recommended where large volume changes are expected. When using welded connections, accessibility and proper procedures for welding must be considered. Welding should be performed in a downhand position.

Design Data

The strength of welds depends on reliable workmanship and the compatibility of welding materials with the metals to be joined. An adequate presentation on the requirements of welding is beyond the scope of this

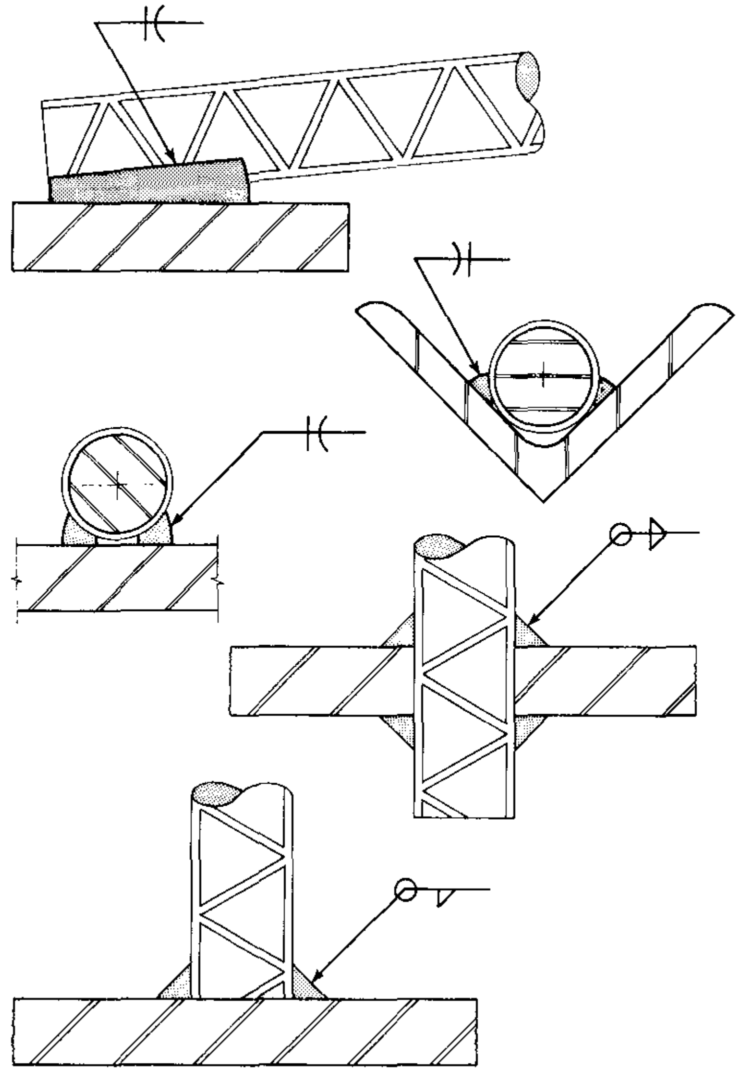


Fig. 9. Typical welds between reinforcing bars and structural steel shapes.

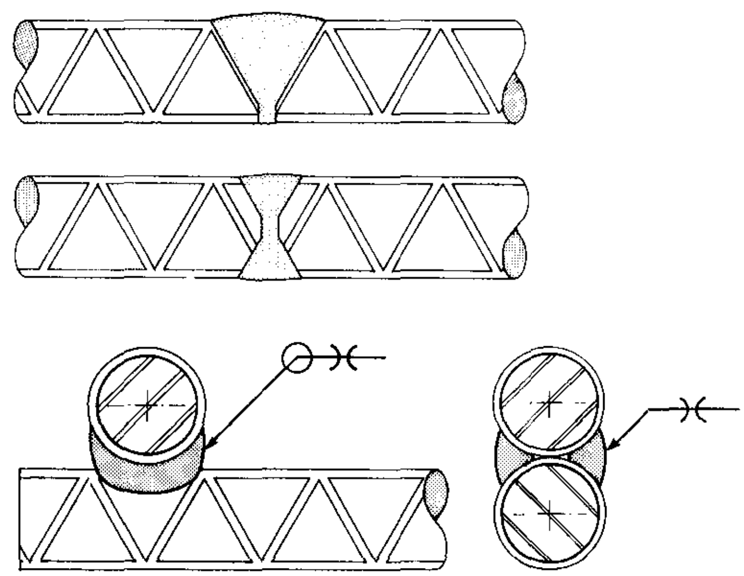


Fig. 10. Typical welds of reinforcing bars.

book. Complete details of structural steel welding are given in AWS D1.1, Structural Welding Code—Steel[15] and design of welds for reinforcement can be found in AWS D1.4-79, Structural Welding Code—Reinforcing Steel.[16]

DOWELS

Description

Dowels are short lengths of steel rods or bars embedded in concrete, welded to steel embedment plates, or grouted into drilled holes.

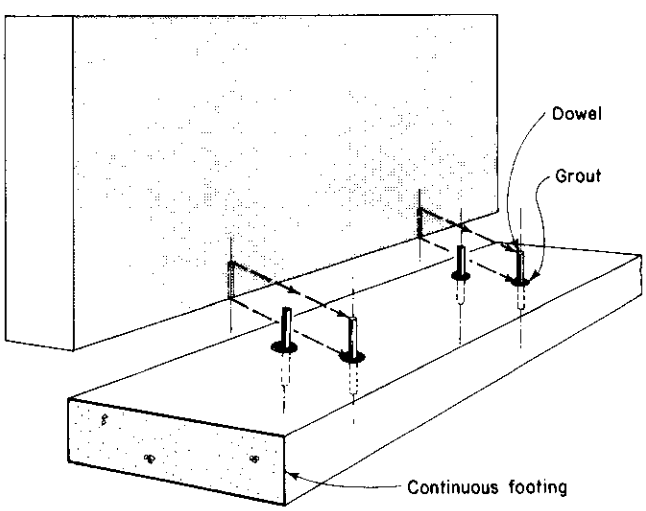
Deformed dowels are cut from conventional steel reinforcing bars. Smooth dowels are cut from smooth bar stock.

Applications

A common application of deformed dowels in tilt-up is to connect wall panels to the building floor slab as shown in the section "Wall Panel to Floor Connection." A common application of smooth dowels is alignment of wall panels as they are tilted and set on a footing or pier. This is illustrated in Fig. 11.

Design Considerations

Dowels connecting wall panels to floors can be provided in various ways. Deformed dowels can either be set in place prior to concreting, welded to a plate embedment, or screwed into preset threaded inserts.



Note: Panel is moved flush against dowel pins for proper alignment

Fig. 11. Application of dowels as alignment pins.

The most common method of connecting panels to floors is through the use of deformed dowels shaped and embedded entirely within the tilt-up panel. After the concrete has hardened, the dowels are bent outward to their final position for splice lapping with floor reinforcement. This practice is limited to bars no larger than No. 5.

Smooth dowels used for alignment of panels should be positioned to meet a specified tolerance.

SAND-CEMENT GROUT

Description

Sand-cement grouts are a mixture of portland cement, sand, and water. Generally they have a high slump and are used for filling small voids where normal concrete cannot be placed. Sand-cement ratios range between 1 to 1 and 3 to 1.

A low-slump grout may be used where high-slump grout cannot be held in place. Low-slump grouts are generally hand-tamped into place.

Applications

Sand-cement grout is generally used to fill the gap between the foundations and wall panels to transfer the bearing loads. Dowels placed in holes that have been drilled in hardened concrete can be anchored with the use of grout.

Design Considerations

High-slump grouts with a water-cement ratio of 0.5 or greater may shrink excessively. Shrinkage can be reduced by using compensating admixtures. The manufacturer's literature should be reviewed and some mixes tested before construction begins.

Dry-pack grouts have a much lower water-cement ratio; as a result, shrinkage generally is not a problem. Moist curing of the grout is recommended.

EPOXY

Description

An epoxy is a two-component system that when combined produces a material used for bonding. Epoxy-based bonding systems are addressed by ASTM C 881, Standard Specification for Epoxy-Resin-Base Bonding System for Concrete.[26] Additives can protect epoxies against moisture.

Applications

Epoxies can be used to connect structural and nonstructural members including concrete, steel, aluminum, copper, and wood. They also provide a means of field repair when mechanical connectors are not practical.

Design Considerations

Although high tensile strengths can be achieved with epoxies, available data on response to cyclic loads is limited. Properties may change with time. Consequently, careful consideration should be given to each application. Epoxies can have thermal expansion of up to seven times that of concrete. They also have a limited time during which they are workable. Temperature, humidity, and surface water may effect performance. It is recommended that design and use data of the manufacturer be thoroughly reviewed.

EPOXY GROUTS

Description

Epoxy grouts consist of a two-component epoxy system and an aggregate filler. Epoxy grouts are addressed by the ASTM specifications given in the section "Epoxy." (26)

Applications

When high bond strength between concrete members is required and a large amount of bonding agent is needed, epoxy grout is an economical alternate to simple epoxy. In tilt-up construction, epoxy grouts are used mostly for grouting dowel connections.

Design considerations

Design considerations for epoxy grout are the same as for epoxy as given in the section "Epoxy," with some exceptions. With the addition of the aggregate filler, the thermal expansion of epoxy grout can be reduced to about twice that of concrete. The amount of aggregate filler affects the strength. Excessive aggregate content reduces the bond strength.

BEARING PADS

Description

Bearing pads are intermediate elements between load-bearing structural members. They are available in a variety of materials:

1. Structural-grade elastomeric (neoprene) pads
2. Laminated steel and neoprene pads
3. Laminated fabric and rubber pads
4. Laminated synthetic fiber pads
5. Teflon pads
6. Multipolymer-plastic bearing strips
7. Tempered hardboard strip

Application

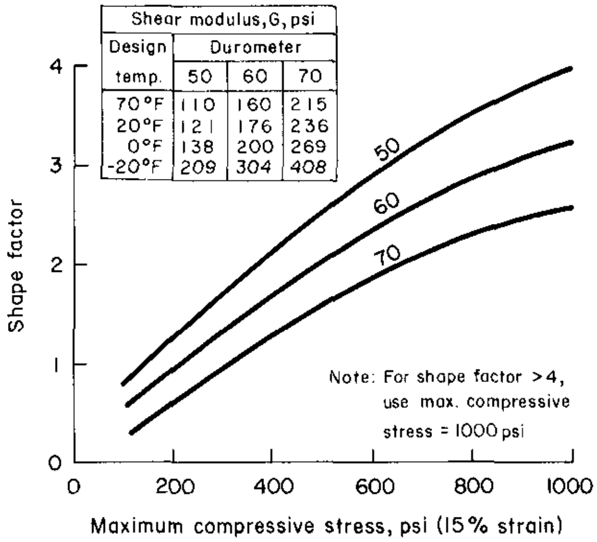
Bearing pads are generally used under simply supported members to distribute loads evenly over a bearing surface. They are designed to allow some displacement and some rotation to occur between the structural members.

Design Considerations

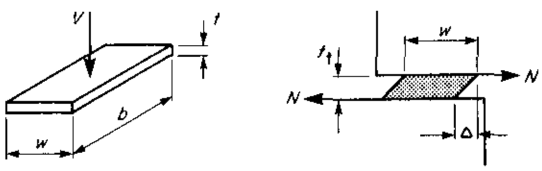
Different types of pads are available to suit different applications. A material may be specified based on compressibility, resilience, frictional characteristics, or response to environmental conditions. Pads are relatively simple and easy to install. However, they can move out of position under repetitive loading. Also, they generally degrade when exposed to fire.

Design Data

Design requirements for structural-grade elastomeric pads are illustrated in Fig. 12. Design data for other types are available from the manufacturer.



Shear modulus, G, psi			
Design temp.	Durometer 50	60	70
70°F	110	160	215
20°F	121	176	236
0°F	138	200	269
-20°F	209	304	408



$$\text{Shape factor} = \frac{wb}{2(w+b)t}$$

$$f = \frac{V}{wb}$$

$$N = \frac{\Delta wbG_t}{t_t}$$

Design recommendations

1. Use unfactored loads for design
2. Maximum compressive stress = 1000 psi
3. Maximum shear stress = 100 psi
4. Maximum shear deformation = $t/2$
5. Maximum compressive strain = 15%
6. $w \geq 5t$ or 4 in.
7. $t_t \geq$ ¼ in. for stems, ⅜ in. for beams

Metric equivalents: 1 in. = 25.4 mm, 1 lb = 4.45 N,
1 psi = 6.89 kPa

Fig. 12. Design of structural-grade elastomeric bearing pads. (Reproduced from Reference 1)

Connection Design

Design of connections for tilt-up requires consideration of strength and serviceability of all materials that may be affected, including concrete, steel, or wood. For further requirements related to these materials, the appropriate codes should be consulted[4,20,27]

Connections can subject concrete to tensile forces causing a brittle, sudden failure without warning. To reduce the possibility of a brittle failure, the strength of the steel embedment should control the design of the anchorage.

The following sections briefly describe some general aspects of connection design that relate to the design considerations and design data included in the section "Connection Elements" and design details included in the section "Connection Details."

