NFBA
NATIONAL FRAME BUILDING ASSOCIATION

POST-FRAME
BUILDING DESIGN MANUAL

Second Edition

NFBA
NATIONAL FRAME BUILDING ASSOCIATION

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Contents

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CHAPTER 1

Introduction to Post-Frame Buildings

Contents

1.1 General

1.1.1 Building Systems

A post-frame building system is one of many types of framing/support systems. In general, a framing/support system is concrete-based, steel-based, wood-based, or a combination of these three. Even though they may contain structural steel or concrete components, post-frame building systems fall under the broad category of wood-based framing systems. From a structural framing perspective, a post-frame building system is analogous to the typical low-rise metal building system. Conventional buildings of both types have two-dimensional primary frames that are connected with secondary framing members, and nomenclature for both building systems is similar. The major difference is that the majority of framing members in a post-frame building are wood-based, and the majority of framing members in a low-rise steel framing building system are steel.

1.1.2 Use

Post-frame buildings are well-suited for many commercial, industrial, agricultural and residential applications. Post-frame buildings offer unique advantages in terms of design and construction flexibility and structural efficiency.

1.2 ANSI/ASABE S618 Definitions

Definitions for post-frame building systems and components are contained in ANSI/ASABE S618 *Post-Frame Building System Nomenclature*. This standard was written to establish uniformity of terminology used in building design, construction, marketing and regulation. All definitions appearing in ANSI/ASABE S618 are repeated here as an introduction to this unique building system.

1.2.1 Building Systems

Post-Frame Building System: A building characterized by primary structural frames of wood posts as columns and trusses or rafters as roof framing. Roof framing is attached to the posts, either directly or indirectly through girders. Posts are embedded in the soil and supported on isolated footings, or are attached to the top of piers, concrete or masonry walls, or slabs-on-grade. Secondary framing members, purlins in the roof and girts in the walls, are attached to the primary framing members to provide lateral support and to transfer sheathing loads, both in-plane and out-of-plane, to the posts and roof framing. See figures 1-1 to 1-3.

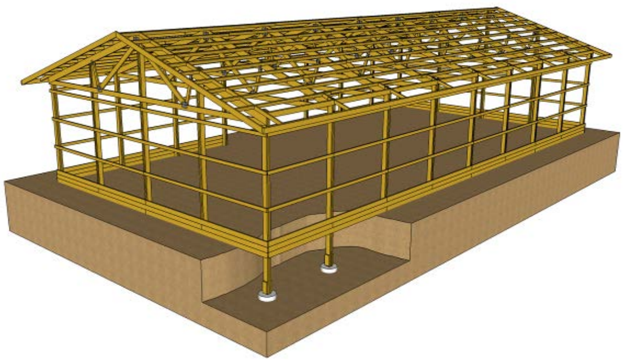

Figure 1-1. Post-frame building with trusses supported directly by embedded posts.

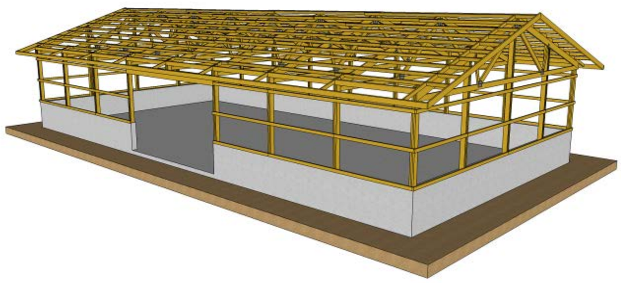

Figure 1-2. Post-frame building mounted on a concrete stem wall.

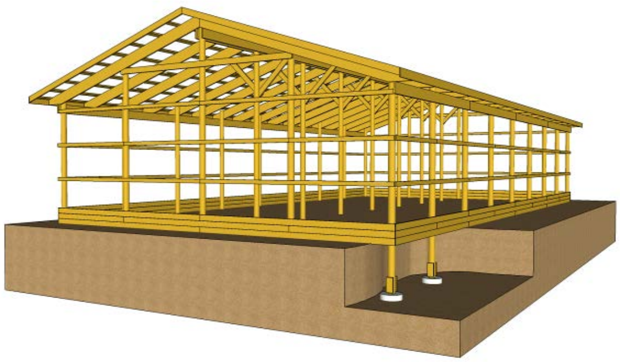

Figure 1-3. Post-frame building featuring girder supported rafters. Since all posts are round poles, this post-frame building could also be identified as a pole building.

Pole-Frame Building System: A post-frame building in which all posts are round poles. Commonly referred to as a *pole building*. See figure 1-3.

1.2.2 Building Subsystems

Primary Frame: The two-dimensional interior frame that is formed by the direct attachment of a roof truss/rafter to its respective posts. Also known as a *post-frame* or a *main frame*. See figures 1-4 to 1-9.

- **Single-Span Primary Frame:** Primary frame without any interior supports. Also known as a clear span primary frame. See figure 1-4.
- **Multi-Span Primary Frame:** Primary frame with one or more interior supports. See figures 1-5 to 1-9.
- **Solid-Web Primary Frame:** Primary frame assembled without using any open-web trusses. See figures 1-6 and 1-8.
- **Open-Web Primary Frame:** Primary frame fabricated with open-web trusses and no solid-web members for roof support. See figures 1-4, 1-5 and 1-7.
- **Hybrid Primary Frame:** Primary frame assembled with both open-web trusses and solid-web members for roof support. See figure 1-9.

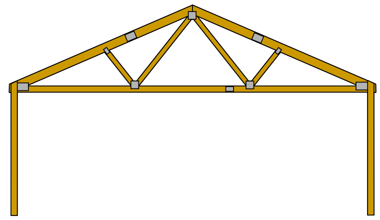

Figure 1-4. A single-span, open-web primary frame.

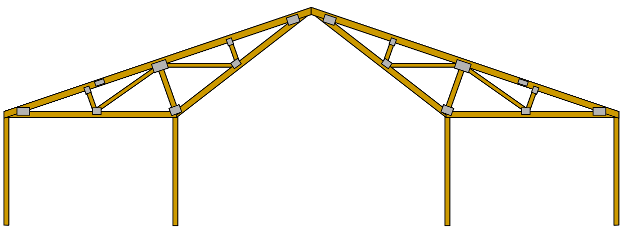

Figure 1-5. A three-span, open-web primary frame featuring twin inverted trusses.

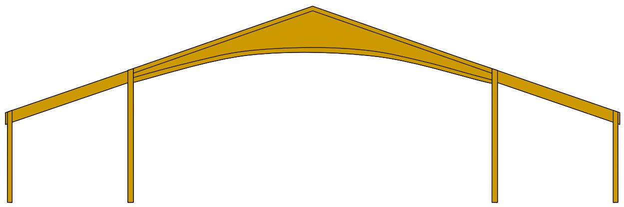

Figure 1-6. A three-span, solid-web primary frame.

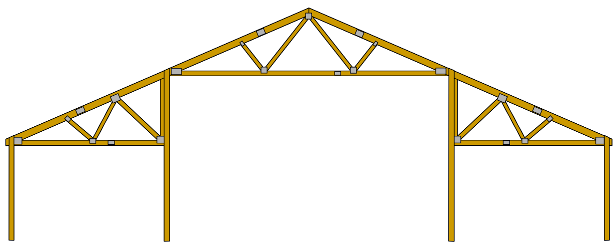

Figure 1-7. A three-span, open-web primary frame featuring a raised center bay.

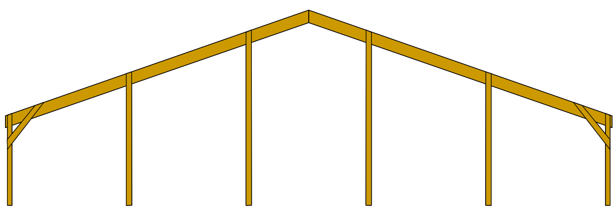

Figure 1-8. A five-span, solid-web primary frame utilizing knee-braces on the sidewall posts. (Knee braces per design requirements, not mandatory.)

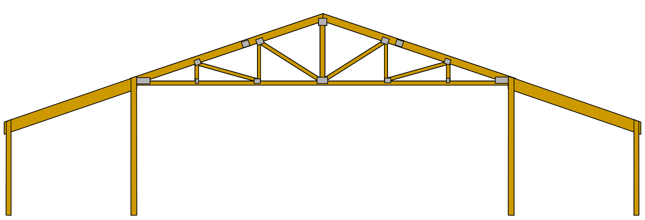

Figure 1-9. A three-span, hybrid primary frame.

Sidewall: An exterior wall oriented perpendicular to individual primary frames.

Endwall: An exterior wall oriented parallel to individual primary frames.

- **Endwall Frame:** Consists of endwall posts and the attached endwall truss or rake rafters.
- **Expandable Endwall:** Endwall frame designed with the load-bearing capability of an interior frame (i.e. primary frame) so it can serve as an interior frame when the building is expanded. See figures 1-1 to 1-3.

Diaphragm: A structural assembly comprised of structural sheathing (e.g., plywood, metal cladding) that is fastened to roof, ceiling, floor or floor framing in such a manner that the entire assembly is capable of transferring in-plane shear forces.

- **Shearwall:** A vertical diaphragm. Any endwall, sidewall, intermediate wall or portion thereof that is capable of transferring in-plane shear forces.

1.2.3 Primary Framing Members

Primary framing members are the main structural framing members in a building. In a post-frame building they include the posts, roof trusses/rafters, and any girders that transfer load between roof trusses/rafters and posts.

Post: A structural wood column. It functions as a major foundation element when it is embedded in the soil. Post-frame building posts include solid-sawn posts, structural composite lumber posts, glulam posts, mechanically-laminated lumber posts, and poles.

- **Solid-Sawn Post:** Post comprised of a single piece of sawn lumber.
- **Structural Composite Lumber Post (SCL Post):** Post comprised of a single piece of structural composite lumber. Structural composite lumber (SCL) includes, but is not limited to: parallel strand lumber (PSL), laminated veneer lumber (LVL), and laminated stand lumber (LSL).
- **Glued-Laminated Post (or Glulam Post):** Post consisting of suitably selected sawn lumber laminations joined with a structural adhesive.
- **Mechanically-Laminated Post (or Mechlam Post):** Post consisting of suitably selected sawn lumber laminations or structural composite lumber (SCL) laminations joined with nails, screws, bolts, and/or other mechanical fasteners.
  - **Nail-Laminated Post (or Nail-Lam Post):** A mechanically laminated post in which only nails have been used to join individual wood layers.
  - **Screw-Laminated Post (or Screw-Lam Post):** A mechanically laminated post in which only screws have been used to join individual wood layers.
  - **Spliced Post:** A mechanically laminated post in which individual laminations are fabricated by end-joining shorter wood members. End joints are generally unreinforced butt joints, mechanically-reinforced butt joints, glued scarf joints, or glued finger joints.
  - **Unspliced Post:** A mechanically laminated post in which individual laminations do not contain end joints.

- **Pole:** A round, naturally tapered, unsawn, wood post. Poles are sometimes slabbed to aid in fastening framing members.
- **Endwall Post:** Post located in an endwall.
- **Sidewall Post:** Post located in a sidewall.
- **Corner Post:** Post that is part of both a sidewall and an endwall.
- **Jamb Post:** Post that frames the side of a door, window, or other framed opening.

Truss: A structural framework, generally two-dimensional (i.e. planar), whose members are almost always assembled to form a series of inter-connected triangles. Perimeter members of the assembly are called truss chords and interior members are called truss webs.

- **Light Wood Truss:** A truss manufactured from wood members whose narrowest dimension is less than 5 nominal inches. Wood members include solid-sawn lumber, structural composite lumber, and glulams. Members may be connected with metal connector plates (MCP), bolts, timber connectors, and screwed- or nailed-on plywood gusset plates.
  - **Metal Plate Connected Wood Truss (MPCWT):** A truss composed of wood members joined with metal connector plates (also know as truss plates). Metal connector plates (MCP) are light-gage, toothed steel plates. The most common type of light wood truss.
- **Heavy Timber Truss:** A truss manufactured from wood members whose narrowest dimension is equal to or greater than 5 nominal inches. Wood members include solid-sawn timber, structural composite lumber, and glulams. Members are generally connected with steel gusset plates that are bolted in place.
- **Ganged Wood Truss:** A truss designed to be installed as an assembly of two or more individual light wood trusses fastened together to act as one.
- **Girder Truss:** Truss designed to carry heavy loads from other structural members framing into it. Frequently a ganged wood truss.
- **Parallel Chord Truss:** Truss with top and bottom chords with equal slopes
- **Roof Truss:** A truss that directly supports a roof.

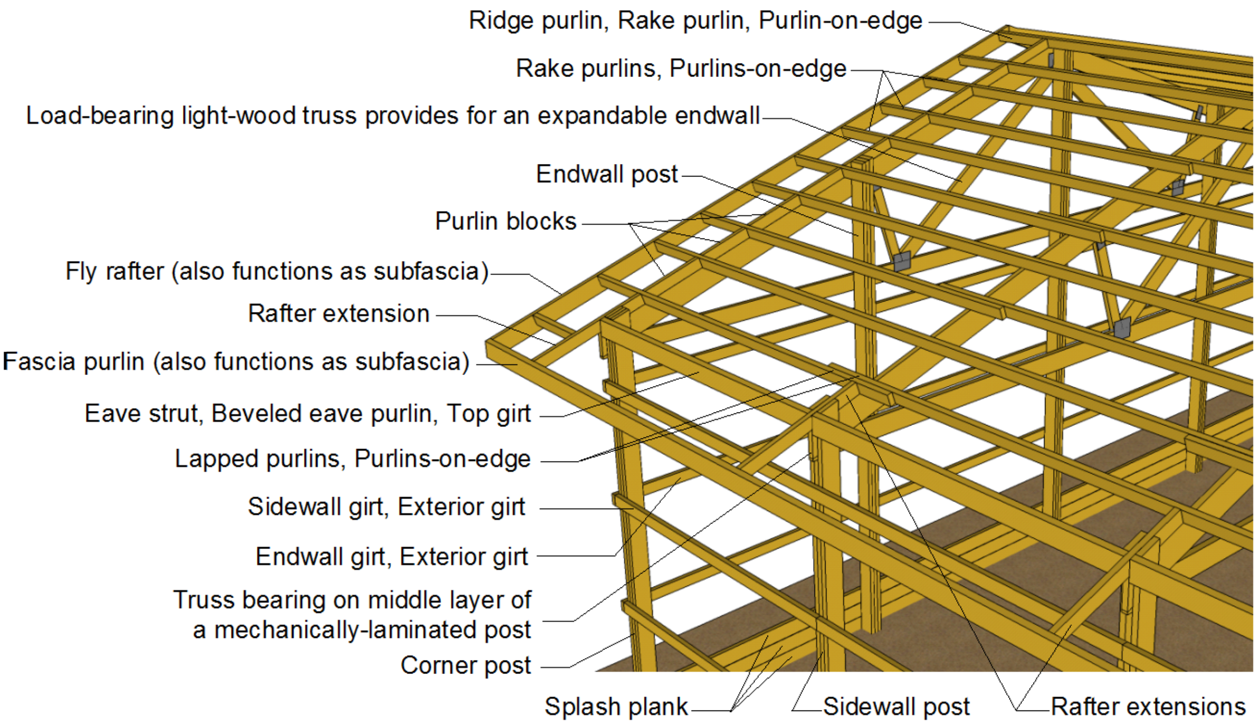

Figure 1-10. Typical corner framing.

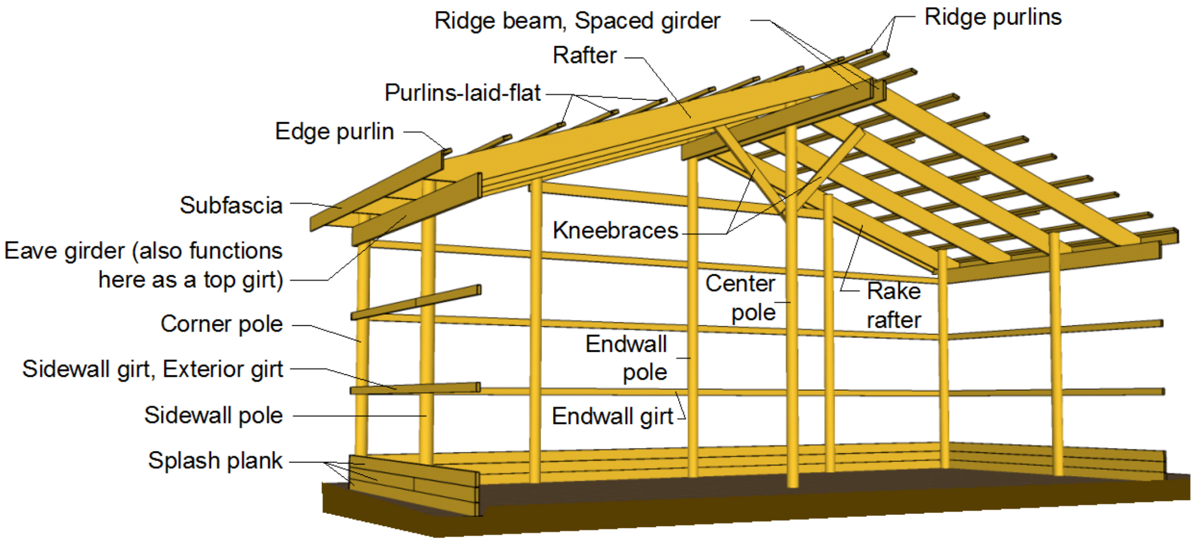

Figure 1-11. Section of a post-frame building featuring girder-supported rafters. Although posts of any type can be used to support girders, this image shows round poles being used as structural wood columns.

Rafter: One of a series of sloped, structural beams that support a roof.

- **Rake Rafter:** A rafter located in an end wall. See figure 1-11.
- **Fly Rafter:** Rafter at the rake overhang that is supported out from the endwall by rake purlins. See figure 1-10.
- **Stacked Rafter:** A narrow, deep rafter made by placing one rafter on top of another and fastening them together. Generally made by fastening dimension lumber together with metal connector plates.

Girder: A large, generally horizontal, beam. Commonly used in post-frame buildings to support trusses whose bearing points do not coincide with a post. Frequently function as headers over large door and window openings.

- **Eave Girder:** Girder located at the eave of a building. See figure 1-11.
- **Ridge Beam:** Girder located at the ridge of a building. See figure 1-11.
- **Truss Girder:** A truss that functions as a girder. Top and bottom chords of a truss girder are generally parallel.
- **Spaced Girder:** A girder composed of two beams that are separated a fixed distance by special spacers and/or the girder supports. See figures 1-11 and 1-12.

Header: Framing member at the top of a window, door or other framed opening. In general, any framing member that ties together the ends of adjacent framing members and may or may not be load bearing. See figure 1-12.

Knee Brace: A diagonally-oriented member used to stiffen and strengthen the connection between a post and the attached roof truss/rafter, or between a post and an attached girder. See figures 1-8 and 1-11.

Bearing Block: A relatively short structural support used to transfer vertical load from one structural member to another. Frequently used to transfer load from a girder to a post or a truss to a post.

Rafter Extension: A framing member attached to the end of a truss or rafter that extends the effective slope length of the roof by supporting additional purlins and/or subfasica. Rafter extensions are commonly used to help form eave overhangs as well as over shot roofs. See figures 1-10, 1-13, 1-14 and 1-15.

Tie-Down Block: A framing member used to attach a roof truss/rafter to a girder. See figure 1-12.

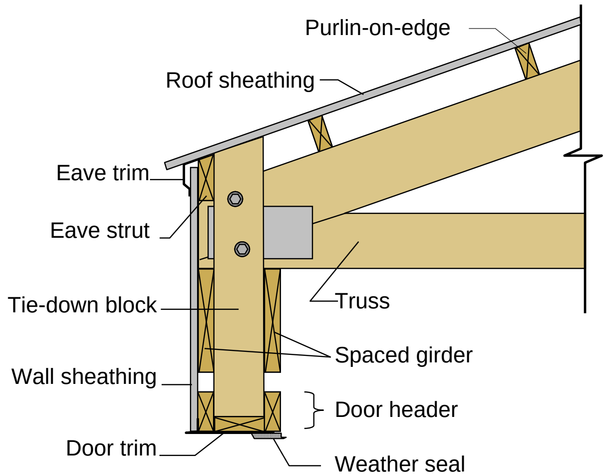

Figure 1-12. Cross-sectional view through an overhead door in the sidewall of a building without an eave overhang.

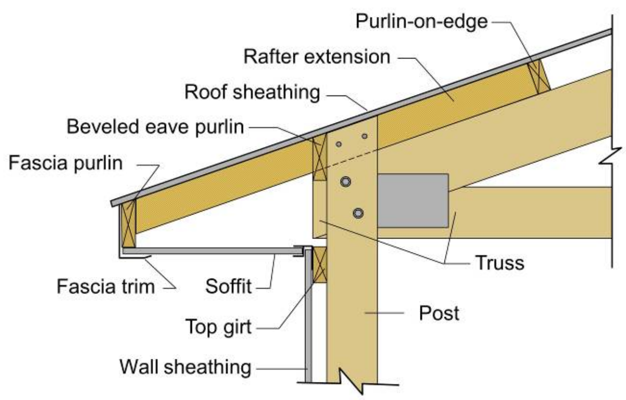

Figure 1-13. Cross-sectional view through an eave overhang that is supported by a rafter extension that runs continuously from the edge purlin/sub-fascia to the first purlin above the post. The rafter extension is fastened to the post, purlin, and top of the truss.

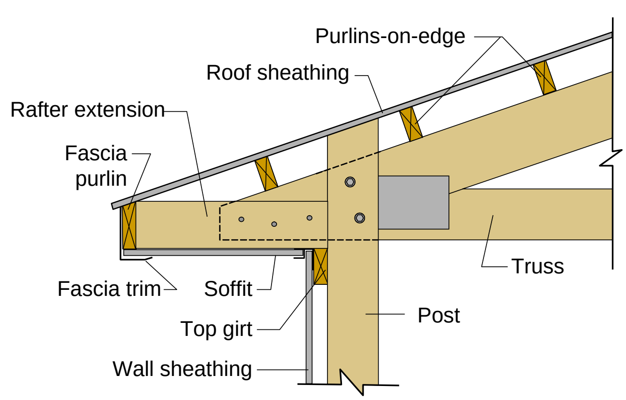

Figure 1-14. Cross-sectional view through an eave overhang that is supported by rafter extensions (one on each side of the truss) that are attached to the tail end of the truss.

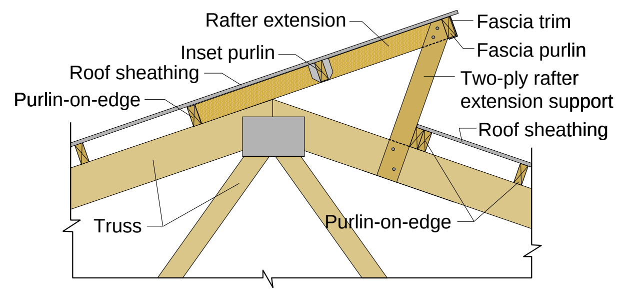

Figure 1-15. Overshot ridge with rafter extension supported by a two-ply rafter extension support. One ply is located between the rafter extension and truss top chord; the second ply extends along the sides of the rafter extension and truss chord.

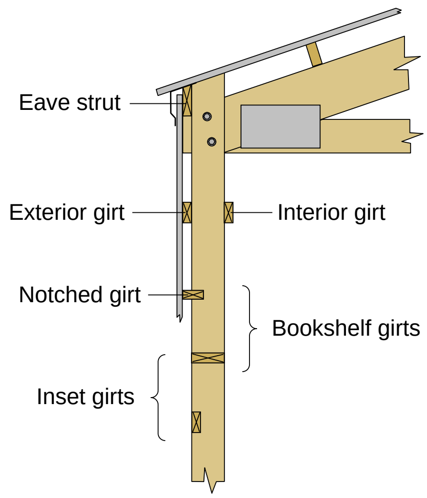

Figure 1-16. Girt types.

1.2.4 Secondary Framing Members

Secondary framing members are structural framing members used to transfer load between exterior sheathing and primary framing members, and/or to laterally brace primary framing members. Secondary framing members in a post-frame building include girts, purlins, eave struts and any structural wood bracing.

Girt: A member attached (typically at a right angle) to posts. Girts laterally support posts and transfer loads between any attached wall sheathing and the posts. See figure 1-16.

- **Exterior Girt:** A girt located entirely on the outside of posts. Also known as an outset girt. See figure 1-16.
- **Inset Girt:** A girt located entirely between adjacent posts. Frequently used to support both exterior and interior wall sheathing and horizontally-placed batt insulation. See figure 1-16.

- **Interior Girt:** A girt located entirely on the inside of posts. Generally used to support interior wall sheathing in buildings with exterior girts. See figure 1-16.
- **Notched Girt:** A girt that is notched to facilitate attachment to a post. Notching places a portion of the girt between adjacent posts, with the remainder located outside or inside the posts. See figure 1-16.
- **Bottom Girt:** The lowest girt. This could be a regular girt, grade girt, or a splash plank. See figures 1-24 and 1-25.
    - **Grade Girt:** A bottom girt located at grade. May also function as a splash plank. See figures 1-22 and 1-24.
- **Splash Plank:** Any decay and corrosion resistant girt that is in soil contact or located near the soil surface, that remains visible from the building exterior upon building completion, and is 2 to 4 inches in nominal thickness. Frequently, multiple rows of tongue and groove (T&G) splash plank are used along the base of a wall. See figures 1-10, 1-11 and 1-24.
- **Top Girt:** The highest girt. A top girt to which both roof and wall sheathing are attached is known as an eave strut. See figures 1-10, 1-11, 1-13 and 1-14.
- **Bookshelf Girt:** A girt with its wide faces horizontally oriented thus enabling it to effectively function as a shelf when left exposed. See figure 1-16.

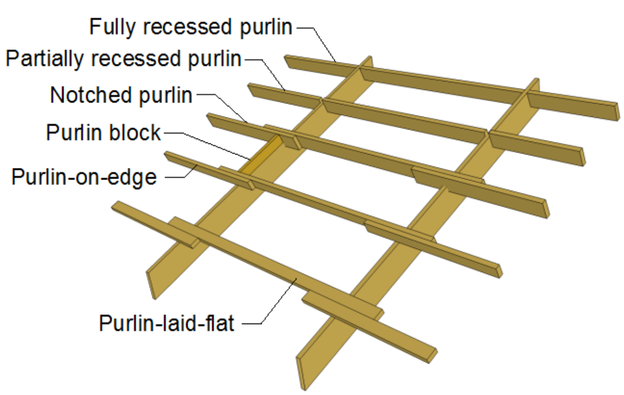

Figure 1-17. Purlin types.

Purlin: A member attached (typically at a right angle) to roof trusses/rafters. Purlins laterally support trusses/rafters and transfer load between roof sheathing and roof trusses/rafters. See figures 1-10, 1-11 and 1-17.

- **Purlin-on-Edge:** A purlin that rests on top of roof trusses/rafters with its narrow face in contact with the trusses/rafters. See figures 1-10 and 1-17.
- **Purlin-Laid-Flat:** A purlin that rests on top of roof trusses/rafters with its wide face in contact with the trusses/rafters. See figures 1-11 and 1-17.
- **Recessed Purlin:** A purlin located entirely between adjacent trusses/rafters. Single-span components that are typically held in place with special metal hangers. Also known as an inset purlin or dropped purlin. See figure 1-17.
    - **Fully Recessed Purlin:** Recessed purlin whose top edge aligns with or is below the top edge of the trusses/rafters to which it is connected. See figure 1-17.
    - **Partially Recessed Purlin:** Recessed purlin whose top edge is above the top edge of the trusses/rafters to which it is connected. See figure 1-17.
- **Notched Purlin:** A purlin that is notched to fit over roof trusses/rafters. See figure 1-17.
- **Lapped Purlins:** Two non-recessed purlins (i.e., purlins-on-edge, purlins-laid-flat, or notched purlins) that bypass each other where they are connected to the same truss/rafter. See figures 1-10 and 1-17.
- **Rake Purlin:** A purlin that overhangs the endwall of a building. See figure 1-10.
- **Ridge Purlin:** A purlin adjacent to the building ridge. See figures 1-10 and 1-11.
- **Eave Purlin:** A purlin located at the eave line of a building. An eave purlin to which both wall and roof sheathing are attached is known as an eave strut. See figure 1-13
- **Fascia Purlin:** A purlin that helps form the fascia of a building. Also known as an edge purlin. See figures 1-13 and 1-14.
- **Edge Purlin:** A purlin in the most outer row of purlins. All fascia purlins are edge purlins but not all edge purlins are fascia purlins. The edge purlins shown in figure 1-11 are not fascia purlins as they do not help form the fascia of the building.
- **Beveled Purlin:** A purlin with an edge that has been cut at an angle, generally to facilitate cladding attachment. See figures 1-12, 1-13 and 1-14.

Eave strut: An eave purlin to which both wall and roof sheathing are attached or a top girt to which both wall and roof sheathing are attached. Simultaneous attachment of an eave strut to both wall and roof sheathing generally provides the strut with effective continuous lateral support to resist bending about both primary axes. See figures 1-12 and 1-16.

Base Plate: A corrosion and decay resistant member that is attached to the top of a concrete floor or wall. A base plate is generally located between posts and may function like a bottom girt. Unlike a girt, primary attachment of a base plate is to the concrete and not the posts. See figures 1-21 and 1-25.

Sill Plate: A corrosion and decay resistant member that is attached to the top of a concrete foundation wall, and upon which posts are attached.

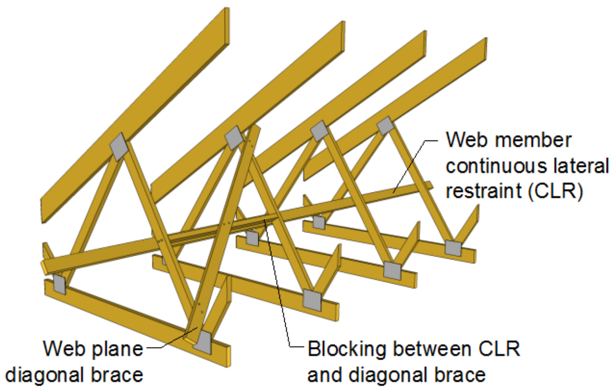

Figure 1-18. Components of a continuous lateral restraint system for web members. For larger truss spacing, individual web member reinforcement may be more economical for lateral bracing of webs than a continuous lateral restraint system.

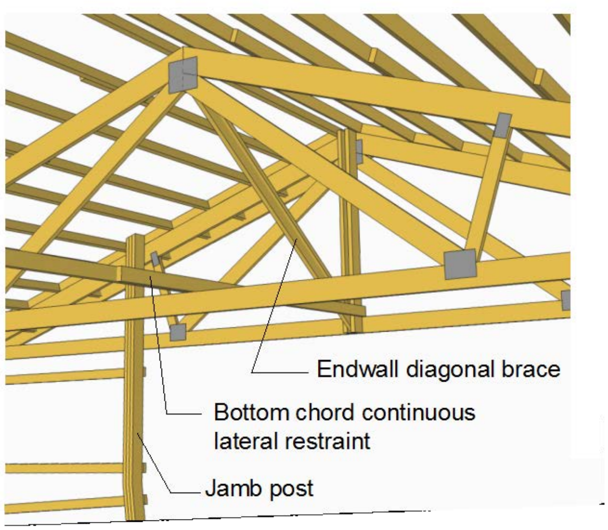

Figure 1-19. Endwall diagonal brace used to transfer endwall forces into roof diaphragm and to keep bottom chord continuous lateral restraint from shifting.

Bracing: Axially-loaded structural members used to help stabilize other structural components. The definitions in this section pertain to permanent bracing. Additional temporary bracing is generally required during construction

- **Continuous Lateral Restraint (CLR):** An uninterrupted row of structural framing members connecting a series of trusses. The row is perpendicular to truss members and thus provides lateral support to the truss members it connects. See figures 1-18 and 1-19.
  - **Bottom Chord Continuous Lateral Restraint:** A row of structural framing members that provides lateral support to the bottom chords of adjacent trusses. See figure 1-19.
  - **Web Member Continuous Lateral Restraint:** A row of structural framing members that provides lateral support to the web members of adjacent trusses. See figure 1-18.
- **Diagonal Brace.** A framing member that runs at an angle to other framing members, and with other framing members generally forms a structurally-stable triangular assembly.
  - **Web plane diagonal brace:** A diagonal brace that lies in the plane formed by the web members of adjacent trusses. The brace generally runs from the roof plane to the ceiling plane, and is required in truss web planes that contain continuous lateral restraints to keep the CLR from shifting. See figure 1-18.
  - **Bottom chord diagonal brace:** A diagonal brace that lies in the plane formed by the bottom chords of adjacent trusses (a.k.a.., the ceiling plane). The braces are used to prevent bottom chord continuous lateral restraints from shifting.
  - **X-brace:** A pair of diagonal braces that cross each other thus forming an “X”. Generally, one brace will be in axial tension while the other brace is loaded in axial compression.
  - **V-brace:** A pair of diagonal braces that meet at one of their ends, thus forming a “V”. Generally, one brace will be in axial tension while the other brace is loaded in axial compression.
  - **Endwall diagonal brace:** A framing member used to transfer load from an endwall to the roof plane. Generally used above large endwall openings or where an endwall post is not continuous from grade to the rake (e.g., an endpost is terminated near the bottom chord of an endwall truss). See figure 1-19.

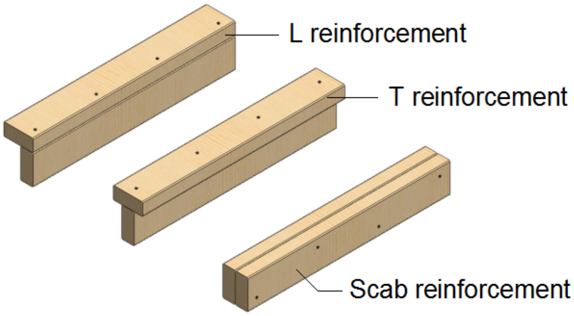

Figure 1-20. Types of individual member reinforcement.

Bracing for Individual Members. The buckling resistance of an individual framing member is often increased by attaching a T-, L-, or scab reinforcement to the side of the member. See figure 1-20.

- **T-Reinforcement:** A member that is attached to a structural framing member such that the cross-section of the two adjoined members forms a tee. See figure 1-20.
- **L-Reinforcement:** A member that is attached to a structural framing member such that the cross-section of the two adjoined members forms an el. See figure 1-20.
- **Scab Reinforcement:** A member whose wide face is attached to the wide face of a structural framing member. See figure 1-20.

Compression-Edge Brace: A brace used to provide lateral support to the compressive edge of a beam or column. More commonly referred to as a flange brace when used to support the compressive edge of an I-shaped section.

Purlin Block: A member placed between purlins to help transfer load from roof sheathing to roof framing, to reduce purlin roll, and/or to eliminate bird perch points. See figures 1-10 and 1-17.

Sub-Fascia: A structural member located under the fascia or eave/fascia trim. In a building with overhangs, the edge purlins and fly rafters generally function as subfasica. In a building without overhangs, the eave strut and rake rafters generally function as sub-fascia. See figures 1-10 and 1-11.

Lookout: A short member in an eave overhang that connects the sub-fascia and wall. Generally used to support soffit. Unlike a rafter extension, a lookout is not used to structurally support purlins or eave sub-fascia.

Track Board: A member to which a sliding door track is directly attached.

Track Board Support: A structural framing member that is used to support a track board.

1.2.5 Diaphragm Components

When post-frame building components (e.g., purlins, girts, purlin blocks, mechanical fasteners, etc.) are positioned and connected in such a way to form a diaphragm (see diaphragm definition in 1.2.2), these components take on additional names as defined in this section.

Diaphragm Structural Framing: Primary and secondary framing members to which structural sheathing panels are attached to form a diaphragm assembly.

Structural Sheathing: Frame coverings that are selected in part for their ability to absorb and transfer structural loads. Common structural sheathings include plywood, oriented strand board, and corrugated (a.k.a. ribbed) steel.

- **Structural Sheathing Panel:** An individual piece of structural sheathing.

Edge Fastener: A sheathing-to-framing connector that is located along the sides or ends of a structural sheathing panel.

Field Fastener: A sheathing-to-framing connector that is not located along the sides or ends of a structural sheathing panel.

Seam (or Stitch) Fastener: An edge fastener that connects two structural sheathing panels thereby adding in-plane shear continuity between the panels.

- **Anchored Seam Fastener:** A seam fastener that penetrates the underlying structural framing a sufficient amount so as to significantly affect the shear characteristics of the connection.

Shear Blocks: Short framing members used to help transfer shear force into or out of the structural sheathing of a diaphragm. For roof diaphragms, properly connected purlin blocks function as shear blocks.

Diaphragm Chords: Diaphragm structural framing members that run perpendicular to the applied load, and thus are subjected to axial forces when the load works to bend the diaphragm.

Drag Strut: A member, typically horizontal, that transfers shear from a floor, roof or ceiling diaphragm to a shear wall.

Structural Ridge Cap: A component that covers the ridge of a building and is capable of transferring shear force between diaphragms located on opposite sides of the ridge.

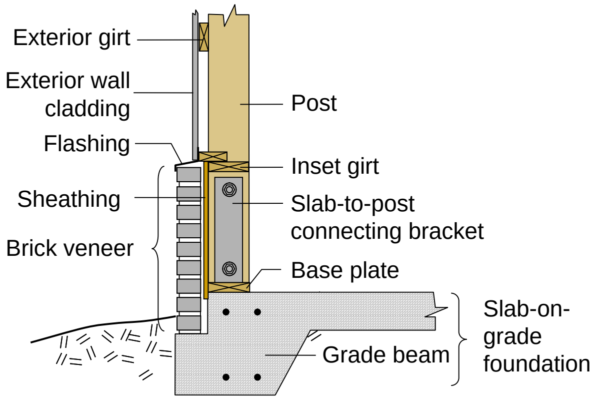

Figure 1-21. Slab-on-grade foundation.

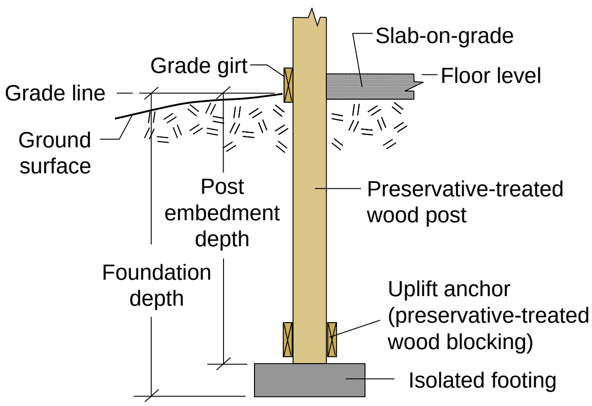

Figure 1-22. Post foundation featuring a preservative-treated wood blocks for uplift anchorage.

1.2.6 Foundation Components

This section contains descriptions of foundation components that are used to define foundation types in Section 1.2.7.

Embedded Pier: A relatively short column embedded in the soil to provide support for an above-grade post, beam, wall, or other structure. Piers include members of any material with assigned structural properties such as solid or laminated wood, steel, or concrete. Embedded piers differ from embedded posts in that they seldom extend above the lowest horizontal framing element in a structure, and when they do, it is often nor more than a foot. See figure 1-24.

Footing: Foundation component at the base of a post, pier or wall that provides resistance to vertical downward forces. When a footing is located below grade and properly attached to a post, pier or wall, it aids in the resistance of lateral and vertical uplift forces. See figures 1-22 to 1-25.

Uplift Anchor: Any element mechanically attached to an embedded post or pier to increase the uplift resistance of the foundation. Common uplift anchors include concrete footings, concrete collars, preservative-treated wood blocks, steel angles, and concrete backfill. See figures 1-22 to 1-24.

Collar: Foundation component attached below grade to an embedded post or pier, and that moves with it to resist lateral and vertical loads. See figure 1-23.

Grade Beam: A corrosion and decay resistant beam located on the soil surface. Also a long, thickened, and more heavily-reinforced portion of a slab-on-grade foundation. See figure 1-21.

1.2.7 Foundations Types

This section defines foundation types that are commonly used to support post-frame building systems.

Post Foundation: A foundation consisting of an embedded post and all attached below-grade elements, which may include a footing, uplift resistance system, and collar. See figures 1-22 and 1-23.

Pier Foundation: A foundation consisting of an embedded pier and all attached below-grade elements, which may include a footing, uplift resistance system, and collar. See figure 1-24.

- **Pier and Beam Foundation:** A pier foundation that supports a grade beam.

Slab-On-Grade Foundation: A reinforced concrete slab that rests on the soil surface. Slab areas located directly beneath structural columns or walls are generally thicker and more heavily reinforced. Long, thickened and reinforced portions are generally referred to as grade beams. See figure 1-21.

Stem Wall Foundation: A foundation consisting of a continuous wall that may be placed on a continuous footing. The base of the foundation is generally located below expected frost penetration depths. See figure 1-25.

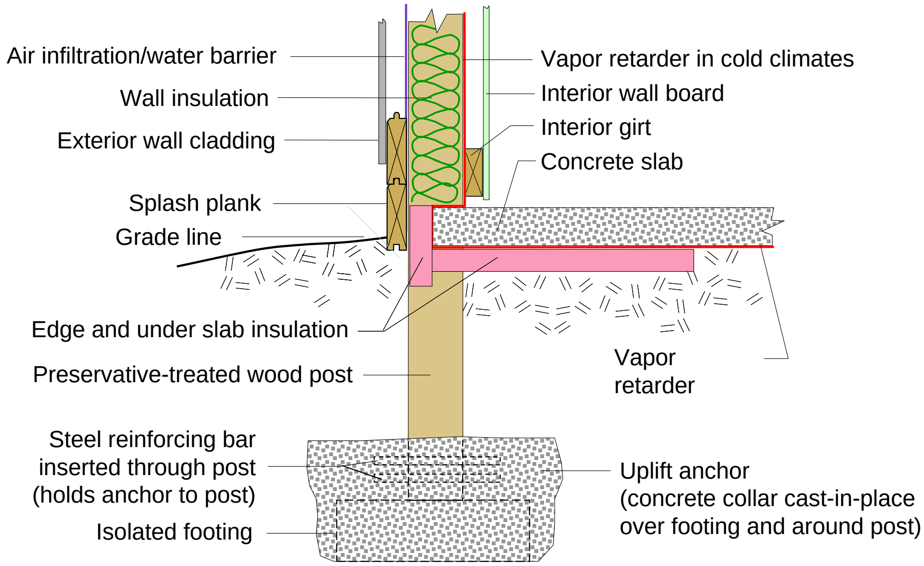

Figure 1-23. Post foundation featuring a cast-in-place concrete collar for uplift anchorage and increased lateral resistance. Concrete collar need not surround footing (as shown above) to be effective in resisting uplift.