BEARING ON PLAIN CONCRETE

Nearly all connections of precast concrete involve the bearing strength of concrete. The ACI Building Code[4] limits the unit design bearing stress to

$$\Phi B_n = \Phi(0.85f'_cA_1)\sqrt{A_2/A_1} \qquad \text{Eq. 1}$$

The relationship $\sqrt{A_2/A_1}$ is limited to a maximum of 2. Fig. 13 describes A_2.

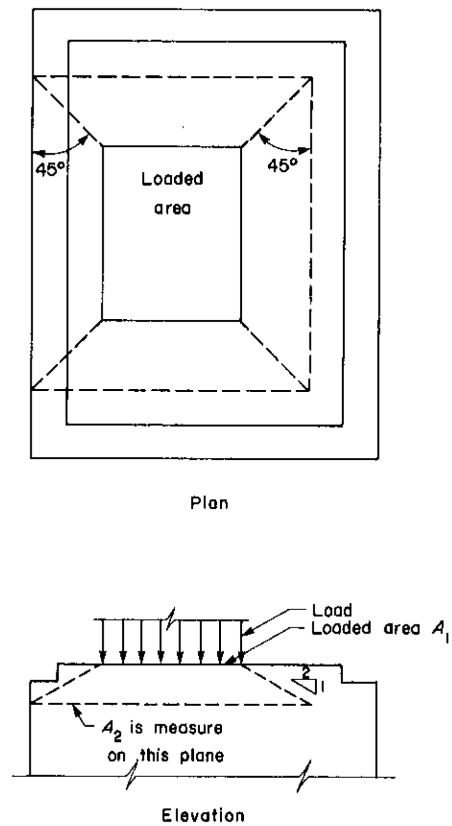


Fig. 13. Application of frustum to find A_2 in stepped or sloped supports.

SHEAR

Shear-Friction Design Using Headed Studs

The shear-friction concept is commonly used in the design of connections between precast elements. In shear-friction design, the nominal shear strength of the connection[1,4] with μ from Table 4 is

$$\Phi V_n = \Phi \mu A_{vf} f_y \qquad \text{Eq. 2}$$

However, in light of some recent findings,[28] the use of the shear-friction concept is questionable when headed studs are involved. If the shear-friction concept is used for special situations, adequate safety factors should be applied.

Shear Strength of Headed Studs (Table 5)

No rational model, including shear-friction, currently exists to accurately determine the capacity of headed stud connections. As a result, empirical equations are presented in this section to predict the capacity. The empirical equations from Reference 28 were used to calculate the values in Table 5 that are governed by

Table 4. Coefficients of Friction for Shear-Friction Connection Design [Reproduced from Reference 4]

Crack interface	Recommended coefficient of friction, μ
Concrete placed monolithically	1.4λ
Concrete placed against hardened concrete with surface intentionally roughened	1.0λ
Concrete anchored to as-rolled structural steel by headed studs or reinforcing bars	0.7λ
Concrete placed against hardened concrete not intentionally roughened	0.6λ

$\lambda = 1.0$ for normal-weight concrete, 0.85 for "sand-lightweight" concrete, and 0.75 for all-lightweight concrete.

Table 5. Design Shear Strength of Single-Headed Studs

	Design shear strength, ΦV_n, limited by concrete strength,* kips					
Edge distance, d_e, in.	Stud diameter, d_b, in.					
	1/4	3/8	1/2	5/8	3/4	7/8
2	1.2	1.2	1.2	1.2	1.2	1.2
3	1.8	2.6	2.6	2.6	2.6	2.6
4	1.8	4.1	4.7	4.7	4.7	4.7
5	1.8	4.1	7.3	7.3	7.3	7.3
6	1.8	4.1	7.3	10.5	10.5	10.5
7	1.8	4.1	7.3	11.4	14.3	14.3
8	1.8	4.1	7.3	11.4	16.5	18.7
≥ 9	1.8	4.1	7.3	11.4	16.5	22.4
	Values in shaded area are controlled by edge distance					
	In the unshaded area $\lambda = 1.0$; for other values of λ multiply tabulated results by λ					

Design shear strength limited by steel strength,** kips						
Stud diameter, d_b, in.	1/4	3/8	1/2	5/8	3/4	7/8
ΦV_n (shear friction)	2.3μ	5.1μ	9.0μ	14.1μ	20.3μ	27.6μ
ΦV_{ns}	2.2	5.0	8.8	13.8	19.9	27.1

* $f'_c = 3000$ psi; for other values of f'_c, multiply tabulated results by $\sqrt{f'_c/3000}$

$d_e < 10d_b$ then $\Phi V_{nc} = \Phi 2\pi d_e^2 \sqrt{f'_c}$

$d_e \geq 10d_b$ then $\Phi V_{nc} = \Phi 800 A_b \lambda \sqrt{f'_c}$

** $f_s = 60{,}000$ psi; for other strengths multiply by $f_s/60{,}000$

$f_y = 0.9f_s = 54{,}000$ psi

Metric equivalents: 1 in. = 25.4 mm, 1 kip = 4.45 kN, 1 psi = 6.89 kPa

concrete strength. The equations from Reference 28 are as follows:

Edge effects are not considered when

$$d_e \geq 10d_b \qquad \text{Eq. 3}$$

Shear capacity when a headed stud is not near a free edge ($d_e \geq 10d_b$):

$$\Phi V_{nc} = \Phi 800 A_b \lambda \sqrt{f'_c} \qquad \text{Eq. 4}$$

where $\Phi = 0.85$

λ = Correction factor found in footnote, Table 4

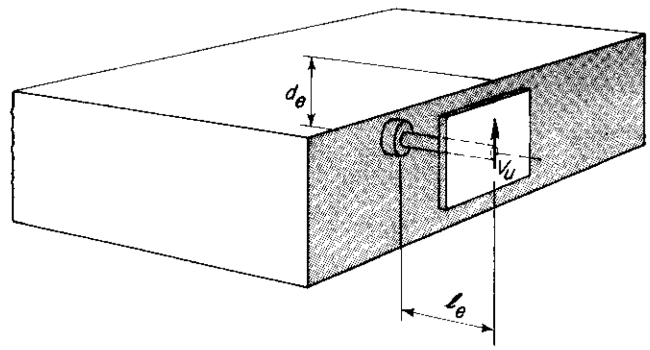


Fig. 14. Shear loading on a stud near a free edge. (Reproduced from Reference 1)

Shear capacity when a headed stud is located near a free edge ($d_e \leq 10d_b$), as shown in Fig. 14:

$$\Phi V_{nc} = \Phi 2\pi d_e^2 \sqrt{f'_c} \qquad \text{Eq. 5}$$

Eq. 5 was developed for normal-weight concrete. Currently no experimental data are available for studs located near a free edge embedded in lightweight concrete; therefore, there is no λ in Eq. 5. It should be modified when used for lightweight concretes.

It should be noted that Eqs. 4 and 5 are empirical and that the length of the stud is not included. If these equations are used to determine an allowable shear force, it is recommended that the length of the stud be at least 70% of that required for full-tension development. If the length of the stud is less than this, the allowable shear force should be reduced.

In no case should the shear capacity of the connection exceed the shear capacity of the studs. Design shear strength governed by steel strength as suggested in Reference 3 is

$$\Phi V_{ns} = \Phi 0.75 A_b f_s \qquad \text{Eq. 6}$$

where $\Phi = 1.0$

Eq. 6 was used to calculate the values in Table 5 that are governed by steel strength.

Note that the values determined by Eq. 6 (design shear strength determined by steel strength) are greater than those determined by Eq. 4 (design shear strength limited by concrete strength) in Table 5 for a concrete strength of 3000 psi. If ductility of the connection is a major consideration, it is recommended that a concrete strength greater than 3500 psi be used or confinement reinforcement that will intersect the assumed failure cones in the concrete be specified.

TENSION

This section covers tension design of headed studs following recommendations offered in Reference 1 and considering information presented in References 1, 2, 3, 28, and 29. Designs involving combined shear and tension are discussed in the section "Combined Shear and Tension."

Table 6. Design Tensile Strength of Single-Headed Studs*

Design tensile strength, ΦP_{nc}, limited by concrete strength and edge distance, kips

Edge distance, d_e	Stud length, ℓ_e	Stud-head diameter, d_h, in. ½	¾	1	1¼	1⅜
2.0	2.5	3.5	3.8	4.1	4.4	4.5
	4.0	5.3	5.6	5.9	6.1	6.3
	5.0	6.4	6.7	7.0	7.3	7.5
	6.0	7.6	7.9	8.2	8.5	8.6
	7.0	8.8	9.1	9.4	9.7	9.8
	8.0	9.9	10.2	10.5	10.8	11.0
3.0	2.5	4.4	4.8	5.1	5.5	5.7
	4.0	7.9	8.3	8.8	9.2	9.4
	5.0	9.7	10.1	10.5	11.0	11.2
	6.0	11.4	11.8	12.3	12.7	12.9
	7.0	13.2	13.6	14.0	14.5	14.7
	8.0	14.9	15.4	15.8	16.2	16.5
4.0	2.5	4.4	4.8	5.1	5.5	5.7
	4.0	10.5	11.1	11.7	12.3	12.6
	5.0	12.9	13.5	14.0	14.6	14.9
	6.0	15.2	15.8	16.4	17.0	17.3
	7.0	17.6	18.1	18.7	19.3	19.6
	8.0	19.9	20.5	21.1	21.6	21.9
5.0	2.5	4.4	4.8	5.1	5.5	5.7
	4.0	10.5	11.1	11.7	12.3	12.6
	5.0	16.1	16.8	17.6	18.3	18.6
	6.0	19.0	19.7	20.5	21.2	21.6
	7.0	21.9	22.7	23.4	24.1	24.5
	8.0	24.9	25.6	26.3	27.1	27.4
6.0	2.5	4.4	4.8	5.1	5.5	5.7
	4.0	10.5	11.1	11.7	12.3	12.6
	5.0	16.1	16.8	17.6	18.3	18.6
	6.0	22.8	23.7	24.6	25.4	25.9
	7.0	26.3	27.2	28.1	29.0	29.4
	8.0	29.8	30.7	31.6	32.5	32.9
7.0	2.5	4.4	4.8	5.1	5.5	5.7
	4.0	10.5	11.1	11.7	12.3	12.6
	5.0	16.1	16.8	17.6	18.3	18.6
	6.0	22.8	23.7	24.6	25.4	25.9
	7.0	30.7	31.7	32.8	33.8	34.3
	8.0	34.8	35.8	36.9	37.9	38.4
≥8.0	2.5	4.4	4.8	5.1	5.5	5.7
	4.0	10.5	11.1	11.7	12.3	12.6
	5.0	16.1	16.8	17.6	18.3	18.6
	6.0	22.8	23.7	24.6	25.4	25.9
	7.0	30.7	31.7	32.8	33.8	34.3
	8.0	39.8	41.0	42.1	43.3	43.9

* f'_c = 3000 psi; for other strengths multiply by $\sqrt{f'_c/3000}$
λ = 1.0 for normal-weight concrete; for other weights multiply tabulated values by other values of λ from Table 4.

Design tensile strength, ΦP_{ns}, limited by steel strength, kips

Stud diameter, d_b, in.	¼	⅜	½	⅝	¾	⅞
ΦP_{ns}	2.7	6.0	10.6	16.6	23.9	32.5

f_s = 60,000 psi; for other steel strengths multiply by f_s/60,000
$f_y = 0.9 f_s$ = 54,000 psi
Φ = 1.0
Metric equivalents: 1 in. = 25.4 mm, 1 kip = 4.45 kN,
1 psi = 6.89 kPa

Typical stud and head diameters, in.

Stud diameter, d_b, in.	¼	⅜	½	⅝	¾	⅞
Stud-head diameter, d_h, in.	½	¾	1	1¼	1¼	1⅜

The tension failure surface for headed studs tends to be conical in shape. For stud groups, the surface is shaped like a truncated pyramid. The angle of the failure surface with respect to the concrete surface for analysis purposes is generally assumed to be 45 degrees. The tensile strength of headed-stud embedments with Φ 0.85 and λ from the footnote of Table 4 is

$$\Phi P_{nc} = \Phi\lambda[f_{tr}A_f + f_{ts}A_s] \quad \text{Eq. 7}$$

Reference 28 suggests a tensile-stress level of $4\sqrt{f'_c}$ for the base area and $(4/\sqrt{2})\sqrt{f'_c}$ for the sloping sides of the failure surfaces.

Design Tensile Strength of Single-Headed Studs (Table 6)

Tensile strength of a headed stud embedded as shown in Fig. 8 and located far from a free edge can be determined by substituting in Eq. 7 the tensile-stress levels and stud configuration. The new equation to determine strength, limited by concrete strength is

$$\Phi P_{nc} = \Phi 12.6 \ell_e \lambda(\ell_e + d_h)\sqrt{f'_c} \quad \text{Eq. 8}$$

To account for the effect for studs located near a free edge ($d_e \leq \ell_e$), as shown in Fig. 14, Eq. 8 with the strength reduction factor becomes

$$\Phi P_{nc} = \Phi 12.6 \ell_e \lambda(\ell_e + d_h)\sqrt{f'_c}C_{es} \quad \text{Eq. 9}$$

where $C_{es} = d_e/\ell_e \leq 1$

The design tensile strength governed by steel strength as presented in Reference 1, for Φ = 1.0, is

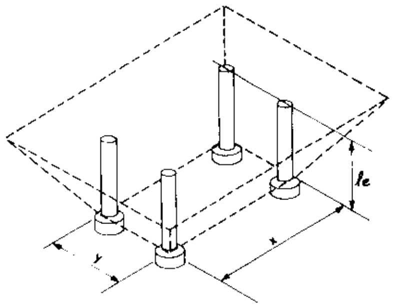
$$\Phi P_{ns} = \Phi A_b f_y \quad \text{Eq. 10}$$

Table 6 shows the design tensile strength based on concrete strength, edge distance, and steel strength considerations. The lowest applicable value from this table is the design tensile strength.

Stud Groups (Tables 7 through 11)

A group of studs is designed differently than a single stud if the shear cones intersect. To account for this, a truncated pyramid is assumed as the failure plane. Design data for groups of headed studs are listed in Tables 7 through 11.[1,28] The equations used to develop the values are provided with the appropriate table.

Table 7. Design Tensile Strength of a Stud Group—Away from a Free Edge



$\Phi P_{nc} = \Phi\lambda[4\sqrt{f'_c}\,xy + (4/\sqrt{2})\sqrt{f'_c}(2\sqrt{2}\,\ell_e(x + y + 2\ell_e))]$
$\Phi \quad = 0.85$

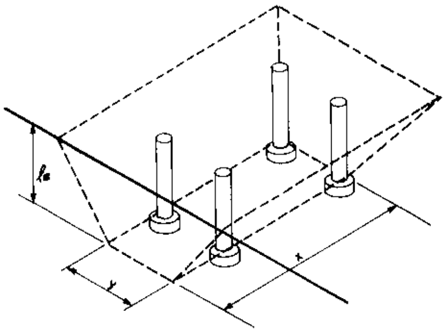
Values shown are for normal-weight concrete with $f'_c = 3000$ psi; for other values of λ and f'_c, multiply tabulated results by $\lambda\sqrt{f'_c/3000}$

Spacing of studs must not exceed $2\ell_e$

ℓ_e, in.	Dimension y, in.	Design tensile strength, ΦP_{nc}, of a stud group, kips											
		Dimension x, in.											
		2	4	6	8	10	12	14	16	18	20	22	24
	0	6	8	10	12	13	15	17	19	21	23	25	27
	2	9	11	14	16	19	22	24	27	29	32	35	37
	4	11	15	18	21	25	28	31	35	38	41	45	48
2.5	6	14	18	22	26	30	34	38	43	47	51	55	59
	8	16	21	26	31	36	41	45	50	55	60	65	70
	10	19	25	30	36	41	47	53	58	64	69	75	81
	12	22	28	34	41	47	53	60	66	72	79	85	91
	0	14	17	20	23	26	29	32	35	38	41	44	47
	2	18	22	26	29	33	37	40	44	48	52	55	59
	4	22	26	31	35	40	44	49	53	58	62	67	71
4.0	6	26	31	36	41	46	52	57	62	67	73	78	83
	8	29	35	41	47	53	59	65	71	77	83	89	95
	10	33	40	46	53	60	67	73	80	87	93	100	107
	12	37	44	52	59	67	74	81	89	96	104	111	119
	0	31	35	40	44	49	53	58	62	67	71	75	80
	2	36	41	46	52	57	62	67	73	78	83	88	93
	4	41	47	53	59	65	71	77	83	89	95	101	107
6.0	6	46	53	60	67	73	80	87	93	100	107	113	120
	8	52	59	67	74	81	89	96	104	111	119	126	134
	10	57	65	73	81	90	98	106	114	122	131	139	147
	12	62	71	80	89	98	107	116	125	134	143	151	160
	0	53	59	65	71	77	83	89	95	101	107	113	119
	2	60	67	73	80	87	93	100	107	113	120	127	134
	4	67	74	81	89	96	104	111	119	126	134	141	148
8.0	6	73	81	90	98	106	114	122	131	139	147	155	163
	8	80	89	98	107	116	125	134	143	151	160	169	178
	10	87	96	106	116	125	135	145	154	164	174	183	193
	12	93	104	114	125	135	146	156	166	177	187	198	208
	0	81	89	96	104	111	119	126	134	141	148	156	163
	2	90	98	106	114	122	131	139	147	155	163	172	180
	4	98	107	116	125	134	143	151	160	169	178	187	196
10.0	6	106	116	125	135	145	154	164	174	183	193	203	213
	8	114	125	135	146	156	166	177	187	198	208	219	229
	10	122	134	145	156	167	178	189	201	212	223	234	245
	12	131	143	154	166	178	190	202	214	226	238	250	262
	0	116	125	134	143	151	160	169	178	187	196	205	214
	2	125	135	145	154	164	174	183	193	203	213	222	232
	4	135	146	156	166	177	187	198	208	219	229	239	250
12.0	6	145	156	167	178	189	201	212	223	234	245	256	268
	8	154	166	178	190	202	214	226	238	250	262	274	286
	10	164	177	189	202	215	227	240	253	265	278	291	303
	12	174	187	201	214	227	241	254	268	281	294	308	321

Metric equivalents: 1 in. = 25.4 mm, 1 kip = 4.45 kN

Table 8. Design Tensile Strength of a Stud Group—Near a Free Edge on One Side



$$\Phi P_{nc} = \Phi\lambda[4\sqrt{f'_c}\,xy + (4/\sqrt{2})\sqrt{f'_c}(\sqrt{2}\,\ell_e(2x + y + 2\ell_e))]$$
$$\Phi \quad = 0.85$$

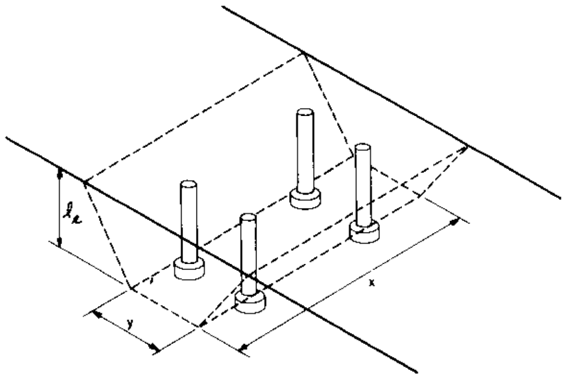
Values shown are for normal-weight concrete with $f'_c = 3000$ psi; for other values of λ and f'_c, multiply tabulated results by $\lambda\sqrt{f'_c/3000}$

Spacing of studs must not exceed $2\ell_e$

		Design tensile strength, ΦP_{nc}, of a stud group, kips											
	Dimension	Dimension x, in.											
ℓ_e, in.	y, in.	2	4	6	8	10	12	14	16	18	20	22	24
2.5	0	4	6	7	9	11	13	15	17	19	20	22	24
	2	5	8	11	13	16	18	21	24	26	29	31	34
	4	7	10	14	17	20	24	27	31	34	37	41	44
	6	9	13	17	21	25	29	33	37	41	46	50	54
	8	10	15	20	25	30	35	39	44	49	54	59	64
	10	12	18	23	29	34	40	46	51	57	62	68	74
	12	14	20	26	33	39	45	52	58	64	71	77	83
4.0	0	8	11	14	17	20	23	26	29	32	35	38	41
	2	11	14	18	22	26	29	33	37	40	44	48	52
	4	13	17	22	26	31	35	40	44	49	53	58	62
	6	15	20	26	31	36	41	46	52	57	62	67	73
	8	17	23	29	35	41	47	53	59	65	71	77	83
	10	20	26	33	40	46	53	60	67	73	80	87	93
	12	22	29	37	44	52	59	67	74	81	89	96	104
6.0	0	17	22	26	31	35	40	44	49	53	58	62	67
	2	20	26	31	36	41	46	52	57	62	67	73	78
	4	23	29	35	41	47	53	59	65	71	77	83	89
	6	26	33	40	46	53	60	67	73	80	87	93	100
	8	29	37	44	52	59	67	74	81	89	96	104	111
	10	32	40	49	57	65	73	81	90	98	106	114	122
	12	35	44	53	62	71	80	89	98	107	116	125	134
8.0	0	29	35	41	47	53	59	65	71	77	83	89	95
	2	33	40	46	53	60	67	73	80	87	93	100	107
	4	37	44	52	59	67	74	81	89	96	104	111	119
	6	40	49	57	65	73	81	90	98	106	114	122	131
	8	44	53	62	71	80	89	98	107	116	125	134	143
	10	48	58	67	77	87	96	106	116	125	135	145	154
	12	52	62	73	83	93	104	114	125	135	146	156	166
10.0	0	44	52	59	67	74	81	89	96	104	111	119	126
	2	49	57	65	73	81	90	98	106	114	122	131	139
	4	53	62	71	80	89	98	107	116	125	134	143	151
	6	58	67	77	87	96	106	116	125	135	145	154	164
	8	62	73	83	93	104	114	125	135	146	156	166	177
	10	67	78	89	100	111	122	134	145	156	167	178	189
	12	71	83	95	107	119	131	143	154	166	178	190	202
12.0	0	62	71	80	89	98	107	116	125	134	143	151	160
	2	67	77	87	96	106	116	125	135	145	154	164	174
	4	73	83	93	104	114	125	135	146	156	166	177	187
	6	78	89	100	111	122	134	145	156	167	178	189	201
	8	83	95	107	119	131	143	154	166	178	190	202	214
	10	88	101	113	126	139	151	164	177	189	202	215	241
	12	93	197	120	134	147	160	174	187	201	214	227	241

Metric equivalents: 1 in. = 25.4 mm, 1 kip = 4.45 kN

Table 9. Design Tensile Strength of a Stud Group—Near a Free Edge on Two Opposite Sides



$\Phi P_{nc} = \Phi\lambda[4\sqrt{f'_c}\,xy + (4/\sqrt{2})\sqrt{f'_c}(2\sqrt{2}\ell_e x)]$
$\Phi \;\; = 0.85$

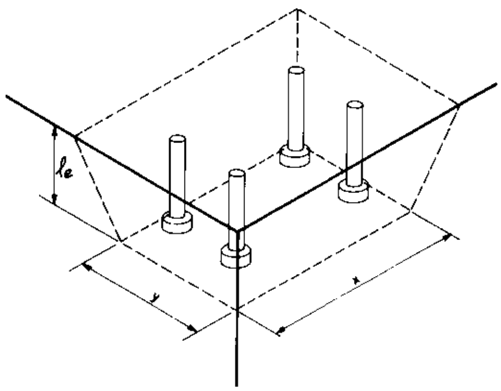
Values shown are for normal-weight concrete with f'_c = 3000 psi; for other values of λ and f'_c, multiply tabulated results by $\lambda\sqrt{f'_c/3000}$

Spacing of studs must not exceed $2\ell_e$

		Design tensile strength, ΦP_{nc}, of a stud group, kips											
	Dimension	Dimension x, in.											
ℓ_e, in.	y, in.	2	4	6	8	10	12	14	16	18	20	22	24
	0	1	3	5	7	9	11	13	14	16	18	20	22
	2	2	5	7	10	13	15	18	20	23	26	28	31
	4	3	6	10	13	16	20	23	26	30	33	36	40
2.5	6	4	8	12	16	20	24	28	32	36	40	45	49
	8	4	9	14	19	24	29	33	38	43	48	53	58
	10	5	11	16	22	27	33	39	44	50	55	61	67
	12	6	12	18	25	31	37	44	50	56	63	69	75
	0	2	5	8	11	14	17	20	23	26	29	32	35
	2	3	7	11	14	18	22	26	29	33	37	40	44
	4	4	8	13	17	22	26	31	35	40	44	49	53
4.0	6	5	10	15	20	26	31	36	41	46	52	57	62
	8	5	11	17	23	29	35	41	47	53	59	65	71
	10	6	13	20	26	33	40	46	53	60	67	73	80
	12	7	14	22	29	37	44	52	59	67	74	81	89
	0	4	8	13	17	22	26	31	35	40	44	49	53
	2	5	10	15	20	26	31	36	41	46	52	57	62
	4	5	11	17	23	29	35	41	47	53	59	65	71
6.0	6	6	13	20	26	33	40	46	53	60	67	73	80
	8	7	14	22	29	37	44	52	59	67	74	81	89
	10	8	16	24	32	40	49	57	65	73	81	90	98
	12	8	17	26	35	44	53	62	71	80	89	98	107
	0	5	11	17	23	29	35	41	47	53	59	65	71
	2	6	13	20	26	33	40	46	53	60	67	73	80
	4	7	14	22	29	37	44	52	59	67	74	81	89
8.0	6	8	16	24	32	40	49	57	65	73	81	90	98
	8	8	17	26	35	44	53	62	71	80	89	98	107
	10	9	19	29	38	48	58	67	77	87	96	106	116
	12	10	20	31	41	52	62	73	83	93	104	114	125
	0	7	14	22	29	37	44	52	59	67	74	81	89
	2	8	16	24	32	40	49	57	65	73	81	90	98
	4	8	17	26	35	44	53	62	71	80	89	98	107
10.0	6	9	19	29	38	48	58	67	77	87	96	106	116
	8	10	20	31	41	52	62	73	83	93	104	114	125
	10	11	22	33	44	55	67	78	89	100	111	122	134
	12	11	23	35	47	59	71	83	95	107	119	131	143
	0	8	17	26	35	44	53	62	71	80	89	98	107
	2	9	19	29	38	48	58	67	77	87	96	106	116
	4	10	20	31	41	52	62	73	83	93	104	114	125
12.0	6	11	22	33	44	55	67	78	89	100	111	122	134
	8	11	23	35	47	59	71	83	95	107	119	131	143
	10	12	25	37	50	63	75	88	101	113	126	139	151
	12	13	26	40	53	67	80	93	107	120	134	147	160