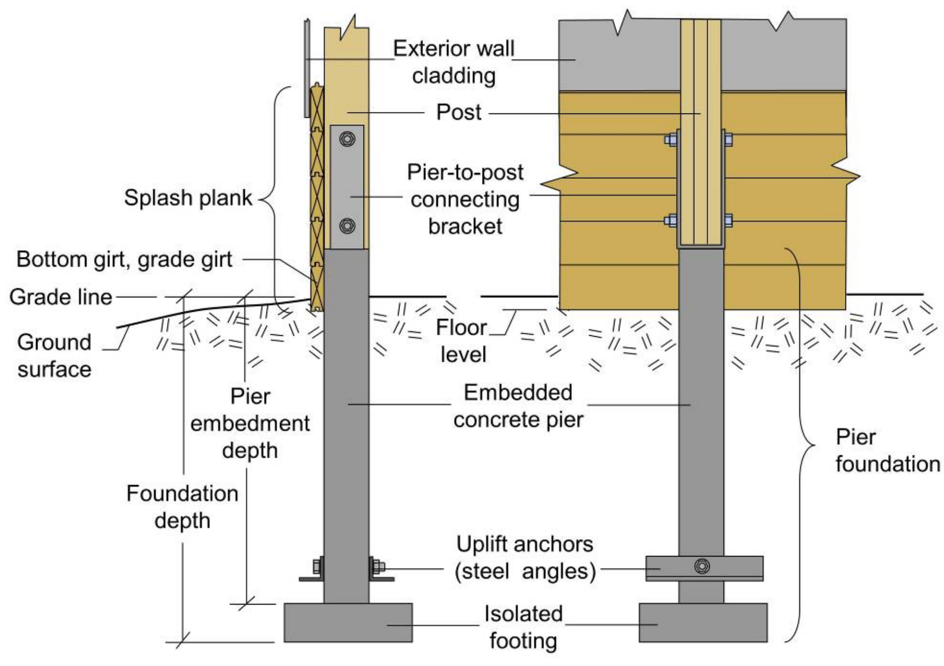

Figure 1-24. Pier foundation featuring steel angles for uplift anchorage.

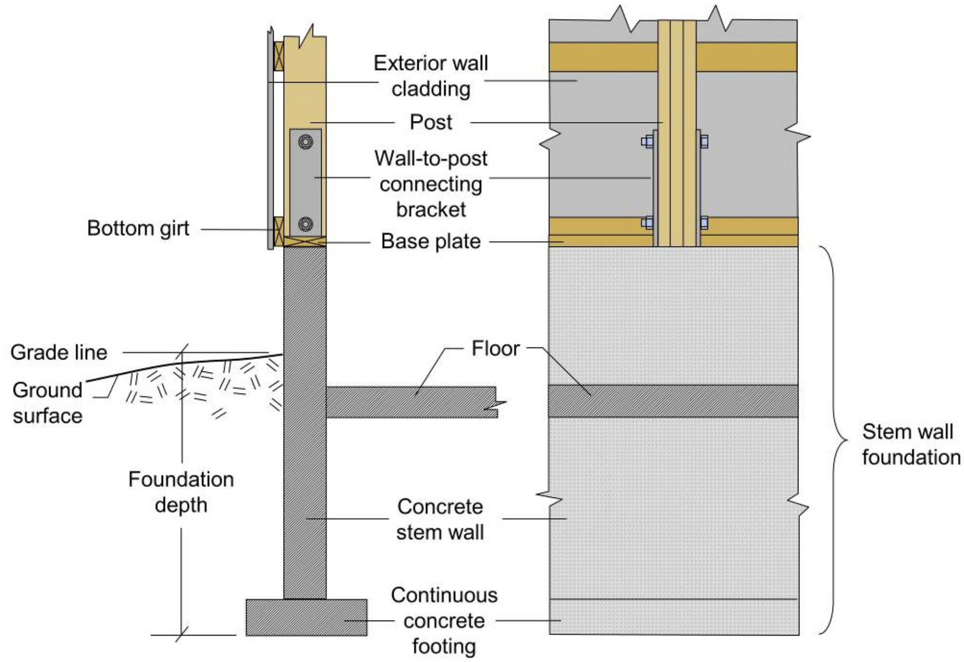

Figure 1-25. Stem wall foundation.

1.2.8 Dimensions

Grade Line (Grade Level): The line of intersection between the building exterior and the finished ground surface and/or top of the pavement in contact with the building exterior. See figures 1-22 to 1-25.

Floor Level: Elevation of the finished floor surface. In the absence of a finished floor, the floor level is taken as the elevation of the bottom edge of the bottom girt. In buildings with stemwall foundations and no finished floor, floor level is taken as the elevation of the unfinished floor. See figure 1-22.

Eave Line: Line formed by the intersection of the plane formed by the top edge of the purlins and the plane formed by the outside edge of the sidewall girts.

Rake Line: Line formed by the intersection of the plane formed by the top edge of the purlins and the plane formed by the outside edge of the endwall girts.

Ridge Line: Line formed by the intersection of the plane formed by the top edge of the purlins on one side of the roof and the plane formed by the top edge of the purlins on the opposite side of the roof. For a mono-slope roof, the ridge line is the line formed by the intersection of the plane formed by the top edge of the purlins and the plane formed by the outside edge of the girt in the tallest sidewall.

Foundation Depth: Vertical distance from the grade line to the bottom of the foundation. Typically the vertical distance from the ground surface to the base of the footing. (a.k.a., foundation embedment depth). See figures 1-22 to 1-25.

Post Embedment Depth: Vertical distance from the grade line to the bottom of an embedded post. Equal to the foundation depth when the post does not bear on a footing or other foundation element. See figure 1-22.

Pier Embedment Depth: Vertical distance from the grade line to the bottom of a pier. Equal to the foundation depth when the footing is part of the pier (i.e., the footing is cast integrally with the pier). See figure 1-24.

Clear Height: Vertical distance between the finished floor and the lowest part of a truss, rafter, or girder.

Post Height: The length of the non-embedded portion of a post.

Eave Height: Vertical distance between the floor level and the eave line.

Building Height: Vertical distance between the floor level and the ridge line. (a.k.a., ridge height).

Building Bay: The area between adjacent post-frames.

Frame Spacing: On-center horizontal spacing of primary frames. Frame spacing may vary within a building. (a.k.a., bay width).

Clear Span: Horizontal distance from the face of one support to the face of the opposite support.

Building Width: Horizontal distance between the outside face of the girts in one sidewall and the outside face of the girts in the opposite sidewall.

Building Length: Horizontal distance between the outside face of the girts in one endwall and the outside face of the girts in the opposite endwall.

Eave Overhang Distance: Horizontal distance from the eave line to the outside of the subfacia.

Rake Overhang Distance: Horizontal distance from the rake line to the outside of the fly rafter.

Girt Spacing: On-center vertical spacing of girts.

Purlin Spacing: On-center spacing of purlins.

1.3 General Building Terminology

The following terms and abbreviations are not specific to post-frame buildings, and thus are defined outside of ASABE S618 *Post-Frame Building System Nomenclature*.

AF&PA: American Forest & Paper Association (formerly National Forest Products Association).

AITC: American Institute of Timber Construction.

ALSC: American Lumber Standard Committee.

ANSI: American National Standards Institute

APA: The Engineered Wood Association (formerly the American Plywood Association)

ASABE: The American Society of Agricultural and Biological Engineers.

ASCE: American Society of Civil Engineers.

ASD: Allowable Stress Design

AWC: American Wood Council. The wood products division of the American Forest & Paper Association (AF&PA).

AWPB: American Wood Preservers Bureau.

Bearing Point: The point at which a component is supported.

Board: Wood member less than two (2) nominal inches in thickness and one (1) or more nominal inches in width.

Board-Foot (BF): A measure of lumber volume based on nominal dimensions. To calculate the number of board-feet in a piece of lumber, multiply nominal width in inches by nominal thickness in inches by length in feet and divide by 12.

Butt Joint: The interface at which the ends of two members meet in a square cut joint.

Camber: A predetermined curvature designed into a structural member to offset the anticipated deflection when loads are applied.

Check: Separation of the wood that usually extends across growth rings (i.e., a split perpendicular-to-growth rings). Commonly results from stresses that build up in wood during seasoning.

Cladding: The exterior and interior coverings fastened to framing.

Components and Cladding: Elements of the building envelope that do not qualify as part of the main wind-force resisting system as defined in ASCE/SEI 7. In post-frame buildings, this generally includes individual purlins and girts, and cladding.

Diaphragm Action: The transfer of load by a diaphragm.

Diaphragm Design: Design of roof and ceiling diaphragm(s), wall diaphragms (shearwalls), primary and secondary framing members, component connections, and foundation anchorages for the purpose of transferring lateral (e.g., wind) loads to the foundation structure via diaphragm action.

Dimension Lumber: Wood members from two (2) nominal inches to but not including five (5) nominal inches in thickness, and 2 or more nominal inches in width.

Eave: The part of a roof that projects over the sidewalls. In the absence of an overhang, the eave is the line along the sidewall formed by the intersection of the wall and roof planes.

Fascia: Flat surface (or covering) located at the outer end of a roof overhang or cantilever end.

Flashing: Sheet metal or plastic components used at major breaks and/or openings in walls and roofs to insure weather-tightness in a structure.

Gable: Triangular portion of the endwall of a building directly under the sloping roof and above the eave line.

Gable Roof: Roof with one slope on each side. Each slope is of equal pitch.

Gambrel Roof: Roof with two slopes on each side. The pitch of the lower slope is greater than that of the upper slope.

Hip Roof: Roof which rises by inclined planes from all four sides of a building.

IBC: International Building Code.

ICC: International Code Council.

Laminated Strand Lumber (LSL): A structural composite lumber (SCL) assembly comprised of wood strands bonded with resins under heat and pressure. Strand fibers are primarily oriented along the length of the member. The least dimension of the strands shall not exceed 0.10 in. (2.54 mm) and the average length shall be a minimum of 150 times the least dimension.

Laminated Veneer Lumber (LVL): A structural composite lumber (SCL) assembly manufactured by gluing together wood veneer sheets. Each veneer is oriented with its wood fibers parallel to the length of the member. Individual veneer thickness does not exceed 0.25 inches.

Loads: Forces or other actions that arise on structural systems from the weight of all permanent construction, occupants and their possessions, environmental effects, differential settlement, and restrained dimensional changes.

- **Dead Loads:** Forces induced by the gravitational attraction between the earth and the mass of the building components.
- **Live Loads:** Loads resulting from the use and occupancy of a building.
- **Seismic Load:** Forces induced in a structure due to the horizontal acceleration and deacceleration of the building foundation during an earthquake.
- **Snow Load:** Forces induced by the gravitation attraction between the earth and any snow that accumulates on the building.
- **Wind Loads:** Loads caused by the wind blowing from any direction.

Lumber Grade: The classification of lumber in regard to strength and utility in accordance with the grading rules of an approved (ALSC accredited) lumber grading agency.

LRFD: Load and Resistance Factor Design

LVL: see Laminated Veneer Lumber.

Main Wind-Force Resisting System: An assemblage of structural elements assigned to provide support and stability for the overall structure. Main wind-force resisting systems in post-frame buildings include the individual post-frames, diaphragms and shearwalls.

Manufactured Component: A component that is assembled in a manufacturing facility. The wood trusses and laminated columns used in post-frame buildings are generally manufactured components.

MBMA: Metal Building Manufacturers Association.

NDS®: ANSI/AWC NDS®-2012 *ASD/LRFD National Design Specification® for Wood Construction*. American Wood Council, Leesburg, VA www.awc.org

Metal Cladding: Metal exterior and interior coverings, usually cold-formed aluminum or steel sheet, fastened to the structural framing.

NFBA: National Frame Building Association.

NFPA: National Fire Protection Association.

Nominal Size: The named size of a member, usually different than its actual size (as with lumber).

Oriented Strand Board (OSB): Structural wood panels manufactured from reconstituted, mechanically oriented wood strands bonded with resins under heat and pressure.

Oriented Strand Lumber (OSL): A structural composite lumber (SCL) assembly comprised of wood strands bonded with resins under heat and pressure. Strand fibers are primarily oriented along the length of the member. The least dimension of the strands shall not exceed 0.10 in. (2.54 mm) and the average length shall be a minimum of 75 times the least dimension.

OSB: See Oriented Strand Board.

Parallel Strand Lumber (PSL): Structural composite lumber (SCL) manufactured by cutting 1/8-1/10 inch thick wood veneers into narrow wood strands, and then gluing and pressing the strands together. Individual strands are up to 8 feet in length. Prior to pressing, strands are oriented so that they are parallel to the length of the member.

Pennyweight: A measure of nail length, abbreviated by the letter d.

Plywood: A wood panel comprised of wood veneers. The grain orientation of adjacent veneers are typically 90 degrees to each other.

Pressure Preservative Treated (PPT) Wood: Wood pressure-impregnated with an approved preservative chemical under approved treatment and quality control procedures.

PSL: See Parallel Strand Lumber.

Rake: The part of a roof that projects over the endwalls. In the absence of an overhang, the rake is the line along the endwall formed by the intersection of the wall and roof planes.

Ridge: Highest point on the roof of a building which describes a horizontal line running the length of the building.

Ring Shank Nail: See threaded nail.

Roof Overhang: Roof extension beyond the endwall/sidewall of a building.

Roof Slope: The angle that a roof surface makes with the horizontal. Usually expressed in units of vertical rise to 12 units of horizontal run.

Self-Drilling Screw: A screw fastener that combines the functions of drilling and tapping (thread forming). Generally used when one or more of the components to be fastened is metal with a thickness greater than 0.03 inches

Self-Piercing Screw: A self-tapping (thread forming) screw fastener that does not require a pre-drilled hole. Differs from a self-drilling screw in that no material is removed during screw installation. Used to connect light-gage metal, wood, gypsum wallboard and other "soft" materials.

SFPA: Southern Forest Products Association

Shake: Separation of annual growth rings in wood (splitting parallel-to-growth rings). Usually considered to have occurred in the standing tree or during felling.

SIP: Structural Insulated Panel.

Siphon Break: A small groove to arrest the capillary action of two adjacent surfaces.

Soffit: The underside covering of roof overhangs.

SPIB: Southern Pine Inspection Bureau.

Structural Composite Lumber (SCL): Reconstituted wood products comprised of several laminations or wood strands held together with an adhesive, with fibers primarily oriented along the length of the member. Examples include LVL and PSL.

Threaded Nail: A type of nail with either annual or helical threads in the shank. Threaded nails generally are made from hardened steel and have smaller diameters than common nails of similar length.

Timber: Wood members five or more nominal inches in the least dimension.

TPI: Truss Plate Institute.

Wane: Bark, or lack of wood from any cause, on the edge or corner of a piece.

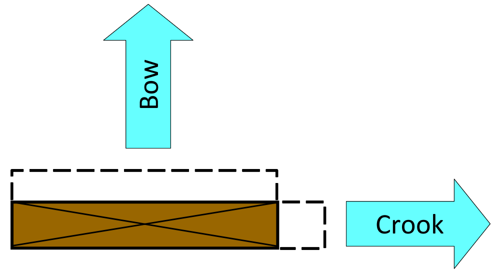

Figure 1-26. Lumber warping terminology: bow versus crook as viewed from end of wood member.

Warp: Any variation from a true plane surface. Warp includes bow, crook, cup, and twist, or any combination thereof.

- **Bow:** Deviation, in a direction perpendicular to the wide face, from a straight line drawn between the ends of a piece of lumber. See figure 1-26.
- **Crook:** Deviation, in a direction perpendicular to the narrow edge, from a straight line drawn between the ends of a piece of lumber. See figure 1-26.
- **Cup:** Deviation, in the wide face of a piece of lumber, from a straight line drawn from edge to edge of the piece.
- **Twist:** A curl or spiral of a piece of lumber along its length. Measured by laying lumber on a flat surface such that three corners contact the surface. The amount of twist is equal to the distance between the flat surface and the corner not contacting the surface.

WCLIB: West Coast Lumber Inspection Bureau

WTCA: Wood Truss Council of America.

WWPA: Western Wood Products Association.

1.3.1 Heavy Timber Construction

Post-frame buildings are frequently and incorrectly referred to as post and beam buildings or as timber frame buildings. Much of the confusion between the framing systems occurs because they are all generally designed around two-dimensional (2-D) frames. In post-frame buildings these 2-D frames are referred to as primary frames, post-frames or main frames. In post and beam buildings and timber frame buildings they are commonly referred to as bents. The key to understanding the difference between the three building systems is to focus on these 2-D frames. If the main member(s) connecting the posts within a 2-D frame fall into the timber category (timbers are defined as members larger than 5 nominal inches in the least dimension), the building would be classified as a post and beam building or a timber frame building.

According to the Timber Frame Business Council or TFBC (http://timberframe.org/faq.html), a timber framed building is a specialized version of post and beam building that utilizes wood joinery such as mortise and tenon, held in place with wooden pegs as shown in figure 1-27. According to this definition, it is not proper to refer to a post and beam building as a timber-frame building when timbers are connected with special metal fasteners, metal plates and other metal connectors.

Figure 1-27. Timber-frame building designed by Steven Knox and built by Connolly & Company, Edgecomb, Maine. Image from http://stknox.com/images/StudioBarnFrame.JPG.

1.4 History

A condensed history of the post-frame building system follows. Early history is based on an accounting provided by James T. Knight (1989) who served as executive director of the National Frame Building Association (NFBA) for nearly three decades.

1.4.1 Ancient History

The concept of pole-type structures is not new. Archeological evidence exists in abundance that pole buildings have been used for human housing for thousands of years in many areas of the world. Although ancient pole buildings have long since disappeared from the landscape, their size and original location are easily determined by the variations in soil color (i.e. soil staining) that occurs when embedded wood poles decay inside the surrounding soil.

1.4.2 Pole Buildings at Jamestown, VA

The very first buildings constructed by English settlers in North America were pole buildings constructed inside the James Fort palisade (i.e. fort wall) at Jamestown, VA. These buildings were erected within a few weeks of the settlers May 13, 1607 arrival on James Island. In an effort to duplicate methods used in construction of these buildings, the barrack's frame shown in Figures 1-28 and 1-29 was erected on the site in 2011. Appearing behind the barrack in figures 1-28 and 1-29 is a replicate section of the palisade which was constructed by embedding wood poles (i.e., pales) alongside each other.

Figure 1-28. Frame of pole building barrack erected in 2011 in an effort to duplicate construction methods used during the colonization of Jamestown. Located behind the replicate barrack is a replicate palisade.

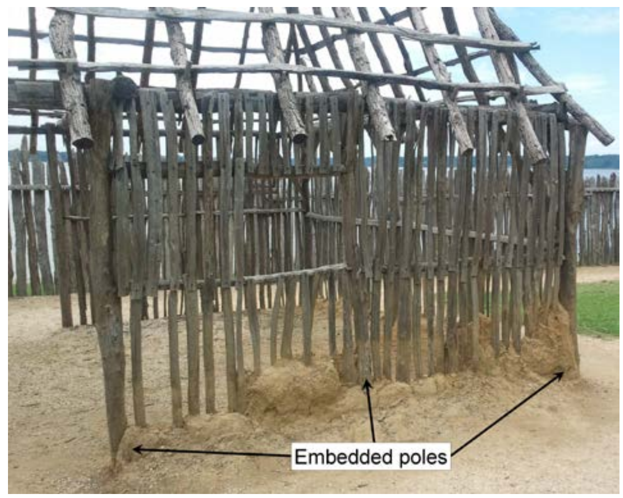

Figure 1-29. Close-up of the barrack shown in figure 1-28. Embedded poles support a girder which in turn supports rafters. Note the similarities between this construction and that shown in figure 1-3.

The most significant of the pole buildings located in James Fort was the church erected in 1608. Although this was the second church built at the site (the first burnt down soon after construction), it is considered the first major Protestant church building in North America. It is famous for housing the wedding of Pocahontas and tobacco planter John Rolfe during the spring of 1614. The exact location of the church was discovered in 2010 by archeologists who uncovered its postholes (Kelso, 2011). Overall building size was found to exactly match the 24 x 60 foot size recorded in 1610 by William Strachey who was the secretary of the colony. Postholes were located exactly 12-foot on center. Their locations are now permanently marked at the site with stub posts (figure 1-30).

Figure 1-30. Stub posts mark the location of the fourteen poles that supported the 1608 church located inside James Fort. A statue of Captain John Smith and the James River appear to the left.

Walls of the Jamestown pole buildings were finished by attaching vertical wood slats to girts (see figure 1-29) and then packing clay around the wood poles, girts and vertical slats. The clay surface was then waterproofed with a mixture of lime and animal fat. Roofs were thatched with marsh grass harvested from surrounding swamps.

It is clear that the English settlers used pole buildings for the same basic reasons we use post-frame buildings today – they can be quickly erected, and they make efficient use of readily-available materials.

Use of pole buildings in America continued throughout the colonization of the country. Norum (1967) reports use of pole buildings on farms in the 19th century.

1.4.3 D. Howard Doane

The modern post-frame structure can trace it roots to D. Howard Doane, founder of Doane Agricultural Service (DAS). Doane had an unwavering vision of a more efficient, productive agriculture, and he worked to improve profitability in all aspects of the farm enterprises that DAS helped manage. Doane's drive led him to explore options to the heavy timber framed buildings that were the mainstay of production agricultural. In the early 1930's, he erected buildings utilizing embedded red cedar poles as their primary supports. The poles supported girders upon which rafters were placed on two-foots centers. Purlins consisted of one-inch thick boards that were laid-flat and spaced 12 to 18 inches apart. This frame was covered with corrugated galvanized steel.

1.4.4 Bernon Perkins

Bernon George Perkins was hired by DAS as a farm manager in the mid-1930s and began using creosote treated poles instead of the more scarce cedar poles. Within a relatively short period of time, preservative treated poles became the mainstay of this new building system. In a textbook titled *Farm Buildings* that was published in 1941, author John C. Wooley introduces "pole-frame" as the simplest type of barn frame. Wooley states that "in many of the newer structures using this type of frame, treated poles are being used, and a footing is provided for each pole." Contrary to this statement by Wooley, the use of footings was actually relatively rare.

Perkins is attributed with two other developments of the modern post-frame building system. He was the first to place 2- by 4-inch members on-edge as purlins, and he was the first to overlap purlins. By installing purlins-on-edge, Perkins was able to increase both rafter and purlin spacing. Overlapping of purlins led to more rapid construction and an improved purlin-to-rafter connection. In 1949, Perkins applied for a patent on a pole-frame building designed for storing and drying hay, grain and other commodities. The patent was granted in June, 1953 (Patent No. 2,641,988). Figure 1 from this patent is reproduced in figure 1-31 and shows the lapped, 2- by 4-inch on-edge purlins characteristic of Perkins' buildings.

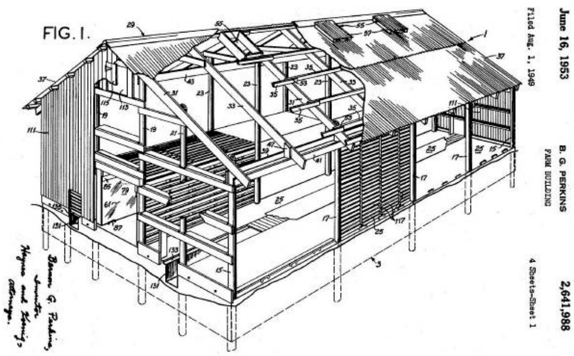

Figure 1-31. Figure 1 from U.S. Patent No. 2,641,988 granted in June, 1953.

Although DAS was the assignee on Perkins' patent, it had no interest in protecting the patent from infringement by others. In fact, D. Howard Doane had Perkins and other employees actively encourage builders and farmers to accept and utilize their ideas. Perkins began a "builders program" in the early 1950's and traveled around the country with Tom Locke, a DAS engineer, sharing plans with builders involved in and/or interested in pole-frame building construction. The program was dissolved in 1954.

1.4.5 Truss Use

The 1950's saw increased use of trusses in farm buildings. By the late 1950's, truss use in pole-frame buildings became the rule rather than the exception. Many trusses were fabricated with the use of ½-inch thick plywood gussets held in place with glue and nails. Fabrication of such trusses was quite labor intensive because of the time required to hand drive numerous nails. Bolted trusses and trusses utilizing timber connectors were also commonly used (trusses using split ring timber connectors were generally referred to as "bolt and ring" trusses). In the early 1960's, buildings with 40-ft clearspans were commonly erected. By the early 1970's, buildings with 60-ft clearspans were routinely built.

1.4.6 Patterson Publication

In 1957 the American Wood Preservers Institute published a document written by Donald Patterson titled *Pole Building Design*. Written for use by engineers, *Pole Building Design* stressed engineering concepts that were "somewhat unusual or unique in pole-type buildings." A major portion of the document was dedicated to methods for determining the depth of embedment of poles - methods based on research funded by the Outdoor Advertising Association of America.

1.4.7 Metal Plate Connected Trusses

The 1960's ushered in the age of the metal plate connected wood truss (MPCWT). Early MPCWTs featured metal connector plates with much larger teeth and lower tooth densities than today's plates. Not until the early 70s did most post-frame building companies begin transitioning from bolted trusses to MPCWTs.

1.4.8. Rectangular Posts

The 1960's also saw the pole-frame building industry begin its transformation into the post-frame building industry as builders began to abandon poles in favor of solid-sawn posts. In some cases the transition was from buildings with round poles to buildings with all slabbed poles, and then to buildings with slabbed poles in all locations except the corners where rectangular posts were used, and finally to buildings with all rectangular posts. Although size-for-size solid-sawn posts typically lacked the bending strength of the poles they replaced, they enabled more rapid and accurate frame erection, as well as the straight-forward installation of quality interior wall finishes (something not easily accomplished with tapered poles). Finished interiors had become more common place as (1) farmers began to utilize thermally-insulated post-frame buildings for shops, offices and certain livestock housing facilities, and (2) the industry began to carve out a niche in the commercial building market.

1.4.9 NFBA Formation

A landmark moment for the industry came in 1969 when the State of Indiana looked to adopt building code provisions requiring "continuous concrete foundations" for all "wood frame commercial buildings". Because of the negative impact this would have on post-frame building, Freemon D. Borkholder organized a meeting which was attended by approximately twenty builders. It was during this 1969 meeting that the decision was made to form an organization of post-frame builders and to call it the National Frame Builders Association (NFBA). In 2007 the word "Builders" was changed to "Building" (i.e., NFBA became the National Frame Building Association) in recognition that the organization is made up of more than just active builders (i.e., it includes suppliers, designers, researchers, etc.).

Use of the term "post frame" does not appear to have been coined during the 1969 Indiana meeting. In his 1941 *Farm Buildings* textbook, author John C. Wooley uses the term "post frame" to refer to buildings with rough, hewed, sawed, or built-up posts placed at 10- or 12-ft intervals and placed on a wood sill or directly on a concrete foundation.

1.4.10 Engineering Infusion

As they expanded into commercial building markets in the late 1960's and early 1970's, larger post-frame building companies began employing their own registered professional engineers for in-house production of all plans, specifications and structural calculations required by code for commercial buildings. This infusion of engineering into the post-frame building industry impacted more than just commercial buildings, as it brought with it the science to safely build larger agricultural structures. Although the importance of properly engineering a building has never been in dispute, few agricultural buildings were fully engineered prior to the 1970's. This is because agricultural buildings were (and still are) exempt from building codes in virtually every jurisdiction in the United States. Although more engineering has been put into agricultural buildings in the past half-century, it is important to realize that many agricultural structures are still constructed with little or no engineering input.

1.4.11 Concrete Footings

Concrete footings which were largely absent from pole-frame buildings erected prior to the 1960's, became standard elements under embedded posts in the 1970's. This can be attributed to significant increases in post bearing pressures, and to commercial building engineering that routinely showed actual soil pressures vastly exceeding allowable soil pressures when footings were absent. Increases in post bearing pressure were directly attributable to increases in clearspan distances (i.e., axial post loads increase in direct proportion to the clearspan distance between posts in a primary frame) and to decreases in post bearing area. In many cases, the butt-end of tapered poles was often greater than the cross-sectional area of the rectangular posts that replaced them.

1.4.12 Penta and CCA Wood Treatments

In the 1940's, pentachlorophenol largely replaced creosote as the preferred preservative treatment for wood, especially for pole-frame building applications. Although creosote was a very effective treatment, the resulting oily surface made it virtually impossible to seal in its strong objectionable odors. Creosote-treated poles were also messy to handle, particularly in warm weather. By the early 1980's chromate copper arsenate (CCA) had effectively supplanted pentachlorophenol in post-frame buildings. As a waterborne preservative, CCA was easier to stain, paint and seal. CCA was also considered to be less of a hazard to humans than penta-treated wood. In fact, in 1984, the EPA banned the use of pentachlorophenol for all indoor applications, except for a few low exposure uses which included embedded poles and posts used in agricultural applications.

1.4.13 Diaphragm Design

In the mid-1980's, post-frame building engineers began discussing a procedure published in 1983 by Hoagland and Bundy for calculating the percentage of horizontal wind load transferred to shear walls by metal-clad roofs in post-frame buildings. This procedure, referred to as diaphragm design, was based on methods outlined for metal-clad steel-framed diaphragms by Bryan (1973), and featured an equation developed by Luttrell (1967) for extrapolating diaphragm test panel data for use in full-scale building design. The first research on metal-clad wood frame diaphragms can be traced to Hurst and Mason who in 1961 published results of tests on two separate (but similar) metal-clad pole buildings that showed that roof and endwall cladding contributed significantly to the overall rigidity of the structure. The first metal-clad wood-frame diaphragm test panels were tested over 15 years later by Hausmann and Esmay (1977) and White (1978).

The diaphragm design procedure published by Hoagland and Bundy in 1983 was slightly modified by Gebremedhin and others (1986), and formed the basis for ASAE EP484 *Diaphragm Design of Metal-Clad, Post-Frame Rectangular Buildings*. Work on the ASAE EP484 commenced in 1986 under the direction of Harvey Manbeck and was approved for publication in 1989 (Manbeck, 1990).

ASAE EP484 requires strength and stiffness values for all load-resisting diaphragms and shearwalls in a building. Consequently, numerous metal-clad wood-frame diaphragm panel tests have been conducted since the 1980's. Sources and a compilation of data from many of these tests are provided in Chapter 8.

A major revision to ASAE EP484 was completed in 1998. This revision included the simplified design approach outlined by Bohnhoff (1992a) and also allowed for more detailed diaphragm analyses including the Force Distribution Method developed by Anderson (1989) and computer program DAFI developed by Bohnhoff (1992).

1.4.14 Mechanically-Laminated Posts

In the early 1980's, builders began switching from solid-sawn posts to nail-laminated posts. This switch was driven by the lack of stress-rated timber, decay issues with solid-sawn posts, and a need for posts that could withstand higher bending stresses. Decay problems became more prevalent when builders switched from poles to sawn posts, primarily because of the difficulty of treating heartwood exposed by sawing operations.

In the mid-1980's, builders began utilizing spliced, nail-laminated posts – posts with preservative-treated wood on one end and untreated wood on the other end. Questions about the bending strength of various spliced post designs led David Bohnhoff to develop a special finite element modeling method for the posts (Bohnhoff et al., 1989) and to conduct numerous tests on both spliced and unspliced posts (Bohnhoff et al., 1991, Williams et al., 1994, Bohnhoff et al., 1997). This ultimately led Bohnhoff to draft ASAE EP599 *Design Requirements and Bending Properties for Mechanically Laminated Columns* which was approved by ASABE in December 1996, and as an American National Standard by ANSI the following February. In 2009, Bohnhoff chaired a major revision of the engineering practice which included a name change to *Design Requirements and Bending Properties for Mechanically Laminated Wood Assemblies*.

1.4.15 Foundation Design Standard

In the late 1980's, Gerald Riskowski and William Friday (1991a, 1991b) developed equations for calculating the embedment depth of collared post foundations. These equations became part of ASAE EP486 *Shallow Post Foundation Design* which was developed under the direction of Friday and released by ASABE in March 1991. Based on research by Neil Meador, the standard was slightly revised in 1999 and approved as an American National Standard by ANSI in October 2000. In 2007, David Bohnhoff began working on a major revision to ASAE EP486. Approved in 2012, the revision contains completely different methods for calculating bearing, lateral, and uplift strengths of both pier and post foundations, and unlike previous versions, it contains safety factors for allowable stress design (ASD) and resistance factors for load and resistance factor design (LRFD). Additionally, the revised EP contains several methods for obtaining soil properties from on-site soil tests.

1.4.16 Screw Fasteners

Prior to the 1980's, metal wall and roof cladding was largely nail-fastened. By the late 1980's, many post-frame builders had made the switch from nails to self-piercing and/or self-drilling screws. In some cases, the adoption of screws was tied to the early adoption of diaphragm design. While some companies still used nails for metal cladding attachment in the 1990's, such usage was pretty much curtailed by the turn of the century.

1.4.17 ASAE EP558 Formation

In 1998, provisions in ASAE EP484 covering diaphragm panel tests were removed and placed in a separate publication titled ASAE EP558 *Load Tests for Metal-Clad Wood-Frame Diaphragms*. At the same time, ASAE EP484 was modified to include additional analysis options, and in August 1998 it was approved by ANSI as an American National Standard.

1.4.18 Construction Tolerances

In 1999, NFBA published the first of two post-frame construction tolerances documents. Like tolerance documents developed by other organizations, the NFBA documents were developed to (1) establish standards of professional conduct for members of the organization (2) enhance the professional reputation of the industry, (3) minimize costly litigation between owners and builders, and (4) maintain regulatory control within the profession. The first of these two documents, titled *Accepted Practice for Post-Frame Building Construction: Framing Tolerances*, includes recommended tolerances for the position/placement of footings, posts, trusses, girders, girts, and purlins. This document was drafted by David Bohnhoff and based on research by Marshall Begel (Bohnhoff and Begel, 2000). The second document, titled *Accepted Practices for Post-Frame Building Construction: Metal Panel and Trim Installation Tolerances*, was released by NFBA in 2005 and covers recommended tolerances for metal panel positioning, metal trim positioning, fastener installation, and surface and edge blemishes. This document was also drafted by Bohnhoff but based on research conducted by David Cockrum (Bohnhoff and Cockrum, 2004).

1.4.19 Glulam Posts

There was a steady increase in glulam post use throughout the 1990's, largely spurred by the launching of several companies fully dedicated to glulam post production for post-frame buildings.

1.4.20 Laminated Veneer Lumber

The development and production of laminated veneer lumber (LVL) in the 1980's resulted in a significant increase throughout the 1990's of buildings with solid-web primary frames featuring LVL rafters. Although solid-web primary frames typically cost more than open-web primary frames, they are preferred because of their cleaner appearance, elimination of bird perch points, and greater durability in corrosive environments (i.e., environments that significantly shorten the life of metal connector plates).

1.4.21 CCA Restrictions

Effective December 31, 2003 the EPA restricted the use of chromate copper arsenate (CCA) treated wood in a number of applications including all post-frame building components except embedded posts. This resulted in a switch to different waterborne copper-based treatments (e.g., copper azole (CA), ammoniacal copper zinc arsenate (ACZA)) and greater use of plastic composite materials for splash plank, grade girts and base plates. These products and further developments in wood treatment, including micronized copper, are the primary products used in the industry today.

1.4.22 Precast Concrete Piers

The turn of the century brought with it an increased emphasis on sustainable building construction, that is, more affordable and environmentally-friendly construction. One manner in which this is achieved is by increasing the functional life of buildings and building components. With respect to post-frame buildings, this has stimulated interest in use of concrete piers (Bohnhoff, 2006), especially a precast concrete pier system developed and patented by Bob Meyer Jr. and marketed nationally under the tradename Perma-Column.

1.4.23 Expansion of Screw Use

Advances in screw fasteners and associated installation equipment continue to have a pronounced impact on post-frame building design and construction. Specifically, self-drilling screws with a diameter near a quarter inch are now used by some builders to attach trusses to posts and to connect other primary-framing components. Also, since the turn of the century, builders have begun to use deck screws to attach girts and other secondary framing members, and to install temporary bracing.

1.5 Advantages

As noted in Section1.1.1, a post-frame building system is one of many types of framing/support systems. A designer who is choosing among various framing systems will assess how each framing system positively and/or negatively affects building **F**unctionality, **A**ffordability, **C**omfort, **A**esthetics, **D**urability, **E**nvironmental-friendliness, and **S**afety (i.e. building FACADES). In each of these areas, a post-frame building system has unique advantages as described in the following paragraphs.

1.5.1 Functionality

To provide a functional building, a designer must have the flexibility to easily and inexpensively (1) alter the interior layout of a building without interference from structural supports, (2) select an optimal clear height and/or vary building clear height, (3) locate exterior doors and windows of any size where they are needed, and (4) apply any combination of horizontal and vertical loads to roof, wall, floor and ceiling elements.

Post-frame building systems provide the aforementioned flexibility by *commonly* and *economically* providing clearspan widths upward of 80 feet and eave heights upward of 20 feet. The relatively large spacing between primary frames enables the installation of large doors and windows at no additional framing cost to the consumer. The ability to easily modify primary frame size and spacing enables designers to easily handle any combination of horizontal and vertical loads.

Indeed, it is the flexibility of the post-frame building system that has resulted in its use in a very broad range of residential, agricultural, commercial and industrial applications. This includes buildings with fully open walls, buildings that are completely sided, and buildings with walls of solid glass or walls comprised entirely of overhead doors. This includes stilt buildings, slab-on-grade buildings, buildings on stem walls, and buildings with full basements. This includes single-story buildings, two-story buildings, structures with high-load capacity lofts and mezzanines, and buildings with stored contents that exert high wall pressure.

1.5.2 Affordability

Significant savings can be obtained with post-frame construction in terms of materials, labor, construction time, equipment and building maintenance. This is directly attributable to the fact that they are among the most efficiently designed structures in the world; and it explains why post-frame buildings dominate markets where cost is the overriding factor in building selection.

When compared to other building systems, the low relative cost of most post-frame buildings is attributable to the use of less extensive foundations and to the fact that wall sections between posts are non-load bearing. As further explained in Section 1.6.2, embedded post foundations commonly used in post-frame buildings require less concrete, heavy equipment, labor, and construction time than conventional perimeter foundations. Additionally, embedded post foundations are better-suited for wintertime construction - times when frozen ground and cold temperatures increase labor and fabrication costs for other foundation types.

1.5.3 Comfort

Building comfort is largely dictated by thermal envelope design, HVAC systems, and natural lighting. Of these three variables, the only one that is significantly affected by frame selection is thermal envelope design.

One of the outstanding features of post-frame building systems is that they allow for a plethora of thermal envelope designs. This stems from the fact that the space between primary frames can be filled with virtually anything. This includes thermal envelopes that rely on more conventional insulations (fiberglass batts and blankets, rigid polystyrene boards, blown-in-blankets, and foam-in-place insulation products), and extends to infill panels featuring bales of organic fiber, structural insulated panels (SIPs), adobe, cordwood, etc..

1.5.4 Aesthetics

Like most other framing systems, post-frame building systems can be clad with virtually any material and thus can be designed to mirror the appearance of most any structure. Unfortunately, many individuals only see post-frame buildings as simple, rectangular structures with an exterior covering of corrugated steel. This results from the simple fact that such buildings are very inexpensive and thus the building of choice for numerous applications where cost is the overriding factor in design decisions. It comes as no surprise that many post-frame buildings go unrecognized as such by the general public simply because they do not feature corrugated steel siding. These include numerous post-frame buildings with wood siding, brick veneer, stone veneer, and stucco (see figures 1-32 through 1-36).

Building occupancy type typically dictates the type of interior finish. For residential and small business applications, interior walls are typically sheathed with gypsum wall board. For extra appeal, wood posts are frequently left exposed (e.g., not covered with wall board). Additionally, exposed glued-laminated and solid-sawn timbers may be substituted for metal plate connected wood trusses (MPCWT).

Figure 1-32. Post-frame boys and girls club with steel siding and brick veneer. An FBi Buildings project.

Figure 1-33. Wood-clad post-frame building constructed for horse housing.

Figure 1-34. Small commercial post-frame building sided with stucco and brick veneer.

Figure 1-35. Residential post-frame building with brick veneer and fiber-cement board siding.

Figure 1-36. Commercial post-frame building clad in stone and smooth metal panels. An FBi Buildings project.

1.5.5 Durability

Durability is dictated by the degree to which degradation of materials due to decay and corrosion is controlled, and the degree to which load levels are maintained within the design strengths of components and connections. The former is managed by using materials that are compatible with the environment(s) to which they are exposed; the latter is controlled with proper structural engineering.

Durability has been a hallmark of post-frame buildings, as is evidenced by the number of post-frame buildings that are still in use years after exceeding their original design life. This can be attributed to proper use of preservative lumber treatments (or concrete where such treatments are not desired), corrosion resistant fasteners, and wood adhesives.

1.5.6 Environmental-Friendliness

The low cost of post-frame buildings is directly attributable to its efficient use of materials, and hence its very low embodied energy relative to structures of similar size. Much of this is attributable to the reduced use of concrete in foundations. To this end, as more accurate and complete life cycle analysis/assessment methods are developed and used in selection of building systems, greater use of post-frame building systems is expected.

Given the durability of post-frame buildings, it is not uncommon for them to exceed their functional design life. For this reason, many older post-frame buildings are now used for purposes other than which they were initially designed.

In situations where post-frame buildings have outlived their initial need, it may be advantageous to move or reconfigure the structure. This is relatively easy to accomplish with modern post-frame buildings, as they are largely assembled using mechanical fasteners (i.e., bolts and screws) that can be quickly removed without damage to components. This ability to "recycle" a post-frame building adds to their reputation as one of the world's most environmentally-friendly structures.

1.5.7 Safety

Outstanding structural performance of post-frame buildings under adverse conditions such as hurricanes is well-documented. Gurfinkel (1981) cites superior performance of post-frame buildings over conventional construction during hurricane Camille in 1969. Harmon et. al (1992) reported that post-frame buildings constructed according to engineered plans generally withstood hurricane Hugo (wind gusts measured at 109 mph). Since post-frame buildings are relatively light weight, seismic forces do not control the design unless significant additional dead loads are applied to the structure (Faherty and Williamson, 1989; Taylor, 1996).

1.6 Ideal Structural Applications

Different structural framing/support systems will have different advantages and disadvantages depending upon the particular application. A post-frame building system is no different than any other structural framing/support system in this regard. In general, a post-frame building system will have inherent advantages where it is advantageous to have wood posts as main, load-bearing, vertical framing elements. Following are thirteen such applications highlighted by Bohnhoff (2008).