Metric equivalents: 1 in. = 25.4 mm, 1 kip = 4.45 kN

Table 10. Design Tensile Strength of a Stud Group—Near Free Edges on Two Adjacent Sides



$\Phi P_{nc} = \Phi\lambda[4\sqrt{f'_c}\,xy + (4/\sqrt{2})\sqrt{f'_c}(\sqrt{2}\,\ell_e(x + y + \ell_e))]$
$\Phi \quad = 0.85$

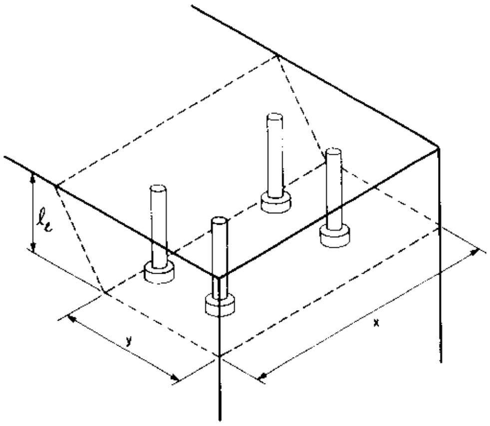
Values shown are for normal-weight concrete with $f'_c = 3000$ psi; for other values of λ and f'_c, multiply tabulated results by $\lambda\sqrt{f'_c/3000}$

Spacing of studs must not exceed $2\ell_e$

ℓ_e, in.	Dimension y, in.	Design tensile strength, ΦP_{nc}, of a stud group, kips											
		Dimension x, in.											
		2	4	6	8	10	12	14	16	18	20	22	24
2.5	0	2	3	3	4	5	6	7	8	9	10	11	12
	2	3	5	7	8	10	12	13	15	17	18	20	22
	4	5	7	10	12	15	17	19	22	24	27	29	32
	6	7	10	13	16	19	22	26	29	32	35	38	41
	8	8	12	16	20	24	28	32	36	40	43	47	51
	10	10	15	19	24	29	33	38	43	47	52	57	61
	12	12	17	22	28	33	39	44	49	55	60	66	71
4.0	0	4	5	7	8	10	11	13	14	16	17	19	20
	2	6	8	11	13	15	17	20	22	24	26	29	31
	4	8	11	14	17	20	23	26	29	32	35	38	41
	6	11	14	18	22	26	29	33	37	40	44	48	52
	8	13	17	22	26	31	35	40	44	49	53	58	62
	10	15	20	26	31	36	41	46	52	57	62	67	73
	12	17	23	29	35	41	47	53	59	65	71	77	83
6.0	0	8	11	13	15	17	20	22	24	26	29	31	33
	2	11	14	17	20	23	26	29	32	35	38	41	44
	4	14	18	22	26	29	33	37	40	44	48	52	55
	6	17	22	26	31	35	40	44	49	53	58	62	67
	8	20	26	31	36	41	46	52	57	62	67	73	78
	10	23	29	35	41	47	53	59	65	71	77	83	89
	12	26	33	40	46	53	60	67	73	80	87	93	100
8.0	0	14	17	20	23	26	29	32	35	38	41	44	47
	2	18	22	26	29	33	37	40	44	48	52	55	59
	4	22	26	31	35	40	44	49	53	58	62	67	71
	6	26	31	36	41	46	52	57	62	67	73	78	83
	8	29	35	41	47	53	59	65	71	77	83	89	95
	10	33	40	46	53	60	67	73	80	87	93	100	107
	12	37	44	52	59	67	74	81	89	96	104	111	119
10.0	0	22	26	29	33	37	40	44	48	52	55	59	63
	2	26	31	35	40	44	49	53	58	62	67	71	75
	4	31	36	41	46	52	57	62	67	73	78	83	88
	6	35	41	47	53	59	65	71	77	83	89	95	101
	8	40	46	53	60	67	73	80	87	93	100	107	113
	10	44	52	59	67	74	81	89	96	104	111	119	126
	12	49	57	65	73	81	90	98	106	114	122	131	139
12.0	0	31	35	40	44	49	53	58	62	67	71	75	80
	2	36	41	46	52	57	62	67	73	78	83	88	93
	4	41	47	53	59	65	71	77	83	89	95	101	107
	6	46	53	60	67	73	80	87	93	100	107	113	120
	8	52	59	67	74	81	89	96	104	111	119	126	134
	10	57	65	73	81	90	98	106	114	122	131	139	147
	12	62	71	80	89	98	107	116	125	134	143	151	160

Metric equivalents: 1 in. = 25.4 mm, 1 kip = 4.45 kN

Table 11. Design Tensile Strength of a Stud Group—Near Three Adjacent Free Edges



$\Phi P_{nc} = \Phi\lambda[4\sqrt{f'_c}\,xy + (4/\sqrt{2})\sqrt{f'_c}(\sqrt{2}\,\ell_e\, x)]$

$\Phi \quad = 0.85$

Values shown are for normal-weight concrete with $f'_c = 3000$ psi; for other values of λ and f'_c, multiply tabulated results by $\lambda\sqrt{f'_c/3000}$

Spacing of studs must not exceed $2\ell_e$

ℓ_e, in.	Dimension y, in.	Design tensile strength, ΦP_{nc}, of a stud group, kips											
		Dimension x, in.											
		2	4	6	8	10	12	14	16	18	20	22	24
2.5	0	0	1	2	3	4	5	6	7	8	9	10	11
	2	1	3	5	6	8	10	11	13	15	16	18	20
	4	2	4	7	9	12	14	16	19	21	24	26	29
	6	3	6	9	12	15	18	22	25	28	31	34	37
	8	3	7	11	15	19	23	27	31	35	39	43	46
	10	4	9	13	18	23	27	32	37	41	46	51	55
	12	5	10	16	21	27	32	37	43	48	54	59	64
4.0	0	1	2	4	5	7	8	10	11	13	14	16	17
	2	2	4	6	8	11	13	15	17	20	22	24	26
	4	2	5	8	11	14	17	20	23	26	29	32	35
	6	3	7	11	14	18	22	26	29	33	37	40	44
	8	4	8	13	17	22	26	31	35	40	44	49	53
	10	5	10	15	20	26	31	36	41	46	52	57	62
	12	5	11	17	23	29	35	41	47	53	59	65	71
6.0	0	2	4	6	8	11	13	15	17	20	22	24	26
	2	2	5	8	11	14	17	20	23	26	29	32	35
	4	3	7	11	14	18	22	26	29	33	37	40	44
	6	4	8	13	17	22	26	31	35	40	44	49	53
	8	5	10	15	20	26	31	36	41	46	52	57	62
	10	5	11	17	23	29	35	41	47	53	59	65	71
	12	6	13	20	26	33	40	46	53	60	67	73	80
8.0	0	2	5	8	11	14	17	20	23	26	29	32	35
	2	3	7	11	14	18	22	26	29	33	37	40	44
	4	4	8	13	17	22	26	31	35	40	44	49	53
	6	5	10	15	20	26	31	36	41	46	52	57	62
	8	5	11	17	23	29	35	41	47	53	59	65	71
	10	6	13	20	26	33	40	46	53	60	67	73	80
	12	7	14	22	29	37	44	52	59	67	74	81	89
10.0	0	3	7	11	14	18	22	26	29	33	37	40	44
	2	4	8	13	17	22	26	31	35	40	44	49	53
	4	5	10	15	20	26	31	36	41	46	52	57	62
	6	5	11	17	23	29	35	41	47	53	59	65	71
	8	6	13	20	26	33	40	46	53	60	67	73	80
	10	7	14	22	29	37	44	52	59	67	74	81	89
	12	8	16	24	32	40	49	57	65	73	81	90	98
12.0	0	4	8	13	17	22	26	31	35	40	44	49	53
	2	5	10	15	20	26	31	36	41	46	52	57	62
	4	5	11	17	23	29	35	41	47	53	59	65	71
	6	6	13	20	26	33	40	46	53	60	67	73	80
	8	7	14	22	29	37	44	52	59	67	74	81	89
	10	8	16	24	32	40	49	57	65	73	81	90	98
	12	8	17	26	35	44	53	62	71	80	89	98	107

Metric equivalents: 1 in. = 25.4 mm, 1 kip = 4.45 kN

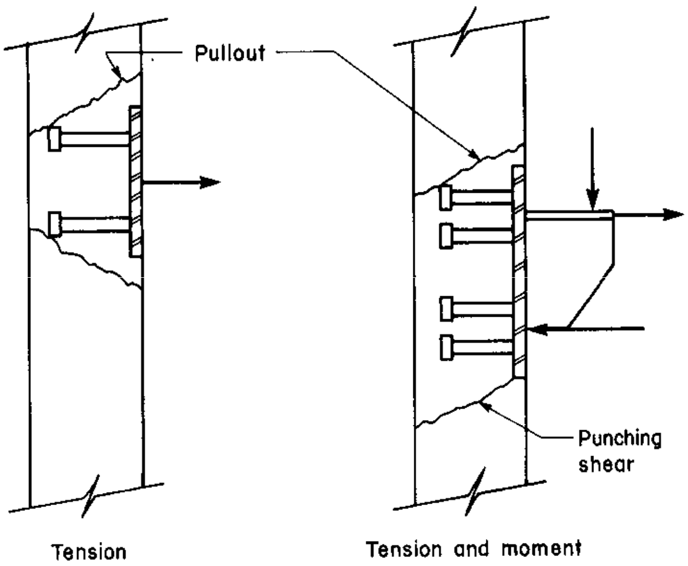


Fig. 15 Stud groups in thin sections under combined tension and moment. (Reproduced from Reference 1)

STUD GROUPS IN THIN SECTIONS

Where plates are anchored to the face of a wall panel, special consideration should be given to stud groups. Since tilt-up panels generally have thin sections, pullout of the anchor plate may produce a localized or global failure. As illustrated in Fig. 15, anchor plates subjected to tension or bending result in two general failure modes. Under tension, the stud group has a typical pullout failure with the shear boundary shaped similar to a truncated pyramid. When the anchor plate resists a force couple, with compression on one side and tension on the other, the tension face "pulls out" while the compression zone "punches through."

Special consideration and design procedures must be used to accommodate either of the above situations. For guidance in design see Reference 28.

COMBINED SHEAR AND TENSION

Interaction of shear and tension on studs will result in strength reduction under combined loading conditions. The following interaction equations should be used to evaluate strength reduction of both concrete and steel:

$$\text{Concrete: } \frac{1}{\Phi}\left[\left(\frac{P_u}{P_{nc}}\right)^2 + \left(\frac{V_u}{V_{nc}}\right)^2\right] \leq 1 \qquad \text{Eq. 11}$$

$$\text{Steel: } \frac{1}{\Phi}\left[\left(\frac{P_u}{P_{ns}}\right)^2 + \left(\frac{V_u}{V_{ns}}\right)^2\right] \leq 1 \qquad \text{Eq. 12}$$

Plate thickness for stud groups subjected to combined shear and tension should not be less than ⅔ the diameter of the studs.

STEEL DESIGN

Design of light steel framing and steel connection elements is beyond the scope of this text. For further information on steel design, refer to Reference 20, the AISC *Manual of Steel Construction.*

WOOD DESIGN

Design of wood joints and support elements is beyond the scope of this text. For further information on wood design, see Reference 27, the NFPA *National Design Specification for Wood Construction.*

Connection Details

In this section connection details are provided that are considered typical. They have been obtained from sources in various areas of the United States and Canada. Due to local environmental conditions such as temperature, humidity, expansive soils, or seismic considerations or because of locally accepted details and practices, some of the details shown may not perform satisfactorily or prove economical in a given area. Each detail should be studied thoroughly before adopting its use. A brief commentary describing each connection followed by a list of advantages and disadvantages is provided. The details are divided into five categories: Wall panels to foundation, floor, roof, wall panel, and steel column connections.

WALL PANEL TO FOUNDATION CONNECTIONS

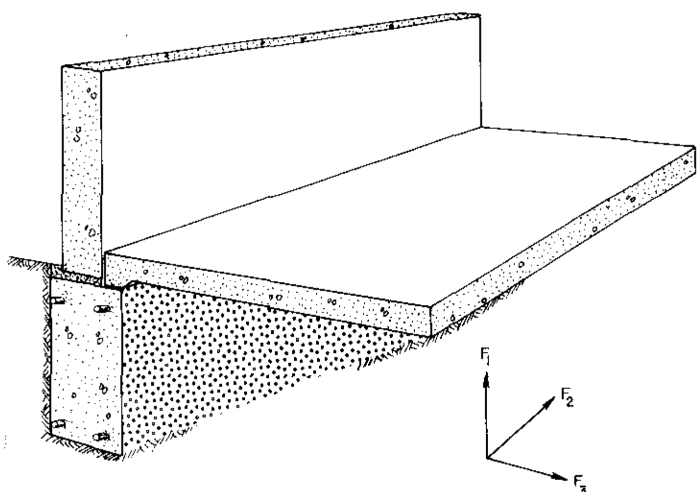
Trenched Footing/Wall Connection

In this connection, the top elevation of the continuous footing is raised to allow direct support of the slab-on-grade. The edge of the slab is used for proper alignment of the wall panel. Lateral support at the base is provided by embedded material in wall and floor.

Advantage: Slab-on-grade facilitates erection of the wall.

Disadvantage: Forming of the slab-on-grade requires tight tolerances.

Restraints: Primary: F_1, F_3
Secondary: F_2

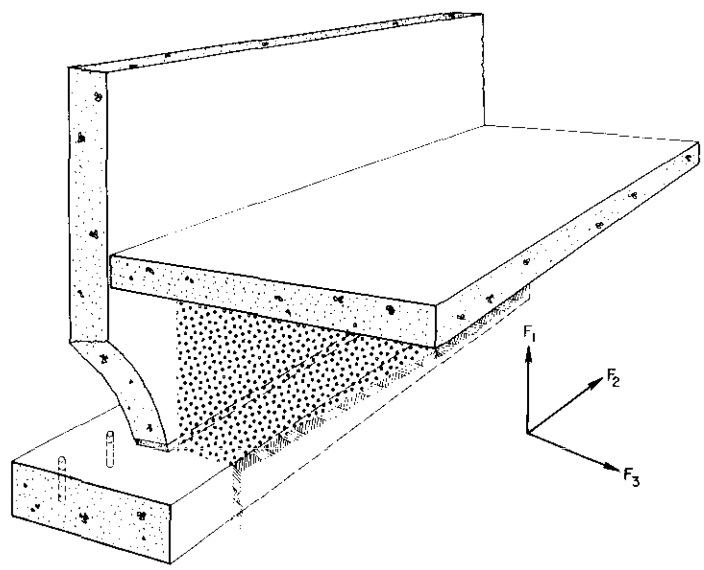


Footing/Wall Connection with Exterior Dowels

In this connection, the wall panel is grouted onto a continuous footing, isolated footings, or drilled pier. Drilled-in dowels are provided on one or both sides of the wall panel. The dowels are used for proper alignment of the wall panel during erection. Leveling shims are placed at each end of the panel. The panel is connected to the slab on grade with dowels or embedments (see Connection Detail for "Slab on Grade/Wall Connection.")

Advantage: Use of the dowel facilitates erection of the wall.

Restraints: Primary: F_1, F_3



WALL PANEL TO FLOOR CONNECTIONS

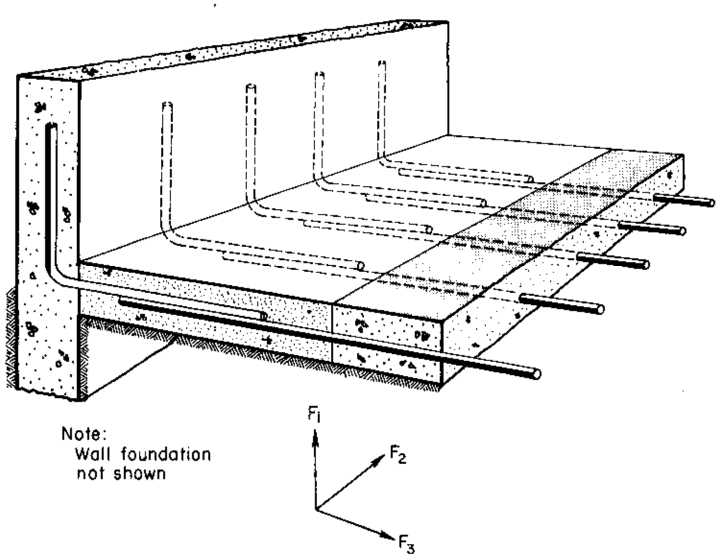
Slab on Grade/Wall Connection

In this connection, the floor slab provides lateral support for a wall panel. The slab on grade is cast, except for a narrow strip (2 to 4 ft wide) adjacent to the wall panel. The wall panel has cast-in-place bent dowels. Threaded inserts can be used in place of bent bars. Dowels or bars are used to form splice with reinforcement in floor slab. Floor area adjacent to the wall panel is then cast. Control joints in closure strips should match control joints in floor.

Advantages: Fabrication is simple
Connection is self-forming.

Disadvantage: Dowels may be damaged during handling.

Restraints: Primary: F_3
Secondary: F_1, F_2



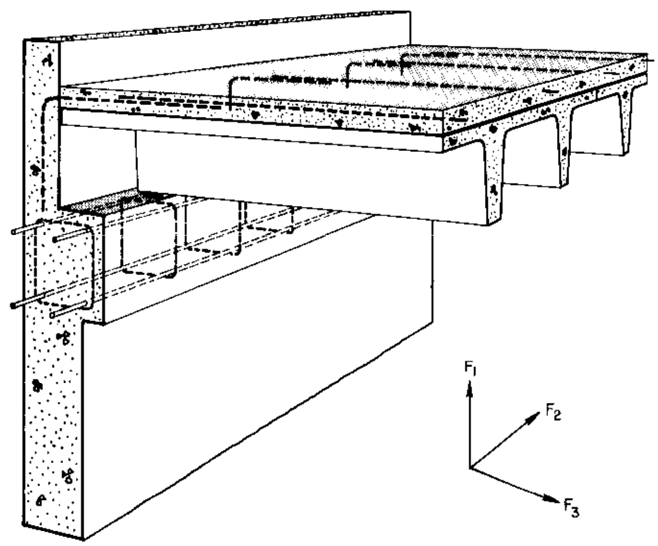
Precast Double Tee/Wall Connection with Ledge

In this connection, a concrete ledge is cast monolithically with a wall panel. The ledge provides vertical support for a double-tee floor system. The double-tee beams rest on bearing pads. The double-tee floor system is topped with a thin layer of reinforced concrete. Deformed dowels in the wall extend into the reinforced concrete topping to provide lateral support for the wall panel. Steel embedments or threaded inserts may be used in place of bent bars.

Advantages: Minimal number of components in connection.
Connection is self-forming.

Disadvantages: Casting of ledge complicates fabrication of wall panel.
Special steps are required to protect dowels during handling.
Placing of the double tee units under the dowels may complicate erection.

Restraints: Primary: F_1, F_3
Secondary: F_2



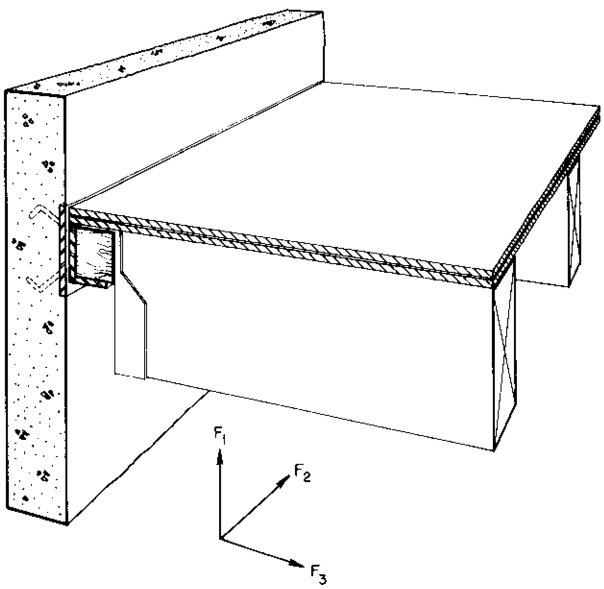
Wood Joist/Wall Connection with Joist Hanger on Wood Ledger

In this connection, a composite ledger provides vertical support for wood joists. The ledger includes a wood nailer bolted to a continuous steel angle using countersunk nuts. The continuous steel angle is welded to steel embedments located at selected intervals. Each joist is connected to the ledger through a steel hanger. Each steel hanger is nailed to the top face of the ledger. The plywood flooring sheets are nailed to the ledger and the wood joist to provide lateral support for the wall panel.

Advantage: Embedment plates allow large construction tolerances.

Disadvantages: Fabrication of the composite ledger requires additional effort.
Welding in the up-hand position may be required.
Consolidation of concrete near embedment plate may be difficult.

Restraints: Primary: F_1, F_3
Secondary: F_2



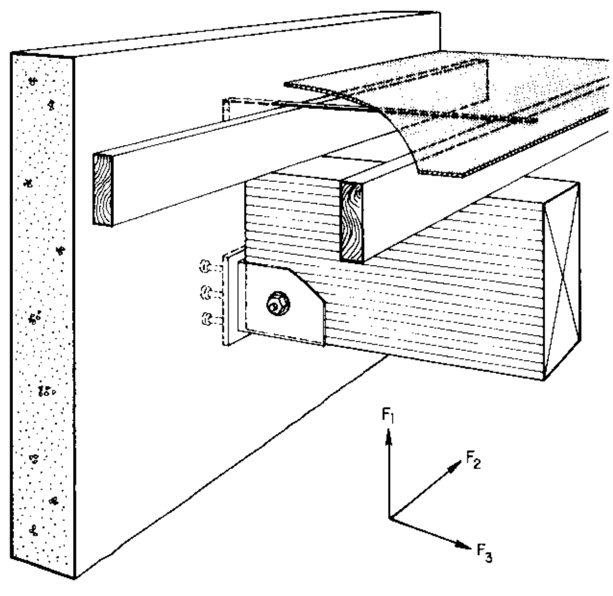
Heavy Timber Beam/Wall Connection with Steel Shoe

In this connection, a steel shoe, welded to a steel embedment, provides vertical support for the heavy timber beam. The beam and the roof diaphragm provide lateral support for the wall panel. Wood joists are secured to the wall panel through embedded steel straps in Seismic Zones 2, 3, and 4.

Advantages: Fabrication and erection are simple.
Steel embedment allows large construction tolerances.

Disadvantages: Bearing-type bolted connection requires tight tolerances.
Consolidation of concrete around embedment may be difficult.
Exposed steel may require fireproofing.

Restraints: Primary: F_1, F_3
Secondary: F_2



WALL PANEL TO ROOF CONNECTIONS

Precast Double Tee Roof/Precast Beam/Wall Connections

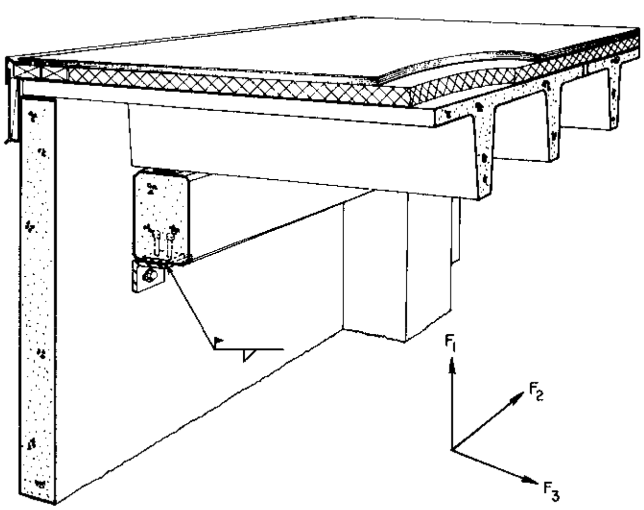
In this connection, a precast concrete beam spans between adjacent reinforced concrete columns. The beam provides vertical support for a double-tee roof system. The precast concrete beam also provides lateral support for the wall panel. One end of the double-tee beam rests on a flexible bearing pad that accommodates movement due to volume change while the opposite end-bearing is rigid. The double-tee flange is cantilevered at the end so that the flange extends over the wall panel. Clip angles are used at selected intervals to tie the wall panel to the beam. Each angle is welded to a steel embedment cast into the beam. Each angle is bolted to the wall panel through embedded threaded parts. Oversized holes are used in the angles to accommodate movement due to the volume change and deflection of the beam.

An alternate supporting detail for the precast double tee is shown as Connection Detail for "Precast Double Tee/Wall Connection with Ledge."

Advantages: Erection is simple.
The connection accommodates movement due to volume change.

Disadvantages: Exposed steel angles may require fireproofing.
The welder must work in the up-hand position.
Placing of cast-in-place inserts requires tight tolerances.