1.6.1 Buildings With Numerous and/or Relatively Large Wall Openings

Windows and doors in a post-frame building that are narrower than the post spacing typically do not require structural headers, since roof trusses/rafters in most post-frame buildings bear directly on the posts. Elimination of structural headers enables elimination of trimmer studs (a.k.a. jack studs, shoulder studs) and other special structural members required to support the headers.

Removing headers and their supports not only saves money, but results in an enhanced thermal envelope when framing members are replaced with thermal insulation. Additionally, fewer framing members mean fewer cracks for air infiltration.

In general, any building with large, regularly-spaced door and window openings is an ideal candidate for post-frame. Mini-warehouses and service garages typically have several equally-spaced and equally-sized overhead doors making them ideal candidates for post-frame (figures 1-37 through 1-39). In these buildings, posts are often used to frame both sides of the doors. Post frame is also ideal for retail stores with large glass facades (figure 1-40).

Figure 1-37. The numerous, equally-spaced overhead doors of mini-warehouses make them ideal for post-frame.

Figure 1-38. Post-frame suburban garage.

Figure 1-39. Post frame readily accommodates the overhead doors required for this automobile repair business in Lafayette, Indiana. An FBi Buildings project.

Figure 1-40. Large, equally spaced windows suit post-frame.

1.6.2 Buildings Without Basements

Many buildings without basements are supported on cast-in-place crawlspace walls or frost walls that rest on continuous cast-in-place concrete footings. The construction time and concrete cost associated with these continuous concrete foundation walls and footings is significantly greater than that associated with a post-frame building that utilizes embedded posts or a post-on-concrete pier system as its foundation system (figure 1-41).

The material and labor savings associated with post/pier foundation systems makes them the most environmentally-friendly foundation system in common use today. Additionally, embedded post and precast pier foundations can be easily removed and reused – a feature which adds to their status as a very environmentally-friendly foundation system.

Figure 1-41. Preservative-treated, mechanically-laminated embedded post (left) and mechanically-laminated post attached to a precast concrete pier (right).

Most buildings without basements feature concrete slab-on-grade floors. More frequently today, these slabs contain radiant heating systems. When post/pier foundation systems are used, the interior concrete slab can be placed after the building shell has been erected. This has two major advantages. First, concrete is much more protected during its placement from wind, precipitation in all forms, and temperature extremes. This can translate into fewer unexpected scheduling delays, less need for costly heat and moisture protection systems, and enhanced concrete surface finish, durability, and strength properties. Second, less preplanning is required for below slab installation of HVAC, plumbing and electrical system components. In fact, no preplanning is required when the interior concrete slab is placed after HVAC, plumbing and electrical system installations have been completed. With respect to utilities, it is also important that insulation must be placed under a slab that contains a radiant heating system, and placement of this insulation requires a very level, properly compacted base – something more easily achieved and maintained in a protected environment.

Some builders may opt to place posts on the thickened edge (i.e., grade beam) of a concrete slab. Such systems generally require more total concrete than systems with concrete pier foundations since the extra concrete required for the grade beams usually exceeds that required to fabricate concrete piers.

1.6.3 Buildings with Tall Exterior Walls

Mechanically- and glue-laminated posts are used in the vast majority of today's post-frame buildings. These posts enable the construction of buildings with relatively large floor-to-ceiling heights at prices much less than they could be fabricated with a comparable wood stud wall.

Laminated posts can be fabricated to any length by splicing shorter pieces of wood together. Laminated posts are also straight and inherently more stable because of the laminating process. The only way to get a tall, relatively straight wall with wood studs is to use more expensive, engineered lumber products (e.g., laminated strand lumber, laminated veneer lumber, parallel strand lumber).

The increased bending moments associated with taller walls may be handled by using higher grade lumber or with larger vertical wall framing elements. Another option is to reduce the spacing of the framing elements so that each element is subjected to less load. These options are easy to accommodate into post-frame building design, which is one more reason why they get the nod over other framing systems in tall wall applications such as that shown in figure 1-42.

The cost advantage that post-frame buildings hold over low-rise steel frame buildings generally starts to disappear once minimum floor-to-ceiling heights move beyond 20 feet. Below these heights, post-frame holds thermal insulation advantages, if not cost advantages, over steel frame structures. This has made post-frame very popular for storage facilities such as the airplane hanger in figure 1-43.

Figure 1-42. Post-frame concrete batch plant with a 45 ft ceiling height, accomplished in part by bracing wood columns back to the main structure of the concrete plant equipment.

Figure 1-43. The ability to construct inexpensive buildings with relatively high eave heights makes post-frame ideal for many machinery storage buildings including this airplane hanger.

Required length of vertical wall framing elements are often significantly different in various locations within a building. Where such length variations occur, structural requirements for the longer elements generally control framing/support system selection. Significant wall framing length variations most commonly occur in the endwall framing of wide buildings with sloped ceilings (the dairy freestall barn in figure 1-44 is one such example). Not surprisingly, the endwalls in many of these buildings are post-frame.

Figure 1-44. This building is typical of many dairy freestall barns. The greater width of these buildings results in (1) tall gable endwalls requiring substantial framing, and (2) the need for interior support posts.

1.6.4 Bulk Storage Buildings

Bulk storage refers to the storage of a relatively large quantity of a material or commodity such as cement, sand, salt (figure 1-45), fertilizer, fruit, vegetable, seed, feed, cotton, straw, and aggregate. If a bulk storage building wall is used to contain the stored material, that wall must be designed to resist the resulting horizontal pressure which is directly dependent on the height of the stored material. Even for stored material heights of only a few feet, this pressure will be several times greater than the environmental design pressure applied to exterior walls by even the highest of winds.

As noted in the previous section, high wall forces are easily accommodated in post-frame building design by altering post size and/or spacing. Post spacing is generally dictated by the spanning capability of the structural material used to contain the bulk material.

Figure 1-45. High wall pressures and resistance of wood to corrosion make post-frame the perfect application for salt storage facilities.

1.6.5 Buildings with Open Walls

Buildings whose only purpose is to provide protection from precipitation and/or solar radiation are generally fabricated with one or more open sides. This would include many commodity (e.g. fertilizer, lumber, feed) storage buildings, animal shelters, and park and other recreational shelters (see figures 1-46 to 1-49). Open sides facilitate quick building access, which can translate into significant cost savings when handling stored materials.

Unless a unique structural support system has been employed, expect the roof above an open wall to be supported by posts with an on-center spacing of 8 or more feet. Since these posts are seldom laterally supported between their base and crown, they must be designed to resist buckling equally in all horizontal directions. For this reason they tend to be round poles, square solid-sawn timbers, or square glulam or parallel-strand lumber members. Nail-laminated posts will typically require the addition of face plates to obtain approximately equal bending strength in all horizontal directions.

Wood posts in open-front buildings are often preservative-treated because of their direct exposure to "the elements." However, in situations where wood posts are supported on concrete piers, or walls are fairly well protected from precipitation with a roof overhang, preservative treatment may be unnecessary.

Figure 1-46. The desire for open sidewalls makes the post-frame building system a popular choice for park and other recreational shelters.

Figure 1-47. Heifer growing facility with open sidewalls.

Figure 1-48. Open front equipment storage building.

Figure 1-49. Hay storage facility with all-around access by. Post spacing on back endwall twice that on front endwall to accommodate machinery access.

1.6.6 Buildings Requiring Interior Posts

When a building has interior columns, it is advantageous to use a post-frame building system for two reasons. First, it increases the likelihood that all building support elements will be on similar footings. This speeds construction and minimizes the likelihood of differential settlement. Second, interior posts may be more effectively incorporated into the framing system since they can be aligned with, and then connected via rafters or header beams to exterior posts to form rugged primary building frames (see figures 1-5 through 1-9).

Interior posts are used in place of interior load-bearing walls, primary because they provide for a more open floor plan. Money may also be saved by switching from bearing walls to posts, since posts utilize isolated footings which require less concrete than the continuous footings used to support bearing walls.

Interior posts are either used to support roofs in wide buildings (figure 1-44) or mezzanines (figure 1-50). In practice, wood-framed roofs that clearspan more than 90 feet and that are subjected to heavy snow loads will generally not be economically competitive with steel roof framing unless interior support is provided.

Figure 1-50. Supporting mezzanines with posts instead of walls provide for more open floor plans.

Interior posts are seldom laterally supported between their base and crown, and thus are designed similarly to posts in open exterior walls.

1.6.6.1 Buildings with Clerestories

A clerestory (a.k.a. clearstory) is a fenestrated (windowed) wall that rises above a roofline (figures 1-51 and 1-52). Because clerestories are used to brighten a building's interior, it is advantageous to support a clerestory wall (and the above roof) with interior posts (and not solid walls).

Clerestories are commonly located above sloping roofs such that the height of the clerestory plus the height of the adjacent sloping roof is roughly equivalent to a single story. Roof slopes associated with clerestories are the same roof slopes common to the typical post-frame building. This combined with the fact that clerestories rely heavily on interior posts explains why buildings with clerestories are commonly post-framed.

Figure 1-52. Commercial building (top) and milking center (bottom) with clerestories.

In many cases, the interior posts not only support the walls and roofs of the clerestory, but they also support a second floor level (i.e., a mezzanine or loft) as shown in figure 1-53. In livestock housing facilities, this second floor is commonly used for hay storage.

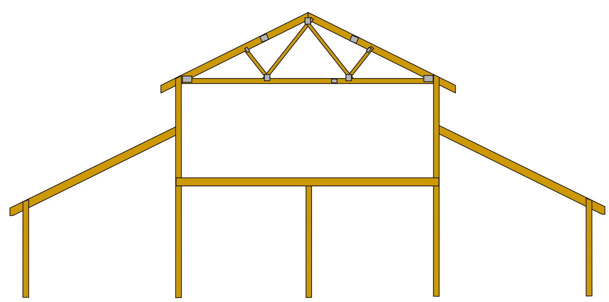

Figure 1-53. Primary frame of a building with clerestory.

Figure 1-51. A clerestory is a common feature on many equestrian facilities.

1.6.7 Buildings with Large, Clearspan Wood Trusses With On-Center Spacing 4 Feet or Greater

Component connections are critical to the structural integrity of a framing system. In buildings with large, clearspan wood trusses, the most critical connections are those between the truss and its supports. In addition to gravity-induced forces (a.k.a. bearing loads), these connections must resist shear forces acting perpendicular to the plane of the truss and uplift forces due to wind. Depending upon overall building design, the connections may also be required to transfer bending moment.

Figure 1-54. Truss resting on outer ply of a laminated post. Steel L-bracket placed on outer face of truss to help reinforce post-to-truss connection.

Figure 1-55. When truss is sandwiched between outer plies of a mechlam post, bolts are placed in double-shear for a very effective connection.

Wood posts enable the fabrication of strong, direct, yet inexpensive connections between large trusses and walls. Exact details for post-to-truss connections vary from designer to designer, and may be influenced by post type. Solid-sawn timber and glulam posts are generally notched to form a truss bearing surface. The truss rests on the notches and is bolted into place. A special plate/bracket like that shown in figure 1-54 may be added to increase connection load transfer capabilities. With mechanically-laminated posts, the truss may rest on a shortened outer-ply or on a shortened inner-ply. The latter scenario, which is shown in figure 1-55, places the bolts in double shear and is a very effective connection.

1.6.8 Buildings Requiring a More Open Structural Frame to Accommodate Non-Structural "Infill" Panels/Materials

Post-frame is the ideal structural support system for straw bale walls (figure 1-56), cordwood or stackwood walls (figure 1-57, light-clay coated organic fiber walls and even earthen walls. Given that straw, cordwood, clay-coated organic fibers and earth are all considered very environmentally-friendly materials, expect the number of post-frame buildings that are constructed with in-fill walls of these materials to grow.

Figure 1-56. Non-structural straw bale walls prior to plastering. Image from the American Society of Agricultural and Biological Engineers. Used with permission.

Figure 1-57. Cordwood (a.k.a. stackwood) infill walls.

For frame openness, the post-frame building system is often a more structurally efficient version of a timber-frame building system. In short, any wall cladding or infill material that has been utilized on or in a timber-frame building may be used on or in a post-frame building. This includes application of structural insulated panels (SIP) to wall and roof surfaces.

1.6.9 Stilt Buildings

Stilt buildings are one of the least expensive options when building in floodplains, over very poor soils or water, on very steep terrain, and in regions of high snow fall (see figures 1-58 to 1-62).

Stilt buildings fall into two categories: those with stilts that only support sill plates and floor headers, and those with stilts that connect to both roof and floor framing. The latter are essentially post-frame buildings with wood-framed floors. Exactly how a post-frame stilt building would be detailed depends largely on desired floor, wall and ceiling finishes as they control the spacing of structural frame components.

Figure 1-58. Deer stand on stilts in Randall, Minnesota.

Figure 1-59. Cabin on stilts in the Missouri Ozarks from http://www.regionslandcompany.com/

Figure 1-60. Sound engineering and construction of this post-frame stilt building on Dauphin Island, Alabama saved it from meeting the same fate as the one deposited in its front yard by Hurricane Katrina. FEMA photo.

Figure 1-61. Stilt Houses in the Amazon Basin from http://gallery.nen.gov.uk/asset75336_1615-.html

Figure 1-62. Stilt building on Assateague Island National Seashore.

1.6.10 Towers and Buildings with Towers

Towers are a natural fit for post frame. When post-frame systems are properly connected and anchored, very strong and relatively inexpensive three-dimensional tower frames may be built, as evidenced by the many pole- and post-supported forest fire lookout towers built in North America during the early 1900's (see figure 1-63).

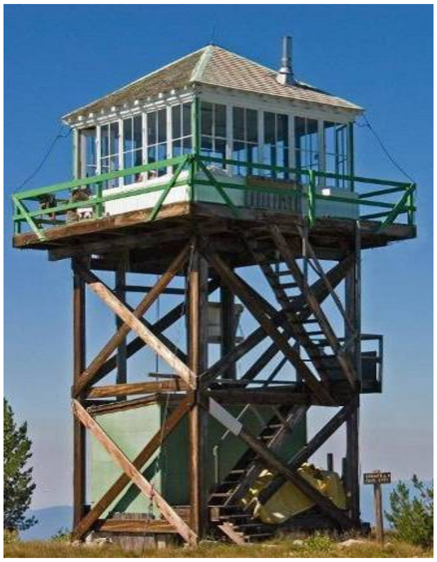

Figure 1-63. Forest fire lookout towers are great examples of how post-frame construction can be used to frame towers. Shown here is the Granite Mountain Lookout, Alpine Lakes Wilderness Area, Washington.

Figure 1-64. Lookout for observing nature, hunting and outdoor recreation. Stands 30 ft. from ground to floor, and 41 ft. to the peak of the roof. The building features 44-ft. long foundation-grade treated four-ply laminated columns on 4 ft. deep embedded footings.

Multi-story towers are becoming popular additions to commercial buildings. In addition to adding flare to a building, they frequently serve as stairwells, sources of natural light, clock towers and observatories. Figure 1-65 shows wood-framed towers and buildings with attached towers.

Figure 1-65. Tower applications are ideal for post-frame. An FBi Buildings project.

1.6.11 Buildings with Post-Supported Porches, Balconies, and Roof Overhangs

The spacing of posts used to support a building's porch, balcony, and/or roof overhang is generally in the 6 to 10 foot range regardless of the building's structural framing/support system. Given that this spacing is typical of the post spacing in most post-frame buildings, there are benefits to using a post-frame building system anytime a building features a relatively long post-supported porch, roof overhang or balcony. First, it increases the likelihood that all building support elements will be on similar footings. This speeds construction and minimizes the likelihood of differential settlement. Second, posts used to support a porch, roof overhang or balcony may be aligned with rafters and then connected via the rafters to posts in the exterior wall to form a more efficient structural frame. See figures 1-66 to1-73.

Figure 1-66. Post-frame convenience store for Byrne Dairy in Galeville, NY. The storefront features a porch and above balcony. A porch formed by a roof overhang extends along the side of the structure.

Figure 1-67. Post-frame storage/workshop with side porch.

Figure 1-68. Post-frame horse barn with post-supported roof overhang.

Figure 1-69. Post-frame horse barn with arcade.

Figure 1-70. Three views of a post-frame garage with side porch and covered end entrance.

Figure 1-71. Post-frame motel with wrap-around balcony.

Figure 1-72. Wrap-around porches on a park shelter (top), institutional building (middle) and equestrian facility (bottom).

Figure 1-73. Suburban garage with a side porch.

1.6.12 Buildings with Bracket-Supported Overhangs

Roof overhangs and eyebrow overhangs are commonly added to buildings to improve building aesthetics and durability. They improve durability by protecting door and window openings and siding from precipitation. They also keep snow slides away from the building and limit intrusion of direct solar radiation during warm periods.

As the distance that an overhang extends from the building wall increases, it is more likely the overhang will be supported by a post (figure 1-68) or wall support bracket (figures 1-74 to 1-76). Whether post supports or wall support brackets are used is largely dependent on overhang height. Normally, post supports are used for lower overhangs because of headroom clearance issues when wall support brackets are used.

With higher overhangs, wall support brackets generally look better than posts and are normally less expensive than post supports because of the added foundation and header beams required with post supports.

Figure 1-74. Wall support brackets used to support an eyebrow overhang.

Wall support brackets are the ideal overhang support system for post-frame buildings in which truss and post spacing are equal. In such buildings, posts and trusses form a series of post-frames as previously described. When wall support brackets are attached to the posts and framing of the overhang, they add rigidity to each post-frame. In situations where the overhang is a roof overhang, the wall support bracket attaches the end of the truss to the post, thus functioning much like an exterior knee brace.

Figure 1-75. Wall support brackets used to support a roof overhang.

Figure 1-76. Tall walls, large regularly-spaced windows and wall brackets make buildings similar to this good candidates for post-frame.

1.6.13 Buildings with Corrosive Contents

With few exceptions, metals are unstable and will corrode in ordinary aqueous environments. The rate of this corrosion depends on the hydrogen-ion concentration (pH) of the solution, the specific nature and concentration of other ions in solution, temperature, and other factors. In general, the more humid an interior building environment, the more likely and frequently moisture will condense on metal surfaces within the building, and the greater will be the rate of metal corrosion. Also, the greater the concentration within a building of acidic gases (e.g., hydrogen sulfide, sulfur oxides, nitrogen oxides, chlorine, hydrogen fluoride), caustic gases (e.g., ammonia), and oxidizing gases (e.g., ozone, nitric acid), the greater will be the rate of metal corrosion within the structure.

Given the higher humidities in livestock housing facilities, and ammonia and hydrogen sulfide gases associated with deposition and decomposition of animal wastes, it is wise to limit direct exposure of metals in such facilities. It is also wise to limit direct exposure of metals in water treatment facilities where chorine is used, and in facilities were bulk fertilizer, salt (figure 1-45) and other corrosive materials are stored. This is accomplished by using wood-framed structures in which mechanical connectors are hidden or specially coated to reduce corrosion and thus enhance overall durability.

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1.8 Acknowledgements

The following companies provided images for this chapter:

CB Structures, 202 Orlan Rd, New Holland, PA 17557. http://www.cbstructuresinc.com/

Eversole Builders, 2495 Election House Road, Lancaster, OH 43130. http://www.eversolebuilders.com/

FBi Buildings, Inc., 3823 W 1800 S, Remington, IN 47977. http://www.fbibuildings.com/

Fingerlakes Construction Company, 10269 Old Route 31 West, Clyde, NY 14433. http://www.fingerlakesconstruction.com/

Fuog Interbuild Inc., PO Box 237, Purcellville, VA 20134. http://www.fuoginterbuildinc.com

Hochstetler Buildings, Inc., 7927 Memorial Drive, Plain City, OH 43064. http://www.hochstetler.net/

Keystone Barn Supply, LLC "Keystone Barns", New Holland, Pennsylvania 17557, http://www.keystonebarns.com/

Lester Building Systems, 1111 Second Avenue South, Lester Prairie, MN 55354. http://www.lesterbuildings.com/

Meyer Buildings, 444 W 1st Avenue, Dorchester, WI 54425. http://www.meyerbuildings.com/

Morton Buildings, Inc., PO Box 399, Morton, IL 61550-0399. http://mortonbuildings.com/

MPB Builders, Inc., 654 E. Oshkosh St., Ripon, WI 54971, http://mpbbuilders.com/

Quality Structures, 167 HWY 59, Richmond, KS 66080. http://www.qualitystructures.com/

RAM General Contracting Inc., 592 Industrial Drive, P.O. Box 660, Winsted, MN 55395. http://ramgeneralcontracting.com/

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CHAPTER 2

Building Regulations

Contents

2.1 Introduction

Building regulations are those technical documents that control or otherwise guide the design process. This includes standards and an array of laws which are largely based on standards. Laws related to buildings can be largely categorized under active building codes, federal regulations that control construction, and local zoning regulations.

2.2 Standards

2.2.1 Introduction

The true foundation of a building project consists of the numerous documents dictating design and construction methodology. These documents, which continually evolve with time, are referred to as "standards" for no other reason than they help standardize the entire building process. Standardization significantly reduces the amount of communication required to complete a project, and this in turn reduces overall cost, and improves overall structural integrity and construction site safety.

Standards associated with the building process cover four principle areas: construction materials, design and engineering requirements, installation methods, and testing practices. Standards relating to construction materials generally address product quality characteristics such as composition, dimensions and uniformity. Design and engineering related standards provide data and formulas for structural load determination, criteria to assist in modeling and structural analyses, and performance characteristics of specific materials or products to aid in component sizing and layout. Installation standards govern the installation of specific products and systems. Testing standards outline procedures for evaluating structural strength, fire resistance and other performance criteria.

2.1.2 Standards Development

Unlike many other countries, standards development in the United States is not government funded, and thus it is the responsibility of manufacturers, designers and fabricators within the U. S. to organize themselves into associations and develop standards where and when it is clear such standards will be mutually beneficial for their firms. Not surprising, there are now hundreds of non-government organizations (NGOs) in the U.S. that write and maintain standards.

Following is an alphabetic listing of NGOs credited with one or more standards of significance to the design and construction of buildings and other structures. Note that the area(s) of coverage listed for a particular NGO may not be the only one(s) associated with that organization.

- ACI: American Concrete Institute (http://www.concrete.org/) - structural design of concrete
- AISC: American Institute of Steel Construction (http://www.AISC.org/) - structural design of steel
- APA: The Engineered Wood Association (http://www.apawood.org/) - specifications for engineered lumber and wood-based panel products
- ASABE: American Society of Agricultural and Biological Engineers (http://www.asabe.org/) - post-frame building design, live loads
- ASCE: American Society of Civil Engineers (http://www.asce.org/) - structural loads
- ASHRAE: American Society for Heating, Refrigeration and Air Conditioning Engineers (http://www.ashrae.org/) - heating, cooling, insulating, ventilating
- ASME: American Society of Mechanical Engineers (http://www.asme.org/) - plumbing
- ASSE: American Society of Sanitary Engineering (http://www.asse-plumbing.org/) - plumbing
- ASTM: American Society for Testing and Materials (http://www.astm.org/) - testing and material specifications
- AWC: American Wood Council (http://www.awc.org/) - wood design specifications
- AWPA: American Wood Protection Association (http://www.awpa.com/)
- NFPA: National Fire Protection Association (http://www.nfpa.org/) - most everything related to fire including sprinkler systems, alarms, electrical wiring and its protection, oil and gas burning equipment and its piping and venting.
- TPI: Truss Plate Institute (http://www.tpinst.org/) - metal plate connected wood trusses
- UL: Underwriters Laboratories, Inc. (http://www.ul.com/) - electrical appliances, furnaces, fireplaces, water heaters, storage of combustibles, chimneys, fire resistant construction, venting

2.2.4 Structural Design Specifications

Those standards that control structural design are categorically referred to as structural design specifications. This includes, for example, the *National Design Specification (NDS) for Wood Construction* published by the American Wood Council.

2.2.5 ANSI

The only organization in the U.S. fully dedicated to standards is the American National Standards Institute (ANSI) formed in 1916 by five major standard developers (i.e., ASME, IEEE, ASCE, AIME, ASTM) along with the U.S. Departments of War, Navy, and Commerce to "enhance the global competitiveness of U.S. business and the American quality of life by promoting and facilitating voluntary consensus standards and conformity assessment systems and promoting their integrity."

ANSI now accredits standard development organizations (SDOs) that voluntarily subscribe to and operate under its requirements. These accredited organizations may submit standards for review and possible acceptance as American National Standards (ANS). There are now over 200 accredited SDOs (including all previously listed NGOs) and over 10,000 American National Standards. Once a standard has been accepted as an ANS, the number of the standard will generally be prefaced with the "ANSI" initialism. For example, ANSI/ASABE EP559 identifies ASABE EP559 after its acceptance as an ANS. Approval of a standard as an ANS increases its level of recognition, which can enhance the standard's adoption into model codes.

ANSI is also the sole U.S. representative and dues-paying member of the International Organization for Standardization (known worldwide as the ISO) and thus organizes the promotion of American National Standards at an international level. The benefits of this to the U.S. economy are obvious.

2.3 Building Codes

2.3.1 Purpose

A building code is a legal document that helps ensure public health and welfare by specifying a minimum level of protection from physical injury, fire and natural environmental forces. Building codes also help minimize factors that adversely affect indoor air quality and overall sanitation.

2.3.1.1 Protection from Physical Injury

Building codes reduce risk of physical injury in numerous ways. For example, stair, handrail and guardrail requirements help prevent dangerous falls, and headroom requirements limit head injuries. Accessibility requirements such as minimum width requirements for doors and hallways, and height requirements for countertops, enable building occupants to more safely move and work in space. Special floor finish requirements help prevent slips. Safety glazing in glass doors and full-height windows prevent glass from shattering and cutting those who accidently strike them.

2.3.1.2 Fire Protection

Codes include several provisions to protect building occupants from injury or death by fire, and to minimize loss of property when a fire breaks out. These provisions can be categorized into four levels of fire protection. From top, or most important, to bottom these levels are:

1. Keep fire from starting. This is primarily done with electrical wiring and fuel burning requirements.
2. Warn people and get them safely out of a structure or to a safe refuge once a fire starts. Warning is accomplished with smoke detector/alarm requirements. Safe exiting results from requirements for strategically located and properly sized exits and exit corridors, as well as from requirements for systems that block circulation of smoke filled air. Providing a safe means of escape or a safe refuge for the occupants during a fire is accomplished with code provisions that: limit building height; ensure adequate location, size, number and visibility of exits and stairways; and control the density and distribution of building occupants.
3. Extinguish the fire before it spreads. This is most effectively accomplished with the proper installation of automatic sprinklers. Where sprinklers are unable to completely extinguish a fire, they will generally help slow the fire, giving occupants more time to move to a safe location.
4. Minimize property loss. This is accomplished by limiting the size of certain structures, protecting adjoining property (e.g., establishing minimum setback distances and requiring fire resistant exteriors) and by limiting the progress and spread of fire and smoke. The latter is accomplished, for example, by eliminating combustible materials within rooms and structural assemblies, limiting use of smoke and gas producing materials, protecting structural members to prevent collapse from the effects of fire, enclosing vertical openings with fire-resistive construction (firestopping and blocking), and subdividing the building into areas through the use of fire-resistive walls, floors and doors.

2.3.1.3 Protection from the Environment

Building codes provide a defense against our natural environment in several ways. Perhaps most importantly, they specify the snow, wind, seismic and other forces that all code compliant buildings must withstand, and they specify the material design standards that must be used in structural design. Building codes also contain the thermal insulation requirements that protect occupants from temperature extremes. Protection from our natural environment also includes constructing durable roof and wall exteriors that repel water and that limit insect insurgencies.

2.3.2 Model versus Active Codes

There are two categories of codes: model and active. A *model* code is any code that is written for general use. In other words, it is not written for use by a specific state, county, town, village, company or individual. An *active* code is a code that has been adopted and is enforced by a regulatory agency. It follows that acceptance of a model code is voluntary in a given jurisdiction until it is adopted through legislation, and enforced by a regulatory agency in that jurisdiction, at which time it becomes an active code in that jurisdiction. Seldom does a regulatory agency adopt and enforce a model code without first appending or otherwise modifying some of its provisions.

Development, adoption, modification and enforcement of building related-codes exist at all levels of government from local municipalities, to county and state governments. As a result, the content and administration of active building codes varies not only between states, but frequently between municipalities within a state. Some states have established a hierarchy structure of state, county and township/village/city building codes. In this situation, more localized governing areas can modify the state (or county) codes, provided the changes result in more strict provisions. In the vast majority of states, building-related codes are adopted and enforced at the state level. This is most commonly done via *adoption by reference* of a series of major model codes into the state's administrative code. While the federal government also adopts and enforces building codes, it only does so for construction occurring on federal land.

2.3.3 Incorporation of Standards

Standards developed by non-governmental organizations are a major part of many codes. A particular standard may be directly embodied in a code (i.e., reprinted word-for-word) or it may be "adopted by reference" in which case only the standard's designation, title, and developer typically appear in the code. Whether by direct embodiment or adoption by reference, the inclusion of a standard in an active code makes compliance with the standard mandatory in that jurisdiction.

2.3.4 Prescriptive Versus Performance-Based Code Requirements

Prescriptive code requirements are those provisions within a code that spell out (i.e., prescribe) exactly how something must be constructed. For example, a prescriptive code requirement would be one that specified the size and grade of lumber for a specific application, or the type and thickness of gypsum wallboard in a particular room, or the diameter and exact location of bolts in a connection.

A performance-based code requirement (a.k.a. a performance specification) is a provision that establishes the minimum performance level for a building material, component or system. Requiring that a building be designed to withstand a balanced roof snow load of 40 pounds per square foot is a performance-based requirement.

Engineers and architects have more latitude in design where performance-based requirements control. They can design anything they want as long as they meet the performance-based requirement. This is obviously not the case with a prescriptive code requirement which places specific restrictions on the material or equipment used and/or how it must be installed and/or tested. In general, residential building codes tend to have more prescriptive requirements than codes dedicated to commercial construction.

2.3.5 History of Model Codes

Understanding the history of major model building codes in the United States is important as many documents still refer to provisions that were established in model codes that are no longer maintained.

The first major U.S. model building code was published in 1905 by the National Board of Fire Underwriters. This group of fire insurance industry representatives developed the code primarily in response to devastating fires that leveled large sections of Boston, New York, Chicago, Baltimore and San Francisco in the late 1800's.

In the early 1920's, building inspectors began forming what eventually became three major regional model code groups, each developing and maintaining its own model building code. The International Congress of Building Officials (ICBO) published the *Uniform Building Code*, the Building Officials and Code Administrators International (BOCA) produced the *National Building Code,* and the Southern Building Code Congress International (SBCCI) published the *Standard Building Code*. In general, most states west of the Mississippi adopted the ICBO code, northeastern states the BOCA code, and states in the southeast adopted the SBCCI code. Wisconsin and New York were the only two states in the country not to adopt one of these three model building codes, opting instead to write and maintain their own building code.

In addition to their model *building* code, the ICBO, BOCA and SBCCI each offered other model codes. For example, SBCCI also produced the *Standard Mechanical Code, Standard Gas Code*, and *Standard Plumbing Code*. In many cases, these other model codes were developed in collaboration with outside organizations. For example, at one time, the International Association of Plumbing and Mechanical Officials (IAPMO) and ICBO jointly published the *Uniform Mechanical Code*.

In December 1994, the three model code agencies (ICBO, BOCA and SBCCI) founded the International Code Council (ICC) with the purpose of developing a single set of comprehensive and coordinated model construction codes for the United States. In less than six years, the ICC completed their task with the publication of eleven *International Codes* (a.k.a. *I-Codes*).

Beginning in 2003, the ICC has released a new version of their codes every three years. In 2013, there were fifteen I-Codes: the *International Building Code*, the *International Residential Code for One- and Two-Family Dwellings*, the *International Mechanical Code*, the *International Plumbing Code*, the *International Fire Code*, the *International Existing Building Code*, the *International Wildland-Urban Interface Code*, the *International Fuel Gas Code*, the *International Energy Conservation Code*, the *International Private Sewage Disposal Code*, the *ICC Performance Code for Buildings and Facilities*, the *International Property Maintenance Code*, the *International Zoning Code*, the *International Green Construction Code*, and the *International Swimming Pool and Spa Code*.

The National Fire Protection Association (NFPA) is another major developer of model building codes. For more than a century, NFPA has been developing and updating codes and standards concerning all areas of fire safety. They are most widely known for the *National Electric Code* or NEC (NFPA 70) which is the quintessential document in the U.S. when it comes to the installation of electric distribution and equipment wiring within facilities. Currently, there are more than 300 NFPA fire codes used throughout the world, and many of these are active in virtually every jurisdiction in the U.S. In addition to the NEC, other well known NFPA codes include the *Life Safety Code* (NFPA 101), the *Fire Prevention Code* (NFPA 1), and the *National Fuel Gas Code* (NFPA 54).

2.4 International Building Code

2.4.1 Adoption

The most significant model building code in the U.S. is the *ICC International Building Code* which is more commonly known by its initialism IBC. The IBC has now been adopted in all 50 States, the District of Columbia, Guam, the U.S. Virgin Islands, and the Northern Marianas Islands. Federal agencies including the Architect of the Capitol, General Services Administration, National Park Service, Department of State, U.S. Forest Service and the Veterans Administration also enforce the I-Codes. The Department of Defense references the IBC for constructing military facilities, including those that house U.S. troops, domestically and abroad.

2.4.2 General Contents

Included in the IBC are sections on occupancy and construction type, allowable heights and areas, fire-resistant construction, fire protection systems, means of egress, accessibility, interior finishes, interior environment; foundations, exterior wall and roof construction; structural loads; and design with wood, concrete, aluminum, steel, masonry, glass, gypsum board, plaster, and plastic. Although the IBC also includes chapters on energy efficiency, electrical systems, mechanical systems, plumbing systems, and elevator and conveying systems, the material actually appearing in these chapters is extremely limited as these chapters adopt other major model codes by reference. Coverage of structural design with wood, steel, concrete and other materials is also extremely limited as most of this material is also adopted by reference.

2.4.3 Occupancy and Construction Types

For fire safety reasons, building codes limit the size of virtually all buildings, with actual size based largely on what the building will have in it and what materials will be used to construct the building. To account for building contents, codes categorize buildings by occupancy type. To account for the materials used in building construction, codes categorize buildings by construction type.

The combination of occupancy and construction type governs allowable building height and area. If buildings with widely different fire hazards were treated equally, regulations providing for fire protection of the more hazardous group would impose a penalty on construction of buildings housing the lesser hazard, thus needlessly increasing building costs or reducing allowable sizes.

2.4.3.1 Occupancy Types

Table 2-1 contains the occupancy types as defined by the IBC. There are 26 occupancy group designations in ten main categories. As should be evident from Table 2-1 descriptions, buildings in the same group will have similar life-safety characteristics, combustible contents, and fire hazards. In addition to building size, occupancy type controls egress requirements.

2.4.3.2 Construction Types

Buildings are classified by construction type in accordance with the fire resistive characteristics of their structural frames (columns, beams, girders, trusses, and spandrels), bearing walls (i.e. walls that have imposed loads on them), nonbearing walls, floors, and roofs.

Fire resistance characteristics are determined via standard laboratory tests. Main tests include ASTM E119, ASTM E136 and ASTM E84. ASTM E119 is used to evaluate the duration that a building element contains a fire, retains its structural integrity, or exhibits both properties during a prescribed fire test exposure. ASTM E136 is used to determine material combustibility; more specifically; it is used to distinguish between materials which do not act to aid combustion and those that add appreciable heat to an ambient fire. ASTM E84 provides comparative measurements of surface flame spread and smoke density measurements with that of select grade red oak and fiber-cement board surfaces under a specific fire exposure condition.

The IBC uses nine construction type categories. These categories are given in Table 2-2 along with the minimum fire resistive ratings (in hours) for their structural frames, exterior bearing walls, interior bearing walls, floors and roofs as determined in accordance with ASTM E119. It is important to realize that there are several factors that can change the requirements in Table 2-2 and the reader is referred to the IBC for these required adjustments.

The NC (non-combustible) designation that accompanies some of the fire resistance ratings means that in addition to meeting the minimum hourly rating, the assembly must also be constructed of material determined to be non-combustible in accordance with ASTM E136. Falling into this NC category would be such materials as stone, concrete, masonry and steel.

With respect to construction, Type I is considered to be both noncombustible and fire-resistive. Type II is also noncombustible, but has little to no fire-resistance. Type III is construction in which the exterior walls are of noncombustible materials and the interior building elements can be of virtually any material (note here that fire-retardant-treated wood framing is permitted within exterior wall assemblies of a 2-hour rating or less). Type IV is construction featuring exterior walls of noncombustible materials and the interior building elements of heavy timber (HT). Timbers must be solid-sawn or glued laminated members (mechanically-laminated assemblies are not allowed). Columns must not be less than 8 nominal inches in thickness, and floor and roof framing must not be less than 6 inches in nominal thickness. Finally, Type V is construction in which all elements can contain combustible materials. Virtually all post-frame buildings would be classified as Type V buildings.

For each construction type there are two levels of fire resistance rating requirements: A and B. Fire resistance rating requirements for level A are one full hour greater than those for level B. For example, as listed in Table 2-2, the fire resistance rating requirements for elements of a Type VA building are 1 hour whereas those for a Type VB building are zero (0) hours.

Table 2-1. International Building Code (IBC) Occupancy Classifications

Group	Group Designation	Description
Assembly	A-1	Assembly uses, usually with fixed seating, intended for the production and viewing of the performing arts or motion pictures
	A-2	Assembly uses intended for food and/or drink consumption
	A-3	Assembly uses intended for worship, recreation, or amusement and other assembly uses not classified elsewhere in Group A
	A-4	Assembly uses intended for viewing of indoor sporting events and activities with spectator seating
	A-5	Assembly uses intended for participation in or viewing outdoor activities
Business	B	The use of a building or structure, or a portion thereof, for office, professional, or service-type transactions, including storage of records and accounts.
Educational	E	The use of a building or structure, or a portion thereof, by six or more persons at any one time for educational purposes through the 12th grade and child care facilities.
Factory	F-1	Factory industrial uses which are not classified as Factory Industrial F-2 Low Hazard
	F-2	Factory industrial uses that involve the fabrication or manufacturing of noncombustible materials which during finishing, packing, or processing do not involve a significant fire hazard
High Hazards	H-1	Buildings and structures containing materials that pose a detonation hazard
	H-2	Buildings and structures containing materials that pose a deflagration hazard or a hazard from accelerated burning
	H-3	Buildings and structures containing materials that readily support combustion or that pose a physical hazard
	H-4	Buildings and structures which contain materials that are health hazards
	H-5	Semiconductor fabrication facilities and comparable research and development areas in which hazardous production materials (HPM) are used.
Institutional	I-1	Buildings, structures, or parts thereof housing more than 16 persons, on a 24-hour basis, who because of age, mental disability, or other reasons, live in a supervised residential environment that provides personal care services. The occupants are capable of responding to an emergency situation without physical assistance from staff.
	I-2	Buildings and structures used for medical, surgical, psychiatric, nursing, or custodial care on a 24-hour basis for more than five persons who are not capable of self-preservation.
	I-3	Buildings and structures that are inhabited by more than five persons who are under restraint or security. A I-3 facility is occupied by persons who are generally incapable of self-preservation due to security measures not under the occupants' control.
	I-4	Buildings and structures occupied by persons of any age who receive custodial care for less than 24 hours by individuals other than parents or guardians, relatives by blood, marriage, or adoption, and in a place other than the home of the person cared for.
Mercantile	M	Buildings and structures or a portion thereof, for the display and sale of merchandise, and involves stocks of goods, wares or merchandise incidental to such purposes and accessible to the public.
Residential	R-1	Residential occupancies containing sleeping units where the occupants are primarily transient in nature
	R-2	Residential occupancies containing sleeping units or more than two dwelling units where the occupants are primarily permanent in nature
	R-3	Residential occupancies where the occupants are primarily permanent in nature and not classified as Group R-1, R-2, R-4, or I
	R-4	Buildings arranged for occupancy as residential care/assisted living facilities including more than five but not more than 16 occupants, excluding staff.
Storage	S-1	Buildings occupied for storage uses that are not classified as Group S-2
	S-2	Buildings used for the storage of noncombustible materials such as products on wood pallets or in paper cartons with or without single thickness divisions; or in paper wrappings. Such products are permitted to have a negligible amount of plastic trim, such as knobs, handles, or film wrapping.
Utility and Miscellaneous	U	Buildings and structures of an accessory character and miscellaneous structures not classified in any specific occupancy shall be constructed, equipped and maintained to conform to the requirements of this code commensurate with the fire and life hazard incidental to their occupancy.

Table 2-2. Construction Types and Associated Fire Resistance Rating Requirements (from IBC Table 601) [a, b, c]

Construction type	Minimum fire resistance rating (hours)[d] Structural frame	Exterior bearing wall	Interior bearing wall	Floor construction	Roof construction
IA	NC-3	NC-3	NC-3	NC-2	NC-1.5 [e]
IB	NC-2	NC-2	NC-2	NC-2	NC-1 [e, f]
IIA	NC-1	NC-1	NC-1	NC-1	NC-1 [e, f]
IIB	NC-0	NC-0	NC-0	NC-0	NC-0 [f]
IIIA	1	NC-2	1	1	1 [e, f]
IIIB	0	NC-2	0	0	0
IV	HT [g]	NC-2	1/HT[g]	HT [g]	HT [g]
VA	1	1	1	1	1 [e, f]
VB	0	0	0	0	0

a Except for special cases, non-load bearing walls typically have a zero (0) hr fire rating.
b Exterior walls (both load bearing and non-load bearing) must meet requirements of Table 2-3.
c See the IBC for other specific requirements/exceptions.
d NC = Non Combustible. Note: with some minor exceptions, any building element with a NC fire resistance rating requirement must be constructed of noncombustible materials.
e Except in Group F-1, H, M and S-1 occupancies, fire protection of structural members shall not be required, including protection of roof framing and decking where every part of the roof construction is 20 feet or more above any floor immediately below. Fire-retardant-treated wood members shall be allowed to be used for such unprotected members.
f In all occupancies, heavy timber shall be allowed where a 1-hour or less fire-resistance rating is required.
g HT = Heavy Timber as defined in IBC 602.4.

As noted in footnote "a" of Table 2-2, non-load bearing walls – both interior and exterior - typically have a zero (0) hour fire rating. Exceptions for interior walls are generally related to occupancy (e.g., walls between sleeping units in the same R-1 building are required to have a minimum 1 hr fire-resistance rating). For exterior walls (both load and non-load bearing), fire resistance ratings are often dictated by the fire separation distance between adjacent buildings (see Table 2-3).