Restraints: Double Tee/Precast Concrete Beam
Primary: F_1
Precast Beam/Wall
Primary: F_3



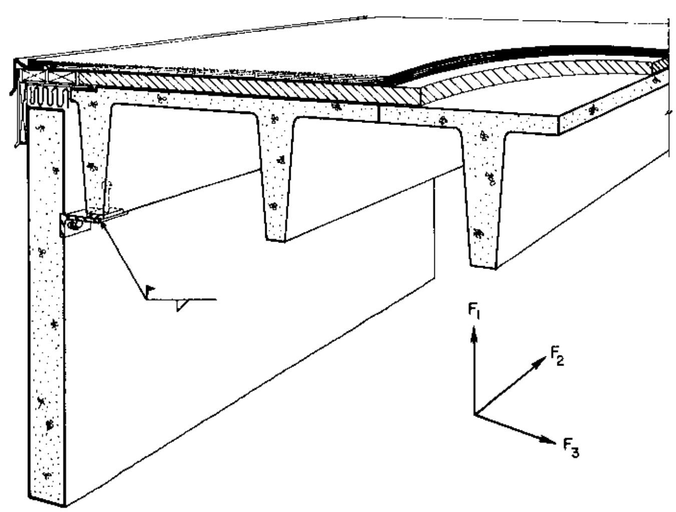
Precast Double Tee Roof/Wall Connections

In this connection, the edge beam of a double-tee roof system provides lateral support for a wall panel. Clip angles are used at selected intervals to tie the wall panel to the edge beam. Each angle is welded to a steel embedment cast into the edge beam. Each angle is bolted to the wall panel through embedded threaded parts. Oversize holes are used in the angles to accommodate movement due to volume change and deflection of the edge beam. Vertical support for the roof should be provided through a separate system, such as the one shown in Connection Detail for "Precast Double Tee Roof/Precast Beam/Wall Connections."

Advantages: Fabrication and erection are simple.
The connection accommodates movement due to volume change and deflection of the edge beam.

Disadvantages: Exposed steel angles may require fireproofing.
The welder must work in the up-hand position.
Placing of cast-in-place inserts requires tight tolerances.

Restraint: Primary: F_3



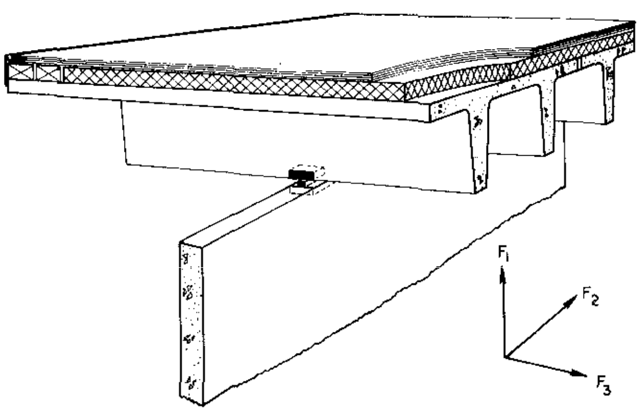
Precast Double Tee Roof/Bearing on Wall Connection

In this connection, the double-tee roof system is supported directly by the tilt-up wall panels. Steel embedments are cast into the underside of the stems of the double tees and into the top edge of the wall panels. A pivot bar is welded between the bearing plates in the tee stems and the wall panel. The double-tee roof provides lateral support at the top of the wall panels.

Advantages: Erection is simple.
Minimal number of components.
Embedments allow large construction tolerances.

Disadvantages: The connection accommodates only minimal displacement or rotation due to volume changes or deflections in the double tee.
Exposed steel may require fireproofing and weather protection.
The welder must work in the up-hand position.
Special reinforcement may be required in the wall panels at bearing points.

Restraints: Primary: F_1, F_3



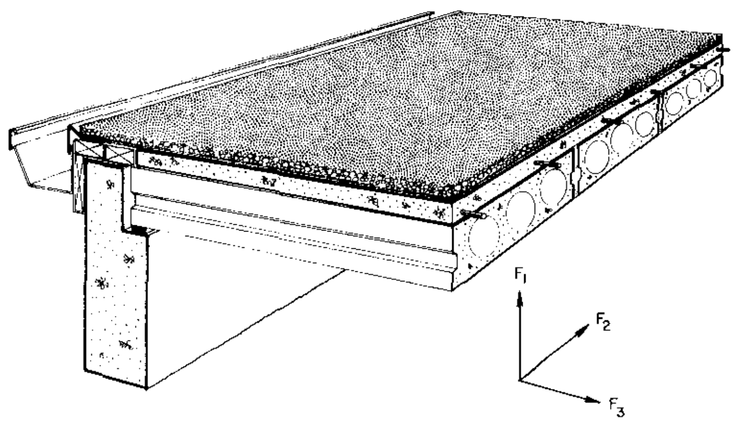
Precast Hollow-Core Roof/Bearing on Wall Connection

In this connection, a recessed ledge in the tilt-up wall provides vertical support for a precast hollow-core roof system. The hollow-core units rest on a flexible bearing pad that accommodates movement due to volume change. Clip angles similar to those shown in Connection Detail for "Precast Double Tee Roof/Precast Beam/Wall Connections" may be needed to provide lateral support to panels.

Advantages: Erection is simple.
Connection accommodates movement due to volume change.

Disadvantage: Special reinforcement and forming may be required at the ledge.

Restraints: Primary: F_1, F_3



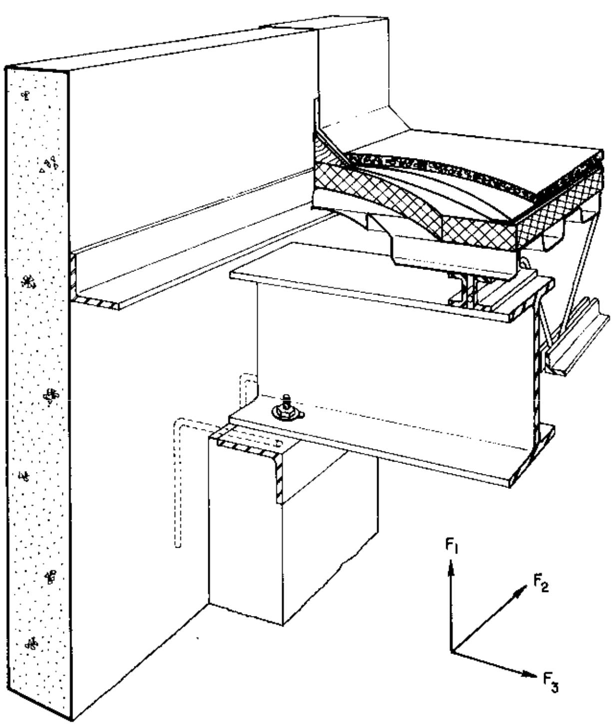
Steel Girder/Pilaster/Wall Connections

In this connection, a reinforced concrete pilaster provides vertical support for a steel girder. The pilaster is cast on the tilt-up panel while in the horizontal position. The pilaster is reinforced as a column with ties projecting from the panel. This connection is used when concentrated roof or floor loads are large and cannot be efficiently transmitted through the wall panel. The vertical load is transmitted through bearing on the steel angle. The steel angle is connected to the pilaster with embedments. Slotted holes are used in the girder to accommodate construction tolerances. The pilaster and the roof diaphragm provide lateral support for the wall panel. Embedded reinforcing bars are extended from the wall panel into the pilaster and welded to the steel angle for anchorage. For comments on the continuous angle, refer to Connection Detail for "In-Plane Wall/Wall Connection with Continuous Steel Chord."

Advantage: Suitable for supporting large, concentrated loads.

Disadvantages: Fabrication of the pilaster and its foundation requires additional effort at the jobsite.
Exposed steel may require fireproofing.

Restraints: Steel Girder/Pilaster
Primary: F_1
Secondary: F_3
Pilaster/Wall
Primary F_3
Secondary F_1, F_2



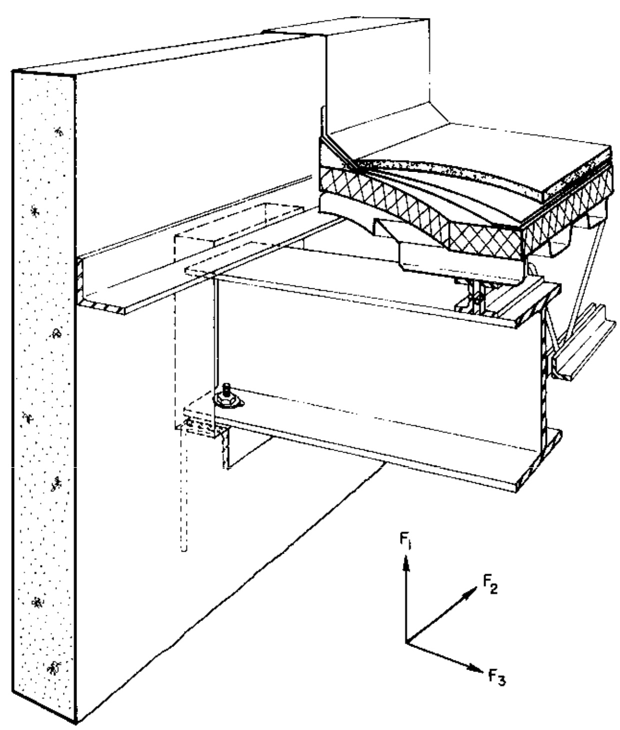
Steel Girder/Wall Connection with Recessed Pocket

In this connection, a recessed pocket in the wall panel provides vertical support for a steel girder. The load is transmitted through bearing on an embedded steel angle. Reinforcing bars are welded to the steel angle for anchorage. The steel girder and the concrete roof diaphragm provide lateral support for the wall panel. The steel girder is bolted to the wall panel through embedded threaded parts. Slotted holes are used in the girder to accommodate construction tolerances. This detail can also be applied to a steel joist/wall connection. For comments on the continuous angle, refer to Connection Detail for "In-Plane Wall/Wall Connection with Continuous Steel Chord."

Advantage: Number of components in connection is minimal.

Disadvantages: Forming of the recessed pocket and embedment of the angle require tight tolerances.
Consolidation of concrete near the pocket may be difficult.
Exposed steel may require fireproofing.
Placing the end of the girder within the recessed pocket may complicate erection procedures.

Restraints: Primary: F_1, F_3
Secondary: F_2



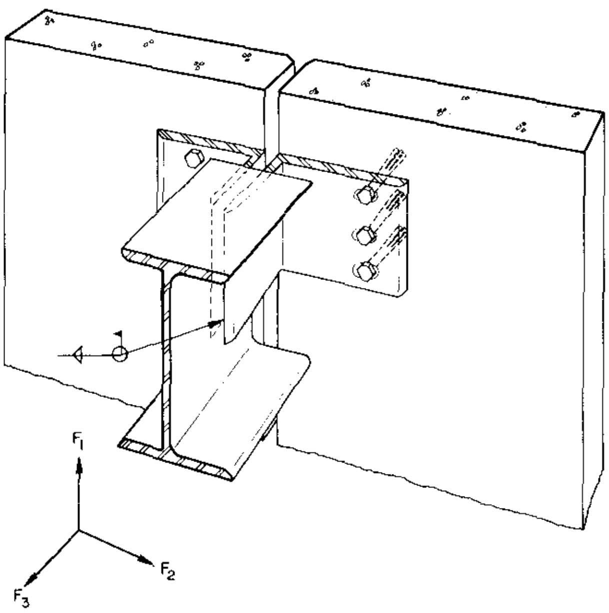
Steel Girder/Clip Angle Wall Connection

In this connection, vertical support for a steel girder is provided by a clip angle attached to the wall panel by threaded inserts. An alternate for attachment is to weld the clip angle to an embedded plate. The clip angle is then welded to the web of the beam. An alternate connection of beam to clip angle is bolting. The clip angle has slotted holes to allow in-plane movement of the panels and accommodate large construction tolerances. It is preferable to locate beam or girder supports away from joints between panels. A minimum of 2 ft is suggested.

Advantages: Number of components in connection is minimal.
Slotted holes (or embedments) allow large construction tolerances.

Disadvantages: Exposed steel may require fireproofing.
Welding of beam to clip angle must be done in overhead position.

Restraints: Primary: F_1
Secondary: F_2, F_3



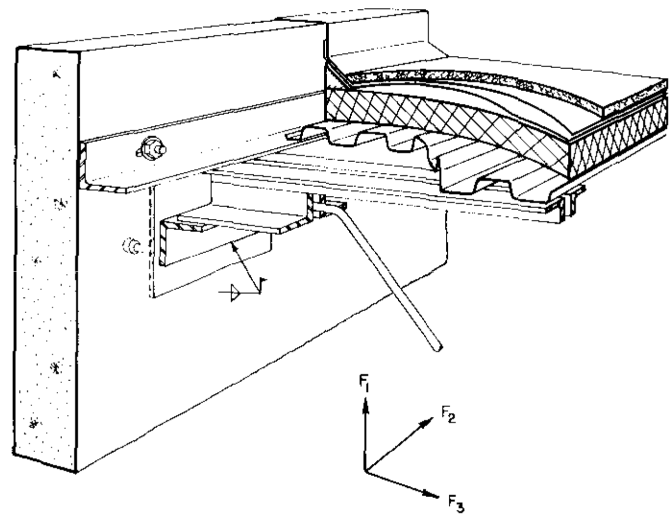
Steel Joist/Wall Connection with Seat Angle

In this connection, a seat angle provides vertical support for a steel joist. The seat angle is welded to a steel embedment after the panel is cast. Alternatively a built-up steel-plate bracket can replace the seat angle. The steel joist is welded or bolted to the seat angle. The steel joist and roof diaphragm provide lateral support for the wall panel. For comments on the continuous angle, refer to Connection Detail for "In-Plane Wall/Wall Connection with Continuous Steel Chord."