Table 2-3. Fire-Resistance Rating Requirements for Exterior Walls Based On Fire Separation Distance (from IBC Table 602) [a]

Fire separation distance, X (ft) [b, c]	Construction type	Occupancy Groups: H	F-1, M, S-1	A, B, E, F-2, I, R, S-2, U
		Minimum fire resistance rating (hours)		
X < 5	All	3	2	1
5 ≤ X < 10	IA Others	3 2	2 1	1 1
10 ≤ X < 30	IA, IB IIB, VB Others	2 1 1	1 0 1	1 0 1
X ≥ 30	All	0	0	0

a See the IBC for special requirements/exceptions.
b The distance measured from the building face to (1) the closest interior lot line, (2) the centerline of a street, an alley or public way, or (3) an imaginary property line between two buildings on the property.
c The fire-resistance rating of exterior walls with a fire separation distance of 10 feet or less shall be rated for exposure to fire from both sides. For X greater than 10 feet, the rated for exposure applies to fire from the inside only (however, this takes a special non-symmetrical rated approved assembly).

2.4.4 Building Height and Stories

The IBC defines building height as the vertical distance between the *grade plane* and the mid-height of the highest roof (do not include overhangs when determining roof mid-height). The *grade plane* is defined as the average elevation of the *finished ground level.* For calculation purposes, the finished ground level is defined as the ground elevation at a location six feet from the exterior wall except (1) wherever a property line is within six feet of the building, use the elevation of the property line, and (2) wherever the finished ground surface slopes toward the building, use the elevation of the location where the ground surface and wall intersect. Grade plane and finished ground level are graphically defined in figure 2-1.

The number of stories in a building is equal to the number of stories with floors entirely above the grade plane plus the basement if the floor above the basement is (1) greater than six feet above the grade plane, or (2) greater than six feet above the finished ground level for more than 50% of the building perimeter, or (3) greater than 12 feet above the finished ground level at any point. Figure 2-2 illustrates the three conditions under which a basement is counted as a story.

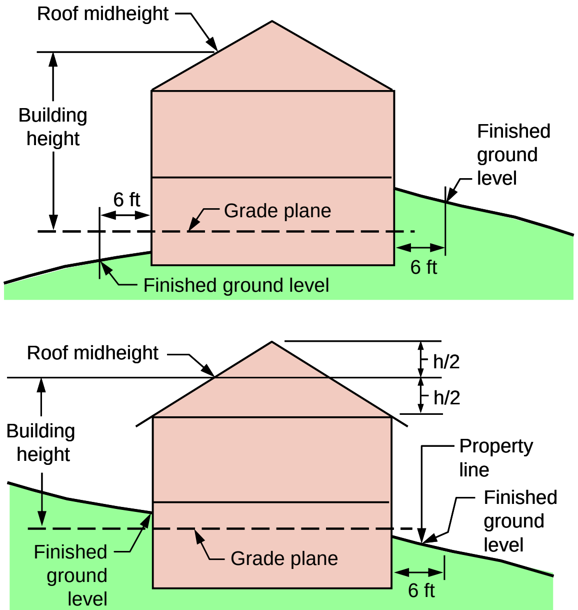


Figure 2-1. Graphical illustration of finished ground level, grade plane, and building height.

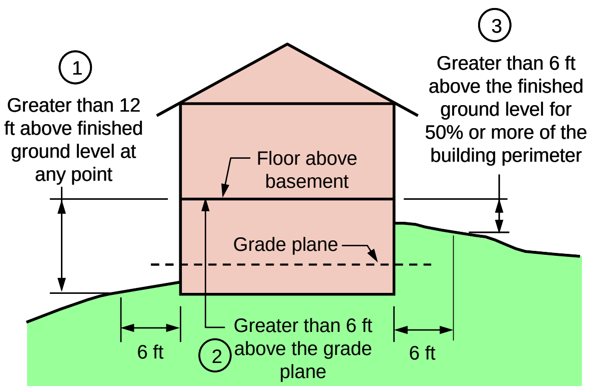


Figure 2-2. Three conditions under which a basement is counted as a building story. Note that only one of the three criteria needs to be met for a basement to be counted as a story.

The maximum height and maximum number of stories in a building is a function of the type of occupancy and type of construction. Table 2-4 contains maximum allowable building heights as specified in the IBC. The limitations in Table 2-4 do not apply to buildings and structures that house special industrial processes that require large areas and unusual building heights to accommodate craneways or special machinery and equipment.

It is important to note that the allowable heights in Table 2-4 can be increased by 20 feet, and the maximum number of stories by one story when the building is protected throughout with an approved automatic sprinkler system. This increase does not apply to Type IIB, III, IV and V buildings with Group I-2 occupancy. Likewise it does not apply to any buildings with H-1, H-2, H-3 or H-5 occupancy.

Roof structures not used for habitation or storage (e.g. towers, spires, steeples) can be unlimited in height if fabricated from noncombustible materials. When such roof structures are fabricated from combustible materials, they cannot extend more than 20 ft above the allowable building height.

2.4.5 Allowable Building Areas

Table 2-5 contains maximum allowable building areas as specified in the IBC. Note that **each part of a building included within "code complying fire walls" shall be considered a separate building.** The limitations in Table 2-5 do not apply to buildings and structures that house special industrial processes that require large areas and unusual building heights to accommodate craneways or special machinery and equipment.

Values in Table 2-5 can be increased due to automatic sprinkler system protection and "street" frontage according to the following formula:

$$A_a = A_t(1 + I_s/100 + I_f/100)$$

where:

A_a = Allowable area per floor, square feet.
A_t = Tabulated area per floor from Table 2-5, square feet.
I_s = Area increase due to sprinkler protection, percent.
= 200% where building is more than one story above the grade plane and protected throughout with an approved automatic sprinkler system.
= 300% where building is no more than one story above the grade plane and is protected throughout with an approved automatic sprinkler system.
I_f = Area increase due to frontage, percent. I_f is equal to zero if less than 25% of the building's perimeter is a public way or accessible open space having a minimum width of 20 feet.
= $100\ (W/30\text{ ft})\ [(F/P) - 0.25]$
W = Minimum width of public way or accessible open space. W must be at least 20 feet and the quantity W divided by 30 ft shall not exceed 1.0.
P = Perimeter of entire building.
F = Building perimeter that fronts on a public way or accessible space having 20 feet open minimum width.

Table 2-4. Maximum Allowable Heights (from IBC Table 503), UL = Unlimited [a]

Use Group	Type of Construction								
	Type I		Type II		Type III		Type IV	Type V	
	A	B	A	B	A	B	HT	A	B
Maximum Height, feet above grade									
	UL	160	65	55	65	55	65	50	40
Maximum Height, stories above grade									
A-1	UL	5	3	2	3	2	3	2	1
A-2	UL	11	3	2	3	2	3	2	1
A-3	UL	11	3	2	3	2	3	2	1
A-4	UL	11	3	2	3	2	3	2	1
A-5	UL	UL	UL	UL	UL	UL	UL	UL	UL
B	UL	11	5	4	5	4	5	3	2
E	UL	5	3	2	3	2	3	1	1
F-1	UL	11	4	2	3	2	3	1	1
F-2	UL	11	5	3	4	3	5	3	2
H-1	1	1	1	1	1	1	1	1	NP
H-2	UL	3	2	1	2	1	2	1	1
H-3	UL	6	4	2	4	2	4	2	1
H-4	UL	7	5	3	5	3	5	3	2
H-5	3	3	3	3	3	3	3	3	2
I-1	UL	9	4	3	4	3	4	3	2
I-2	UL	4	2	1	1	NP	1	1	NP
I-3	UL	4	2	1	2	1	2	2	1
I-4	UL	5	3	2	3	2	3	1	1
M	UL	11	4	4	4	4	4	3	1
R-1	UL	11	4	4	4	4	4	3	2
R-2	UL	11	4	4	4	4	4	3	2
R-3	UL	11	4	4	4	4	4	3	3
R-4	UL	11	4	4	4	4	4	3	2
S-1	UL	11	4	3	3	3	4	3	1
S-2	UL	11	5	4	4	4	5	4	2
U [b]	UL	5	4	2	3	2	4	2	1
U [c]	UL	12	4	2	4	2	4	3	2

a In almost all cases, the allowable height can be increased by 20 feet, and the maximum number of stories by one story when the building is protected throughout with an approved automatic sprinkler system.

b Non-agricultural Group U buildings.

c Group U Agricultural Buildings (from IBC Appendix C) which includes buildings with the following uses: livestock shelters or buildings (including structures and milking barns), poultry buildings or shelters, barns, storage of equipment and machinery used exclusively in agriculture, horticulture structures (including detached production greenhouses and crop protection shelters), sheds, grain silos, stables

.

Based on preceding equations, the allowable floor area, A_a, of a sprinklered building with a tabulated floor area, A_t, of 26,000 ft^2 and a 25 ft wide public way and/or accessible open space along three-quarters of the building's perimeter is 88,833 ft^2 (note: I_s = 200 and I_f = 41.67).

The last line in Table 2-5 gives allowable floor areas for Group U agricultural buildings. These tabulated floor areas are from Appendix C of the IBC.

Group U agricultural buildings and groups F-2 and S-2 buildings, can have unlimited floor area if they are (1) one-story in height, and (2) surrounded and adjoined by public ways or yards not less than 60 feet in width.

Group U agricultural buildings and groups B, F, M and S buildings, can have unlimited floor area if they are (1) protected throughout with an automatic sprinkler, (2) no more than two stories in height, and (3) surrounded and adjoined by public ways or yards not less than 60 feet in width.

A mezzanine that is less than one-third of the floor area,

Table 2-5. Maximum Allowable Building Area (from IBC Table 503)

Use Group	Type of Construction								
	Type I		Type II		Type III		Type IV	Type V	
	A	B	A	B	A	B	HT	A	B
	Maximum Allowable Building Area, Thousand of Square Feet per Story (UL = Unlimited)								
A-1	UL	UL	15.5	8.5	14	8.5	15	11.5	5.5
A-2	UL	UL	15.5	9.5	14	9.5	15	11.5	6
A-3	UL	UL	15.5	9.5	14	9.5	15	11.5	6
A-4	UL	UL	15.5	9.5	14	9.5	15	11.5	6
A-5	UL	UL	UL	UL	UL	UL	UL	UL	UL
B	UL	UL	37.5	23	28.5	19	36	18	9
E	UL	UL	26.5	14.5	23.5	14.5	25.5	18.5	9.5
F-1	UL	UL	25	15.5	19	12	33.5	14	8.5
F-2	UL	UL	37.5	23	28.5	18	50.5	21	13
H-1	21	16.5	11	7	9.5	7	10.5	7.5	NP
H-2	21	16.5	11	7	9.5	7	10.5	7.5	3
H-3	UL	60	26.5	14	17.5	13	25.5	10	5
H-4	UL	UL	37.5	17.5	28.5	17.5	36	18	6.5
H-5	UL	UL	37.5	23	28.5	19	36	18	9
I-1	UL	55	19	10	16.5	10	18	10.5	4.5
I-2	UL	UL	15	11	12	NP	12	9.5	NP
I-3	UL	UL	15	10	10.5	7.5	12	7.5	5
I-4	UL	60.5	26.5	13	23.5	13	25.5	18.5	9
M	UL	UL	21.5	12.5	18.5	12.5	20.5	14	9
R-1	UL	UL	24	16	24	16	20.5	12	7
R-2	UL	UL	24	16	24	16	20.5	12	7
R-3	UL	UL	UL	UL	UL	UL	UL	UL	UL
R-4	UL	UL	24	16	24	16	20.5	12	7
S-1	UL	48	26	17.5	26	17.5	25.5	14	9
S-2	UL	79	39	26	39	26	38.5	21	13.5
U [a]	UL	35.5	19	8.5	14	8.5	18	9	5.5
U [b]	UL	60	27.1	18	27.1	18	27.1	21.1	12

a Non-agricultural Group U buildings.

b Group U Agricultural Buildings (from IBC Appendix C) which includes buildings with the following uses: livestock shelters or buildings (including structures and milking barns), poultry buildings or shelters, barns, storage of equipment and machinery used exclusively in agriculture, horticulture structures (including detached production greenhouses and crop protection shelters), sheds, grain silos, stables.

or an equipment platform that is less than two thirds the floor area of the room or space in which it is located, does not contribute to the building area or number of stories. Where a room contains both a mezzanine and an equipment platform, the two raised floor levels can be ignored in allowable area calculations as long as their aggregate area is less than two-thirds the floor area of the room or space in which they are located.

2.4.6 Buildings with Mixed Occupancy

Many buildings are used for multiple purposes and thus have areas assigned to different occupancy groups. With a few exceptions, different occupancies within the same building must be separated by *fire barriers* and/or *horizontal assemblies* in accordance with the fire-resistance rating requirements in Table 2-6. Each separated occupancy must comply with the building height limitations for the type of construction used. Additionally, in each story of such buildings, the sum of the ratios of the actual building area of each separated occupancy divided by the allowable building area of each occupancy can not exceed 1.0. For a single-story Type VB building that has a total floor area of 10,000 square feet with 30% under a Group B occupancy and the other 70% under a Group F-2 occupancy, this ratio would be an acceptable 0.87 if the allowable area for Group B and Group F-2 are 9000 and 13,000 sq ft, respectively (i.e., 3000/9000 + 7000/13,000 = 0.87 $\leq$ 1.0).

Table 2-6. Required Separation of Occupancies (from IBC Table 508.4)

Occupancy	A, E	I-1, I-3, I-4	I-2	R[b]	F-2, S-2[d], U	B, F-1, M, S-1	H-1	H-2	H-3, H-4	H-5
	Minimum fire resistance rating (in hours) for buildings NOT equipped throughout with an automatic sprinkler system [a]									
A, E	N	—	—	—	—	—	—	—	—	—
I-1, I-3, I-4	2	N	—	—	—	—	—	—	—	—
I-2	NP	NP	N	—	—	—	—	—	—	—
R[b]	2	NP	NP	N	—	—	—	—	—	—
F-2, S-2[c], U	1	2	NP	2[c]	N	—	—	—	—	—
B, F-1, M, S-1	2	2	NP	2	2	N	—	—	—	—
H-1	NP	NP	NP	NP	NP	NP	NP	—	—	—
H-2	4	NP	NP	NP	4	3	NP	NP	—	—
H-3, H-4	3	NP	NP	NP	3	2	NP	NP	NP	—
H-5	NP	NP	NP	NP	NP	NP	NP	NP	NP	NP
	Minimum fire resistance rating (in hours) for buildings equipped throughout with an automatic sprinkler system [a]									
A, E	N	—	—	—	—	—	—	—	—	—
I-1, I-3, I-4	1	N	—	—	—	—	—	—	—	—
I-2	2	2	N	—	—	—	—	—	—	—
R[b]	1	1	2	N	—	—	—	—	—	—
F-2, S-2[d], U	N	1	2	1[c]	N	—	—	—	—	—
B, F-1, M, S-1	1	1	2	1	1	N	—	—	—	—
H-1	NP	NP	NP	NP	NP	NP	N	—	—	—
H-2	3	3	3	3	3	2	NP	N	—	—
H-3, H-4	2	2	2	2	2	1	NP	1	1[e]	—
H-5	2	2	2	2	2	1	NP	1	1	N

a N = No separation requirement, NP = Not permitted.
b See IBC Section 420.
c The required separation from areas used only for private or pleasure vehicles shall be reduced by 1 hour but to not less than 1 hour.
d See IBC Section 406.3.4.
e Separation is not required between occupancies of the same classification.

Some mixed occupancy buildings consist of a main occupancy and several smaller ancillary occupancies. A separation between the main occupancy and ancillary occupancies (i.e. accessory occupancies) is not needed if (1) the main occupancy covers more than 90% of the building area of the story in which the accessory occupancies are also located, and (2) none of the occupancies exceeds its allowable floor area.

2.4.7 Fire Barriers, Walls and Partitions

Fundamental to the fire code provisions of the IBC are definitions for fire barrier, fire wall and fire partition. While these terms may seem equivalent, they are not.

2.4.7.1 Fire Barrier

A fire barrier is defined as "a fire-resistance rated wall assembly of materials designed to restrict the spread of fire in which continuity is maintained." It is important to note that a fire barrier is defined as a wall. Similar functioning assemblies that are run in a horizontal position are defined by the IBC as *horizontal assemblies*.

A fire barrier is used for mixed use occupancy separation in accordance with Table 2-6, and is also used to separate a single use occupancy within a building into smaller *fire areas* (generally in an effort to eliminate the requirement for sprinklers). Other common applications include shaft/exit enclosures, and protection of horizontal exits and exit passageways.

Table 2-7 contains fire-resistance rating for fire barriers and horizontal assemblies. For fire barriers and/or horizontal assemblies separating *fire areas* of mixed occupancies, the highest *fire-resistance rating* of the involved occupancies shall apply.

Table 2-7. Fire-Resistance Rating Requirements for Fire Barriers and Horizontal Assemblies (from IBC Table 707.3.10)

Occupancy Group	Fire-Resistance Rating (hours)
H-1, H-2	4
F-1, H-3, S-1	3
A, B, E, F-2, H-4, H-5, I, M, R, S-2	2
U	1

2.4.7.2 Fire Walls

A fire wall is defined as "a fire-resistance-rated wall having protected openings, which restricts the spread of fire and extends continuously from the foundation to or through the roof, with sufficient structural stability under fire conditions to allow collapse of construction on either side without collapse of the wall."

Each portion of a building separated by one or more code compliant fire walls can be considered a separate building. Thus fire walls are commonly used to separate a building into two separate buildings where it would not be possible to meet maximum allowable area requirements as a single building. Additionally, a fire wall can be used to separate a building into two separate buildings having two different construction types, or to separate an un-sprinklered area of a building from a sprinklered area, therefore, allowing for a fully sprinklered building.

Table 2-8 contains fire resistance requirements for fire walls. Where a fire wall also separates occupancies that are required to be separated by a fire barrier, the most restrictive of each separation applies.

Table 2-8. Fire Wall Fire-Resistance Ratings (from IBC Table 706.4)

Group	Fire-Resistance Rating (hours)
A, B, E, H-4, I, R-1, R-2, U	3[a]
F-1, H-3[b], H-5, M, S-1	3
H-1, H-2	4[b]
F-2, S-2, R-3, R-4	2

a In Type II or V construction, walls shall be permitted to have a 2-hour fire-resistance rating.

b For Group H-1, H-2 or H-3 buildings, also see IBC Sections 415.6 and 415.7.

2.4.7.3 Fire Partitions

A fire partition is defined as "a vertical assembly of materials designed to restrict the spread of fire in which openings are protected." A fire partition wall runs from the foundation or floor/ceiling assembly below to the underside of the floor or roof sheathing, deck or slab above, or to the fire-resistance rated floor-ceiling or roof/ceiling assembly above.

Fire partition walls must have a minimum fire resistance rating of 1 hour. They are required between dwelling units in the same building and between sleeping units in the same building. Corridor walls and walls separating tenant spaces in covered and open mall buildings must also meet fire partition wall requirements.

2.5 Federal Codes

Codes existing at a federal level are embodied in the Code of Federal Regulations (CFR) accessible at http://www.gpoaccess.gov/cfr/. The CFR is divided into 50 titles that represent broad areas subject to federal regulation. Various federal departments and agencies are responsible for the development and enforcement of different CFR sections.

Some CFR provisions have a very direct and/or major impact on structural design and construction. Chief among these is the Occupational Safety and Health Administration (OSHA) section containing safety and health regulations for construction (CFR Title 29, Part 1926). Other departments and agencies with code provisions that influence structural design and construction include the U.S. Department of Commerce (DOC), U.S Department of Energy (DOE), U.S. Department of Agriculture (USDA), U.S. Department of Housing and Urban Development (HUD), Environment Protection Agency (EPA), National Institute of Standards and Technology (NIST), and National Institutes of Health (NIH).

2.6 NFBA Sponsored Fire Tests

2.6.1 One-Hour Post-Frame Fire Wall

In January of 1990, the National Frame Builders Association had Warnock Hersey International, Inc., conduct a one-hour fire endurance test on the exterior wall shown in figure 2-3. The wall met all requirements for a one-hour rating as prescribed in ASTM E-119-88. The wall sustained an applied load of 10,400 lbf per column throughout the test. Copies of the fire test report can be obtained from the NFBA.

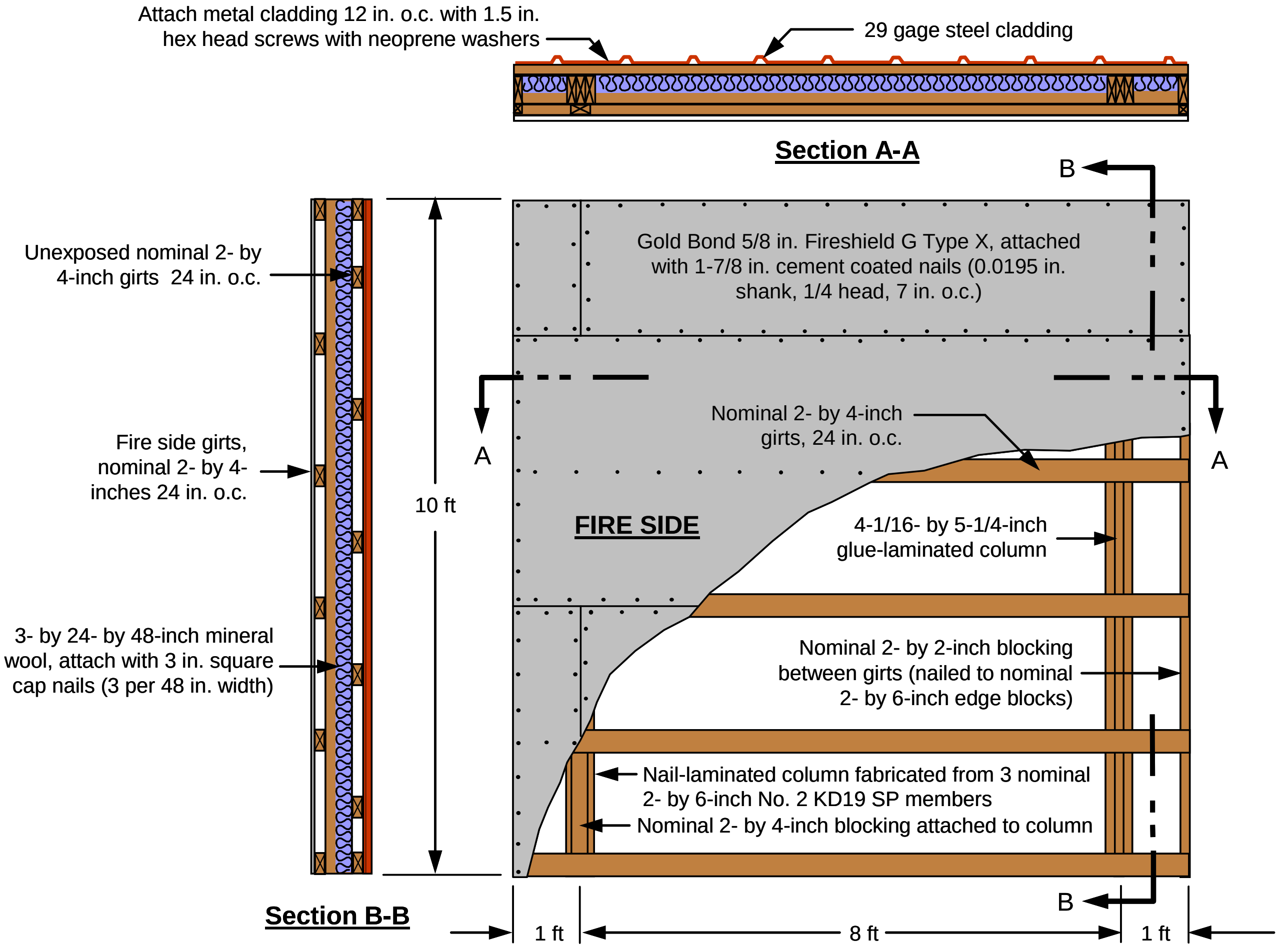


Figure 2-3. Construction details for exterior wall that obtained a one-hour fire-resistance rating during a January 1990 test conducted for the National Frame Builders Association by Warnock Hersey International, Inc. Details of the test are available from NFBA upon request.

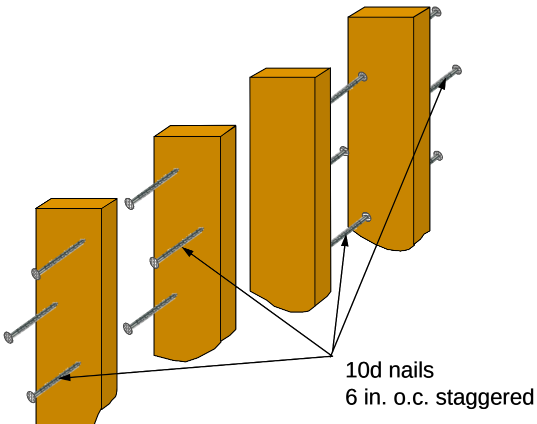
2.6.2 3½-Hour Post-Frame Fire-Wall

In December, 2011 UL conducted a full-scale test for NFBA on the fire wall assembly shown in figure 2-4 (UL, 2012). The goal of the test was to obtain an assembly with a minimum fire-resistance bearing wall rating of three hours. The assembly obtained a 3.5 hr *bearing* wall rating. Use of the term "bearing" in this instance means that under the ASTM E119 fire-resistance test conditions, the wall continued to support its maximum design load for 3.5 hours without passage of flame or gases hot enough to ignite cotton waste, and without the temperature on the unexposed surface (non fire side) increasing more than 250 F.

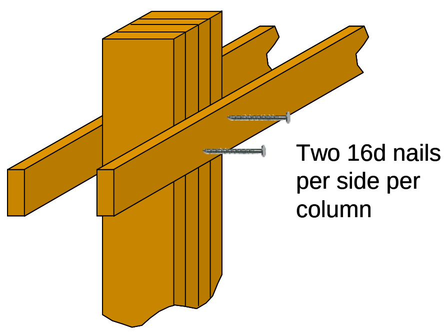
The assembly in figure 2-4 is designated as UL Design Number V304. In addition to the 3.5 h bearing wall rating, the assembly was also assigned a 2 hr "finish rating". The finish rating is the time required to obtain an average temperature rise of 250 degrees (or a single point rise of 325 degrees) between the material on the exposed surface (gypsum wallboard in this case) and the substrate being protected (underlying wood girts).

2.6.2.1 Advantages

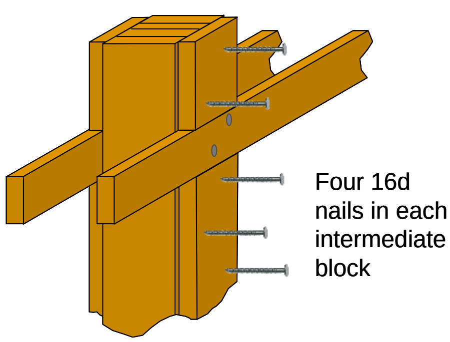
Because post frame buildings are typically classified as Type V buildings, they have some of the lowest allowable areas (Table 2-5). While automatic fire-suppression systems (sprinklers) can be used to increase allowable areas, they can be very costly and impractical, especially in rural areas where access to the large amounts of water required for a properly sized sprinkler system can be financially prohibitive. In such cases it is often more practical to split a building into two separate buildings via a fire wall. For all occupancies except F-2, S-2, R-3 and R-4, the IBC requires that such a fire wall have a minimum fire resistance rating of 3 hours (Table 2-8). Only H-1 and H-2 occupancies are required to have a 4-hour fire resistance rated fire wall.



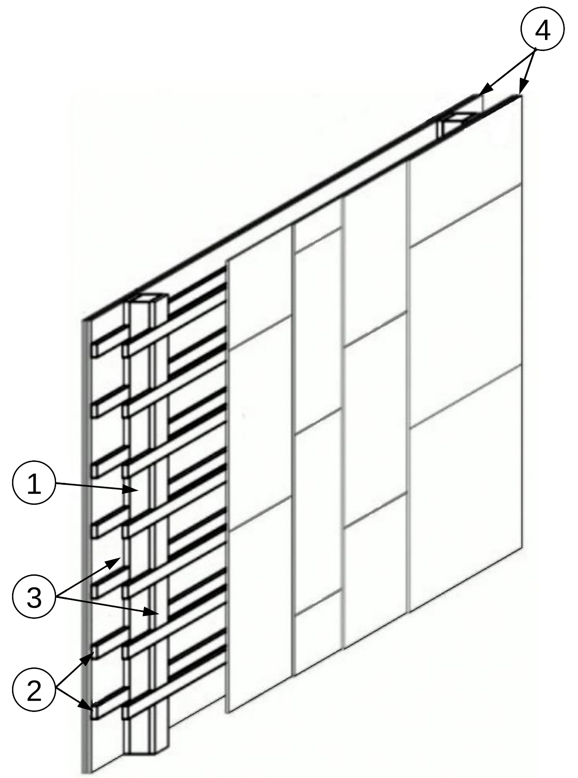
1 Nail-laminated column detail



2 Girt to Column Connection



3 Blocking to Column Connection



1. **Wood Columns** – Four-ply nail-laminated column constructed by nailing two 2x6 studs together with 10d nails spaced 6 in. oc and staggered. Columns spaced a max 96 in. oc and oriented so that the 2 in. side of each member is perpendicular to the horizontal girts.
2. **Wood Girts** - Nominal 2 in. by 4 in. girts are applied horizontally to the face of the columns at 16 in. on center. The girts are applied with two 16d nails per column. The joints are aligned on the vertical center of the columns.
3. **Wood Blocking** - 2 in. by 6 in. blocking vertically-applied to column face between each girt and fastened with four equally-spaced 16d nails
4. **Gypsum Wallboard (GWB)** - Nominal 5/8 in. thick GWB applied horizontally. Joints in adjacent layers are staggered a min. 16 in. and secured to the wood girts as follows: First layer fastened with 2 in. long Type W coarse threaded screws spaced max 24 in. oc. along the horizontal edge and max 8 in. o.c. along the vertical edge to the columns. Second layer fastened with 2-1/2 in. long Type W coarse threaded screws spaced max 24 in. o.c. Third layer fastened with 3 in. Type W coarse threaded screws spaced max 24 in. o.c. The fourth layer fastened with 3-1/2 in. long Type W coarse threaded screws spaced max 12 in. o.c. All screws are offset min. 6 in. from adjacent layers.

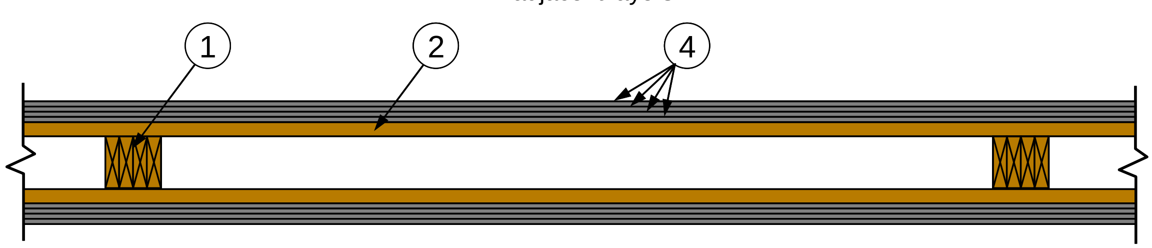


Figure 2-4. Post-frame fire wall with 3.5-hour bearing wall fire resistance rating. UL Design Number V304 (UL, 2012).

Many post-frame buildings are constructed to house occupancy groups S-1, M and F-l (Royer and Stauffer, 2012) – all of which require a minimum 3-hour fire wall. Prior to the approval of UL Design No. V304, only non-wood assemblies constructed using concrete masonry units, or steel studs with Type X gypsum wallboard on both sides were available to designers who required a 3-hour wall.

Because the structural frame of UL Design V304 is typical post-frame, post-frame builders no longer need to hire specialty trades people to build fire walls. This combined with the fact that the fire wall would likely have a foundation identical to that of the rest of the building facilitates more rapid and less expensive assembly.

2.7 Zoning Regulations

2.7.1 Purpose

Zoning laws are established to accomplish one or more community objectives or goals. These include lessening traffic congestion; providing safety from fire, flooding, panic, or other dangers; promoting community health standards; providing adequate lighting; preventing pollution of streams, lakes, and air; preventing unregulated land fills or other disposal activities; and lessoning the cost to the public of transportation, water distribution, sewage, school, park, and other public services.

2.7.2 Typical Contents

Most zoning regulations will:

- Specify height, size, and situation of buildings on lots with reference to streets and property boundaries.
- Regulate the percentage of a lot that may be occupied by a building.
- Specify the size of yards, courts, and other open spaces.
- Control the density of population by residency use in relation to lot size.
- Regulate location and use of buildings, structures and land for trade, industry, residence, or other purposes.
- Divide municipalities into districts of such number, shape, and area as may best be suited to carry out the purposes of land-use planning.

2.7.3 Development and Enforcement

Zoning laws are developed by municipalities. They (and building codes) are principally enforced by the granting of building permits and inspection of construction work in progress. Certificates of occupancy are issued when completed buildings satisfy all regulations.

2.8 Codes and Farm Buildings

2.8.1 Applicable Codes

With few exceptions, model codes written for commercial buildings include verbiage for farm buildings. This includes the IBC which includes farm building under Group U occupancies.

2.8.2 Exemptions

Although the IBC has been adopted in all 50 states, most state and local governments have verbiage in their active codes that exempts "buildings used exclusively for farming purposes" from provisions of the IBC. It is important to note that these same governments seldom exempt farm buildings from other regulations such as the provision of the *National Electric Code*.

The exemption of agricultural buildings from building codes has existed since building codes were first established. This is because the costly conflagrations in the late 1800's that drove development of building codes were unique to urban areas so there was no need to enact them in rural areas.

In modern times, the exemption of farm buildings has more to do with that fact that the loss of an agricultural building seldom presents significant risk to human life. Additionally, many agricultural buildings remain isolated, consequently, if they do start to burn, the fire is relatively easy to contain, and thus loss is generally limited to the building in which the fire started.

Regardless of whether or not a particular agricultural building is exempt from the local code, it should still be fully engineered. As buildings increase in size, a point is reached at which a fully-engineered structure will cost less than a non-engineered structure, or be much safer than the non-engineered structure. This is because non-engineered buildings are often "over-designed" in as many areas as they are "under-designed" in others resulting in no net savings to the consumer who is trying to avoid engineering costs.

The obvious problem with purchasing any non-engineered structure is that they have a higher probability of failure than their engineered counterparts. This probability of failure increases as building size increases. This is because the total number of structural elements in a building is directly related to the size of the structure. With more structural elements in a building, there is an increased probability that one component within the structure may fail. While the failure of a single component in a large building might seem minor, it is not. Failure of a single member or connection in the primary roof framing often triggers the collapse of the entire roof/ceiling assembly.

2.9 Significant Design Documents

Table 2-9 contains the source and a description for several documents related to post-frame building design. The two primary documents in this compilation are the American Wood Councils' (AWC) *National Design Specification® (NDS) for Wood Construction* and the American Society of Civil Engineers' (ASCE) *Minimum Design Loads for Buildings and Other Structures*. Much of this post-frame building design manual is tied to methodology appearing in these two documents.

In addition to the NDS, there are three other major "design specifications" listed Table 2-9. These are the ACI *Building Code Requirements for Structural Concrete,* the AISC *Steel Construction Manual*, and AISI's *North American Specification for the Design of Cold-Formed Steel Structural Members*. None of these three design documents will be overviewed in this manual. They are listed here because one or more of them may be required to complete the design of a post-frame building depending on the inclusion of structural concrete, hot-rolled steel sections, built-up steel sections, and/or cold-formed steel sections in the structure.

A number of shorter documents have been included in Table 2-9. In many cases, these documents contain in-depth information on a subject covered in less detail in one of the more major publications appearing in the table. Most can be downloaded at no cost.

The *Wood Handbook: Wood as an Engineering Material* is an outstanding publication written by employees of the USDA Forest Products Laboratory. It is available for free download and should be part of the library of any individual associated with the use of wood in construction.

The American Society for Testing and Materials (ASTM) is the largest developer of standards in the world. Virtually any material or product used in building construction is subject to requirements of one or more ASTM standards. Despite this fact, Table 2-9 does not contain references to a single ASTM standard. This is because those ASTM standards are either embodied in, or referenced by, other documents listed in the table. Note that it is not uncommon for ASTM standards to appear in a list of building specifications, especially when they govern material characteristics that influence long-term, in-service durability.

Table 2-9. Partial list of technical references related to post-frame building design and construction.

ACI	**American Concrete Institute** 38800 Country Club Drive, Farmington Hills, MI 48331 Phone: 248-848-3700; Fax: 248-848-3701; Website: http://www.concrete.org/ 1. *ACI 318 Building Code Requirements for Structural Concrete and Commentary* Concrete inspection; materials; durability requirements; concrete quality, mixing, and placing; formwork; embedded pipes; construction joints; reinforcement details; analysis and design; strength and serviceability; flexural and axial loads; shear and torsion; development and splices of reinforcement; slab systems; walls; footings; precast concrete; composite flexural members; prestressed concrete; shells and folded plate members; strength evaluation of existing structures; provisions for seismic design; structural plain concrete; strut-and-tie modeling in Appendix A; alternative design provisions in Appendix B; alternative load and strength reduction factors in Appendix C; and anchoring to concrete in Appendix D. 2. *ACI DCCM: Design and Control of Concrete Mixtures* Fundamentals of freshly mixed and hardened concrete including sustainability; durability; materials for making concrete, such as portland cements, supplementary cementing materials, aggregates, water, admixtures, fibers, and reinforcement; procedures for mix proportioning, batching, mixing, transporting, handling, placing, consolidating, finishing, and curing concrete. 3. *ACI 360R-10 Guide to Design of Slabs-on-Ground* Planning, design, and detailing of slabs. Background information on design theories is followed by discussion of the types of slabs, soil-support systems, loadings, and jointing. Design methods are given for unreinforced concrete, reinforced concrete, shrinkage-compensating concrete, post-tensioned concrete, fiber-reinforced concrete slabs-on-ground, and slabs-on-ground in refrigerated buildings, followed by information on shrinkage and curling.

Table 2-9 cont. Partial list of technical references related to post-frame building design and construction.

AISC	**American Institute of Steel Construction** One East Wacker Drive Suite 700, Chicago, IL 60601-1802 Phone: 312-670-2400; Fax: 312-670-5403; Website: http://www.aisc.org/ 1. *AISC Steel Construction Manual* Covers both ASD and LRFD for hot-rolled and built-up steel sections. Includes the following chapters: dimensions and properties; general design considerations; design of flexural members; design of compression members; design of tension members; design of members subject to combined loading; design considerations for bolts; design considerations for welds; design of connecting elements; design of simple shear connections; design of flexible moment connections; design of fully restrained moment connections; design of bracing connections and truss connections; design of beam bearing plates, column base plates, anchor rods, and column splices; design of hanger connections, bracket plates, and crane-rail connections; specifications and codes; miscellaneous data and mathematical information; index and general nomenclature.
AISI	**American Iron and Steel Institute** 25 Massachusetts Ave. NW, Suite 800 Washington, DC 20001 Phone: 202-452-7100; Websites: http://www.steel.org and http://www.mysteelworks.org 1. *North American Specification for the Design of Cold-Formed Steel Structural Members* Covers LRFD and ASD for cold-formed steel members. Includes the following sections: general provisions; elements; members; structural assemblies; connections and joints; tests for special cases; and design of cold-formed steel structural members and connections for cyclic loading (fatigue). Appendices A, B and C cover provision applicable to the United States, Canada, and Mexico, respectively.
AITC	**American Institute of Timber Construction** 7012 S. Revere Parkway, Suite 140, Englewood, CO 80112 Phone: 303-792-9559; Fax: 303-792-0669; Website: http://aitc-glulam.org 1. *AITC Timber Construction Manual* Definitive design and construction industry source for building with wood, both sawn lumber and structural glued laminated timber. 2. *AITC 104 Typical Construction Details* Includes detailed sketches and descriptive information for connections of glued laminated timber members including: beams to masonry; cantilever beams; beam and purlin hangers; beam to columns; column anchorage; arch anchorage; arch connections; truss connections; suspended loading connections. Describes various connections that should be avoided and details the protection of glulam from decay. 3. *AITC 109 Standard for Preservative Treatment of Structural Glued Laminated Timber* The uses of preservative treatments for the protection of glued laminated timber to resist decay are discussed. Topics discussed are design considerations, species of wood, types of preservative treatment, requirements for retention, penetration, certification and marking, incising, treatment prior to bonding, adhesives and bonding processes, fabrication and machining, care after treatment, field treatment and exudation of natural wood resin. 4. *AITC 112 Standard for Tongue-and-Groove Heavy Timber Roof Decking (free download)* This standard applies to solid sawn tongue-and-groove heavy timber decking. Its provisions are not applicable to laminated timber decking. Discusses species, sizes and patterns, lengths, moisture content, applications, specifications, weights of installed decking and allowable load tables for nominal 2, 3 and 4 inch thickness decking.

Table 2-9 cont. Partial list of technical references related to post-frame building design and construction.

AITC

5. ANSI/*AITC A190.1 American National Standard, Structural Glued Laminated Timber*
   The primary reference standard for manufacturing and quality control requirements for glued laminated timber. Topics addressed are lumber, adhesives, grading, end jointing, face laminating, finishing, marking, qualification of manufacturers and process quality control. Recommended for all manufacturers of glulam assemblies.

APA

APA - The Engineered Wood Association

P.O. Box 11700, 7011 South 19th Street, Tacoma, WA 98466-5333
Phone: 253-565-6600; Fax: 253-565-7265; Website: http://www.apawood.org/

1. *Form E30: APA Engineered Wood Construction Guide* (free download)
   Comprehensive guide to engineered wood construction systems for both residential and commercial/industrial buildings. Includes much of the material contained in other APA documents. Covers selection and specification of panels, glulam, I-Joist, and SCL. Also floor, wall and roof construction details and building requirements and related panel systems.
2. *Form C415 Glulam Floor Beams* (free download)
   Span tables for glulam floor beams in residential construction, tables for substituting glulam for steel beams, and design details.
3. *Form D485 Technical Note: Corrosion-Resistant Fasteners for Construction* (free download)
   Basic recommendations for fastener used in plywood siding and foundations, and fire-retardant-treated plywood.
4. *Form D510 Panel Design Specification* (free download)
   Design capacities (ASD) and design methods for wood structural panels manufactured under Voluntary Product Standards PS1 (Form L870) and PS2 (Form S350), and APA Performance Standard PRP-108 (Form E445).
5. *Form D710 I-Joist Construction Details - Performance Rated I-Joists in Floor and Roof Framing* (free download)
   Recommended construction details for I-joists in floor and roof applications. Also includes details for cantilevers and web holes.
6. *Form E445 Performance Standards and Qualification Policy for Structural-Use Panels* (free download)
7. APA PRP-108 - Describes requirements and test methods for APA Performance Rated Panels.
8. *Form EWS T300 EWS Technical Note: Glulam Connection Details* (free download)
   Illustrations of correct and incorrect ways to make a connection involving glulam members.
9. *Form EWS X440 Glulam Product Guide* (free download)
   Describes APA EWS trademarked glulam, addresses important design considerations, and includes a specification guide.
10. *Form EWS X720 PRI-400 Performance Standard for APA EWS I-Joists*
    Performance criteria, qualification requirements, and quality assurance information for APA Performance Rated I-Joists.
11. *Form EWS Y117 Glulam Design Specification* (free download)
    Glulam layup principles, allowable stresses, specification guidelines and design values
12. *Form G310 Wind-Rated Roofs: Designing Commercial Roofs to Withstand Wind Uplift Forces* (free download)
    Tested assembly details for roof systems with APA wood structural panels used as substrate that meet classifications of FM Approvals or of Underwriters Laboratories.

Table 2-9 cont. Partial list of technical references related to post-frame building design and construction.

APA	13. *Form H335 Technical Note: Wood Structural Panel Sheathing or Siding Used to Resist Combined Shear & Uplift* (free download) Determination of the number and location of uplift fasteners that must be used in combination with fasteners used to resist shear. 14. *Form K325 Designing for Combined Shear and Wind Uplift* (free download) Introduces a straightforward solution for builders to meet building code requirements for structural wall systems in high wind areas. The APA system for combined shear and wind uplift is a prescribed three-step design process that builders can follow with minimal added engineering. 15. *Form L350 Diaphragms and Shear Walls* (free download) Design and construction recommendations for engineered diaphragm systems in floor, shear wall, and roof systems. 16. *Form L870 Voluntary Product Standard, PS 1, Structural Plywood* (free download) Covers Voluntary Product Standard PS 1 - the national standard for producing, marketing, and specifying plywood for construction and industrial uses. 17. *Form Q225. Technical Note: Load-Span Tables for APA Structural-Use Panels* (free download) Uniform load design capacities for various span ratings, section properties and panel thicknesses calculated using Panel Design Specification (Form D510) 18. *Form S350 Performance Standard for Wood-Based Structural-Use Panels* (free download) Covers Voluntary Product Standard PS 2 and describes the requirements for producing and testing wood-based structural-use panels. 19. *Form S475 Glued Laminated Beam Design Tables* (free download) Glued laminated beam design tables provide recommended preliminary design loads for two of the most common glulam beam applications: roofs and floors. The tables include values for section properties and capacities, and allowable loads for simple span and cantilevered beams. 20. *Form SR-101 APA System Report: Design for Combined Shear and Uplift from Wind* (free download) Designs for utilizing wind uplift resistance capabilities from wood structural panels, in addition to resisting lateral shear forces and wind pressure perpendicular to the wall. 21. *Form T325 Data File: Roof Sheathing Fastening Schedules for Wind Uplift* (free download) Nailing schedules for wood structural panel roof sheathing for high wind areas. 22. *Form TT-111 Technical Topics: Wood Moisture Content and the Importance of Drying in Wood Building Systems* (free download) Guidance on how to avoid potential moisture problems that could lead to costly and hazardous deterioration as well as health risks when using wood structural products with impermeable materials. 23. *Form X305 Introduction to Lateral Design* (free download) Explains how to design wood-frame buildings to withstand the lateral loads typical of high wind and seismic zones. 24. *Form X505 Panel Handbook & Grade Glossary* (free download) Panel and construction terminology explained in easy-to-understand language. 25. *Form Y391 Technical Note: Structural Adhesives for Plywood-Lumber Assemblies* (free download) Covers structural adhesives for strength and stiffness, and semi-structural adhesives for stiffness only. 26. *Form Z416 Data File: Nailed Structural-Use Panel and Lumber Beams* (free download) Load-span tables and fabrication information for beams fabricated by nailing structural panels to dimension lumber. 27. *Form Z725 APA Performance Rated I-Joists* (free download) Includes information on span ratings, installation details, cantilever designs, architectural specifications and engineering design properties for APA Performance Rated I-Joists

Table 2-9 cont. Partial list of technical references related to post-frame building design and construction.

ASABE

American Society of Agricultural and Biological Engineers
2950 Niles Road, St. Joseph, MI 49085-9659
Phone: 269-429-0300; Fax: 269-429-3852; Website: http://www.asabe.org/

1. *ANSI/ASABE S618 Post Frame Building System Nomenclature*
   Establishes uniformity in terms used in the design, construction, marketing and regulation of post frame building systems.
2. *ANSI/ASAE EP484.2 Diaphragm Design of Metal-Clad, Wood-Frame Rectangular Buildings*
   Analysis and design of single story, rectangular metal-clad wood-frame buildings using roof and ceiling diaphragms, alone or in combination. The roof (and ceiling) diaphragms, endwalls, intermediate shearwalls, and building frames are the main structural elements of a structural system used to efficiently resist the design lateral (wind) loads.
3. *ANSI/ASAE EP486.2 Shallow Post and Pier Foundation Design*
   Design procedure for shallow post and pier foundations that resist moments and lateral and vertical forces acting on them. The design procedure provides necessary definitions, material requirements, and design equations for post and pier foundations.
4. *ASAE EP558 Load Tests for Metal-Clad Wood-Frame Diaphragms*
   Test method for determination of the in-plane strength and stiffness of a metal-clad wood-frame diaphragm assembly.
5. *ANSI/ASAE EP559.1 Design Requirements and Bending Properties for Mechanically-Laminated Wood Assemblies*
   Guidelines for designing and calculating allowable bending properties of 3- and 4-layer mechanically laminated wood assemblies subjected to uni-axial bending about their strong axis
6. *ANSI/ASAE EP378.4 Floor and Suspended Loads on Agricultural Structures Due to Use*
   Recommended design loads resulting from livestock, suspended caged poultry, vehicles, and manure stored on a floor.
7. *ASAE EP393.3 Manure Storages*
   Recommendations for siting, design, and construction of both earthen and fabricated manure storage units.
8. *ANSI/ASAE EP433 Loads Exerted by Free-Flowing Grain on Bins*
   Methods of estimating the grain pressures within centrally loaded and unloaded bins used to store free-flowing, agricultural whole grain.
9. *ANSI/ASAE EP446.3 Loads Exerted by Irish Potatoes in Shallow Bulk Storage Structures*
   Guidelines for calculating loads on vertical and inclined walls, partitions, bin fronts, ducts, and appurtenances that are to resist lateral pressure of potatoes stored in bins that are wider than deep and not deeper than 5.5 m (18 ft)
10. *ASAE EP538.2 Design Loads for Bunker (Horizontal) Silos*
    Design loads for the walls of bunker (horizontal) silos for storing whole plant silages. Does not include hydrostatic pressures that may occur when the silage becomes saturated.
11. *ANSI/ASAE EP545 Loads Exerted by Free-Flowing Grain on Shallow Storage Structures*
    Methods of estimating the grain pressures within shallow storage structures used to store free-flowing, agricultural whole grains.

ASCE

American Society of Civil Engineers
1801 Alexander Bell Drive, Reston, Virginia 20191-4400
Phone: 800-548-2723; Website: http://www.asce.org/

1. ASCE/SEI 7 Minimum Design Loads for Buildings and Other Structures
   Means for determining dead, live, soil, flood, wind, snow, rain, atmospheric ice, and earthquake loads, as well as their combinations.

Table 2-9 cont. Partial list of technical references related to post-frame building design and construction.

AWC

American Wood Council

222 Catoctin Circle SE, Suite 201 Leesburg, VA 20175
Phone: (202) 463 2766, Fax: (202) 463-2791, Website: http://www.awc.org/

1. *ANSI/AWC NDS-2012 National Design Specification (NDS) for Wood Construction with Commentary*
   The NDS is adopted in all model building codes in the U.S. and is used to design wood structures worldwide. ANSI/AWC NDS-2012, was approved as an ANSI American National Standard on August 15, 2011. The 2012 NDS was developed by the American Wood Council's (AWC) Wood Design Standards Committee and is referenced in the 2012 International Building Code.
2. *NDS Supplement – Design Values for Wood Construction, 2012 Edition*
   Design provisions in the NDS are integral with design values in the NDS Supplement. As such, it is not appropriate to mix design values and provisions from different editions of the NDS. For example, the 2001 NDS Supplement contains increased shear design values for sawn lumber to reflect changes in ASTM D245 and provisions of the 2001 NDS were revised to address these increases.
3. *Special Design Provisions for Wind and Seismic (SDPWS) Standard with Commentary* (free view only download)
   The AWC SDPWS-08 covers materials, design and construction of wood members, fasteners, and assemblies to resist wind and seismic forces. Engineered design of wood structures to resist wind or seismic forces is either by allowable stress design (ASD); or load and resistance factor design (LRFD).
4. *ASD/LRFD Manual for Engineered Wood Construction, 2012 Edition* (free download)
   The ASD/LRFD Manual contains design information for structural lumber, glued laminated timber, structural-use panels, shear walls and diaphragms, poles and piles, I-joists, structural composite lumber and over 40 details are included in the chapter on connections.
5. *Structural Wood Design Solved Example Problems, 2005 Edition*
   Structural Wood Design Solved Example Problems is intended to aid instruction on structural design of wood structures using both allowable stress design and load and resistance factor design. Forty example problems allow direct side-by-side comparison of ASD and LRFD for wood structures.
6. *ANSI / AF&PA PWF-2007 - Permanent Wood Foundation Design Specification*
   Structural design requirements for load-bearing wood-frame wall and floor systems designed for both above and below-grade use as a foundation for light frame construction.
7. *DA 1 - Application of Technical Report 12 for Lag Screw Connections* (free download)
   Example calculations of reference lateral design values for lag screws and wood screws that account for the differences in applied moment and bearing resistance of the threaded and unthreaded portions of the fastener
8. *DA 4 - Post Frame Rink Shank Nails* (free download)
   Reference design values for post frame ring shank nails manufactured in accordance with ASTM F1667. Tabulated values are calculated in accordance with the 2005 NDS yield limit equations.
9. *DA 6 - Beam Design Formulas with Shear and Moment Diagrams* (free download)
   Shear and moment diagrams with accompanying formulas for design of beams under various static loading conditions. Configurations include simple span, cantilever, and 2-span continuous beams.
10. *DA 8 - Interior Shear Walls* (free download)
    This document describes how interior shear walls are designed to resist lateral loads
11. *DCA 1 - Flame Spread Performance of Wood Products* (free download)
    Provides building code accepted flame spread ratings for various wood products and species which are normally used as interior finishes for walls, ceilings, and floors in buildings.
12. *DCA 2 - Design of Fire-Resistive Exposed Wood Members* (free download)
    Illustrates how exposed heavy timber and glued laminated columns and beams can be designed to meet building code fire resistance requirements
13. *DCA 3 - Fire Rated Wood Floor and Wall Assemblies* (free download)
    Describes how interior and exterior wood-frame walls and wood I-joist floors can be used to meet building code requirements for fire resistive assemblies.

Table 2-9 cont. Partial list of technical references related to post-frame building design and construction.

AWC

14. *DCA 4 - CAM for Calculating and Demonstrating Assembly Fire Endurance* (free download)
Describes a procedure to calculate the fire endurance rating of a wood-frame wall, roof, or floor/ceiling assembly. The procedure is based on combining previously determined fire resistance time values of each separate component of the assembly without the need for additional fire testing.
15. *DCA 5 - Post-Frame Buildings* (free download)
Provides guidance to post-frame building designers for meeting the requirements of the 2000 International Building Code and to confirm that a properly designed post-frame building is in fact code compliant.
16. *DCA 6 - Prescriptive Residential Deck Construction Guide* (free download)
Includes guidance on provisions of the International Residential Code (IRC) pertaining to single level residential wood deck construction. Provisions contained in this document that are not included in the IRC are considered good practice recommendations.
17. *TR 10 - Calculating the Fire Resistance of Exposed Wood Members* (free download)
Provides information for adjusting section properties and allowable stresses to account for a reduced member cross-section due to charring.
18. *TR 12 - General Dowel Equations for Calculating Lateral Connection Values* (free download)
Calculation of lateral values for single dowel type fastener connections using a generalized and expanded form of the NDS yield limit equations. These general dowel equations apply to NDS connection conditions, but also permit rational and consistent treatment of gaps and fastener moment resistance, and consideration of various connection limit states
19. *TR 14 - Designing for Lateral-Torsional Stability in Wood Members* (free download)
Describes the basis of the current effective length approach used in the NDS and summarizes the equivalent uniform moment factor approach; provides a comparison between the two approaches; and proposes modification to NDS design provisions.
20. *WCD 2 - Tongue and Groove Roof Decking* (free download)
Contains everything needed to design and construct tongue and groove wood roof decking, including span and load tables.
21. *WCD 5 - Heavy Timber Construction* (free download)
Defines minimum requirements for heavy timber construction, and provides illustrations of good construction details
22. *WCD 6 - Design of Wood Frame Structures for Permanence* (free download)
Recommendations for control of moisture and protection against decay and insect infestations

AWPA

American Wood Preservers Association
P.O. Box 361784, Birmingham, AL 35236-1784
Phone: 205-733-4077; Fax: 205-733-4075; Website: http://www.awpa.com/

1. *AWPA U1-10 Use Category System: User Specification for Treated Wood*
Preservative treatment levels for wood as a function of end use (i.e., use category designation), wood species and preservative treatment type.
2. *AWPA T1-10 Use Category System: Processing and Treatment Standard*
Governs preservative retention and penetration requirements, processing limitations, and quality control and inspection requirements for treated wood.

Table 2-9 cont. Partial list of technical references related to post-frame building design and construction.

FPL	**Forest Products Laboratory** One Gifford Pinchot Drive, Madison, WI 53726 Phone: 608-231-9200; Fax: 608-231-9592; Website: http://www.fpl.fs.fed.us/ 1. *Wood Handbook: Wood as an Engineering Material* (free download) Presents properties of wood and wood-based products of particular concern to the architect and engineer. Includes discussion of designing with wood and wood-based products along with some pertinent uses.
GA	**Gypsum Association** 6525 Belcrest Road, Suite 480, Hyattsville, MD 20782 Phone: 301-277-8686; Fax: 301-277-8747; Website: http://www.gypsum.org/ 1. *GA-530 Design Data -Gypsum Board* A complete collection of current Gypsum Association publications compiled into a handy 3-ring binder. 2. *GA-600-09 Fire Resistance Design Manual* A compilation of tested gypsum-board designed fire-rated protection systems. Contains almost 400 systems that may be used for fire-rated walls and partitions, floor/ceiling systems, roof/ceiling systems, and to protect columns, beams, and girders.
ICC	**International Code Council** 500 New Jersey Avenue, NW, 6th Floor, Washington, DC 20001 Phone: 888-422-7233; Fax: 202-783-2348; Website: http://www.iccsafe.org/ 1. *I-Codes* See Section 2.35
NFBA	**National Frame Building Association** 8735 W Higgins Road, Chicago, IL 60631 Phone: 800-557-6957; Fax: 847-375-6495; Email: nfba@nfba.org; Website: http://www.nfba.org/ 1. *NFBA Post Frame Building Design Manual* Introduction to post-frame buildings and their design with chapters specifically dedicated to diaphragm design, metal-clad wood-frame diaphragm properties, post properties, and post foundation design. 2. *NFBA - Accepted Practices for Post-Frame Building Construction: Metal Panel and Trim Installation Tolerances* Controls and/or limits for the relative orientation and spacing of panel and trim, mechanical fastener placement, and panel and trim surface blemishes for post-frame buildings. Provisions only apply to installation of exterior metal panel and exterior metal trim with a nominal base metal thickness less than 0.05 inches (1.20 mm). Fastener criteria only apply to exposed (i.e. through-panel) fasteners. 3. *NFBA - Accepted Practices for Post Frame Building Construction: Framing Tolerances* Recommended tolerances for construction of primary and secondary wood framing in post-frame buildings. 4. *Post-Frame Construction guide* (free download) Introduction to the advantages and structural components of post-frame building systems.

Table 2-9 cont. Partial list of technical references related to post-frame building design and construction.

NRCS

Natural Resources Conservation Service
National Design, Construction, and Soil Mechanics Center
501 W Felix Street, Bldg 23, Fort Worth, TX 76115
Phone: 817-509-3752, Fax: 817-509-3753, Website: www.nrcs.usda.gov/wps/portal/nrcs/main/national/ndcsmc

1. *Conservation Practice Standard 313 – Water Storage Facilities* (free download)
   Location, size, foundation liquid tightness, structural loading and structural design requirements for for facilities used to temporarily store wastes such as manure, wastewater, and contaminated runoff as part of an agricultural waste management system.
2. *Conservation Practice Standard 367 – Roofs and Covers* (free download)
   Material, design life, structural loads, and structural design requirements for rigid and semi-rigid roofs and flexible covers used to protect facilities used for: water quality improvement; diversion of clean water from barnyard, feedlot, and animal exercise areas; waste storage facilities; capture of biogas for energy production; and air quality improvement and odor reduction.

NFPA

National Fire Protection Association
1 Batterymarch Park, Quincy, MA 02169-7471
Phone: 617-770-3000; Fax: 617 770-0700; Website: http://www.nfpa.org/

1. *NFPA 1 Fire Prevention Code*
   Includes, but is not limited to: building inspection; fire investigation; building plan review for life safety and fire systems; fire and life safety education; design of existing means of egress; design, maintenance, and testing of fire protection systems and equipment; fire access requirements; regulation of special events; interior finish requirements; storage, use, processing and handling of flammable and combustible gases, liquids, and solids and hazardous materials.
2. *NFPA 13 Installation of Sprinklers*
   Minimum requirements for the design and installation of automatic fire sprinkler systems, along with acceptable sprinkler systems and components and design development alternatives.
3. *NFPA 70 National Electrical Code*
   Installation requirements for electrical conductors, equipment, and raceways; signaling and communications conductors, equipment, and raceways; and optical fiber cables and raceways.
4. *NFPA 54 National Fuel Gas Code*
   Installation requirements for fuel gas piping systems, appliances, equipment, and related accessories.
5. *NFPA 72 National Fire Alarm and Signaling Code*
   Application, installation, location, performance, inspection, testing, and maintenance of fire alarm systems, supervising station alarm systems, public emergency alarm reporting systems, fire warning equipment and emergency communications systems, and their components.
6. *NFPA 101 Life Safety Code*
   Addresses those construction, protection, and occupancy features necessary to minimize danger to life from the effects of fire, including smoke, heat, and toxic gases created during a fire. This includes minimum criteria for the design of egress facilities so as to allow prompt escape of occupants from buildings or, where desirable, into safe areas within buildings.

SPIB

Southern Pine Inspection Bureau
4709 Scenic Highway, Pensacola, Fl 32504-9094
Phone: 850-434-2611; Website: http://www.SPIB.org/

1. *Standard Grading Rules for Southern Pine Lumber*
   Rules under which the production of Southern Pine is classified and sold
2. *Standard Southern Pine Patterns (free download)*
   Standard patterns for panel, flooring, decking siding.

Table 2-9 cont. Partial list of technical references related to post-frame building design and construction.

SFPA	**Southern Forest Products Association** and **Southern Pine Council** 6600 Riverside Drive, Suite 212, Metaire, LA 70003 Phone: 504/443-4464, Website: http://www.southernpine.com/ or http://www.SFPA.org/ 1. *Southern Pine Use Guide* (free download) Complete grade descriptions, design values and sample specifications for a wide range of uses. Also includes treated Southern Pine, storage, fire and sound data. 2. *Maximum Spans for Southern Pine Joists and Rafters* (free download) 48 tables providing a convenient reference of joist and rafter spans for specific grades of Southern Pine visually and mechanically graded lumber 3. *Raised Floor Design/Construction Guide* (free download) Comprehensive guide to building a raised floor system including site prep, soil analysis, materials specification, foundation options, span tables, design loads, moisture control and cost comparisons. Includes 25 construction details. 4. *Southern Pine Headers and Beams* (free download) Comprehensive series of size selection and allowable load tables for choosing the proper header or beam in specific applications. Southern Pine glued laminated timber is included as well as single and multiple-member Southern Pine lumber. 5. *Pressure-Treated Southern Pine* (free download) A guide to specification of treated wood for various uses and exposures, supplemented with detailed preservative tables by specific product or application. Details of approved industry standards, proper grade and quality marks for treated lumber are included, plus guidance on suitable fasteners and connectors. 6. *Permanent Wood Foundation Design and Construction Guide* (free download) Design specifications and structural requirements for using PWF systems. Typical applications are illustrated

TPI	**Truss Plate Institute** 218 North Lee Street, Suite 312, Alexandria, VA 22314 Phone: 703-683-1010; Fax: 866-501-4012; Website: http://www.tpinst.org/; E-mail:info@tpinst.org 1. *ANSI/TPI 1-2007: National Design Standard for Metal Plate Connected Wood Truss Construction* Describes materials used in a truss, both lumber and steel, and design procedures for truss members and joints. Also includes professional responsibilities, methods for evaluating metal connector plates, and manufacturing quality assurance for trusses. 2. *TPI/WTCA Building Component Safety Information (BCSI): Guide to Good Practice for Handling, Installing, Restraining & Bracing of Metal Plate Connected Wood Trusses* (free download) Information regarding the handling, installation, restraining and bracing of metal plate connected wood trusses. Includes information on hoisting and placement of truss bundles, long span truss installation, hip set assembly and bracing, design and installation of permanent restraint/bracing/reinforcement for trusses and individual truss members, and toe-nailed connections for attaching trusses at bearing locations. 3. *DSB-89: Recommended Design Specification for Temporary Bracing of Metal Plate Connected Wood Trusses* A specification developed primarily for technical audiences such as engineers and architects of record and truss design engineers. The specification covers the determination of temporary bracing requirements for symmetrical dual pitched triangular, mono pitched triangular, scissors, and 2x4/2x6 parallel chord metal plate connected wood trusses spaced four feet on center or less. Includes a supplement with temporary bracing tables for use in lieu of specific design analysis for a particular metal plate connected wood truss installation system.

Table 2-9 cont. Partial list of technical references related to post-frame building design and construction.

UL	**Underwriters Laboratories, Inc.** 333 Pfingsten Road, Northbrook, IL 60062-2096 Phone: 847-412-0136; Website: http://www.ul.com/ 1. *Fire Resistance Directory* Volume 1 covers hourly fire ratings for beams, columns, floors, roofs, walls and partitions. Volumes 2A and 2B include fire rated systems; such as construction joints, through penetration fire stops, perimeter (curtain walls) fire containment systems, electrical circuit protective systems, and fire resistive duct assemblies. Volume 3 includes dampers, fire doors, hardware and frames, glass blocks and glazing materials, and leakage rated door assemblies.

WWPA	**Western Wood Products Association** 1500 SW First Ave., Suite 870, Portland, Oregon 97201 Phone: 503-224-3930; Fax: 503-224-3934; Email: info@wwpa.org; Website: http://www.wwpa.org/ 1. *Western Lumber Product Use Manual* Comprehensive technical manual on WWPA products, featuring basic categories of lumber, grades, sizes and species groups, base values and adjustment factors for dimension lumber, along with design values for structural decking, MSR lumber, structural-glued products, posts & timbers. Also includes end-use recommendations and specification guidelines, section properties, relative mechanical properties, appearance lumber grades and sizes and industrial products. 2. *Western Lumber Grading Rules* Grading rules for Douglas Fir, Engelmann Spruce, Hemlock-True Firs, Idaho White Pine, Incense Cedar, Lodgepole Pine, Ponderosa Pine, Sugar Pine, Western Larch and Western Red Cedar. Includes Base Design Values.

2.10 Additional References

ICC. (2012). International building code. International Code Council, Washington, DC.

Royer, T.R. & Stauffer, A. (2012). A new post-frame fire wall. *Frame Building News, 24*(2):58-61.

UL. (2012). Design No. V304, Design No. V304, BXUV.V304 Fire resistance ratings - ANSI/UL 263. Online certifications directory. Retrieved February 17, 2013.

CHAPTER 3

Structural Load and Deflection Criteria

Contents

3.1 Introduction

Code compliant structures must be designed to safely carry loads calculated in accordance with the applicable governing building code. When a structure is located in a jurisdiction where it is exempt from all governing codes, it is still wise to engineer the building to withstand the minimum design loads that would be applicable to the structure if it was not code exempt.

3.1.1 Load Variations

Most structural loads exhibit some degree of random behavior. For example, weather-related loads such as snow, wind and rain fluctuate over time and locations. Extensive research has been conducted to characterize this load variation, and to refine procedures for determining design loads within the context of the intended building occupancy and use.

3.1.2 Chapter Limitations

It is impractical to describe all load calculation procedures in this chapter in detail because some are quite complex, they change frequently, and at any given time it is not uncommon for different active codes to reference different versions of the same load standard. For these reasons, general concepts and key references related to structural loads and deflection criteria are presented instead, with an emphasis on issues that apply to post-frame buildings.

3.2 Load Standards

3.2.1 ANSI/ASCE 7 Standard

The National Bureau of Standards published a report titled *Minimum Live Load Allowable for Use in Design of Buildings* in 1924. The report was expanded and published as ASA Standard A58.1-1945. This standard has undergone several revisions to become the current ASCE Standard ANSI/ASCE 7 *Minimum Design Loads for Buildings and Other Structures*. At the time this design manual was written, the most recent revision of ASCE 7 was 2010 (ASCE, 2010). The ASCE 7 standard is periodically revised and balloted through the ANSI consensus approval process. Design professionals should check the governing building code for the latest adopted edition. For clarity of presentation, this manual uses and will refer to ASCE 7-10.

ASCE 7 is the primary technical source used by model codes (e.g., IBC, IRC) concerning dead, live, snow, wind, rain and seismic loads. In some cases, the model codes attempt to distill the rigorous ASCE 7 procedures into a simpler, easy-to-use format.

3.2.2 Metal Building Systems Manual

The Metal Building Systems Manual, published by the Metal Building Manufacturers Association (2012), is recognized by model building codes as an excellent technical resource document for calculating structural loads on low-rise buildings (e.g. post-frame buildings). It contains step-by-step examples for calculating wind, snow and seismic design loads per ANSI/ASCE 7-2010 (as referenced in the 2012 IBC), along with information for designing roof drainage systems for metal buildings, per the 2012 International Plumbing Code (IPC). Tabulated in the manual is wind, snow, seismic and rainfall design data for every U.S. county based on the 2012 IBC, ASCE 7-10, USGS and NOAA data. The manual also contains procedures for calculation of loads resulting from monorail, jib, underhung bridge, top-running bridge, and single-leg gantry cranes. Additionally, the manual contains recommended drift and deflection criteria for the design of metal buildings that use masonry, metal and other wall systems, as well as tolerances for manufacturing and erection.

3.2.3 ASABE Standards

A few ASABE engineering practices (EPs) have been developed to help engineers calculate loads specific to agricultural structures. In general, these EPs cover live loads not addressed by ANSI/ASCE 7. For a further explanation of these live load EPs, see Section 3.9.

For a long time, ASABE maintained ASAE EP288 entitled *Agricultural Building Snow and Wind Loads*. In the 1990's, ASABE members voted to rewrite EP288 and adopt (by reference) the snow and wind load provisions of ANSI/ASCE 7. After this was done, it did not make much sense to maintain the standard and it was subsequently dropped. For this reason, any references to ASAE EP288 should be replaced with ANSI/ASCE 7.

3.3 Building Risk Categories

3.3.1 Single Occupancy Type Buildings

For the purposes of applying flood, wind, snow, and earthquake provisions, ANSI/ASCE 7 classifies buildings and other structures into four different categories based on the nature of occupancy. The categories range from I to IV. As shown in Table 3-1, Category I represents buildings and structures with a low hazard to human life in the event of failure and Category IV represents essential facilities.

A rational basis should be used to determine the risk category for structural design, which is primarily based on the number of persons whose lives would be endangered or whose welfare would be affected in the event of a failure. ANSI/ASCE 7 provides the plot in figure 3-1 of the approximate relationship between the number of lives placed at risk by a failure and the Risk Category

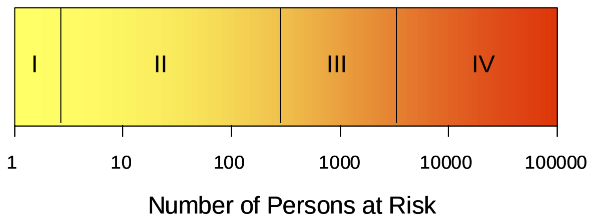


Figure 3-1. ANSI/ ASCE 7 Risk Category as a function of the number of lives placed at risk by a failure.

3.3.2 Multiple Use Buildings

When buildings or structures have multiple uses (occupancies), the independence of the structural systems associated with the different uses must be examined. If the failure of a structural system in one use area will impact the structural system in another use area, then the structural systems in both areas must be designed as if both areas house the highest usage group (the highest usage group is the usage group associated with the highest applicable building category). For example if one end of a structure is a large greenhouse – a usage associated with a Category I building, and the other end of the structure is used to store petrochemicals that are dangerous to the public if released – a usage associated with a Category III building, the entire structure would have to be classified as a Category III building if the failure of the structural system in the greenhouse area increases the probability of failure of the structural system in the petrochemical storage area.

3.4 Load Types

3.4.1 Notations and Definitions

The following symbols for different load types are from ANSI/ASCE 7.

D = *Dead Load*. Vertical loads due to the mass of the components that comprise the structure. This includes any permanently attached equipment (e.g., lights, fans, water lines). Dead loads exist even if the structure is not being occupied or used.

Table 3-1. ANSI/ASCE 7 Risk Categories (from ANSI/ASCE 7-10 Table 1.5-1)

Category	Use or Occupancy of Buildings and Structures	Examples
I	• Buildings and other structures that represent a low hazard to human life in the event of failure.	• Agricultural buildings • Minor storage facilities • Certain temporary facilities
II	• All buildings and other structures except those listed in Categories I, III and IV	• Residential buildings • Most commercial and industrial buildings
III	• Buildings and other structures, the failure of which could pose a substantial risk to human life. • Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure. • Buildings and other structures not included in Risk Category IV containing toxic or explosive substances where their quantity exceeds a threshold established by the authority having jurisdiction and is sufficient to pose a threat to the public if released	• Buildings and other structures where more than ~300 people congregate in one area • Buildings and other structures with day-care facilities with capacity greater than ~150 • Buildings and other structures with elementary or secondary school facilities with capacity greater than ~150 • Buildings and other structures with a capacity greater than ~500 for colleges or adult education facilities • Health care facilities with ~50 or more resident patients but not having surgery or emergency treatment facilities • Jails and detention facilities • Power generating station and other public utilities not included in Category IV
IV	• Buildings and other structures designated as essential facilities. • Buildings and other structures, the failure of which could pose a substantial hazard to the community. • Buildings and other structures required to maintain the functionality of other Risk Category IV structures. • Buildings and other structures containing sufficient quantities of highly toxic substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released and is sufficient to pose a threat to the public if released.	• Hospitals and other health care facilities having surgery or emergency treatment facilities • Fire, rescue and police stations and emergency vehicle garages • Designated earthquake, hurricane, or other emergency shelters • Communications centers and other facilities required for emergency response • Power generating stations and other public utility facilities required in an emergency • Aviation control towers, air traffic control centers and emergency aircraft hangars • Water storage facilities and pump structures required to maintain water pressure for fire suppression • Buildings and other structures having critical national defense functions

E = *Earthquake Load*. Combined effect of horizontal and vertical earthquake induced forces.

F = *Fluid Load.* Loads due to fluids with well-defined pressures and maximum heights. This includes forces due to flooding.

H = Loads due to lateral earth pressure, ground water pressure, or pressure of bulk materials.

L = *Live Load.* All static and dynamic loads, except roof live loads, that are due to use and occupancy of a structure. This includes loads due to movable objects (e.g., animals, stored products, unbolted equipment), movable partitions and loads temporarily supported by the structure during maintenance.

L_r = *Roof Live Load.* Accounts for higher loads during roof maintenance, and where roof is used as a walkway, gardens, assembly area, etc.

R = *Rain Load.* Accounts for (1) ponding (retention of water due solely to the deflection of a relatively flat roof), (2) load of rainwater that will accumulate on a roof if the primary drainage system for that portion is blocked, and (3) uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow.

S = *Snow Load*. Vertical loads from the mass of snow accumulating on a structure.

T = *Self-Straining Load.* Forces arising from contraction or expansion resulting from temperature changes, shrinkage, moisture changes, creep in component materials, movement due to differential settlement, or combinations thereof:

W = *Wind Load.* Forces due to wind.

3.4.2 Nominal Loads

Nominal loads represent an engineer's best estimate of the actual loads to which a structure may be subjected. In the case of loads that are highly variant (e.g., snow, wind, earthquake) nominal loads represent an engineers best estimate of the actual maximum load (a.k.a. extreme load) that could be applied to the structure during its design life. Nominal loads are also known as *service loads* because they are the loads to which a structure may be subjected while it is "in service" (i.e., being used).

Loads calculated in accordance with ANSI/ASCE 7 and the IBC are nominal loads. For allowable stress design (ASD), nominal loads are used without adjustment except when applied in combinations for which it is unlikely that all loads will simultaneously act at their maximum design levels.

3.4.3 Factored Loads

Nominal loads are multiplied by load factors to obtain factored loads. Factored loads are used in load and resistance factor design (LRFD). Not surprisingly, in LRFD jargon, loads which have not been factored (i.e., nominal loads) are frequently referred to as *unfactored loads*.

Load factors introduce safety into design by accounting for: (1) uncertainty in the estimation of nominal loads, (2) uncertainties and simplified assumptions in structural analysis, and (3) the improbability that more than one extreme load will occur simultaneously.

3.5 Load Combinations

Different load types act simultaneously. The load combination that produces the highest force(s) in a component must be used in the design of that component. This means, for example, if the combination of dead load plus live load produces the greatest forces in a component -- even greater than does a combination that includes three or more different load types (e.g., dead + live + wind load) --it must be used in the design of the component. It is very important to note that different load combinations may govern the design of different components in the same building/structure.

3.5.1 Basic Load Combinations for ASD

Where allowable stress design (ASD) is used, ANSI/ASCE 7 and the IBC require that structures (and portions thereof) be designed to resist the most critical effects from the following combinations of nominal loads.

1. D
2. $D + L$
3. $D + (L_r$ or S or $R)$
4. $D + 0.75\,L + 0.75(L_r$ or S or $R)$
5. $D + (0.6\,W$ or $0.7\,E)$

6a. $D + 0.75\,L + 0.75(0.6\,W) + 0.75(L_r$ or S or $R)$

6b. $D + 0.75\,L + 0.75(0.7\,E) + 0.75\,S$

7. $0.6\,D + 0.6\,W$
8. $0.6\,D + 0.7\,E$

Notes:

a. In combinations (4) and (6) the companion load *S* shall be taken as either the flat roof snow load (p_f) or the sloped roof snow load (p_s). Drifting and unbalanced snow loads, as primary loads, are covered by combination (3).

b. Where fluid loads *F* are present, they shall be included in combinations (1) through (6) and (8) with the same factor as that used for dead load *D*.

c. Where *H* is present, it shall be included with a load factor of 1.0 if it adds to the primary variable load effect. Where *H* resists the primary load effect, include *H* with a load factor of 0.6 where the load is permanent or with a load factor of zero (0) for all other conditions.

3.5.2 Basic Load Combinations for LRFD

Where load and resistance factor design (LRFD) is used, ANSI/ASCE 7 and the IBC require that structures (and portions thereof) be designed to resist the most critical effects from the following combinations of factored loads.

1. $1.4{\cdot}D$
2. $1.2{\cdot}D + 1.6{\cdot}L + 0.5{\cdot}(L_r$ or S or $R)$
3. $1.2{\cdot}D + 1.6{\cdot}(L_r$ or S or $R) + (\mathrm{x}L$ or $0.5{\cdot}W)$
4. $1.2{\cdot}D + 1.0{\cdot}W + {\cdot}\mathrm{x}L + 0.5{\cdot}(L_r$ or S or $R)$
5. $1.2{\cdot}D + 1.0{\cdot}E + {\cdot}\mathrm{x}L + 0.2{\cdot}S$
6. $0.9{\cdot}D + 1.0{\cdot}W$
7. $0.9{\cdot}D + 1.0{\cdot}E$

Notes:

a. x = 1.0 for garages, areas of public occupancy, and values of L greater than 100 lbf/ft^2. When L is less than or equal to 100 lbf/ft^2, set x equal to 0.5.

b. Where H is present, it shall be included with a load factor of 1.0 if it adds to the primary variable load effect. Where H resists the primary load effect, include H with a load factor of 0.6 where the load is permanent or with a load factor of zero (0) for all other conditions.

c. In combinations (2), (4), and (5), the companion load S shall be taken as either the flat roof snow load (p_f) or the sloped roof snow load (p_s). Drifting and unbalanced snow loads, as primary loads, are covered by combination (3).

d. Where fluid loads F are present, they shall be included with the same load factor as dead load D in combinations (1) through (5) and (7)

3.5.3 General Comments on Load Combinations

Most loads, other than dead loads, vary significantly with time. When more than one variable load is considered, it is extremely unlikely that they will all attain their maximum values at the same time. Accordingly, some reduction in the total of the combined loads is appropriate. For the ASD load combinations, this reduction is accomplished through the 0.75 load combination factor. Note that the 0.75 factor applies only to the variable loads, not to the dead loads. In the LRFD load combinations this reduction is accomplished by using a 0.5 load factor (instead of the 1.6 load factor) on one of the variable loads.

Dead load is always present, thus it appears in every load combination.

The last two ASD load combinations and the last two LRFD load combinations apply when the structural actions of horizontal and gravity forces counteract each other (e.g., overturning of a structure). In such cases it is conservative to underestimate the dead load of the structure, which explains the 0.6 multiplier used on dead load in the ASD combinations, and the relatively low LRFD dead load factor of 0.9.

Wind events occur for a relatively short duration as do earthquakes. The probability of these two different loads producing high forces simultaneously is virtually non-existent, and thus they never appear together in a load combination.

In addition to the preceding loads and load combinations, the authority having jurisdiction may require a specific structure be capable of withstanding the effects of extraordinary (i.e., low-probability) events such as fires, explosions and vehicular impact.

Uncertainties in lateral forces from bulk materials (H) are higher than those in fluids (F), particularly when dynamic effects are introduced as the bulk material is set in motion by filling or emptying operations. Accordingly, the LRFD load factor for such loads is set equal to 1.6.

3.6 Tributary Area

3.6.1 Load Path

When a load is applied to a particular area of a structure, any number of components may be involved in resisting it. If you track all components that are involved in resisting a particular load from its point of application to the building's foundation, you have defined the *load path* for that load. Load paths begin with a single building element (i.e., the element to which the load is applied) and can expand to hundreds of building elements. To accurately determine load paths in all but the simplest of structures requires special structural analysis techniques.

3.6.2 Tributary Area Definition

The first elements in most load paths are surfaces (i.e., floors, walls, roofs, ceilings). Generally, each of these surfaces is directly supported by several components. Exactly how much load is transferred from a surface to each of these supports depends on several factors, the most critical being: (1) the relative stiffness of the supports, (2) the relative flexibility of the surface, and (3) the relative location of the supports. The total load that is carried by each support is some percentage of the load applied to the entire surface. It follows, that the surface can be divided up into areas such that the total load on a particular area is equivalent to the total load transferred to a particular support. This particular area is defined as the *tributary area* for that support. In other words, when an entire surface is loaded, each support supports a total load equal to the load applied to its tributary area.

3.6.3 Approximation

A good first-order approximation of tributary areas (and hence support loads) for most applications is to draw a dividing line half way between adjacent supports and assume everything on one side of the line is tributary to the support on that side of the dividing line, and everything on the other side of the line is tributary to the other support as shown in figure 3-2. Note, if supports are uniformly spaced a distance, S as shown in figure 3-2, the width of the tributary area for each interior support will also be equal to S.

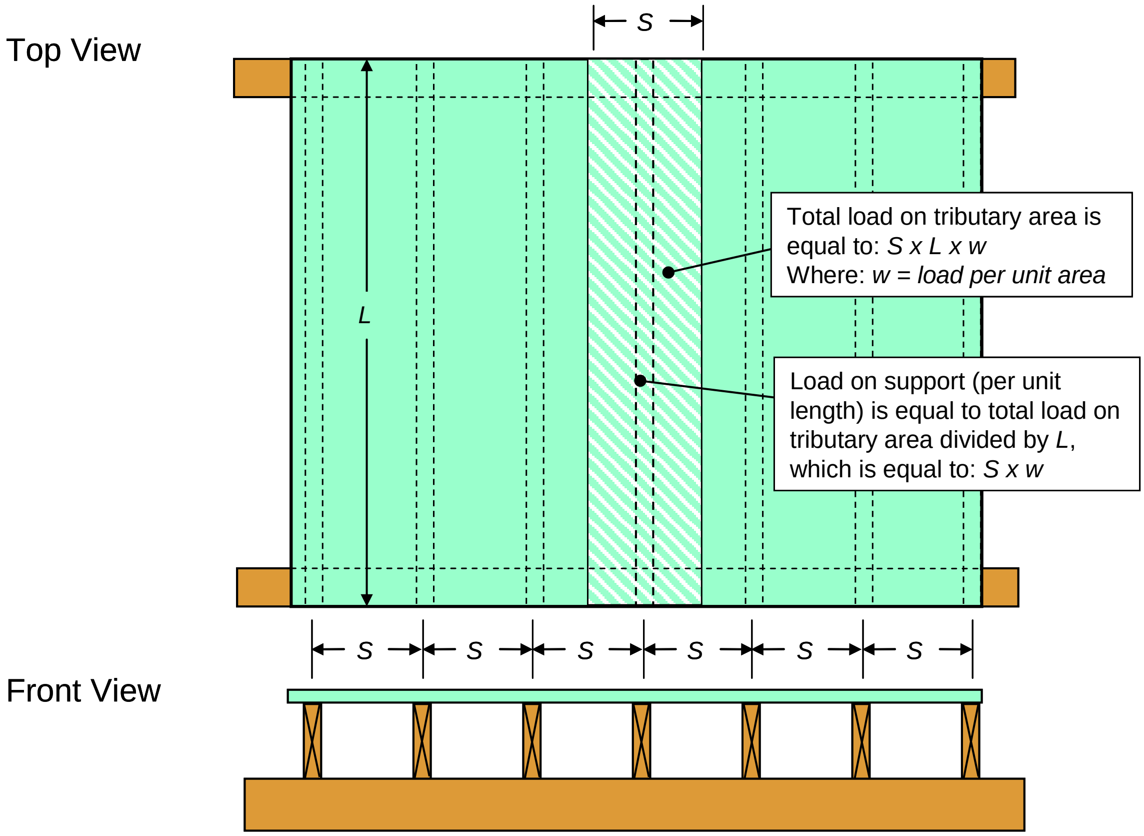


Figure 3-2. Approximate tributary area for uniformly spaced supports.