Advantages: Erection is simple.
Steel embedment allows large construction tolerances.

Disadvantages: Exposed steel may require fireproofing.
Consolidation of concrete near steel embedment may be difficult.

Restraints: Primary: F_1, F_3
Secondary: F_2



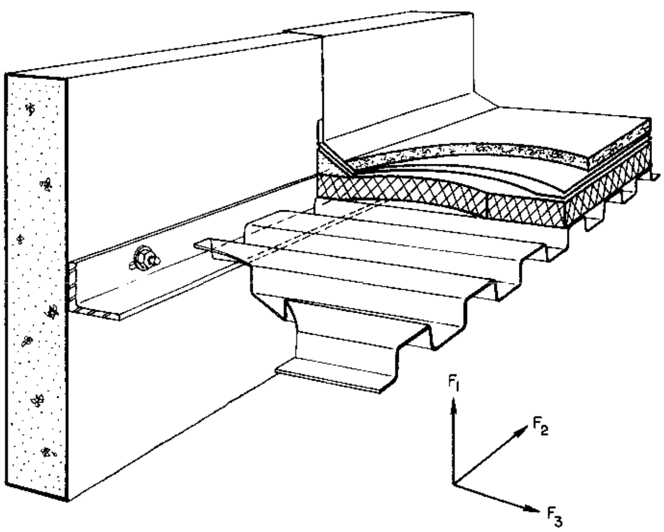
Metal Deck/Wall Connection

In this connection, a continuous steel angle provides vertical support for a corrugated metal roof deck and can be used as the chord for the roof diaphragm. The roof deck is tack or puddle welded to the angle. The angle is bolted to the wall through embedded threaded parts. Alternatively, the steel angle can be welded to steel embedments at selected intervals. The roof diaphragm provides lateral support for the wall panel. For additional comments on continuous angle refer to Connection Detail for "In-Plane Wall/Wall Connection with Continuous Steel Chord."

Advantage: Fabrication and erection are simple.

Disadvantages: Installation of the continuous angle may require tight tolerances.
Exposed steel may require fireproofing.

Restraints: Primary: F_1, F_3
Secondary: F_2



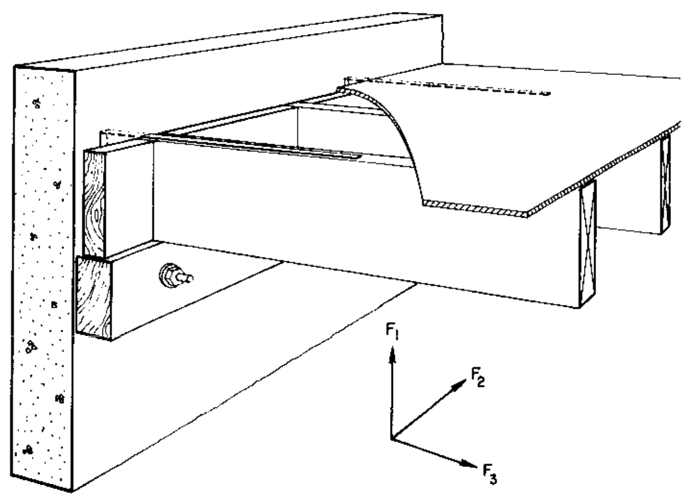
Wood Joist/Wall Connection with Wood Ledger

In this connection, a wood ledger provides vertical support for a wood-joist roof system. The ledger is connected to the wall panel using embedded threaded parts. The embedments can be cast in the concrete by using the predrilled ledger as a template. Embedded steel straps are nailed to wood joists to provide lateral support for the wall panel in Seismic Zones 2, 3, and 4. Bearing between the wall panel and the wood joists provides lateral support when an inward horizontal force is applied.

Advantage: Fabrication and erection are simple.

Disadvantage: Close placement tolerances are required on cast-in-place steel straps.

Restraints: Primary: F_1, F_3
Secondary: F_2

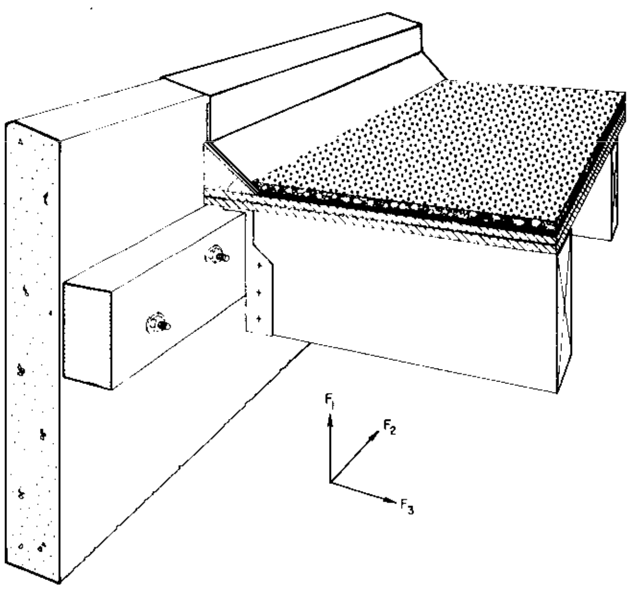


Wood Joist/Wall Connection with Joist Hanger on Wood Ledger

In this connection, a wood ledger provides vertical support for wood joists. Each joist is connected to the ledger through a steel hanger. Each steel hanger is nailed to top face of the ledger. The ledger is connected to the wall panel through embedded threaded parts. The embedments can be cast in the concrete by using the predrilled ledger as a template. An alternate attachment for the wood-ledger connection to the wall panel is shown in Connection Detail for "Wood Joist/Wall Connection with Joist Hanger on Wood Ledger." The plywood roof sheets are nailed to the ledger and the wood joists to provide lateral support for the wall panel. Embedded steel-strap ties are required in Seismic Zones 2, 3, and 4, as shown in Connection Detail for "Wood Joist/Wall Connection with Wood Ledger."

Advantage: Fabrication and erection are simple.

Restraints: Primary: F_1, F_3
Secondary: F_2

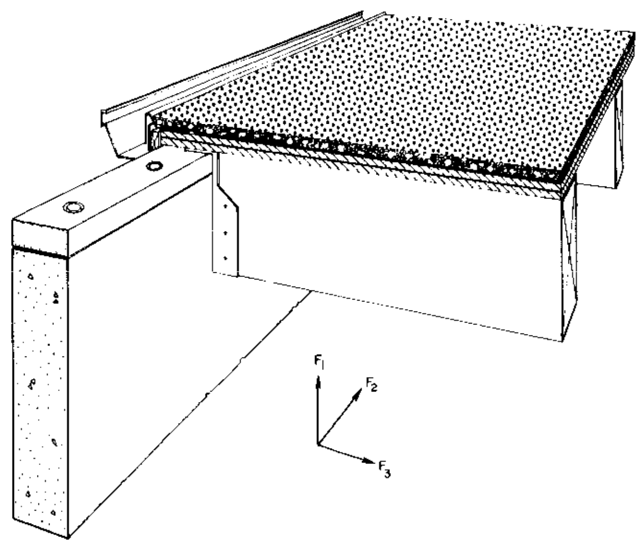


Wood Joist/Wall Connection with Joist Hanger on Panel Top

In this connection, the top of a wall panel provides vertical support for the wood joists. Each joist is connected to top of the wall panel through a steel hanger. Each hanger is nailed to a wood nailer. The wood nailer is bolted to the top of the wall panel using embedded threaded parts with countersunk heads. The wood nailer can be utilized as part of the formwork for casting the wall panel, as a template for the embedments. The plywood roof sheets are nailed to the wood nailer and the wood joists to provide lateral support for the wall panel.

Advantage: Fabrication and erection are simple.

Restraints: Primary: F_1, F_3
Secondary: F_2

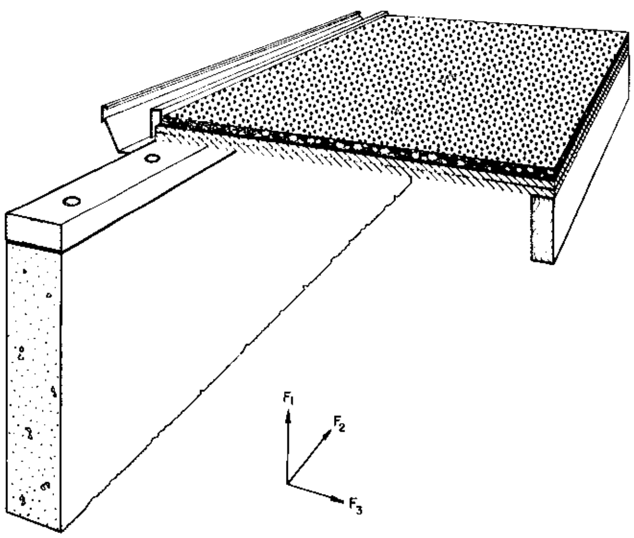


Plywood Roof Deck/Wall Connection with Wood Ledger on Panel Top

In this connection, the top of a wall panel provides vertical support for a narrow strip of the roof adjacent to the wall panel. The wood joists span parallel to the wall panel. The plywood sheets are nailed to a wood nailer. The wood nailer is bolted to the top face of the wall panel using embedded threaded parts with countersunk heads. The wood nailer can be efficiently utilized as part of the formwork for casting the wall panel. The plywood sheets provide lateral support for the wall panel.

Advantage: Fabrication and erection are simple.

Restraints: Primary: F_1, F_3
Secondary: F_2



WALL PANEL TO WALL PANEL CONNECTIONS

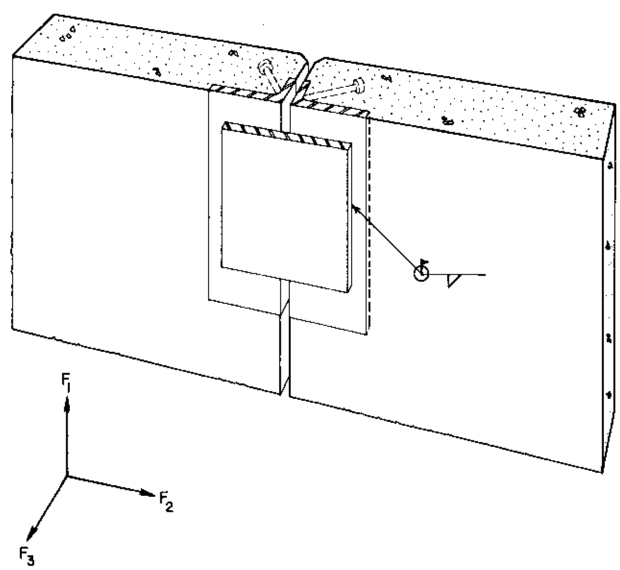
In-Plane Wall/Wall Connection with Steel Embedments

In this connection, steel angles are embedded at the interior face of the wall panels. A steel plate is welded to both embedments to form the connection. It is recommended that no more than two or three panels be connected without a free joint. This detail can also be used with a recessed pocket and then plastered for esthetics.

Advantages: Number of components in connection is minimal.
Fabrication and erection are simple.
Steel embedments allow large construction tolerances.

Disadvantages: Exposed steel may require fireproofing.
Consolidation of concrete near steel embedments may be difficult.
Very rigid connection, can cause local distress due to volume changes.

Restraints: Primary: F_1, F_2
Secondary: F_3



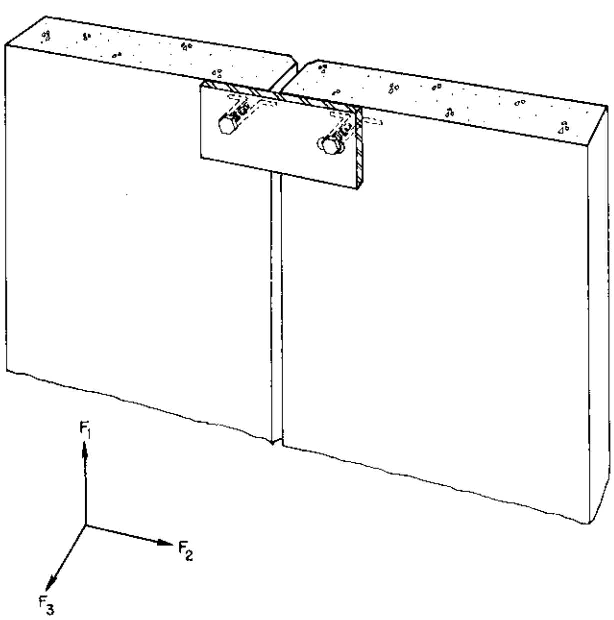
In-Plane Wall/Wall Connections with Threaded Inserts

In this connection, threaded inserts are embedded at the interior face of the wall panels. A steel plate is bolted to both panels. One of the bolt holes in the connection plate is slotted to allow movement due to volume change and to make alignment easier.