30	40	50	40	30	20
30	40	50	40	30	20
20	30	50	30	20	20
20	30	40	30	20	20
20	20	30	30	20	20

Figure 3-3. Variable wind load distribution on a roof surface. In this case, any small block could have a design pressure of 50/unit area, whereas the entire area may seldom see an average greater than 30/unit area.

3.6.4 Use in Applied Load Calculations

The probability of some loads acting at their maximum value over an entire surface decreases as the size of the surface increases. For example, while it may be quite likely for some portion of a roof to be subjected to a wind pressure of 50 lbf/ft^2, the probability of the entire roof being subjected to a wind pressure of 50 lbf/ft^2 will generally be much less (figure 3-3). In such situations, the applied load used in design will generally be a function of tributary area. With respect to the wind pressure example in figure 3-3, this translates into pressure of 50 lbf/ft^2 for roof framing members with a small tributary area and something less for framing members and other roof supports that have a measureable larger tributary area. In addition to wind, many live loads are a function of tributary area.

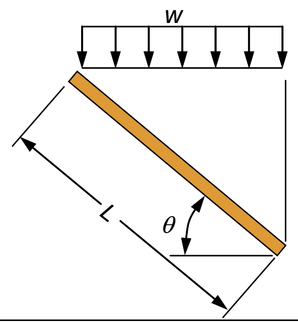
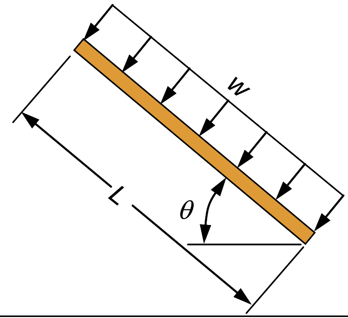
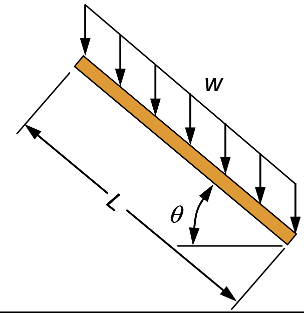
3.7 Load Representations

Schematics in Table 3-2 show how various loads are represented. Specifically, Table 3-2 shows load diagrams for forces applied to a projected horizontal plane (e.g. snow load), forces applied normal to a surface (e.g., wind load), and a Y-directed load applied to a member (e.g. dead load).

3.7.1 Horizontal Uniform Dead Load Calculation

Some structural analysis programs require that the dead load associated with a sloping surface be represented as a uniform load, w_{DL}, acting on a horizontal plane as shown in figure 3-4. For a given horizontal distance, b_H, a sloping roof surface contains more material and is heavier than a flat one. Thus, w_{DL} increases as roof slope increases.

Table 3-2. Load Representations

			
	Load applied to a projected horizontal plane	Load applied normal to surface	Y-directed load applied to surface
Total load in Y direction	$L\ w \cos\ \theta$	$L\ w \cos\ \theta$	$L\ w$
Total load in X direction	0	$L\ w \sin\ \theta$	0
Example Load Type	Snow	Wind	Dead

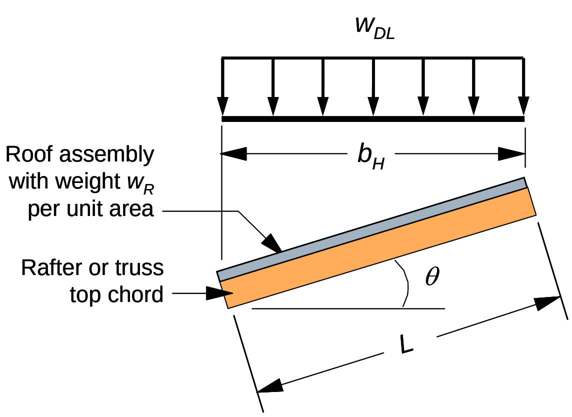


Figure 3-4. Roof dead load represented by an equivalent uniform load acting on a horizontal plane.

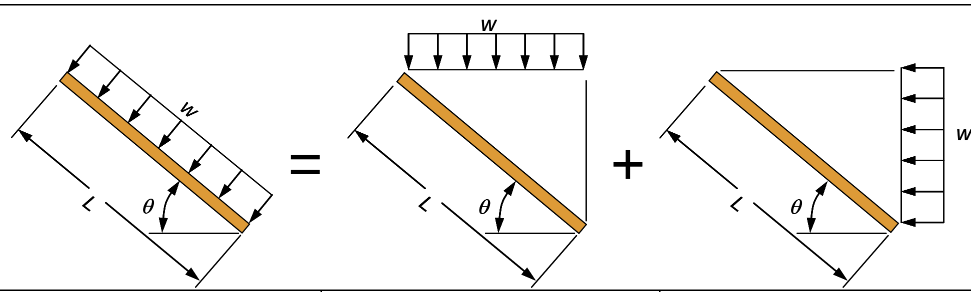
Load w_{DL} is obtained by multiplying the unit weight of the roof assembly, w_R, by the slope length, L, and dividing the resulting product by the horizontal length, b_H. Numerically, this is equivalent to dividing w_R by the cosine of the roof slope. For example, for a roof at a 4:12 slope, with materials weighing 4 lbm for each square foot of roof surface area, the equivalent load, w_{DL}, to apply to the horizontal plane would be:

$$w_{DL} = (4 \text{ lbm/ft}^2)/(\cos 18.4°) = 4.21 \text{ lbm/ft}^2$$

3.7.2 Components of Normal Loads

For some structural analyses (especially those involving hand calculations) it is beneficial to break a uniformly distributed load acting normal to a sloping surface into its horizontal and vertical components. This is a very straight forward process as the magnitude of the uniformly distributed load acting normal to a surface is equal in magnitude to the uniformly distributed load applied to both the vertically- and horizontally-projected planes as depicted in Table 3-3. A common mistake is to multiply the normal pressure by the sine and cosine of the roof slope to obtain the two components.

Table 3-3. Components of a Uniform Distributed Load Acting Normal to a Sloping Surface



	Load applied normal to surface	Load applied to a projected horizontal plane	Load applied to a projected vertical plane
Total load in Y direction	$L\ w \cos\ \theta$	$L\ w \cos\ \theta$	0
Total load in X direction	$L\ w \sin\ \theta$	0	$L\ w \sin\ \theta$

Table 3-4. Approximate Weights of Construction Materials (from Hoyle and Woeste, 1989)

Material	Weight (lb/ft^2)	Material	Weight (lb/ft^2)
Ceilings		**Roofs (continued)**	
Acoustical fiber tile	1.0	Plywood (per inch thickness)	3.0
Gypsum board (see Walls)		Roll roofing	1.0
Mechanical duct allowance	4.0	Shingles	
Suspended steel channel system	2.0	Asphalt	2.0
Wood purlins (see Wood, Seasoned)		Clay tile	9.0-14.0
Light gauge steel (see Roofs)		Book tile, 2-in.	12.0
		Book tile, 3-in	20.0
Floors		Ludowici	10.0
Hardwood, 1-in. nominal	4.0	Roman	12.0
Plywood (see Roofs)		Slate, ¼ in.	10.0
Linoleum, 1/4-in.	1.0	Wood	3.0
Vinyl tile, 1/8-in.	1.4		
		Walls	
Roofs		Wood paneling, 1-in.	2.5
Corrugated Aluminum		Glass, plate, 1/4-in.	3.3
14 gauge	1.1	Gypsum board (per 1/8-in. thickness)	0.55
16 gauge	0.9	Masonry, per 4-in. thickness	
18 gauge	0.7	Brick	38.0
20 gauge	0.6	Concrete block	20.0
Built-Up		Cinder concrete block	20.0
3-ply	1.5	Stone	55.0
3-ply with gravel	5.5	Porcelain-enameled steel	3.0
5-ply	2.5	Stucco, 7/8-in.	10.0
5-ply with gravel	6.5	Windows, glass, frame, and sash	8.0
Corrugated Galvanized steel			
16 gauge	2.9	**Wood, Seasoned**	Density (lbm/ft^3)
18 gauge	2.4		
20 gauge	1.8	Cedar	32.0
22 gauge	1.5	Douglas-fir	34.0
24 gauge	1.3	Hemlock	31.0
26 gauge	1.0	Maple, red	37.0
29 gauge	0.8	Oak	45.0
Insulation, per inch thickness		Poplar, yellow	29.0
Rigid fiberboard, wood base	1.5	Pine, lodgepole	29.0
Rigid fiberboard, mineral base	2.1	Pine, ponderosa	28.0
Expanded polystyrene	0.2	Pine, Southern	35.0
Fiberglass, rigid	1.5	Pine, white	27.0
Fiberglass, batt	0.1	Redwood	28.0
Lumber (see Wood, Seasoned)		Spruce	29.0

3.8 Dead Loads

3.8.1 Definition

Dead loads are the gravity loads due to the combined weights of all permanent structural and nonstructural components of the building, such as sheathing, trusses, purlins, girts and fixed service equipment. These loads are constant in magnitude and location throughout the life of the building.

3.8.2 Weights of Construction Materials

Tables C3-1 and C3-2 in ANSI/ASCE 7-10 contain minimum design dead loads, and minimum densities for design loads from materials, respectively. Table 3-4 contains approximate weights of materials commonly used in post-frame construction as compiled by Hoyle and Woeste (1989).

3.8.3 Special Considerations

Design dead loads that exceed the weights of construction materials and permanent fixtures are permitted, except when checking building stability under wind loading. Using inflated design dead loads may lead to conservative designs for gravity load conditions; however, it would not be a conservative assumption for designing anchorage to counteract uplift, overturning and sliding due to wind loads. In the cases of wind uplift and overturning, the dead load used in design must not exceed the actual dead load of the construction.

3.9 Live Loads

3.9.1 Definition

Live loads are loads imposed by the construction, maintenance, use and occupancy of the structure. Any loads that are or could be applied to an unoccupied structure (e.g. wind. snow, dead) are not live loads.

3.9.2 Agricultural Related Live Loads.

The only structural load standards maintained by ASABE are live load standards. These include:

- ASAE EP378 *Floor and Suspended Loads on Agricultural Structures Due to Use.* Recommended design loads resulting from livestock, suspended caged poultry, vehicles and manure on a floor.
- ASAE EP393 *Manure Storages.* Recommendations for the design, construction, and location of manure storage units. Included are design loads for interior and exterior walls, floors, footings and covers.
- ANSI/ASAE EP433 *Loads Exerted by Free-Flowing Grains on Bins.* Methods of estimating the grain pressures within centrally loaded and unloaded bins used to store free-flowing, agricultural whole grain.
- ASAE EP538 *Design Loads for Bunker (Horizontal) Silos.* Provides design loads for the walls of bunker (horizontal) silos for storing whole plant silages. Does not include hydrostatic pressure that may occur when silage becomes saturated.
- ANSI/ASAE EP545 *Loads Exerted by Free-Flowing Grain on Shallow Storage Structures.* Presents methods for estimating grain pressures within shallow storage structures used to store free-flowing agricultural whole grains.

Note that all standards adopted by ASABE prior to July 2005 (the month the society name changed from ASAE to ASABE) retain their ASAE designation, even when revised. This is to avoid someone from confusing or misinterpreting a revision to an old standard as a completely new and totally different ASABE document.

3.9.3 Non-Agricultural Related Live Loads

Non-agricultural related live loads are tabulated in Table 4-1 of ANSI/ASCE 7-10. A handful of values extracted from this table are compiled in Table 3-5. The values in this table are minimum values. It is generally wise to select design values that are greater than the tabulated minimums.

Table 3-5 contains both uniform and concentrated loads. Where both uniform and concentrated loads are given, components must be designed to support the load that produces the greatest effect (i.e., stress and/or deflection). Unless otherwise specified, the indicated concentrated load shall be assumed to be uniformly distributed over an area 2.5 feet square and shall be located to produce the maximum load effect in the structural members.

The live loads given in Table 3-5 include allowances for normal impact conditions. Additional allowances are generally required for unusual vibration and impact forces. For the purpose of design, the weight of machinery and moving loads shall be increased as follows to allow for impact: (1) elevator machinery, 100%; (2) light machinery, shaft- or motor-driven, 20%; (3) reciprocating machinery or power-driven units, 50%; (4) hangers for floors or balconies, 33%.

3.9.4 Live Load Reductions

In some cases, reductions are allowed for uniform loads to account for the low likelihood of the loads simultaneously occurring over the entire tributary area of a component. For example, ANSI/ASCE 7-10 allows a reduction in live loads in accordance with the following equation when the value of $K_{LL}A_T$ is 400 ft^2 or more:

$$L = L_o\,[\,0.25 + (15\text{ ft})/(K_{LL}A_T)^{0.5}\,] \qquad (3\text{-}1)$$

Where:

L = Reduced design live load per ft^2 of area supported by the member
L_o = Unreduced design live load per ft^2 of area supported by the member (from Table 3-5 or ASCE Table 4-1)
K_{LL} = Live load element factor (from Table 3-6 or ASCE Table 4-2)
A_T = Tributary area in ft^2

With respect to application of equation 3-1:

- L shall not be less than $0.50L_o$ for members supporting one floor and L shall not be less than $0.40L_o$ for members supporting two or more floors.
- Live loads that exceed 100 lbf/ft^2 shall not be reduced unless the loads are for members supporting two or more floors in which case a 20% reduction is allowed.

Table 3-5. Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Loads

Occupancy or Use	Uniform, lbf/ft^2	Concentrated. lbf
Fire escapes	100	
Manufacturing		
Light	125	2000
Heavy	250	3000
Office Buildings (File and computer rooms shall be designed for heavier loads based on anticipated occupancy)		
Lobbies and first floor corridors	100	2000
Offices	50	2000
Corridors above first floor	80	2000
Roofs		
Ordinary flat, pitched, and curved roofs	20	
Roofs used for roof gardens	100	
Sidewalks, vehicular driveways, and yards, subject to trucking. Note: conc. load acting on area of 20 in.^{2}	250	8000
Stairs and exits		
One- and two-family dwellings	40	
All other	100	
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	125	
Heavy	250	
Stores		
Retail, first floor	100	1000
Retail, upper floors	75	1000
Wholesale, all floors	125	1000
Walkways and elevated platforms (other than exit ways)	60	
Yards and terraces, pedestrians	100	

Table 3-6. Live Load Element Factor, K_{LL}, Values from Table 4.2 of ANSI/ASCE 7-10

Element	K_{LL}
Interior Columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
All other members not identified, including: one- and two-story slabs, cantilever beams, edge beams with cantilever slabs	1

3.10 Snow Loads

3.10.1 Ground Snow Load, p_g

Roof snow loads in the United States are based on ground snow loads. ANSI/ASCE 7-10 presents ground snow load maps that correspond to a mean recurrence interval of 50 years. These maps do not give snow load values for areas that are subject to extreme variations in snowfall, such as western mountain regions. In some regions, the best and only reliable source for ground snow loads is local climatic records.

3.10.2 Roof Snow Loads

Roof snow loads are influenced by a number of factors besides ground snow load. These factors include roof slope, temperature and coefficient of friction of the roof surface, and wind exposure. Snow loads are also adjusted by an importance factor to account for risk to property and people. The basic form of the snow load calculation found in ASCE 7-10 is:

$$p_s = 0.7\, C_e\, C_t\, I_S\, C_s\, p_g \qquad (3\text{-}2)$$

where:

p_s = snow load on a sloped roof,
0.7 = factor that relates ground snowpack to snow load on a flat roof for the contiguous United States,
C_e = exposure factor,
C_t = thermal factor,
I_S = importance factor for snow loads,
C_s = roof slope factor, and
p_g = ground snow load (50-yr mean recurrence).

The exposure factor ranges from 0.7 to 1.2 and accounts for the effects of wind and is a function of roof exposure and surrounding topography. The thermal factor ranges from 0.85 to 1.3 and accounts for the likelihood of snow melting from the roof surface. The snow importance factor accounts for the building risk category as defined in Section 3.3. Specifically, I_s equals 0.8, 1.0, 1.1 and 1.2 for Category I, II, III and IV buildings, respectively. The roof slope factor varies from 0 to 1 and depends on roof slope as well as the slipperiness and temperature of the roof surface.

3.10.3 Special Considerations

Several factors can cause snow and ice to accumulate unevenly on roofs. These factors must be considered in design. Drifting snow is particularly an issue near the ridge of wide buildings, at changes in roof height, around roof projections, in valleys of multi-gable roofs, and in valleys formed by intersecting roof planes. Specific recommendations and calculation procedures for these special considerations are given in ANSI/ASCE 7-10. Design examples are provided in the Metal Building Systems Manual (MBMA, 2012).

3.11 Wind Loads

3.11.1 Controlling Factors

Wind loading on structures is a complex phenomenon as wind loads are influenced by wind speed, building orientation and geometry, surrounding structures and topography, and building openings.

3.11.2 Building Definitions for Wind Loads

For determination of wind loads, buildings are broken into three different categories (i.e., enclosed, open, and partially enclosed) based on the size and location of building openings. A building opening is any area in the building envelope (wall, roof surfaces) that does not have a permanently attached means for *effective* closure.

An ***enclosed building*** is a building that does not comply with the requirements for open or partially enclosed buildings.

An ***open building*** is a structure having all walls at least 80% open.

A ***partially enclosed building*** is a structure in which (1) the total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope by more than 10%, AND (2) the total area of openings in a wall that receives the positive external pressure exceeds 4 ft^2 or 1% of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20%. If a building by definition complies with both the “open” and “partially enclosed” definitions, it shall be classified as an “open” building.

Proper classification of a building for wind loads is critical as partially enclosed buildings can be subjected to significant variation in internal pressures. Where there is doubt as to how effective a specific closure is under high wind load situations, two separate wind load analyses should be conducted, one with the area assumed closed, the other with the area treated as an opening. In cases where it is likely that a door may be kept open during a high wind, two separate wind load analyses should be conducted, one with the door assumed open, the other with the door assumed closed.

3.11.3 Design Wind Speed

Design wind pressure calculations begin with selection of wind speed, V, from the ANSI/ASCE 7-10 maps. There are different maps for the four ANSI/ASCE 7-10 risk categories defined in Table 3-1. Maps for risk categories I and II are associated with mean recurrence intervals (MRI) of 300 and 700 years, respectively (have annual probabilities of being exceeded of 0.00333 and 0.00143, respectively). Wind speeds on ANSI/ASCE 7-10 maps for Category III and IV structures have a mean recurrence interval of 1700 years (annual probability of being exceeded of 0.000588).

Map values are 3-second gust speeds recorded 33 feet (10 m) above the ground in an ANSI/ASCE 7-10 Exposure C terrain. For the central U.S., the basic exposure C wind speed at 33 feet above the ground for Category I and II structures is 105 and 115 mph, respectively. For Category III and IV structures it is 120 mph.

3.11.4 Effective Wind Velocity Pressure, q_z.

If a fluid with a velocity V is completely converted to a static pressure, the static pressure exerted by the fluid is equal to $\rho V^2/(2g)$ where ρ is the density of the fluid and g is the gravitational constant. If ρ is taken as 0.0766 lbm/ft^3 [the dry air mass density at 59 F and 29.92 inch Hg] and g is given as 32.17 ft·lbm/(s^2·lbf) then:

$$P = 0.00256 \cdot V^2 \qquad (3\text{-}3)$$

Where P is the static pressure in lbf/ft^2 and V is wind speed in miles per hour.

To estimate the effective wind pressure, q_z, acting at a particular height on a particular building due to wind velocity V, equation 3-3 is multiplied by (1) a velocity pressure exposure coefficient K_z that adjusts the basic wind speed values from the ANSI/ASCE 7-10 maps to design heights other than 33 feet and exposures other than C, (2) a wind directionality factor K_d that accounts for the reduced probability of maximum winds coming from any given direction and the reduced probability of the maximum pressure coefficient occurring for any given wind direction, and (3) a K_{zt} factor that accounts for the effect of wind speed-up over nearby hills and escarpments. With these factors, equation 3-3 appears as:

$$q_z = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \qquad (3\text{-}4)$$

Where q_z is in lbf/ft^2 and V is in miles per hour.

3.11.5 Design Wind Pressure, *p*

The design wind pressure applied to a surface depends on the calculation method. ANSI/ASCE 7-10 has three different methods: (1) a directional procedure for buildings of all heights, (2) an envelope procedure for low rise buildings (buildings with a mean roof height h that is less than 60 feet and does not exceed the least horizontal dimension of the building), and (3) a wind tunnel procedure for any building or structure. The envelope procedure is widely used in post-frame building design.

Separate design wind pressures are calculated for elements of the "main wind force resisting system" (MWFRS) and "components and cladding" (C&C). The MWFRS is taken to include trusses, posts, girders, shearwalls and diaphragms. Components and cladding include members such as purlins, girts, curtain walls, sheathing, roofing and siding. Since wind pressures are higher on small areas due to localized gust effects, design wind pressures are higher for components and cladding.

For *enclosed* and *partially enclosed* low-rise building systems, ANSI/ASCE 7-10 provides the following equations for design wind pressure:

For elements of the main wind-force resisting system:

$$p = q_h\,[(GC_{pf}) - (GC_{pi})] \qquad (3\text{-}5)$$

For components and cladding:

$$p = q_h\,[(GC_p) - (GC_{pi})] \qquad (3\text{-}6)$$

For overhangs:

$$p = q_h\,(GC_{po}) \qquad (3\text{-}7)$$

Where:

p = Design wind pressure
q_h = q_z calculated at the mean roof height (see figure 3-5)
GC_{pf} = External pressure coefficient for main wind-force resisting systems
GC_{pi} = Internal pressure coefficient
= 0.00 for open buildings
= +0.55 and -0.55 for partially enclosed buildings
= +0.18 and -0.18 for enclosed buildings
GC_p = External pressure coefficient for components and cladding
GC_{po} = Combined pressure coefficient for overhangs

ANSI/ASCE 7-10 contains figures, tables, and equations for determining pressure coefficients (i.e., GC_{pf}, GC_p, GC_{po}). These coefficients are dependent on the tributary area for the component being designed, and its location relative to the geometric discontinuities in the surfaces of the building (i.e., its location relative to wall corners, eave lines, the ridge, etc.). To account for the influence of location, building surfaces are broken into zones. Pressure coefficients within each of these zones are assumed constant.

The design wind pressure applied to a wall surface is the sum of the pressures applied to both sides of the surface (i.e. external and internal pressures) as calculated according to equations 3-5 and 3-6, and shown in figure 3-5.

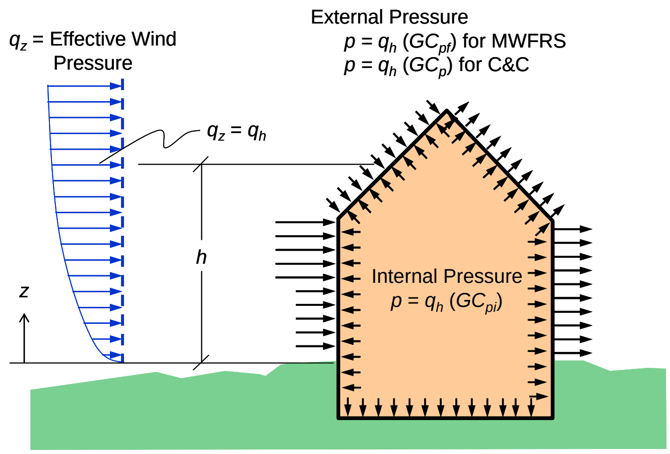


Figure 3-5. Total pressure on a surface is sum of external and internal pressures. For calculation purposes, pressures (both internal and external) acting toward a surface are considered positive.

3.11.6 Minimum Wind Loads

ANSI/ASCE 7-10 specifies minimum design wind loads for both the MWFRS and C&C. It is not uncommon for these minimum wind loadings to be used in the design of agricultural post-frame buildings that are exempt from building codes.

For the MWFRS, the ANSI/ASCE 7-10 minimum design wind load for an enclosed or partially enclosed building is 16 lbf/ft^2 multiplied by the wall area of the building and 8 lbf/ft^2 multiplied by the roof area of the building projected onto a vertical plane normal to the assumed wind direction. For C&C, the minimum design wind pressure is 16 lbf/ft^2 acting in either direction normal to surfaces.

3.12 Seismic Loads

3.12.1 Cause

Earthquakes produce lateral forces on buildings through the sudden movement of the building's foundation. Building response to seismic loading is a complex phenomenon and there is considerable controversy as to how to translate knowledge gained through research into practical design codes and standards.

3.12.2 Lateral Force

Basic concept of seismic load determination for low-rise buildings is to calculate an equivalent lateral force at the ground line as follows:

$$V = C_s W \tag{3-8}$$

where:

- V = total lateral force, or shear, at the building base
- W = total dead load, plus other applicable loads specified in ANSI/ASCE 7-10. For most single-story post-frame buildings, the only other minimum applicable load is a portion (20% minimum) of the flat roof snow load. If the flat roof snow load is less than 30 psf, the applicable load to be included in W is permitted to be taken as zero.
- C_s = seismic response coefficient = $S_{DS}/(R/I_c)$
- S_{DS} = design spectral response acceleration parameter
- R = response modification factor
- I_c = importance factor based on risk category (Table 3-1)

ANSI/ASCE 7-10 is the most complete source for determination of all of the proceeding variables.

3.12.3 Seismic and Post Frame

Seismic loads rarely control post-frame building design because of the relatively low building dead weight as compared with other types of construction (Taylor, 1996; Faherty and Williamson, 1989). For post-frame buildings, lateral loads from wind usually are much greater than those from seismic forces.

3.13 Deflection

3.13.1 Code Application

Post-frame building components must meet deflection limits specified in the governing building code.

3.13.2 Exception to Code Requirements

Girts supporting corrugated metal siding are typically not subjected to deflection limitations unless their deflection compromises the integrity of an interior wall finish. Because of the inherent flexibility of corrugated metal siding, girt deflections present no serviceability problems, and consequently, girt size is generally only stress dependent.

3.13.3 Time Dependent Deflection

In certain situations, it may be necessary to limit deflection under long term loading. Published modulus of elasticity, E, values for wood are intended for the calculation of immediate deflection under load. Under sustained loading, wood members exhibit additional time-dependent deformation (i.e. creep). It is customary practice to increase calculated deflection from long-term loading by a factor of 1.5 for glued-laminated timber and seasoned lumber, or 2 for unseasoned lumber (see Appendix F, AWC, 2012). Thus, total deflection is equal to the immediate deflection due to long-term loading times the creep deflection factor, plus the deflection due to the short-term or normal component of load. For applications where deflection is critical, the published value of E (which represents the average) may be reduced as deemed appropriate by the designer. The size of the reduction depends on the coefficient of variation of E. Typical values of E variability are available for different wood products (see Appendix F, AWC, 2012).

3.13.4 Shear Deflection

Shear deflection is usually negligible in the design of steel beams; however, shear deflection can be significant in wood beams. Approximately 3.4 percent of the total beam deflection is due to shear for wood beams of usual span-to-depth proportions (i.e. 15:1 to 25:1). For this reason, the published value of E in the Supplement to the National Design Specification is 3.4 percent less than the true flexural value (AF&PA, 1993). This correction compensates for the omission of the shear term in handbook beam deflection equations. For span-to-depth ratios over 25, the predicted deflection using the published E value will exceed the actual deflection. Similarly, for span-to-depth ratios less than 15, predicted deflections will be significantly less than actual. This could lead to unconservative designs (with respect to serviceability) for post-frame members such as door headers. Practical information on the effects of shear deformation on beam design is given in Appendix D of

Hoyle and Woeste (1989) for rectangular wood beams and Triche (1990) for wood I-beams.

3.14 References

3.14.1 Non-Normative References

AF&PA. (1993). *Commentary to the national design specification for wood construction*. American Forest and Paper Association (AF&PA), Washington, D.C.

Faherty, K.F. & Williamson, T.G. (1989). *Wood engineering and construction handbook*. McGraw-Hill, New York, NY.

Hoyle, R.J. & Woeste, F.E. (1989). *Wood technology in the design of structures.* Ames, IA: Iowa State University Press.

MBMA. (2012). *Metal building systems manual*. . Metal Building Manufacturers Association (MBMA), Cleveland, OH.

Taylor, S.E. (1996). Earthquake considerations in post-frame building design. *Frame Building News, 8*(3):42-49.

Triche, M.E. (1990). Shear deflection effect on I-joist design. *Wood Design Focus*, *1*(2).

3.14.2 Normative References

ANSI/AWC ASD/LRFD NDS - *National Design Specification for Wood Construction*.

ANSI/ASCE 7-10 *Minimum design loads for buildings and other structures.*

CHAPTER 4

Structural Design Overview

Contents

4.1 Introduction

The aim of this chapter is to give an overview of post-frame building design from a structural perspective, beginning with a broad overview in Section 4.2, followed by Sections 4.3 through 4.11 which look at specific post-frame building components. These sections are followed by two sections which address the durability of materials: Section 4.12 covers decay resistance of wood, and Section 4.13 covers electrochemical corrosion resistance of metals. These two sections have been included in this chapter because of the tremendous impact material durability has on long-term structural integrity (and hence long-term performance) of a building.

4.2 Broad Overview

4.2.1 Primary Framing

Primary framing is the main structural framing in a building. In a post-frame building, this includes the columns, trusses (or rafters), and any girders that transfer load between trusses and columns. As defined ANSI/ASABE S618 (see Section 1.2.2), each truss and the post(s) to which it is attached form an individual *primary frame*, also referred to as a *post-frame* or *main frame*. Primary frames collect and transfer load from roof purlins and wall girts (i.e. secondary framing) to the foundation. In the context of wind loading in standards and building codes, primary frames are an integral part of the main wind-force resisting system (MWFRS). Specific sections dedicated to primary framing include: Section 4.3 Posts, Section 4.4 Trusses, Section 4.5 Girders, and Section 4.6 Knee Braces.

4.2.2 Secondary Framing

As defined in Section 1.2.4, secondary framing includes any framing member used to transfer load between cladding and primary framing members, and/or laterally brace primary framing members. The secondary framing members in a post-frame building include the girts, purlins and any structural wood bracing such as permanent truss bracing. Specific sections dedicated to secondary framing include: Section 4.7 Roof Purlins, Section 4.8 Wall Girts, and Section 4.9 Large Doors.

4.2.3 Diaphragms and Shearwalls

When cladding is fastened to the wood frame of a post-frame building, large shearwalls and roof and ceiling diaphragms are formed that can add considerable rigidity to the building. In many post-frame buildings, diaphragms and shearwalls are carefully designed and become an integral part of the main wind-force resisting system. Roof and Ceiling Diaphragms are covered in Section 4.10 and Shearwalls in Section 4.11.

4.2.4 Specific Details

The post-frame building system is a special type of light wood-frame construction. Although model and active building codes extensively cover light wood-frame construction, they seldom cover specific details of post-frame building design. Specifically, they lack coverage of post foundation design, metal-clad wood-frame diaphragm design, and interaction between post-frames and diaphragms. Hence, Chapters 5, 6, 7 and 8 focus on these topics in more detail.

4.2.5 Limitations

The structural design of buildings involves making many judgments, such as determining design loads, structural analogs and analyses, and selecting materials that can safely resist the calculated forces for the life of the structure. New research or testing could justify a change of design procedure for the industry or for an individual designer. To this end, it is important to note that the considerations presented herein are not exhaustive, and thus should never be considered or treated as exhaustive.

4.3 Posts

4.3.1 General

The function of the wood post is to carry axial and bending loads to the foundation. Posts are embedded in the ground or attached to either a conventional masonry or concrete wall or a concrete slab-on-grade. Posts can be solid-sawn, structural composite lumber, mechanically-laminated, or glued-laminated. Any portion of a post that is embedded or exposed to weather must be pressure-treated with preservative chemicals to resist decay and insect damage (see Section 4.12).

4.3.2 Controlling Load Combinations

The load combination that usually controls post design is full dead plus 3/4 wind plus 3/4 snow for ASD (e.g., equation 6a in Section 3.5.1) and full dead plus full wind plus one-half snow for LRFD (e.g., equation 4 in Section 3.5.2); however, local codes may stipulate different load combinations. It is possible for any one of the combinations to be critical; therefore, they all should be considered for a specific building design. For example, maximum gravity load will govern truss-to-post bearing and post foundation bearing; whereas wind minus dead load will govern the truss-to-post connection (for uplift).

4.3.3 Force Calculations

The diaphragm analysis method presented in Chapter 6 is the most accurate method to determine design moments, and axial and shear forces in posts. Historically, some designers calculated the maximum post moment for embedded posts by using the simple structural analog of a propped cantilever (i.e. fixed reaction at the post bottom and pin reaction at the top). The implicit assumption of this analog is that the roof diaphragm and shearwalls are infinitely stiff. This model may be adequate for buildings with extremely stiff roof diaphragms and for conservatively estimating shear forces in the roof diaphragm; however, it may underestimate the maximum post moment for many post-frame buildings. The analysis procedures described in Chapter 6 are more reliable since they account for the flexible behavior of roof diaphragms and shear walls.

If posts are embedded, generally two bending moments must be calculated - one slightly below the groundline and the other above ground. For posts attached to a concrete surface with a flexible connection, the post-to-concrete surface connection is typically modeled as a pin (leaving the post moment equal to zero at that point), and thereby limiting maximum post bending moment calculations and checks to a single point.

The bending moment and shear forces in the post at groundline are frequently used in embedded post foundation design calculations.

4.3.4 Combined Stress Analysis

Forces involved in post design subject the posts to combined stresses (bending and axial), and thus they must be checked for adequacy using the appropriate interaction equation from the NDS. In theory, every post length increment must satisfy the interaction equation, but in practice, a minimum of two locations are checked: the point of maximum interaction near the ground level (column stability factor, C_p, equal to 1.0) and the upper section of the posts where the maximum moment occurs in conjunction with column action (C_p<1.0).

4.3.5 Shear Stress

The shear stress due to lateral loading (wind or seismic) rarely controls post design, but should always be checked as a matter of good practice. Other loads such as bulk loads from stored materials may influence final post design.

4.3.6 Deflection

A post deflection limit is not normally specified for post-frame buildings, but interior finishes may require it. Refer to the deflection criteria in Chapter 3.

4.3.7 Connections

Truss-to-post connections must be designed for bearing as well as uplift. Connection design procedures are given in the NDS. Truss-to-post connections should be modeled as pins unless moment-carrying capacity can be justified. Direct end grain bearing is desirable and is often achieved by notching the post to receive the truss (or in the case of a mechanically-laminated post, by placing the truss on a shortened lamination). When designing the truss-to-post connection for uplift, it is important to accurately estimate the weights of construction materials if any counteracting credit is to be taken.

For surface-attached posts, the bottom connection needs to be checked for maximum shear and uplift forces.

For embedded posts attached to collars or footings, the below-grade connections must be properly designed to withstand gravity and uplift loads, and corrosion-resistant fasteners must be used (see Section 4.13).

4.3.8 Construction Alternatives

The posts in post-frame buildings can be solid sawn, mechanically-laminated, glued-laminated or wood composite. Allowable design stresses are published in the NDS or are available from the manufacturers. Treated wood (Section 4.12) is used for the embedded part of the post, but no treatment is required on the parts that are not in contact with the ground and are protected by the building envelope.

4.3.9 Foundation

Post-frame building foundations include embedded post and pier foundations as well as conventional concrete frost wall, concrete basement walls, and slab-on-grade foundations.

Embedded post foundations must be designed to resist lateral forces induced by wind and seismic loads, as well as wind uplift, and gravity loads. Post and pier foundation design is an important aspect of post-frame building design that is not well known in the structural engineering design community, and therefore Chapter 6 is dedicated to this subject. If a concrete slab is used along with an embedded post/pier foundation system, it only need be designed for interior loads since exterior building loads are transferred directly to the ground through the posts.

4.4 Trusses

4.4.1 General

Together with posts, wood trusses are primary structural elements of post-frame buildings. Two excellent sources of technical information on trusses are the Truss Plate Institute (TPI) and the Wood Truss Council of America (WTCA). Trusses must be properly designed, handled and installed. These responsibilities are shared by the building owner, contractor and designer, and the truss designer and manufacturer. The importance of a clear understanding of responsibilities among these parties cannot be overstated, and is covered in WTCA 1-1995 *Standard Responsibilities in the Design Process Involving Metal Plate Connected Wood Trusses* and ANSI/TPI 1-2007: *National Design Standard for Metal Plate Connected Wood Truss Construction*.

4.4.2 Design Loads

The load combination that controls the design of most truss components in much of the U.S. is full snow plus dead load. When applying ANSI/ASCE 7-10 loads combinations involving snow, note that different load combinations apply for balanced and unbalanced snow loadings (see notes following load combination equations in Section 3.5).

It is important to check all applicable load combinations because different load combinations may control the design of different truss elements. For example, a wind load combination may cause stress reversal in some truss elements as discussed in Section 4.4.5.

Truss manufacturers commonly express allowable truss design loads with a series of four or a series of three hyphenated numbers (e.g., 20-4-0-1 or 20-4-1). If there are four numbers, the first number is the sum of the allowable "non-dead" loads on the top chord, the second is the allowable dead load on the top chord, the third is the sum of the allowable "non-dead" loads on the bottom chord, and the fourth is the allowable dead load on the bottom chord. Frequently, a truss is designed without placing non-dead loads on the bottom chord. In such cases, it is common to only use the three number designation as shown in the above example. Unless otherwise specified, it is assumed that the loads are expressed in lbf per square foot (psf). To this end, it is imperative that the designated truss design loading be accompanied by the truss spacing, since it is the product of the allowable load in psf and the spacing in feet that dictates the allowable load that can be applied to a lineal foot of the truss.

When reviewing truss design specifications, be aware that truss manufacturers commonly categorize loads as either dead or live, in which case, they consider snow and wind loads to be types of live loads. This runs counter to the ANSI/ASCE 7-10 definition of a live load. Because loads like wind and snow are associated with different ASD load duration factors and different LRFD time effect factors, it is important that a designer know what portion of an allowable top chord "live" load is due to snow, what portion is due to wind, what portion is due to roof live loading, etc.

4.4.3 Design

This design manual does not present specifics of roof truss design. Metal-plate-connected wood trusses in the United States are designed according to the provisions of ANSI/TPI 1-2007. Other designs are based on proprietary test information, along with design criteria from the NDS.

Section 6.1.1.2 of ANSI/TPI 1-2007 requires that an accepted structural analysis method for analyzing statically indeterminate structures, such as the matrix stiffness method, be used to determine the design moments and axial forces for each truss member. Section 6.1.1.1 of ANSI/TPI 1-2007 requires that this model closely approximate the geometry and properties of the truss members and connections. Regardless of analysis methods, structural modeling assumptions are important and can dramatically influence the design (Brakeman, 1994). For example, partial fixity at truss plate joints as well as eccentricity at heel joints, can be modeled a variety of ways. The heel joint usually gets the most attention since heel joint modeling decisions can greatly influence truss design.

The size, and in some cases the orientation, of truss plates is dependent on proprietary design values. These values may be available from the manufacturer. Some are available in the form of an ICC-ES Evaluation Report (see http://www.icc-es.org/reports/)

Trusses can be obtained pre-engineered from the manufacturer. It is important to consider wind loading on trusses as stress reversals can occur and overstress some members. This design is complicated by the fact that wind loads are influenced by building geometry, so this information must be communicated to the truss designer. Any structural bracing (e.g. knee braces) or redundant supports must be included in the truss design.

4.4.4. Truss-to-Post Connection

The connection between the truss and post is critical. Designers must consider both gravity forces and uplift forces. With some truss-to-post connection designs, it might be necessary to examine the impact of the connection on the forces induced in the truss chords, heel joints, and post. Observations from several building investigations revealed that the individual trusses and posts were designed properly, but the connection between the two units was not. Many different methods and hardware have been used to design the connection, such as bolts, nails, truss anchors, and combinations of the same. Unless otherwise governed by a specific code, the design of this connection should meet NDS requirements.

4.4.5 Stress Reversal

The trusses used in post-frame buildings are typically long span and, consequently, have long webs. When the truss becomes part of a post-frame building, it is possible, under certain loading conditions, for a tension web in the truss design to become a compression web.

Stress reversal can also occur in truss chords due to a wind uplift loading combined with dead load. This load case may not frequently control the size of the truss chord lumber, but it makes compression in the bottom chord possible. This situation is one reason that lateral bracing of the bottom chord is required (SBCA/TPI, 2013).

4.4.6 Temporary Bracing

Temporary bracing is required to ensure stability of trusses during their installation and until permanent bracing for trusses and the building are in place. This area is the most difficult to manage in the field.

According to WTCA 1-1995 and ANSI/TPI 1-2007, determination and installation of temporary bracing is the responsibility of the building contractor. This work should be done in accordance with the *Building Component Safety Information (BCSI) Guide to Good Practice for Handling, Installing, Restraining and Bracing of Metal Plate Connected Wood Trusses* – a publication jointly produced by the Structural Building Components Association (SBCA) and the Truss Plate Institute (TPI). The BCSI guide contains the following ten major chapters:

- BCSI-B1:*Guide for Handling, Installing, Restraining & Bracing of Trusses*
- BCSI-B2:*Truss Installation & Temporary Restraint/Bracing*
- BCSI-B3: *Permanent Restraint/Bracing of Chords & Web Members*
- BCSI-B4: *Construction Loading*
- BCSI-B5:*Truss Damage, Jobsite Modifications & Installation Errors*
- BCSI-B7: *Temporary & Permanent Restraint/Bracing of 3x2 and 4x2 Parallel Chord Trusses*
- BCSI-B8: *Using Toe-Nailed Connections to Attach Trusses At Bearing Locations*
- BCSI-B9: *Multi-Ply Girders*
- BCSI-B10: *Post Frame Truss Installation, Restraint & Bracing*
- BCSI-B11: *Fall Protection & Trusses*

The temporary bracing requirements in BSCI-B10 (the chapter specifically dedicated to post-frame building trusses) is similar in content to BCSI-B2. The primary difference is that BCSI-B2 applies to trusses spaced up to 2 ft on-center and up to 80 ft in length, and BSCI-B10 applies to trusses between 2 and 12 ft on-center and up to 81 ft in length.

4.4.7 Permanent Bracing

Permanent truss bracing is critical to the performance of the roof system. In fact, improper design and/or improper installation of permanent bracing are leading causes of post-frame building structural failures.

Permanent bracing for trusses in post-frame buildings is typically specified in accordance with BCSI-B3 and BCSI-B10 (SBCA/TPI, 2013). If properly planned, the temporary bracing applied during truss installation can be used as permanent bracing, making the completion of the permanent bracing more efficient.

4.5 Girders

4.5.1 General

Girders are heavy beams used to span large openings (e.g., doors) and to support trusses located between posts. For example, when roof truss spacing is less than the post spacing, girders (sometimes called headers) are needed to carry the intermediate trusses. This is a common occurrence over large door openings. These beams are considered main wind-force resisting members. Vertically nail-laminated lumber, structural composite lumber, glued-laminated beams and steel I-beams are all commonly used as girders. There is an abundant supply of structural-composite lumber products from manufacturers who publish their own allowable stresses. Often, the critical load combination is dead plus snow load, although all applicable load combinations must be checked.

4.5.2 Design Criteria

Girders are designed as bending members. Any one of the four criteria used for the design of bending members can control design (i.e. bending, shear, compression perpendicular to grain, and deflection). Shear can often control girder design. Also note that formulae found in most handbooks account for bending but not shear deflection. Designers should consider the impact of shear deflection on the total deflection of a girder. Hoyle and Woeste (1989) provide formulae for calculating shear deflection of wood beams.

4.5.3 Vertically Laminated Lumber

The design of girders for a post-frame building is routine structural design except when a girder is fabricated by vertically laminating three or more pieces of dimension lumber. In this case, the allowable bending stress can be increased using the repetitive member factors published in ANSI/ASAE EP559 (ASABE, 2012).

4.5.4 Connections

Girder attachment to posts and individual roof trusses is a fundamental part of girder design. When designing girder-to-post connections, both uplift and gravity must be considered. When designing truss-to-girder connections, special consideration must be given to situations in which trusses are hung off the side of the girder. In such cases truss-to-girder connections should be designed to prevent rotation between the trusses and girder, the girder must be sized to handle additional stresses due to torsion, or the girder must be braced to reduce rotation.

4.6 Knee Braces

4.6.1 General

Knee braces are intended to supplement the resistance of post frames under lateral loads, and can influence the unsupported length of columns. They have been used less and less in recent years.

4.6.2 Effectiveness

Knee brace effectiveness is highly dependent on the stiffness of its connections to the post and truss. If the connections at the ends of the brace are flexible or not very stiff due to the use of a few nails, the roof diaphragm carries the bulk of the load, and the brace is ineffective (Gebremedhin and Woeste, 1986). If the brace connections are made very stiff (by installing many nails or bolts) the brace could effectively resist the wind loading but could overload the truss.

4.6.3 Analysis

Knee braces induce primary bending moments in truss chords if attached between panel points. Knee braces induce secondary bending moments when attached directly to panel points. If knee braces are to be used in post-frame design, load sharing among the truss, post, knee brace, connections, and diaphragm (when applicable) must be included in the structural analysis.

4.7 Roof Purlins

4.7.1 General

Roof purlins are typically 2- by 4-inch or 2- by 6-inch lumber, and are key structural elements of the roof assembly. They resist gravity loads, wind loads, roof diaphragm chord forces, and provide lateral bracing to truss top chords (or rafters). To fulfill the chord-bracing role, the purlins must be supported against lateral movement by attachment to sheathing or metal cladding that provides the needed roof diaphragm strength. Not all roof cladding materials provide diaphragm strength and/or purlin lateral support; one example is standing seam roofing, which is fastened with clips that allow adjacent sheets to slide.

4.7.2 Classification

Purlins in post-frame buildings fall into the ANSI/ASCE-7 wind load category of "component and cladding." Components and cladding collect the loads and distribute them to the primary structural elements, identified as the main wind-force resisting system. Wind loads are much greater at eaves, ridges, edges, corners and other discontinuities. Purlin spacing and fasteners are critical in these areas. If these areas fail under extreme wind loading, the building envelope will be breached, and internal wind pressures will change dramatically.

4.7.3 Orientation

Purlins are installed on-edge or flat. When they are used on-edge, they may be either placed on top of the truss or recessed between the trusses. Purlins placed on-edge are frequently overlapped and fastened together at the overlap. When used flat, purlins are installed on top of the trusses.

4.7.4 Truss Chord Bracing

Purlin spacing is a factor in truss design since purlins provide lateral support to the truss top chord. In some cases, the slenderness ratio for weak-axis truss chord buckling between purlins can be greater than that for strong-axis buckling. Therefore, when specifying trusses, the building designer should inform the truss-design engineer of the planned purlin spacing.

4.7.5 Design Loads

Purlin design often is controlled by the dead plus snow load combination, or dead plus wind load (especially in the edge zones of the roof). Dead loads used for design may exceed actual weights for gravity load calculations; however, inflated dead loads cannot be used to offset wind uplift or wind overturn moments. In these cases, offsetting loads cannot exceed actual weights of materials.

4.7.6 Design Criteria

Purlins members should be checked for bending strength, shear capacity, and deflection. If the roof assembly is functioning as a structural diaphragm, purlins will also be subjected to axial forces. Purlins shall be designed to carry bending about both axes. Weak axis bending may be omitted if it can be demonstrated by test or analysis that the roof sheathing provides support. The connections between the purlins and rafters should be designed for both gravity loads and wind uplift forces. Purlin hangers are often used when purlins are recessed, and their capacity should be verified for the various loading cases. In general, the provisions of the NDS apply for the connections and stress analysis.

4.8 Wall Girts

4.8.1 General

Girts are used to collect wind-induced wall loads and distribute them to the post frames. For end walls, the wind loads are distributed to structural end-wall posts.

4.8.2 Classification

Girts belong to the "component and cladding" category for determining the design wind load.

4.8.3 Orientation

Girts are either installed flat on post faces or recessed between the posts. Girts recessed between posts are almost always orientated with the narrow edge facing the cladding, and in this position, are frequently used to support both interior and exterior cladding/sheathing.

4.8.4 Post Bracing

Girts provide lateral support to side-wall columns. With girts securely installed, the slenderness ratio of the post weak axis is greatly reduced. Therefore, posts can usually be designed to carry the axial loads using the slenderness ratio of the strong axis.

4.8.5 Design Loads

Girts are normally designed to resist only wind load. Wind loads are much greater at corners and other discontinuities. Girt spacing and fasteners are critical in these areas. If these areas fail, the building envelope will be breached, and internal wind pressures will change dramatically.

The dead load of the girt and attached steel is normally negligible for girt design. Cladding is attached to the girts by nails or screws, and the stiffness of these connections does not allow the girts to undergo significant bending stress or deflection from the action of the small dead loads present. However, the wall dead load should be included in total dead load calculations for the post foundation.

Girts must be designed to resist forces induced by stored materials, especially granular materials such as fertilizer or seeds/grain. Care should betaken to assure that the capacity of wall panels, fasteners and girts are not exceeded by these forces.

4.8.6 Design Criteria

Girts are designed as bending members for which the usual bending-member design criteria apply. The critical connections between the girts and the post should be checked for both wind pressure and suction. The top wall girt may be constructed to carry chord forces from the roof diaphragm and, if so, must be checked for the appropriate axial loads. The NDS provisions apply for the connections and stress analysis.

4.9 Large Doors

4.9.1 General

Large doors are common in post-frame buildings. Door components must be designed to withstand design wind loads, and are treated as "components and cladding" for such calculations.

4.9.2 Open Doors

It is not uncommon for building owners to leave large doors open, even during periods of high wind. If an owner anticipates that this will occur, the building must be designed accordingly. Note that a large opening on one side of the structure is generally associated with increased internal wind pressure coefficients, and thus can significantly increase roof uplift forces.

4.10 Roof and Ceiling Diaphragms

4.10.1 General

Roof and ceiling diaphragms are used to resist lateral (sidesway) forces applied to the building by wind, earthquake and stored material. Under lateral load, roof and ceiling diaphragms act as large stiff plates. These plates support and distribute loads to wall posts. Conceptually, diaphragm design is easy to understand, but the application of the procedure requires analysis tools and data.

Diaphragms made from plywood are well documented, as well as those made entirely from steel. Less information is available about wood-framed, metal-clad diaphragms which are prevalent in the post-frame building industry. This is a major factor in post-frame building design and is covered in more depth in Chapters 6 and 7.

4.10.2 Design Properties

Diaphragm performance depends on factors such as the steel, steel sheet-to-sheet fasteners, steel-to-wood fasteners, and the wood frame. There is no standard steel panel construction, so diaphragm strength and stiffness depend on the specific construction used. Strength and stiffness data on laboratory test panels are generally required to derive design values. Most post-frame buildings have much greater spans than laboratory test panels; therefore, test data must be extrapolated to practical building sizes as explained in Chapter 7.

4.11 Shearwalls

4.11.1 General

A large portion of the shear forces induced in roof and ceiling diaphragms is transferred to the building foundation by shearwalls. In many post-frame buildings, the only walls available to transfer this shear are exterior walls (i.e., endwalls and sidewalls). Where present, interior partition walls can be designed to transfer additional shear.

4.11.2 Endwalls

Endwalls in post-frame buildings resist wind loads perpendicular to the building end wall and simultaneously help transmit roof shears (due to parallel-to-end wall wind components) to the ground. In the diaphragm design procedure described in Chapter 6, maximum roof shears occur at the endwalls. The roof shear is transferred into the top truss chord or rafter of the endwall, through the endwall to the ground level, and finally to the ground by posts or to posts connected to a concrete slab. In addition to shear forces, the end wall is subject to overturning forces. Wirt et al. (1992) have published procedures for analyzing and designing end-wall foundations.

4.11.3 Wall Openings

Allowances must be made for openings in shearwalls. One common practice in post-frame construction is to place large doorways in the building endwalls. Procedures for accounting for the opening and ways to reinforce the remaining wall are given in Chapter 6.

4.11.4 Partitioning

Partitioning of the building into structural segments is one method to reduce maximum roof shears and endwall shears. For example, if it is not practical to reinforce an endwall that has a large door installed, the alternative is to install a structural partition in the center of the building. The structural partition must meet the shear requirements delivered by the roof diaphragm. Buttresses, inside or outside the walls, can be used to reduce the effective length of the building with respect to maximum roof and end-wall shears.

4.12 Decay Resistance of Wood

When wood moisture content exceeds 20% on a dry weight basis in the presence of oxygen, it is vulnerable to attack by insects and decay fungi. Although some wood species (and the heartwood of other species) are naturally resistant to these types of attack, most structural woods used in North America are not. These structural wood species must be chemically treated to protect them from decay and insect attack.

4.12.1 Pressure Preservative Treatment (PPT)

Chemicals used for preserving wood are impregnated into the wood using pressure. These preservative chemicals abate wood decay by altering the wood as a potential food source for insects and fungi.

4.12.2 PPT Use in Post-Frame Buildings

Wood that is in direct soil contact must be preservative treated. This includes embedded wood posts and any girts in direct ground contact (e.g. splash plank). Structural wood elements that are not in contact with the soil but are directly exposed to the outdoor environment should also be preservative treated.

4.12.3 Treatment Types

The type of preservative treatment and the required amount of retention by the wood depends on the end use application (service life and environmental conditions) of the wood component.

The preservatives typically used in North America for ground contact are waterborne copper-based, or oil-type. Waterborne copper-based preservatives include chromated copper arsenate (CCA), ammoniacal copper zinc arsenate (ACZA), copper azole (CA-C), and alkaline copper quat (ACQ). Oil-type preservatives include pentachlorophenol (penta), coal tar creosote and copper naphthenate (CuN).

As noted in Section 1.4.21, effective December 31, 2003 the EPA registered label for use of CCA no longer permits use in a number of applications including all post-frame building components except embedded posts. Consequently, non CCA treatments are required for splash plank, grade girts, base plates, sill plates, etc.

Waterborne copper-based preservatives increase the potential for metal-fastener corrosion and thus their use will often dictate fastener type (see Section 4.13.5).

4.12.4 Treatment Levels

For adequate protection from insects and decay fungi, it is imperative that American Wood Protection Association (AWPA) preservative retention guidelines be followed. These retention levels are published for various "Use Categories" in Section 6 of AWPA Standard U1. Retention levels are considered minimums and are expressed in lbm/ft^3 (pcf). While treating to levels greater than the AWPA specified minimums can ensure better protection against decay, the cost of this extra treatment in terms of enhanced durability should be weighed any negative environmental impacts.

4.12.5 Incising

The rate at which treatments are absorbed into wood, and the depth of penetration of the treatment, varies from wood species to wood species. Whereas southern pine species take treatment quite well, most western species must be incised to comply with AWPA retention requirements. Incising may reduce lumber strength on dimensions less than 4 inches thick. Consult the American Wood Council (AWC) National Design Specification (NDS) for Wood Construction regarding the use of incised wood in structural applications.

4.12.6 PPT Quality Assurance

Quality assurance is critical to the performance of treated wood. The treating industry has developed a quality control and treatment quality marking program accredited by the American Lumber Standards Committee. Any treated members specified for use in a post frame building should be labeled by an approved agency (e.g., Southern Pine Inspection Bureau (SPIB), Timber Products Inspection (TPI), Bode, etc.) to assure that the members have been treated in accordance with AWPA Standard U1 and to the appropriate retention level.

4.12.7 MSDS

Treated wood suppliers provide Material Safety Data Sheets (MSDS) with the product. These sheets contain special instructions about the care, handling and disposal of treated wood. Federal law dictates that these sheets must be provided to all employees exposed to the materials.

4.12.8 Cutting/Drilling Treated Wood

Saw cuts or drilled holes made after treatment may expose untreated wood. This problem is especially critical if the newly exposed wood is in the splash zone or in contact with the ground. When using mechanically-laminated posts, the cut end of the treated lumber should be placed upward, above the ground level; otherwise, brush-applied, soaked, or dipped field treatments are recommended. AWPA Standard M4 outlines procedures for field treatment; some chemicals require a certified pesticide applicators license to apply. The chemical suppliers should be consulted for application restrictions.

4.13 Electrochemical Corrosion Resistance of Metals

Electro-chemical corrosion is the breaking down of a material due to chemical reactions with its surroundings. Most commonly it begins with the loss of electrons from a metal via a reaction with surrounding water and oxygen, and ultimately produces oxide(s) and/or salt(s) of the original metal. For example, rust is a general term for a series of iron oxides formed by the reaction of iron with oxygen in the presence of water. Other non-iron based metals undergo equivalent corrosion, but the resulting oxides are not commonly called rust.

The rate of corrosion is affected by water and accelerated by electrolytes, as illustrated by the effects of road salt (calcium chloride) on the corrosion of automobiles. Given sufficient time, oxygen and water, any iron mass eventually converts entirely to rust and disintegrates.

The corrosion of aluminum is much slower than that of iron because the resulting aluminum oxide forms a coating which protects the remaining aluminum in a process known as passivation.

4.13.1 Galvanic Corrosion

Galvanic corrosion is a common form of corrosion that occurs when dissimilar metals or metal alloys are brought into electrical contact by immersion in a conductive electrolyte. In the case of building materials, this conductive electrolyte is generally impure water (e.g., rainwater, groundwater). When electrically connected, one of the dissimilar metals becomes the anode and corrodes faster than it would all by itself in the conductive electrolyte, while the other metal becomes the cathode and corrodes slower than it would alone in the conductive electrolyte. Which metal becomes the anode and which becomes the cathode depends on their relative electrical potential within the conductive electrolyte. Table 4-1 contains a list of the electrical potentials of metals in flowing seawater. When arranged in order of their electrical potential, the list of metals is referred to as a galvanic series. Within a galvanic series, the metal closer to the anodic (or active) end of the series will be the anode and thus will corrode faster, while the one toward the cathodic (or noble) end will corrode slower. The greater the electrical potential difference between two metals, the more rapidly the anode will corrode when the metals are electrically connected.

4.13.2 Building Material Selection Guidelines

Galvanic corrosion is minimized by (1) using metals that are not dissimilar, (2) preventing dissimilar metals from becoming electrically connected, and (3) keeping small anodes from contacting large cathodes. With respect to the latter, rate of corrosion is dependent on the surface area of the anode relative to the cathode. The smaller the surface area of the anode relative to the cathode, the more concentrated the flow of electrons at the anode (i.e., the higher the current), and the faster the rate of corrosion. Conversely, the larger the anode's surface area in relation to the cathode, the more spread out the flow of electrons and the slower the rate of corrosion. For example, if there is a window frame made of stainless steel and it is attached with carbon steel screws, the screws will probably corrode. If the window frame is made of carbon steel and it is attached with stainless steel screws there will be very little, if any, corrosion.

Table 4-1. Galvanic Series in Flowing Seawater [a]

	Metal or Metal Alloy	Electrical Potential Range of Alloy vs. Reference Electrode, volts [b]
Anodic or Active End	Magnesium	-1.60 to -1.63
	Zinc	-0.98 to -1.03
	Aluminum Alloys	-0.70 to -0.90
	Cadmium	-0.70 to -0.76
	Cast Irons	-0.60 to -0.72
	Steel	-0.60 to -0.70
	Aluminum Bronze	-0.30 to -0.40
	Red Brass, Yellow Brass, Naval Brass	-0.30 to -0.40
	Tin	-0.29 to -0.31
	Copper	-0.28 to -0.36
	Lead-Tin Solder (50/50)	-0.26 to -0.35
	Admiralty Brass, Aluminum Brass	-0.25 to -0.34
	Manganese Bronze	-0.25 to -0.33
	Silicon Bronze	-0.24 to -0.27
	Stainless Steel – Type 410, 416 [c]	-0.25 to -0.36
	90-10 Copper-Nickel	-0.21 to -0.28
	80-20 Copper-Nickel	-0.20 to -0.27
	Stainless Steel – Type 430	-0.20 to -0.32
	Lead	-0.19 to -0.25
	70-30 Copper-Nickel	-0.13 to -0.22
	Silver	-0.09 to -0.14
	Stainless Steel–Types 302,304,321,347	-0.05 to -0.10
	Stainless Steel – Type 316, 317 [c]	-0.00 to -0.10
Cathodic or Noble End	Titanium and Titanium Alloys	+0.06 to -0.05
	Platinum	+0.25 to +0.18
	Graphite	+0.30 to +0.20

(a) Source: Stephen Dexter, University of Delaware Sea Grant Marine Advisory Service

(b) These numbers refer to a Saturated Calomel Electrode. Measured in seawater with flow rates between 8 and13 ft/s and temperatures between 50 and 80 F (10-27 C)

(c) Values listed are for a passive state. In low-velocity or poorly aerated water, or inside crevices, these alloys may start to corrode and exhibit potentials near -0.5 V

In high moisture environments, components that are in direct contact should not have an electric potential difference (from Table 4-1) that exceeds 0.20 volts. Metals listed in Table 4-1 have been color-coded into groups that fall within the potential difference range of roughly 0.20 volts. Using this as a guideline, there should be no problem with zinc coated components contacting aluminum components, nor a problem with tin and copper components contacting each other. Conversely, allowing a zinc- or aluminum-components to contact copper or tin will result in more rapid degradation of the zinc and aluminum. Lead should not be used in any construction where Galvalume or aluminum is utilized. Note that if a component has a metallic coating, the metallic coating and not the base metal determines the electric potential of the component.

Materials should be selected so rainwater does *not* flow from the cathode in a metal pair to the anode in a metal pair. For example, in a metal pair of copper and galvanized steel, copper is the cathode and galvanized steel (i.e., zinc) is the anode. Water from a copper roof will contain dissolved copper that will result in unwanted corrosion of a galvanized steel gutter. Alternatively, water runoff from a galvanized steel roof will not corrode a copper gutter.

4.13.3 Sacrificial and Barrier Coatings

Application of a metallic coating to a panel is done to protect the base metal from oxidation. When the coating is measurably more active/anodic (i.e., higher up on the galvanic series) than the base metal, it will provide galvanic protection to the base metal. Such is the case with a zinc-coated (a.k.a. galvanized) steel panel. When steel is exposed by cutting or scratching of the panel, galvanic corrosion will take place with the anodic zinc moving to cover the exposed cathodic steel. A coating that protects base metal in this manner is referred to as a sacrificial coating.

The extent to which a sacrificial coating can continue to protect base metal is directly proportional to the amount (i.e., thickness) of the coating. Metallic coatings that are not sacrificial can only protect base metal by preventing moisture and oxygen from reaching the base metal. Such metallic coatings, along with paint, are referred to as barrier coatings.

Prevention of base metal oxidation by sacrificial and barrier coatings is important. Not only is such oxidation unsightly, but it compromises panel strength and eventually weather tightness.

Application of paint coatings to metallic surfaces, or placement of plastic or other non-metallic barriers between dissimilar metals, can significantly reduce galvanic corrosion. When protecting an underlying metallic coating or base metal with a paint coating, it is important to realize that a small accidental scratch in the coating can result in rapid corrosion of the newly exposed metal if the exposed metal becomes the anode in a reaction with a nearby dissimilar metal that has a large surface area.

4.13.4 Mechanical Fastener Selection

Due to their small surface area relative to the materials they fasten, fasteners that take on the role of the anode will be at risk of rapid corrosion and thus should be avoided. Table 4-2 contains a guideline for selection of fasteners based on galvanic action. In general, zinc-coated fasteners should only be used to connect galvanized and aluminum-zinc alloy coated steel. Do not use zinc- or aluminum-coated fasteners to attach copper or stainless-steel panels. Whenever possible, match the surface metal on the fasteners with that on the panels and trim they will attach.

Table 4-2. Common Fastener Recommendations from Manufacturers

Panel/Trim Surface Material	Fastener Metal					
	Electro-Plated Screws [a]	Hot-Dip Galvanized Nails [b]	Zinc Capped Screws [c]	Aluminum	Copper	Stainless Steel
Zinc (Galvanized)	Yes	Yes	Yes	No	No	No
Aluminum-Zinc Alloy (e.g. Galvalume)	No	Yes	Yes	No	No	No
Aluminum	No	No	No	Yes	No	No
Copper	No	No	No	No	Yes	Yes[d]
Stainless Steel	No	No	No	No	No	Yes

(a) Screws with an electrodeposited coating of zinc applied in accordance with ASTM B633.
(b) Nails with a zinc coating that meets or exceed ASTM A153 Class D thickness specifications.
(c) ASTM B633 electroplated screws with a special zinc or zinc-aluminum alloy cap.
(d) Austentic stainless steels (302/304, 303, 305) may increase the corrosion of copper whereas martensitic stainless steel (410) fasteners will not.

4.13.5 Corrosion Due to Wood Preservatives

Most waterborne wood preservatives contain copper. This includes chromated copper arsenate (CCA), alkaline copper quat (ACQ), copper azole (CA), and ammoniacal copper zinc arsenate (ACZA). To avoid galvanic corrosion in wood containing a copper-based treatment and used in a moist condition, use fasteners that are comprised of, or coated with: copper, a material more noble than copper such as silicon bronze and types 304 and 316 stainless steel, or plastic (Baker, 1992). Do not use aluminum fasteners or aluminum-coated fasteners in lumber containing copper-based wood treatments (Baker, 1992; AWC, 2004). Fasteners with a sufficient zinc coating (i.e., fasteners that meet ASTM A153 Class D for hot-dip galvanizing) can generally be used in wood containing copper-based treatments as long as the wood is not regularly exposed to moisture or other environments considered extremely corrosive. This means that any fastener with only an electrodeposited coating of zinc (e.g., an electro-plated screw) should not be used in wood containing copper-based treatments if the wood is regularly exposed to moisture or an environment considered extremely corrosive.

Do not allow aluminum, aluminum-coated, and galvalume-coated panels and trim to come into direct contact with wood preservatives containing copper, mercury or fluorides. Galvanized steel is generally compatible with chromated copper arsenate (CCA) treatments but not with alkaline copper quat (ACQ) and copper azole (CA) in damp conditions. Avoid direct contact between bare metal panels and treated lumber where (1) condensation will frequently form on the metal surface in contact with the lumber, and (2) the wood treatment is more noble (cathodic) than the metal surface. Direct contact between metal panels and treated lumber can be avoided by separating them with a barrier proven suitable for the application.

4.14 References

4.14.1 Non-Normative References

AWC. (2004). *Fastener corrosion*. American Wood Council Fact Sheet. AWC. 1111 Nineteenth Street, NW, Suite 800, Washington, DC 20036 http://www.awc.org/HelpOutreach/faq/CorrosionFactSheet.pdf

Baker, A. J. (1992). Corrosion of nails in CCA- and ACA-treated wood in two environments. *Forest Products Journal, 42*(9):39-41

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Hoyle, R. J. & Woeste, F. E. (1989). *Wood technology in the design of structures.* Fifth edition. Iowa State University Press, Ames, IA.

SBCA/TPI. (2013). *Building component safety information (BCSI): Guide to good practice for handling, installing, restraining and bracing of metal plate connected wood trusses.* Jointly produced by the Structural Building Components Association (SBCA) and the Truss Plate Institute (TPI). http://www.sbcindustry.com/bcsi.php

Wirt, D. L., Woeste, F. E., Kline, D. E., and McLain, T. E. (1992). Design procedures for post-frame end walls. *Applied Engineering in Agriculture, 8*(1):97-105.

4.14.2 Normative References

ANSI/ASAE EP 559.1 *Design requirements and bending properties for mechanically-laminated wood assemblies.*

ANSI/ASABE S618 *Post frame building system nomenclature*.

ANSI/ASCE-7 *Minimum design loads for buildings and other structures*

ANSI/AWC-2012 *National design specification for wood construction.*

ANSI/TPI 1-2007 *National design standard for metal plate connected wood truss construction.*

ASTM A153 *Standard specification for zinc coating (hot-dip) on iron and steel hardware.*

AWPA U1-13 *User specification for treated wood.*

WTCA 1-1995 *Standard responsibilities in the design process involving metal plate connected wood trusses*.

CHAPTER 5

Post and Pier Foundation Design

Contents

5.1 Introduction

A distinct advantage of post-frame construction is the opportunity to transfer structural loads to the soil via post and pier foundations, thereby eliminating the need for a traditional foundation.

Post and pier foundations are a more environmentally-friendly option to concrete frost walls because they use considerably less concrete and can be quickly and easily removed. Most post and pier foundations can be reused.

5.1.1 Governing Design Standard

Soil properties, safety factors, analysis methods and design equations presented in this chapter are from ANSI/ASAE EP486.2 *Shallow Post and Pier Foundation Design*.

5.1.2 Classification

Depending on their width-to-length ratio, and the surrounding soil type, foundations are categorized as either shallow or deep. The vast majority of post and pier foundations are *shallow foundations*, and exhibit a behavior quite different from that of deeper systems such as pilings.

In many respects ANSI/ASAE EP486.2 is a blend of commonly published procedures for determining allowable vertical loads on shallow spread footings, and commonly published procedures for determining allowable lateral loads on short piles. Thus the term “shallow” is included in the title of the engineering practice. As is common with shallow foundation design, ANSI/ASAE EP486.2 ignores any foundation-soil friction that would help a pier/post foundation transfer gravity loads into the soil ore resist uplift forces.

Soil deformation around a post is a three-dimensional phenomena. Figure 5-1 shows the lines of constant soil pressure (in a horizontal plane of soil) that form when a post moves laterally. The greater the distance between two posts, the less influence one post will have on the soil pressure near the other. For design purposes, individual embedded posts are considered *isolated* foundations by ANSI/ASAE EP486.2 when post spacing is at least 4.5 times greater than post/pier width or at least 3 times greater than the maximum dimension of a footing or attached collar.

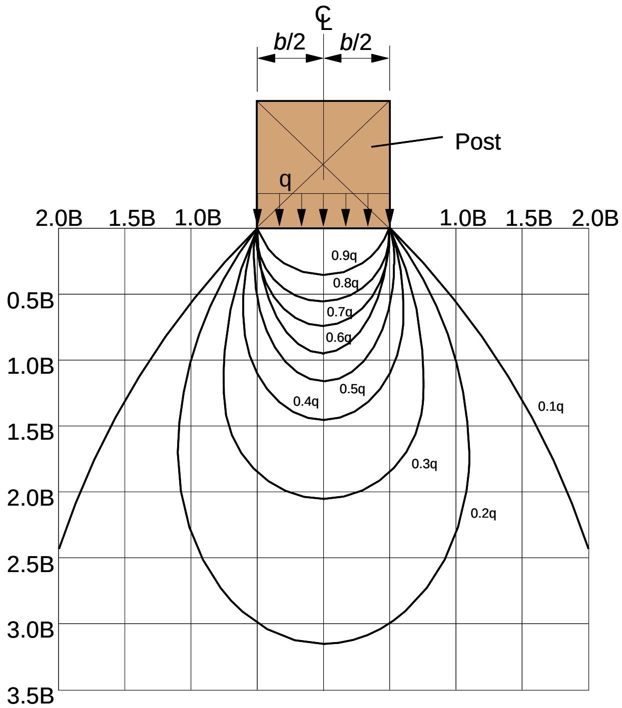


Figure 5-1. Constant pressure lines in a horizontal plane of soil.

5.1.3 Chapter Overview

The layout of this chapter is similar to that for ANSI/ASAE EP486.2. This chapter leads with definitions (Section 5.2), soil characteristics, classification and use (Section 5.3), engineering properties of soil (Section 5.4), and foundation material properties (Section 5.5). This is followed by Section 5.6 on structural analysis and Section 5.7 on governing strength equations. Sections 5.8, 5.9 and 5.10 cover bearing strength, lateral strength, and uplift strength assessment, respectively. The last two sections cover frost heave considerations and installation requirements.

5.2 Definitions

5.2.1 Post versus Pier Foundations

Post and pier foundations were previously defined in Section 1.2.7 as follows:

Post foundation: An assembly consisting of an embedded post and all below-grade elements, which may include a footing, uplift resistance system, and collar. See figure 5-2.

Pier foundation: An assembly consisting of a pier and all below-grade elements, which may include a footing, uplift resistance system, and collar. See figure 5-3.

A pier was defined in Section 1.2.6 as a relatively short column partly embedded in the soil to provide lateral and vertical support for a building or other structure. Piers include members of any material with assigned structural properties such as solid or laminated wood, steel, or concrete.

Piers differ from embedded posts in that they seldom extend above the lowest horizontal framing element in a structure, and when they do, it is often only a few inches. Conversely, embedded posts typically extend up to and generally past the main roof supports. It follows that a post foundation and a pier foundation can appear identical below-grade, and that the only way to differentiate between them is to identify how far above grade the major foundation element extends.

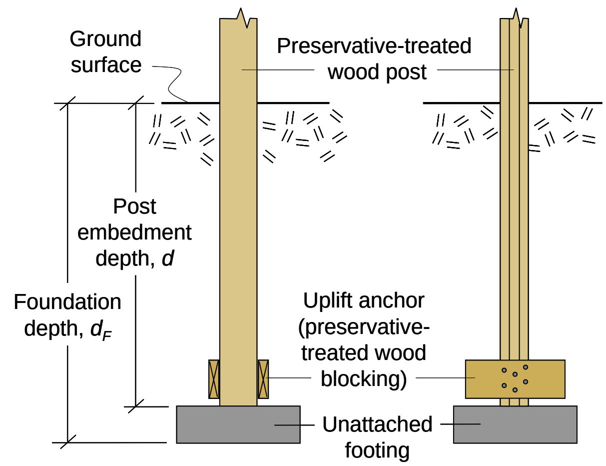


Figure 5-2. Preservative-treated wood post foundation.

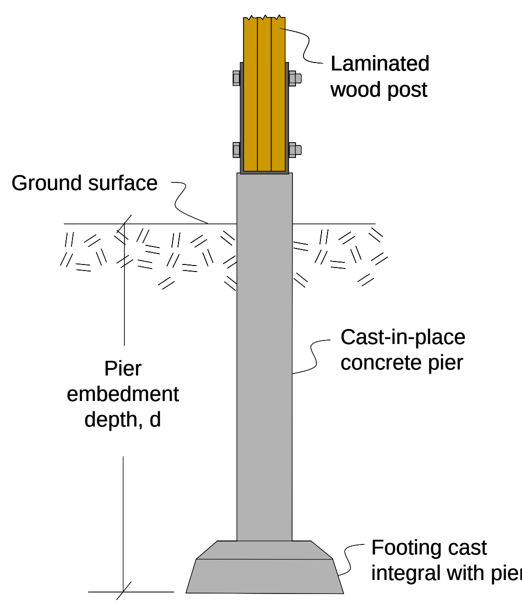


Figure 5-3. Cast-in-place concrete pier foundation. Footing functions as uplift resisting system.

Whereas a post and pier foundations are identical in every respect below grade (same materials, size, uplift resisting system, etc), there will be no difference in "below-grade" performance when they are subjected to equal loads at grade.

5.2.2 Foundation Components

In addition to embedded posts and piers, other major foundation elements include footings, collars, uplift anchors, and in some cases, even backfill.

Footing: Foundation component at the base of a post or pier that provides resistance to vertical downward forces. When properly attached to the post/pier, a footing aids in the resistance of lateral and vertical uplift forces.

Collar: Foundation component that surrounds a post or pier and aids in the resistance of lateral loads. When a collar is mechanically attached to a post or pier such that it can not slide up or down relative to the post/pier, it will also aid in resisting vertically-applied loads.

Uplift Anchor: Any element mechanically attached to an embedded post or pier to increase the uplift resistance of the foundation.

Backfill: Material filling the excavation around a post or pier foundation.

Footings and collars are distinctly different elements that may or may not be present in a particular foundation. Both footings and collars can function as uplift anchors. Uplift anchors can also be special elements such as preservative-treated wood blocks (figure 5-2) or steel angles (figure 5-3).

Where concrete or controlled low-strength material (CLSM) is used as backfill, the backfill will effectively function as a collar.

5.2.3 Foundation Dimensions

Foundation dimensions of importance to design are illustrated in figure 5-4 and defined as follows:

Foundation Face Width, *b*: width of the face of the post/pier, footing, or collar that applies load to the soil when the foundation moves laterally.

Post (or Pier) Side Width, *w*: dimension of a post/pier measured parallel to the direction of applied lateral load. Equal to width b for a round or square pier/pole.

Footing Breadth, *B*: diameter of a round footing or side length of a square footing.

Uplift Resisting System Width, B_U: diameter of a circular uplift resisting system or the smaller of the two dimensions characterizing a rectangular uplift resisting system.

Uplift Resisting System Length, L_U: length of a rectangular uplift resisting system with a width B_U.

Foundation Depth, d_F: Vertical distance from the ground surface to the bottom of a post or pier foundation. Typically the vertical distance from the ground surface to the base of the footing.

Post (or Pier) Embedment Depth, *d*: Vertical distance from the ground surface to the bottom of the embedded post or pier. Includes the thickness of the footing when the footing is rigidly attached to the post/pier or is cast integrally with the post/pier.

Uplift Resisting System Depth, d_U: Vertical distance between soil surface and top of the foundation uplift resisting system.

Water Table Depth, d_W: distance between soil surface and top of the water table.

Depth to Point of Rotation, d_R: Depth from ground surface to the point about which a non-constrained foundation rotates below grade. The rotation point is the point below grade at which the foundation does not move laterally under the applied loads.

Depth to Ultimate Point of Rotation, d_{RU}: Depth from ground surface to the point about which a non-constrained foundation rotates below grade when loaded to capacity.

5.2.4 Foundation Constraint

If a post or pier foundation is not restrained from moving horizontally at or just above the ground surface it is said to be ***non-constrained***. Conversely, if a post or pier foundation pushes against (or is attached to) an "immovable" structural element such that the lateral displacement at some point at or just above the ground surface is essentially equal to zero, the foundation is said to be ***constrained***. An example of a constrained post or pier foundation is one that bears against a concrete slab-on-grade.

A single post can be both constrained or non-constrained, depending on the load case. Using the previous example of a concrete slab-on-grade, and assuming that the post is not attached to the slab, if the wind loading was such that the post was pushing on the slab, the post would be considered constrained. However, if the wind were blowing in the opposite direction, the post would not be supported by the slab; hence, the post would be analyzed for that load case as non-constrained.

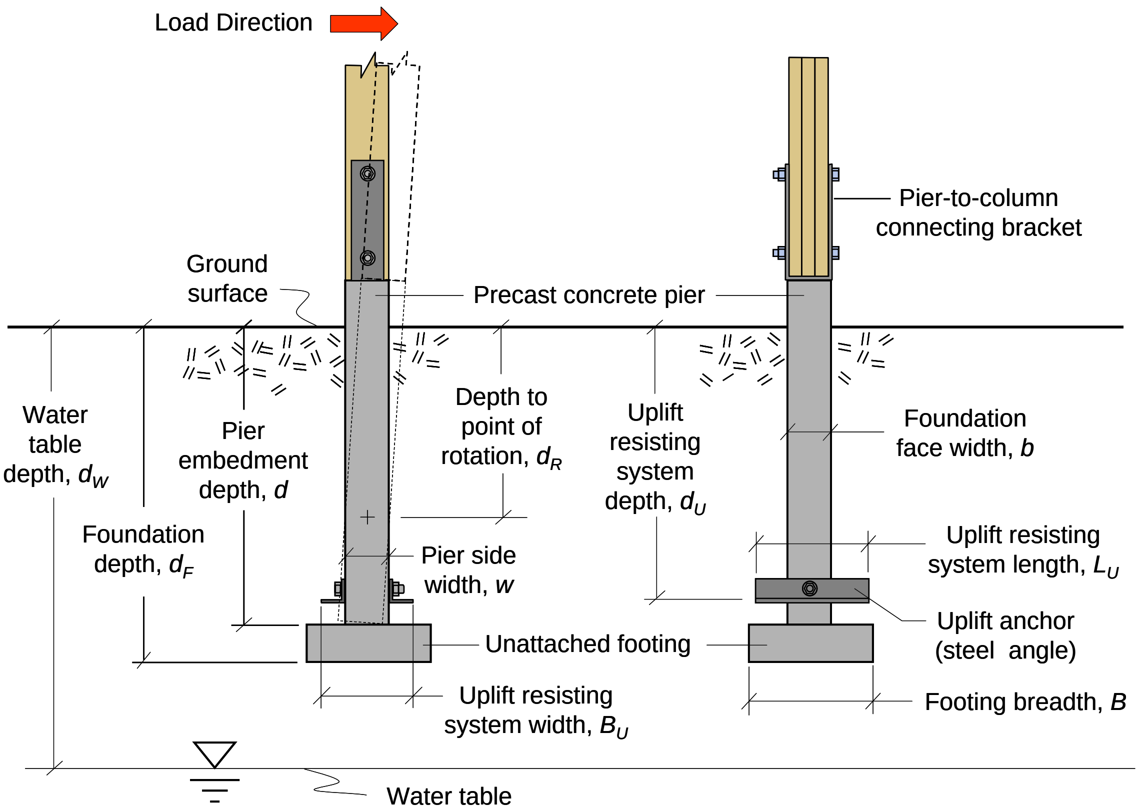


Figure 5-4. Side view (left) and face view (right) of a pier foundation with an unattached footing and an uplift resistance provided by steel angles.

5.3 Soil Characteristics, Classification and Use

5.3.1 Overview

Soil is characterized and classified in different ways for different purposes. In this section, soil characteristics that are of interest in engineering design are defined, and the Unified Soil Classification System (USCS) is presented. The USCS is used by ANSI/ASAE EP486.2 to assign engineering properties for post/pier foundation design when such properties have not been directly determined by laboratory or in-situ soil tests.

This section also contains information on soils that should be avoided when constructing a building.

5.3.2 Soil Separates

Soil separates are specific ranges of particle (a.k.a. grain) sizes. In the U.S., the four main soil separates are gravels, sands, silts and clays. Virtually all soil classification systems define clays as particles having diameters less than 0.00008 inches (0.002 mm). The Unified Soil Classification System defines silts as particles with diameters less than 0.003 inches (0.075 mm), sands as particles with diameters between 0.003 and 0.08 inches (2 mm), and gravels as particles with diameters between 0.08 and 3.0 inches (76 mm). Particles less than 0.003 inches in diameter can not be distinguished with the naked eye.

5.3.3 Particle Size Distribution

The single most important factor dictating soil behavior is the size distribution of the particles comprising the soil. Grain-size distribution is generally determined by a combination of mechanical sieving and hydrometer analyses. The finest mechanical sieve is a No. 500 which retains particles larger than 0.0010 in. (25 µm). Thus you can't use mechanical sieving to study particles smaller than the largest silt-sized grains. Figure 5-5 shows a grain-size distribution curve along with soil size classifications, U.S. standard sieve sizes, and D_{60}, D_{30}, and D_{10} particle sizes. The D_{10} size is referred to as the effective size of the soil. The ratio of D_{60} to D_{10} is defined as the uniformity coefficient, C_U.

Dividing the square of D_{30} by the product of D_{60} and D_{10} yields the coefficient of curvature, C_C.

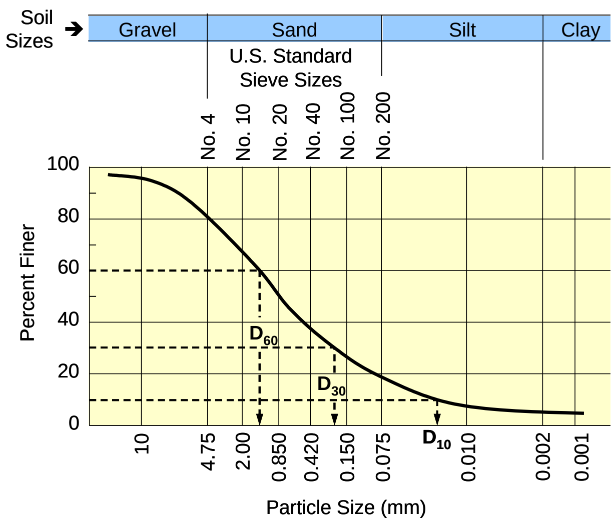


Figure 5-5. Grain size distribution curve showing soil size classifications, U.S. standard sieve sizes, and D_{60}, D_{30}, and D_{10} particle sizes.

5.3.4 Fine- Versus Coarse-Grained Soils

If more than 50% of a soil (by mass) is comprised of gravel- and sand-sized particles, the soil is categorically referred to as a ***coarse-grained soil***. Conversely, if more than half of the soil mass consists of clay- and silt-sized particles, the soil is considered a ***fine-grained soil***.

5.3.5 Cohesive Versus Cohesionless Soils

Soils with individual grains that stick together are called ***cohesive soils***. Those soils whose particles do not stick together are referred to as ***cohesionless soils***.