Advantages: Fabrication and erection are simple. Slotted holes allow movement due to volume change.

Disadvantages: Exposed steel may require fireproofing. Consolidation of concrete near steel embedments may be difficult.

Restraints: Primary: F_3



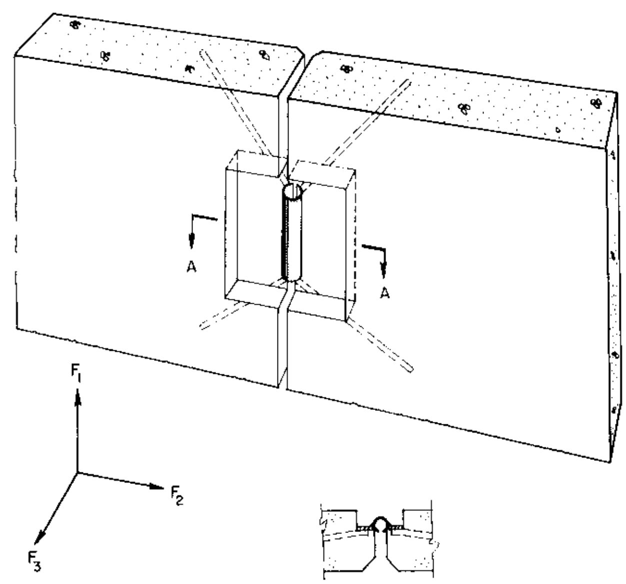
In-Plane Wall/Wall Connections with Slitted Pipe

In this connection, two adjacent wall panels are connected through slitted pipes. Each pipe includes one longitudinal slit. At either side of the slit, the pipe is welded to embedments to form a connection. Generally the pipe is placed into a recessed pocket. After erection, a bond-breaking agent is applied on the pipe and the recessed pocket is grouted for esthetics.

Advantages: Connection accommodates some horizontal differential movement between wall panels. Connection can be "hidden."

Disadvantages: Consolidation of concrete near steel embedments may be difficult. Pocket complicates fabrication of wall panel. Welding of pipe may be difficult.

Restraints: Primary: F_1, F_3



Section A-A

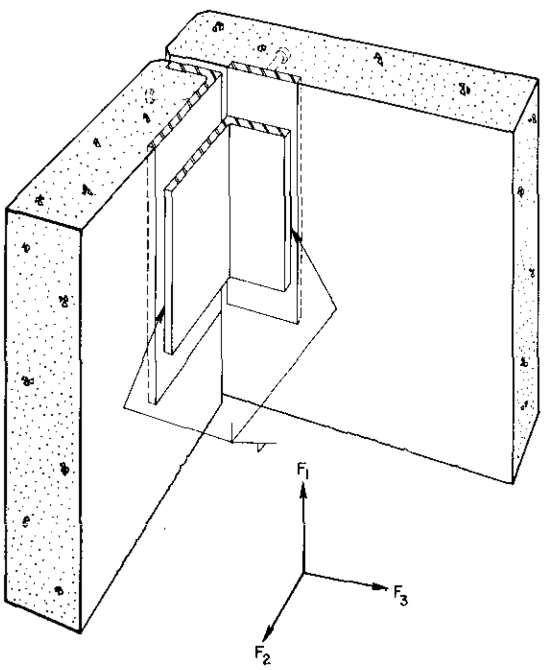
Corner Wall/Wall Connections with Steel Embedments

In this connection, wall panels are connected through clip angles. Each clip angle is welded to steel embedments cast into the wall panels. Alternatively the clip angles can be bolted to the wall panels through embedded threaded parts. Rigid, welded corner connection is less critical than Connection Detail for "In-Plane Wall/Wall Connection with Steel Embedments" for distress due to volume change.

Advantages: Number of components in connection is minimal.
Fabrication and erection are simple.
Steel embedments allow large construction tolerances.

Disadvantages: Exposed steel may require fireproofing.
Consolidation of concrete near embedment may be difficult.
Connection may require welding in the up-hand position.

Restraints: Primary: F_1, F_2, F_3



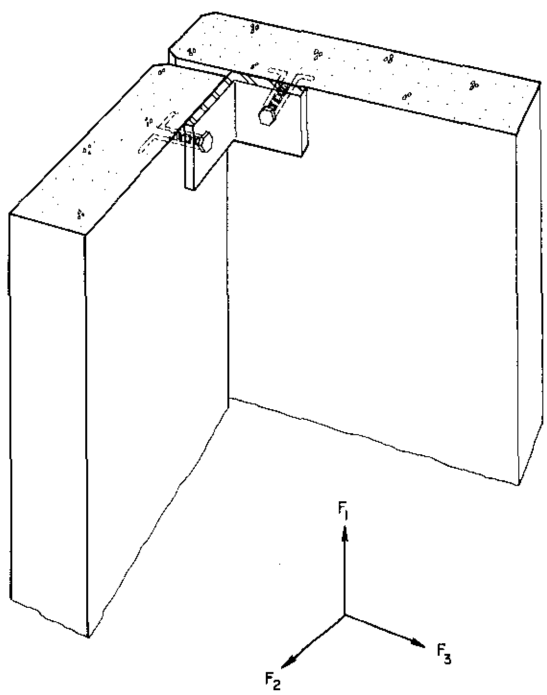
Corner Wall/Wall Connections with Threaded Inserts—Type 1

In this connection, wall panels are connected through clip angles. The clip angles are bolted to the panels with threaded inserts cast into the panels.

Advantages: Number of components in connection is minimal.
Fabrication and erection are simple.

Disadvantages: Exposed steel may require fireproofing.
Close construction tolerances required.

Restraints: Primary: F_2, F_3



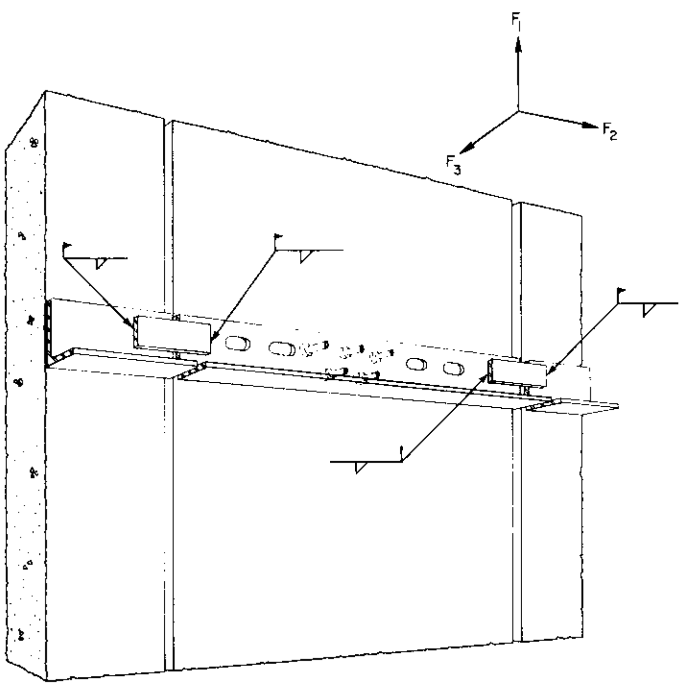
In-Plane Wall/Wall Connections with Continuous Steel Chord

In this connection, a continuous angle is embedded in the surface with headed studs anchoring the steel section over one quarter of the panel length centered on the centerline of the panel. In the remaining three quarters of the panel width the chord is bolted to the panel through slotted holes. At each joint, a steel plate is welded to the angles attached to the adjacent panels to form the connection. The continuous steel section may provide vertical support for a portion of a roof or floor system and also act as chord for the roof diaphragm.

Advantages: Fabrication and erection are simple. Steel embedments allow large construction tolerances.

Disadvantages: Exposed steel may require fireproofing. Connection may require welding in the up-hand position. Consolidation of concrete near embedment may be difficult.

Restraints: Primary: F_1, F_2
Secondary: F_3



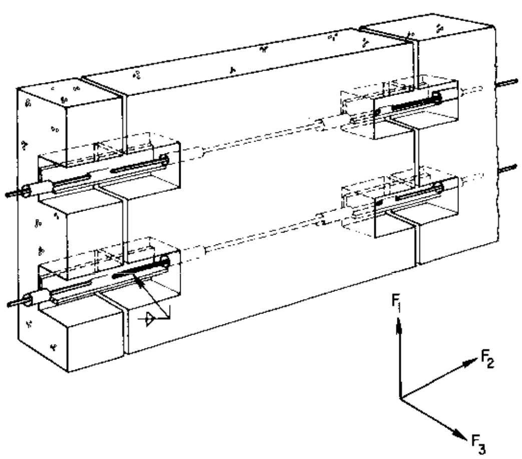
In-Plane Diaphragm Chord Wall/Wall Connection

In this connection, wall panels are connected continuously by means of continuous reinforcement. Reinforcement is mechanically anchored to the center one-third of the panel. The outer one-third of the top chord of reinforcement is unbonded by encasement in cardboard or plastic sleeves. A recessed pocket is provided to allow weldment of an angle to reinforcement in adjacent wall panels, thus forming the continuous chord. This detail is commonly used with wood roof systems.

Advantages: Erection is simple. Accommodates movement due to volume change.

Disadvantages: Additional material and labor costs in forming recessed pocket and forming unbonded length of reinforcement. Welding in up-hand position.

Restraint: Primary: F_2



WALL PANEL TO STEEL COLUMN CONNECTIONS

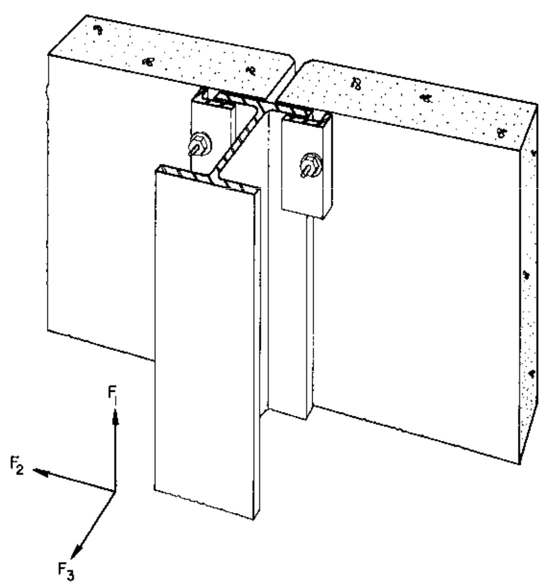
Steel Column/Wall Connection with Bolted Steel Angles

In this connection, each wall panel is connected to a steel column through bolted steel angles. Each steel angle bears against the interior face of a column flange and is bolted to the wall panel through embedded threaded parts.

Advantages: Fabrication and erection is simple. Connection accommodates movement due to volume change. Large tolerances can be specified.

Disadvantages: Special steps may be required to protect exposed threaded parts during handling. Exposed steel may require fireproofing.

Restraint: Primary: F_3



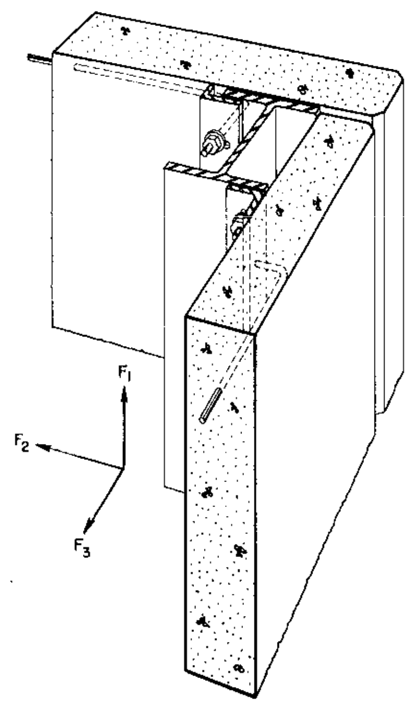
Wall/Steel Column/Wall Connection with Bolted Steel Angles

In this connection, the steel column to wall Connection Detail "Wall Panel to Steel Column Connections" is applied to corner wall panels adjacent to a steel column.

Advantages: Fabrication and erection is simple. Connection accommodates movement due to volume change. Large tolerances can be specified.

Disadvantages: Special steps may be required to protect exposed threaded parts during handling. Exposed steel may require fireproofing.

Restraints: Primary: F_2, F_3



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KEYWORDS: bolts, connections, design, details, ductility, durability, embedments, grout, inserts, load path, restraint, studs, threaded rods, tilt-up, tolerances, welding.

ABSTRACT: A reference for connections commonly used in tilt-up construction with 28 perspective drawings of details between wall panels and foundation, floor, adjacent wall panels, and roof. Details were obtained from engineering offices experienced in tilt-up design and from a review of current literature. Connection elements such as standard bolts, high-strength bolts, coil bolts, threaded inserts, expansion inserts, embedments, welding, dowels, grout, and bearing pads are all discussed.

REFERENCE: *Connections for Tilt-Up Wall Construction* (EB110.01D), Portland Cement Association, 1987.

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