The properties of cohesive soils are influenced by clay minerals (do not confuse clay minerals with clay-sized particles). Clay minerals result from the weathering of other minerals, mainly feldspar, mica and ferromagnesian minerals. Since they are the end-products of the weathering of different rock minerals, clay minerals are quite resistant to further change by weathering. Clay minerals consists of a large number of very tiny flat plates, stacked together but separated by thin layers of water that contain dissolved ions. The most common clay minerals are kaolinite, illite, and smectites. It is important to note that these three clay minerals have very different properties.

Cohesionless soils (a.k.a. non cohesive soils) are those that do not exhibit cohesion. Sand- and gravel-sized particles do not exhibit cohesion, and thus most coarse-grained soils are cohesionless soils. Whereas the density of cohesive soils is largely dependent on the type of clay mineral present and water moisture content, the density of cohesionless soils depends on grain shape, grain size distribution, and the relative position (packing) of the grains.

5.3.6 Atterberg Limits

In 1911, in an effort to better characterize the behavior of fine-grained soils, Swedish chemist Albert Atterberg defined three soil moisture contents (dry basis, %) that he called the liquid limit, plastic limit, and shrinkage limit. These limits where later refined by Arthur Casagrande. The ***liquid limit*** (**LL**) is the moisture content (MC) above which soil behaves as a viscous liquid, the ***plastic limit*** (**PL**) is the MC below which soil behaves as a semisolid, and the ***shrinkage limit*** (**SL**) is the MC below which there is no more volume change. The difference between the liquid and plastic limits is called the ***plasticity index*** (**PI**). The relationships between Atterberg limits as a function of moisture content are illustrated in figure 5-6. Procedures for defining Atterberg limits are covered in ASTM D4943 and ASTM D 4318.

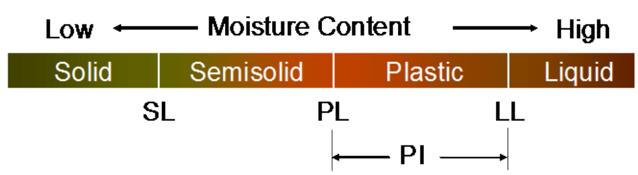


Figure 5-6. Relationship between Atterberg Limits.

A large difference between the liquid and plastic limits (i.e., a high plasticity index) means that the soil behaves as a plastic over a wide range of moisture contents. This is an indication that the soil contains more smectite type clay minerals and will undergo measurable expansion and contraction as it absorbs and desorbs water. Clays with a high plasticity index (PI) are called fat clays, and those with a low PI are called lean clays (figure 5-7).

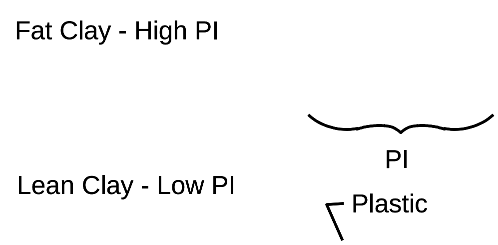


Figure 5-7. The difference between fat and lean clays is defined by the magnitude of their plasticity index.

5.3.7 Organic Soils

Soils that contain vegetative matter are called organic soils. While organic soils are great for growing crops, never place a building foundation on them.

Organic soils are dark or drab shades of gray, brown & black. They have a very distinct odor that can be revived upon wetting. The liquid limit of a soil containing substantial amounts of organic matter decreases dramatically when the soil is oven-dried before testing. Thus a comparison of the liquid limit of a sample before and after oven-drying is used as a qualitative measure of a soil's organic matter content (see Table 5-1).

5.3.8 Expansive Soils

Soils that shrink and swell to extremes are called expansive soils, swelling soils, heaving soils, and/or volume change soils. In regions that are generally wet, such soils are referred to as shrinkable soils since problems occur in wet areas when things start to dry out. In general, soils with a plasticity index (PI) between 20 and 40 have moderate expansive properties. Those with PIs between 40 and 60 are highly expansive, and those with a PI greater than 60 are very expansive.

ANSI/ASAE EP486.2 states that soil with an expansion index greater than 20, as determined in accordance with ASTM D 422, is considered expansive and should be avoided. A soil is also considered expansive if it meets both of the following criteria.

1. Plasticity index (PI) of 15 or greater, determined in accordance with ASTM D 4318.
2. More than 10 per cent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D 422.

5.3.9 Unified Soil Classification System (USCS)

The USCS separates soils into 15 major groups in accordance with the criteria outlined in Table 5-1. The diagram in figure 5-8 is required for this classification. Each group has a two letter designation. The first letter identifies the predominate soil type: G = gravel, S = sand, M = silt, C = clay, O = organic, and PT = peat. The second letter provides additional information: W = well graded, P = poorly graded, M = coarse material with nonplastic fines or fines with low plasticity, C = coarse material with plastic fines, L = liquid limit less than 50%, and H = liquid limit above 50%.

An overview of Table 5-1 shows that a particle size distribution curve, liquid limit and plastic limit are all that are needed to classify soils in accordance with the Unified Soil Classification System.

5.3.10 Backfill Materials

Backfill properties can have a significant impact on post/pier foundation behavior. Common backfill materials include excavated soil, coarse-grained soils, concrete, and controlled low-strength material (CLSM).

Except as excluded in section 5.3.8, excavated soil can generally be used for backfill. In the special case where holes are drilled in clay, it may be preferable to backfill with the excavated clay instead of a coarse-grained material for frost heave reasons. In all cases, excavated material used as backfill should be compacted to its pre-excavation density and should be free of organic material and construction debris.

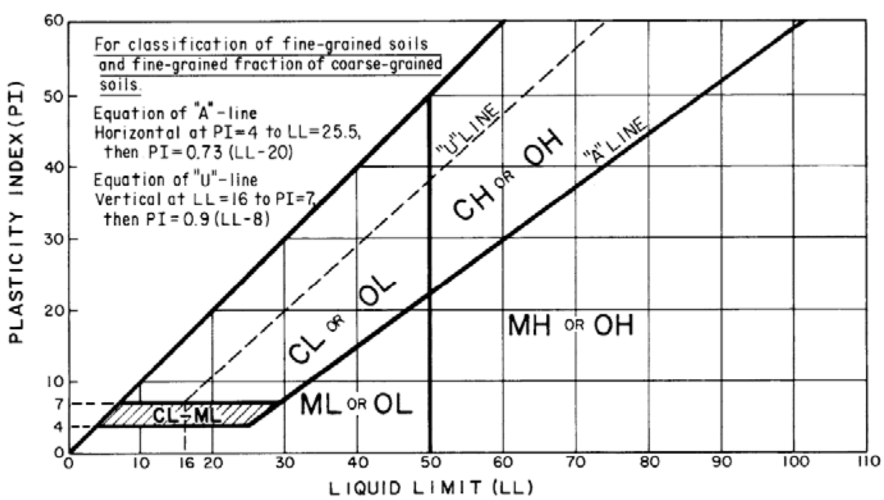


Figure 5-8. Plot used to differentiate between high and low PI soils for the Unified Soil Classification System (see Table 5-1).

Table 5-1. Unified Soil Classification System (USCS) from ASTM D 2487

Criteria for Assigning Group Symbols and Names Using Laboratory Tests (Based on material passing the 3-inch (75 mm) sieve)				Group Symbol	Typical Names
Course-Grained Soils More than 50% retained on the No. 200 sieve	**Gravels** 50% or more of course fraction retained on the No. 4 sieve	Clean Gravels (Less than 5% fines[A])	$C_U \geq 4$ and $1 \leq C_C \leq 3$ [B]	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			$C_U < 4$ and/or $1 > C_C > 3$ [B]	GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines (more than 12% fines[A])	Fines classify as ML or MH	GM[D]	Silty gravels, gravel-sand-silt mixtures
			Fines classify as CL or CH	GC[D]	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of course fraction passes the No. 4 sieve	Clean Sands (Less than 5% fines[C])	$C_U \geq 6$ and $1 \leq C_C \leq 3$ [B]	SW	Well-graded sands and gravelly sands, little or no fines
			$C_U < 6$ and/or $1 > C_C > 3$ [B]	SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines (More than 12% fines[C])	Fines classify as ML or MH	SM[D]	Silty sands, sand-silt mixtures
			Fines classify as CL or CH	SC[D]	Clayey sands, sand-clay mixtures
Fine-Grained Soils 50% or more passes the No. 200 sieve	**Silts and Clays** Liquid Limit less than 50%	Inorganic	PI > 7 and plots on or above "A" line	ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
			PI < 4 or plots below "A" line	CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays
		Organic	Oven dried liquid limit divided by regular liquid limit is less than 0.75	OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit 50% or more	Inorganic	PI plots on or above "A" line	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			PI plots below "A" line	CH	Inorganic clays or high plasticity, fat clays
		Organic	Oven dried liquid limit divided by regular liquid limit is less than 0.75	OH	Organic clays of medium to high plasticity
Highly Organic Soils	Primarily organic matter, dark in color, and organic odor			PT	Peat, muck, and other highly organic soils

[A] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt; GW-GC well-graded gravel with clay; GP-GM poorly graded gravel with silt; GP-GC poorly graded gravel with clay.

[B] C_U = Uniformity Coefficient = D_{60}/D_{10} C_C = Coefficient of Curvature $=(D_{30})^2/(D_{60}\ D_{10})$

[C] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt; SW-SC well-graded sand with clay; SP-SM poorly graded sand with silt; SP-SC poorly graded sand with clay.

[D] If fines classify as CL-ML, use dual symbol GC-GM or SC-SM

Replacing excavated material with a gravel or well-graded sand may be necessary where greater soil strength and stiffness are needed. Compact all backfill by tamping layers that do not exceed a thickness of 8 inches (0.2 m).

Cast-in-place concrete and controlled low-strength material (CLSM) are more expensive, but can significantly enhance the lateral strength of a foundation. This is because the width, *b*, of a foundation for lateral strength analysis is equated to the diameter of the concrete or CLSM backfill. Note that concrete and CLSM placed against soil may increase the potential for frost heaving.

Where CLSM is used to increase the effective width of a post/pier for lateral strength of a post/pier foundation, a CLSM unconfined compressive strength greater than 150 lbf/in^2 (1 MPa) is essential. CLSM with an unconfined compressive strength less than 150 lbf/in^2 can generally be excavated (broken up) using hand tools (e.g. shovels, picks) and machinery (e.g. excavators, backhoes) fitted with conventional buckets. Percussive devices such as jackhammers, impact hammers and rotary drills are generally required to break up CLSM with unconfined compressive strengths greater than 150 lbf/in^2.

5.4 Engineering Properties of Soil

5.4.1 General

The primary soil properties utilized in post and pier foundation design include: Young's modulus for soil, E_S; undrained soil shear strength, S_U; drained soil friction angle ϕ'; and moist unit weight, γ. In this section presumptive values for these properties are presented along with equations for calculating the properties from standard laboratory and in-situ tests.

5.4.2 Presumptive Soil Properties

Table 5-2 contains presumptive soil properties as tabulated in ANSI/ASABE EP486.2. These values may be used in the absence of satisfactory soil test data or specific building code requirements.

Data tabulated in Table 5-2 are unfactored values for use with the resistance and safety factors presented in Section 5.7. Because the values in Table 5-2 have not been pre-adjusted to account for a margin of safety in design, they will appear to be less conservative than data appearing in many presumptive soil property tables.

Since the range of possible void ratios in silts (types ML and MH soils) and gravels (types GW and GP soils) is relatively small, the unit weights for these soils do not largely change with variations in consistency, and thus have been assigned constant values in Table 5-2.

5.4.3 Soil Tests

Either laboratory or in-situ testing or a combination of laboratory and in-situ testing can be used to obtain all necessary information needed for post/pier foundation design.

Site-specific soil test results almost always result in higher design values than would be obtained using Table 5-2 values. This is because presumptive values are the lowest values associated with a broad classification of soils, each at their minimum strength conditions. Additionally, soil tests remove uncertainty associated with use of presumptive soil properties, and thus lower factors of safety are associated with calculations where soil characteristics have been ascertained through test.

Since certain soil tests are more accurate than others for obtaining a specific soil property, factors of safety are a function of soil test method. Test procedures deemed the most accurate for obtaining various soil properties can be determined by a comparison of factor of safety values presented in Section 5.7.

When establishing soil properties, assume that all cohesive soils will be loaded undrained, even under long-term static loadings, and that all cohesionless soils will be loaded drained, even under rapid loadings such as those resulting from earthquakes and wind forces.

5.4.3.1 Sampling Location and Depth

For uplift and lateral strength assessments, soil sampling and in-situ soil tests should cover the distance between one-third and 100% of the anticipated foundation depth. For bearing strength assessment, in-situ soil tests should be taken at a location between the anticipated footing base and a distance *B* below the anticipated footing depth.

A minimum site investigation generally includes at least three borings, usually combined with standard penetration testing. For a rectangular structure, a boring at each corner and one in the center of the structure is recommended, with more required depending on soil complexity and variability, and the size and importance of the structure.

5.4.4 Young's Modulus for Soil, E_S.

Young's modulus, E_S, is used to predict the lateral movement of a foundation. E_S can be determined from laboratory tests or from in-situ soil tests. In general, it is best to reserve laboratory tests for backfills; that is, highly disturbed materials without a stress history. E_S for non-backfill materials is generally best estimated using field (in-situ) tests because of the significance of stress history on E_S and the difficulty of obtaining undisturbed soil samples for laboratory testing.

Common laboratory tests include triaxial compression tests conducted in accordance with ASTM D2166 and D2850. E_s for most cohesive soils can also be determined using an unconfined compression test in accordance with ASTM D3080.

In-situ tests for E_S include prebored pressuremeter tests (PBPMT), cone penetration tests (CPT), and standard penetration tests (SPT).

When a prebored pressuremeter test is used, E_S can be calculated as:

$$E_S = (E_O + E_R) / 2$$

where: E_O is the pressuremeter first load modulus and E_R is the pressuremeter reload modulus calculated in accordance ASTM D4719.

When a cone penetrometer test is used, E_S for sandy soils can be calculated as:

$E_S = 1.5\ q_{cr}$ for silts, sands and silty sands

$E_S = 2\ q_{cr}$ for young, normally consolidated sands

$E_S = 3\ q_{cr}$ for aged, normally consolidated sands

$E_S = 4\ q_{cr}$ for sand and gravel

where: q_{cr} is the average cone resistance determined in accordance with ASTM D3441.

Table 5-2. Presumptive Soil Properties for Post and Pier Foundation Design from ANSI/ASAE EP486.2

Soil Type	Unified Soil Classifi-cation	Consistency	Moist unit weight, γ	Drained soil friction angle[a], ϕ'	Undrained soil shear strength[b], S_U	Young's modulus for soil[c][d], E_S	Increase in Young's modulus per unit depth below grade[c][d][e], A_E		Pois-son's ratio[f], ν
			lbf/ft^3	deg	lbf/in^2	lbf/in^2	$\frac{\text{lbf}}{\text{in}^2\text{-ft}}$	$\frac{\text{lbf}}{\text{in}^3}$	
Homogeneous inorganic clay, sandy or silty clay	CL	Soft	125	NA	3.5	3920	-	-	0.5
		Medium to Stiff	130		7	6160	-	-	
		Very Stiff to Hard	135		14	8400	-	-	
Homogeneous inorganic clay of high plasticity	CH	Soft	110	NA	3.5	1680	-	-	0.5
		Medium to stiff	115		7	2800	-	-	
		Very Stiff to Hard	120		14	4480	-	-	
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand of low plasticity	ML	Soft	120	NA	3.5	3920	-	-	0.5
		Medium to stiff			7	6160	-	-	
		Very Stiff to Hard			14	8400	-	-	
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand of high plasticity	MH	Soft	105	NA	3.5	1680	-	-	0.5
		Medium to stiff			7	2800	-	-	
		Very Stiff to Hard			14	4480	-	-	
Silty or clayey fine to coarse sand	SM, SC, SP-SM, SP-SC, SW-SM SW-SC	Loose	105	30	NA	-	440	37	0.3
		Medium to Dense	110	35		-	660	55	
		Very Dense	115	40		-	880	73	
Clean sand with little gravel	SW, SP	Loose	115	30	NA	-	880	73	0.3
		Medium to Dense	120	35		-	1320	110	
		Very Dense	125	40		-	1760	147	
Gravel, gravel-sand mixture, boulder-gravel mixtures	GW, GP	Loose	135	35	NA	-	2640	220	0.3
		Medium to Dense		40		-	3520	293	
		Very Dense		45		-	4400	367	
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay	GW-GC GC, SC	Loose	120	35	NA	-	1320	110	0.3
		Medium to Dense	125	40		-	1760	147	
		Very Dense	130	45		-	2200	183	

(a) Rapid undrained loading will typically be the critical design scenario in these soils. Laboratory testing is recommended to assess clay friction angle for drained loading analysis.

(b) Loading assumed slow enough that sandy soils behave in a drained manner.

(c) Estimate of stiffness at a rotation of 1^{o} for use in approximating structural load distribution. Use values that are 1/3 of the tabulated values for serviceability limit state evaluations.

(d) Constant values of stiffness used for calculation of clay response. Stiffness increasing with depth from a value of zero used for calculation of sand response.

(e) Assumes soil is located below the water table. Double the tabulated A_E value for soils located above the water table.

(f) Poisson ratio of 0.5 (no volume change) assumes rapid undrained loading conditions.

When a standard penetrometer is used, E_S in lbf/in^2 can be assumed to equal 56 $(N_1)_{60}$ for silts, sandy silts, and slightly cohesive soils; 97 $(N_1)_{60}$ for clean fine to medium sands and slightly silty sands; 140 $(N_1)_{60}$ for coarse sands and sands with little gravel; and 170 $(N_1)_{60}$ for sandy gravel and gravels. $(N_1)_{60}$ is the N_{60} blow count normalized with respect to vertical effective stress and is given as:

$$(N_1)_{60} = N_{60}\,(p_A/\,\sigma'_v)^{1/2}$$

where: N_{60} is the N_{SPT} blow count corrected for field procedures and equipment; p_A is atmospheric pressure (2090 lbf/ft^2); and σ'_v is vertical effective stress. The SPT blow count N_{SPT} is determined for clayey soils in accordance with ASTM D1586 and for sandy soils in accordance with ASTM D6066.

Young's modulus for soil can also be estimated from undrained shear strength, S_U. For soft sensitive clay, E_S ranges from 400 S_U to 1000 S_U. For medium stiff to stiff clay, E_S ranges from 1500 S_U to 2400 S_U. For very stiff clay, E_S ranges from 3000 S_U to 4000 S_U

The presumptive values in Table 5-2 assume that E_S is constant with depth for silts and clays, and increases linearly with depth for sands and gravels. To calculate E_S for sands and gravels, multiple the A_E value in the second last column of Table 5-2 by depth, z. In equation form:

$$E_{S,z} = A_E\, z \qquad (5\text{-}1)$$

where:

$E_{S,z}$ = E_S that is equal to zero at grade and increases linearly with depth z below grade, lbf/in^2 (kPa)
A_E = increase in Young's modulus per unit increase in depth z below grade, lbf/in^3 (kN/m^3)
z = depth below grade, in (m)

5.4.5 Undrained Shear Strength, S_U

Undrained soil shear strength is used to calculate bearing capacity, uplift resistance and lateral strength in cohesive soils. Like Young's modulus, undrained shear strength can be determined from both laboratory and in-situ testing.

For a cohesive soil, undrained soil strength is determined using an unconfined compressive strength test in accordance with ASTM D2166 or an unconsolidated-undrained triaxial compression test in accordance with ASTM D2850. The primary result of ASTM D2166 is the unconfined compressive strength of the soil, q_u. The undrained shear strength, S_U, as determined using ASTM D2166, is equal to one-half the unconfined compressive strength q_u. ASTM D2850 does not directly produce the value for undrained shear strength S_U. To determine S_U using ASTM D2850, several (typically three) tests are required at different confining pressures, and S_U is equal to the cohesion intercept of the failure envelope drawn tangent to the Mohr's circle for all individual tests.

Three in-situ tests used to determine undrained soil shear strength are the prebored pressuremeter test, the cone penetration test and the field vane test.

When the prebored pressuremeter test is used, S_U is equated to 0.41 $p_L^{0.75}$ where both S_U and p_L are given in lbf/in^2 and p_L is the limit pressure determined in accordance with ASTM D4719.

Where a cone penetrometer is used, S_U is equated to 0.037 q_{cr} where q_{cr} is average cone resistance determined in accordance with ASTM D3441.

When a field vane test is conducted, S_U of cohesive soils is determined directly from the torque applied to the vane shear device in accordance with ASTM D2573.

5.4.6 Soil Friction Angle, ϕ

Soil friction angle largely controls the strength of cohesionless soils. It is required to calculate the uplift resistance, U, provided by a cohesionless soil. When ultimate bearing capacity, q_B, is not determined via in-situ tests, ϕ is used in the general bearing capacity equation to determine q_B of cohesionless soils. Likewise, ϕ is used to calculate the ultimate lateral resistance pressure, p_U, where p_U has not been determined by in-situ testing.

The primary laboratory tests used to determine friction angle ϕ of a cohesionless soil are the direct shear test which is conducted in accordance with ASTM D3080 and the consolidated-drained (CD) triaxial compression test.

In-situ tests for soil friction angle include the standard penetration test and the cone penetration test.

When the standard penetration test is conducted on sandy soils, the soil friction angle is calculated as:

$$\phi = [20\,(N_1)_{60}]^{0.5} + 20$$

where:

$$(N_1)_{60} = N_{60}\,(p_A/\,\sigma'_v)^{1/2}$$

and $(N_1)_{60}$ is the N_{60} blow count normalized with respect to vertical effective stress; N_{60} is the N_{SPT} blow count corrected for field procedures and equipment; p_A is atmospheric pressure (2090 lbf/ft^2); and σ'_v is vertical effective stress.

When the cone penetrometer is used in sandy soils, friction angle is calculated as:

$$\phi = 17.6 + 11.0 \log\,[q_{cr}/(p_A\,\sigma'_v)^{0.5}]$$

where: q_{cr} is average cone resistance; p_A is atmospheric pressure (2090 lbf/ft^2); and σ'_v is vertical effective stress.

5.4.7 Moist Unit Weight, *γ*

Vertical and lateral soil resisting pressures increase with increases in soil unit weight (i.e., density) and depth. This is because soil confinement pressures increase as both of these variables increase.

Soil moist unit weight is generally determined in accordance with ASTM D2937 *Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method* or ASTM D1587 *Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes.*

Moisture content should always accompany any measurement of unit weight. The two most common procedures for determining soil moisture content are ASTM D2216 *Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass* and ASTM D4643 *Test Method for Determination of Water (Moisture) Content of Soil by Microwave Oven Heating*

5.5 Foundation Material Properties

5.5.1 General

This section contains material requirements for post and pier foundation elements. Elements not specifically addressed by the following requirements shall be designed in accordance with applicable normative references, building codes, standards, and good engineering judgment.

5.5.2 Minimum Concrete Compressive Strength

ANSI/ASAE EP486.2 requires that all concrete used in footings, posts and piers have a minimum 28-day compressive strength of 3000 lbf/in^2. This requirement is consistent with ACI 318 *Building Code Requirements for Structural Concrete* and is important for application of the prescriptive minimum plain concrete footings sizes presented in Sections 5.5.3 and 5.5.4.

5.5.3 Cast-in-Place Concrete Footings

The minimum nominal thickness allowed for an unreinforced (plain) footing that is cast-in-place on a compacted base is 8 inches. Load-induced forces may dictate a thicker footing. Also, the thickness of a footing shall be such that the concrete provides a minimum cover of 3 inches above and below any required steel reinforcement.

Cast-in-place concrete footings do not require steel reinforcement when the actual maximum distance from a footing edge to the nearest post/pier edge is less than the nominal thickness of the footing. This is because under such geometric conditions, arch action ensures footing tensile stresses are minimal under critical bearing loads. For example, if a post with actual dimensions of 4.5- by 5.5-inches is centered on a footing with a diameter of 14 inches, reinforcement would not be required as long as the footing had a nominal thickness of at least 7 inches – 4.5 inches/2 = 4.75 inches (i.e., the footing radius minus half the narrow dimension of the post). In this case, the 4.75 inches is guaranteed by the required minimum nominal thickness of 8 inches for plain cast-in-place footings.

Where the preceding edge distance requirement is not met, the need for reinforcement shall be determined in accordance with ACI 318 Chapter 15.

5.5.4. Precast Concrete Footings

In accordance with ANSI/ASAE EP486.2, the actual thickness of unreinforced (plain) precast footing that is placed on a flat, compacted base shall not be less than 4 inch. Load-induced forces may dictate a thicker footing. Also, the thickness of a footing shall be such that the concrete provides a minimum cover of 1.5 inches above and below any required steel reinforcement.

Precast concrete footings do not require steel reinforcement when the actual maximum distance from a precast footing edge to the nearest post/pier edge is less than 1.25 times the actual thickness of the footing. This is because under such geometric conditions, arch action ensures footing tensile stresses are minimal under critical bearing loads. For example, if a post with actual dimensions of 4.5- by 5.5 inches is centered on a precast footing with a diameter of 14 inches, reinforcement would not be required as long as the footing had a nominal thickness greater than (7 inches – 4.5 inches/2)/1.25 = 3.8 inches. In this case, the 3.8 inches is guaranteed by the required minimum actual thickness of 4 inches established for precast footings.

The 1.25 factor in the preceding calculation compensates for the fact that the *maximum* distance from a footing edge to the nearest post/pier edge is used in the calculation. This maximum distance is generally measurably greater than the *average* distance between the edge of the footing and the nearest post/pier edge. The 1.25 factor is not allowed in the design of cast-in-place footings because of greater variation in the actual size of cast-in-place footings, and because once they have been cast, cast-in-place footings cannot be shifted to improve alignment with the posts/piers they support.

When sizing reinforcement for larger precast footings, the design must consider the extent and location of contact between the base of the placed footing and the underlying compacted base.

5.5.5 Concrete Piers

Axial, shear and bending forces in most concrete piers are such that the assemblies must be treated as structural columns. ACI 318 clause 22.2.2 requires that all structural columns contain reinforcement and thus be designed in accordance with Chapters 10, 11 and 12 of the ACI code.

ACI 318 clause 10.9.1 requires that the cross-sectional area of longitudinal reinforcement not be less than 1.0 percent of the gross cross-sectional area of the concrete. ACI 318 clause 10.9.2 requires that no less than four longitudinal bars be used within rectangular or circular ties, no less than three longitudinal bars be placed within triangular ties, and no less than six longitudinal bars be enclosed with spirals.

The location and size of shear reinforcement in concrete piers is determined in accordance with ACI 318 Chapters 11. Shear reinforcement is not required where tests show that the required bending strength and shear strengths can be developed when the shear reinforcement is omitted.

When a concrete pier is formed by casting concrete directly against earth, a minimum concrete cover of 3 inches is required on all steel reinforcement. When concrete is cast on site but not directly against the earth (e.g., the concrete is cast into cardboard forming tubes), the minimum concrete cover on steel reinforcement can be reduced to 2 inches for bars No. 6 or larger (bars 19 mm or greater in diameter) and 1.5 inches for No. 5 or smaller bars (bars 13 mm or smaller in diameter). Minimum required concrete cover on reinforcement in precast concrete piers (i.e., piers manufactured under plant control conditions) is 1.5 inches for No. 6 or larger bars and 1.25 inches for No. 5 or smaller bars (ACI 318 clause 7.7.1 and 7.7.3).

5.5.6 Embedded Wood Posts and Piers

Wood used for embedded posts and piers must be preservative treated in accordance with AWPA U1 Use Category UC4B.

Mechanically-laminated wood posts and piers shall be sized in accordance with ANSI/ASAE EP559. All other wood posts and piers shall be sized in accordance with ANSI/AWC NDS.

Fasteners used below grade in mechanically-laminated wood posts and piers shall meet the requirements of ANSI/ASAE EP559.

5.5.7 Anchor Attachments

Fasteners used below grade to attach collars, footings and other devices to resist uplift forces shall have a durability equal to the service life of the structure.

5.5.8 CLSM Base for Precast Concrete and Wood Footings

A controlled low-strength material (CLSM) placed between the bottom of a precast concrete or wood footing and the underlying soil can be used to increase the effective bearing area of the footing when its unconfined compressive strength exceeds the ultimate bearing capacity of the underlying soil.

In lieu of using a CLSM base for footings, some builders have compacted a non-hydrated (i.e., dry) concrete mix in the base of holes drilled for pier/post foundation placement. Tests conducted by Bohnhoff et al. (2003) have shown that non-hydrated concrete mixes that are compacted within a soil mass and allowed to self-hydrate, will obtain unconfined compressive strengths that more than double the 1160 lbf/in^2 (8 MPa) limit for classification as a controlled low-strength material.

Non-hydrated concrete mixes that are confined below a footing can be assumed to have the same high bearing strength as any other dry soil. Thus there is no need to be concerned about forces placed on the footing prior to hydration of the concrete mix.

5.6 Structural Analysis

5.6.1 Introduction

Structural analysis is the determination of forces induced in building components by applied structural loads. For post-frame buildings featuring post/pier foundation systems, such structural analyses can be measurably influenced by horizontal deformation of the soil in contact with the posts/piers.

To account for soil deformation during the structural analysis of a building frame with a shallow post/pier foundation, the soil in contact with the foundation is modeled with a series of horizontal springs. This consists of determining the effective Young's modulus for the soils surrounding the foundation (Section 5.6.2), selecting locations for the springs that will be used to model the surrounding soil (Section 5.6.3), and then assigning properties to the soil springs (Section 5.6.4). An example structural analysis that utilizes soil springs is presented in Section 5.6.5.

Section 5.6.6 contains equations that can be used to approximate the lateral movement of a post/pier foundation due to the application of a groundline bending moment M_G and groundline shear force V_G. Use of these equations is restricted to assumption inherent in their development. These assumptions are overviewed in Section 5.6.6. An application of Section 5.6.6 equations is presented in Section 5.6.7.

5.6.2 Effective Young's Modulus for Soil, E_{SE}

The key soil property required for structural analysis of a pier/post foundation is the effective Young's modulus for the soil, E_{SE}. At a given depth, E_{SE} is a function of Young's modulus of the backfill material, $E_{S,B}$, and the Young's modulus for the surrounding unexcavated soil $E_{S,U}$. In equation form:

$$E_{SE} = \frac{1}{I_S / E_{S,B} + (1\text{-}I_S)/ E_{S,U}} \quad \text{for } 0 < J < 3b \qquad (5\text{-}2a)$$

$$E_{SE} = E_{S,B} \quad \text{for } J \geq 3b \qquad (5\text{-}2b)$$

$$E_{SE} = E_{S,U} \quad \text{for } J = 0 \qquad (5\text{-}2c)$$

Where:

$E_{S,B}$ = E_S for backfill at depth z
$E_{S,U}$ = E_S for the unexcavated soil surrounding the backfill at depth z
I_S = strain influence factor, I_S, dimensionless
= $[\ln(1 + J/b)]/1.386$ for $0 < J < 3b$ (5-3)
J = distance (measured in the direction of laterally foundation movement) between the edge of the backfill and the face of the foundation component at depth z (see figure 5-9)
b = width of the post, collar, footing that is surrounded by the backfill at depth z

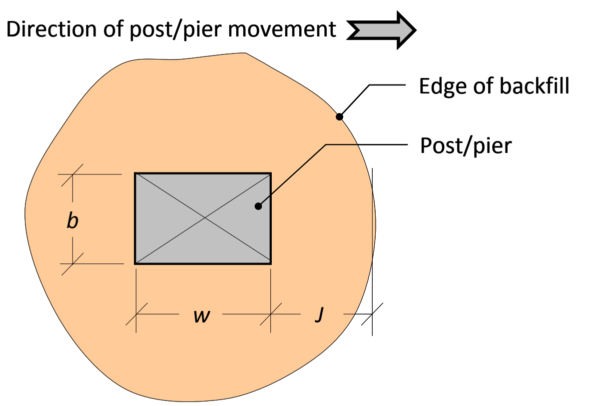


Figure 5-9. Top view of foundation showing distance *J* between the post/pier/footing/collar and the edge of the backfill.

The strain influence factor is the fraction of total lateral displacement that is due to soil straining within a distance J of the face of the foundation.

When the foundation is surrounded by unexcavated soil, $J = 0$ and $E_{SE} = E_{S,U} = E_S$ of the unexcavated soil (equation 5-2c). Such is the case when a post is driven into the soil, or a helical pier is turned into the soil.

Equation 5-2c also applies to those portions of a foundation that are entirely backfilled with concrete or controlled low-strength material (CLSM). In this case, E_{SE}, is equated to E_S for the soil surrounding the concrete or CLSM.

5.6.2.1 Example E_{SE} Calculation

Problem Statement

A post foundation consists of a 3-ply post fabricated from 2- by 10-inch lumber resting on a precast concrete footing. Backfill is 18 inches in diameter and classified as a medium to dense SW-SM soil. The surrounding unexcavated soil is classified as a medium to stiff ML soil. The water table is 2 feet below the footing. What is E_{SE} for the foundation at a location 3 feet below grade?

Effective Young's Modulus for Soil, E_{SE}

From Table 5-2, the backfill material (medium to dense SW-SM soil) has an A_E value of 1320 lbf/in^2/ft (note: the Table 5-2 value of 660 lbf/in^2/ft is doubled because the soil is located above the water table). Young's modulus at a depth of 3 feet for this backfill, $E_{S,B}$, is equal to the product of A_E and 3 feet or 3960 lbf/in^2. The unexcavated soil (medium to stiff ML soil) has an $E_S = E_{S,U}$ value of 6160 lbf/in^2 that is constant with depth.

With a backfill diameter of 18 inches and post side width of 9.25 inches, the distance J between the edge of the backfill and center of the post face is (18 in. – 9.25 in.)/2 = 4.375 in. Substituting this into equation 5-3 along with a post face width b of 4.5 inches yields a strain influence factor I_S of 0.49. Substituting this into equation 5.2a, with an $E_{S,B}$ of 3960 lbf/in^2 and $E_{S,U}$ of 6160 lbf/in^2 yields an effective Young's modulus E_{SE} of 4842 lbf/in^2

5.6.3 Soil Spring Location

To locate soil springs, first draw horizontal sectioning lines wherever there is an abrupt change in soil type, backfill type, and/or width of the post/pier foundation. Each layer resulting from this sectioning must be modeled with at least one soil spring. To determine if more than one spring is required for a particular layer, follow the ANSI/ASAE EP486.2 recommendation that soil spring spacing t should not exceed $2w$ where w is the side width of a rectangular post/pier (see figure 5-9) or diameter of a round post/pier.

Locating soil springs is illustrated in Figures 5-10 and 5-11. Figure 5-10 shows six soil springs being used to model a non-constrained post in a multi-layered soil. Since the footing is not attached to the post, there is no need for a soil spring to model soil in contact with the footing. It is important to note that such an assumption ignores friction between the post and footing. Although three springs are being used to model the resistance provided by each soil layer, two per layer would be sufficient given that the thickness of each soil layer does not appear to exceed more than 4 times the width w.

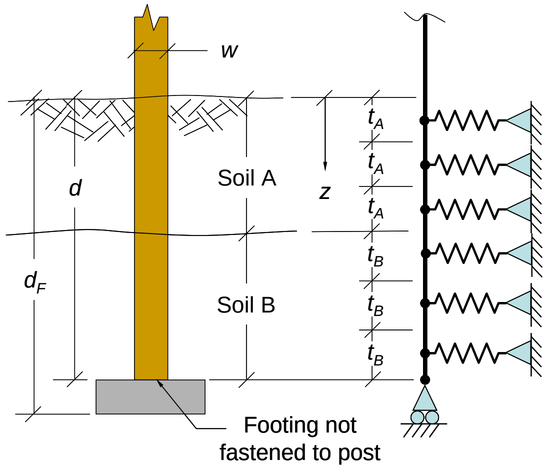


Figure 5-10. Modeling a non-constrained post in a layered soil. Footing not attached to rest of foundation.

Figure 5-11 shows the modeling of a non-constrained post that has an attached footing and an attached collar. Note that individual springs are required for both the footing and the collar because they each have different widths relative to the post.

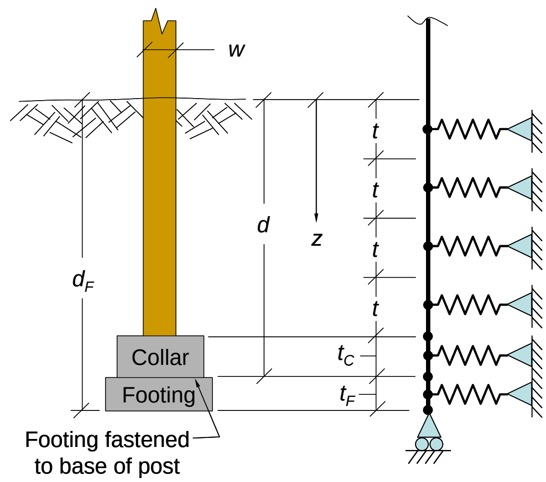


Figure 5-11. Modeling soil behavior when both footing and collar are attached to the post

In addition soil springs, other restraints ssociated with post/pier foundation modeling include placement of a horizontal roller support at the foundation base as shown in Figures 5-10 and 5-11. Such a support ignores friction between the foundation and underlying soil.

Resistance provided by surface restraint(s) must also be modeled. Figure 5-12 shows an embedded post that abuts a slab-on-grade. To model the restraint that the slab provides when the post moves toward the slab, the slab is modeled as a vertical roller support (figure 5-12(a)). Because the slab only abuts the inside of the post and is not attached to the post, it does not apply a force to the post when the post moved away from the slab, and thus is simply modeled as a non-constrained post (figure 5-12(b)).

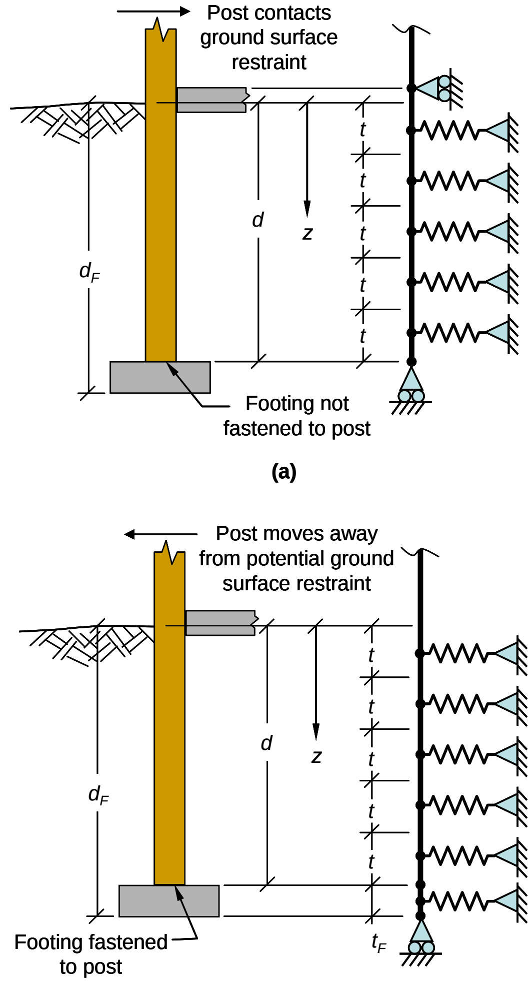


Figure 5-12. Modeling an embedded post abutting a slab-on-grade when the post moves (a) toward the slab, and (b) away from the slab.

5.6.4 Soil Spring Properties

All soil springs are assumed to exhibit linear-elastic behavior until a point of soil failure is reached, at which point the force in the soil spring stays constant as the spring undergoes additional deformation. A graphical depiction of this behavior is shown in 5-13.

The initial stiffness, K_H, of an individual soil spring located at depth z is given as

$$K_H = 2.0\ t\ E_{SE} \tag{5-4}$$

Where:

t = thickness of the soil layer represented by the spring, in. (m)

E_{SE} = effective Young's modulus for soil at depth z, lbf/in^2 (kN/m^2)

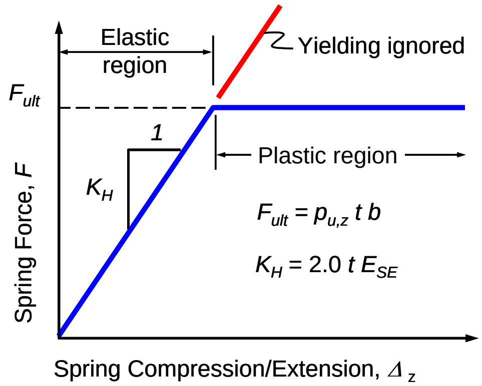


Figure 5-13. Load-displacement relationship for a soil spring.

For structural analyses used to determine load distribution within a building frame, yielding of soil springs is ignored. In other words, the stiffness of a soil spring is assumed to equal K_H regardless of the load applied to the spring. Thus, the value of F_{ult} displayed in figure 5-13 is not needed during the structural analysis phase of the design process.

Ignoring soil spring yielding during structural analyses is consistent with the modeling of steel frame members and all other components that do not exhibit linear-elastic behavior at high loads. The sole purpose of a structural analysis is to determine load distribution under service load conditions. When properly sized, no component (soil, steel, or otherwise) should be loaded to levels near those associated with plastic behavior or failure.

The equation 5-4 relationship between soil spring stiffness and the effective Young's modulus for soil is purely empirical. Selection of the 2.0 value appearing in equation 5-4 is explained in the ANSI/ASAE EP486.2 Commentary.

5.6.5. Example Structural Analysis with Soil Springs

Problem Statement

A nominal 6- by 6-inch No.2 SP post is embedded 4 feet. It rests on a concrete footing but is not attached to the footing. Two nominal 2- by 6-inch wood blocks, 12 inches in length, are bolted to each side of the base of the post to increase the uplift resistance and lateral strength capacity of the foundation. The top 2.5 feet of soil are classified as medium to stiff ML silts. The next several feet of soil below this clay layer are classified as medium to dense SW sands. The water table is located 7 to 8 feet below grade. Backfill is a mixture of the ML silt and SW sand removed by the 18-inch diameter auger used to form the post hole. The mixture is compacted by hand in 6- inch lifts.

If a bending moment of 20,000 in-lbf and a shear force of 1000 lbf are applied to the post foundation at the groundline, what is the resulting rotation and lateral displacement of the post foundation at the groundline?

Spring Location

Two depths are associated with an abrupt change in soil and/or post design properties that will affect spring location: a change in soil type at a depth of 30 inches, and a change in foundation width from 5.5 inches to 12 inches at a depth of 42.5 inches.

The post side width w of 5.5 inches equates to a maximum spring spacing of 11 inches (i.e., $t \leq 2w$). To meet this maximum spacing requires a minimum of six springs located as shown in Figure 5-14.

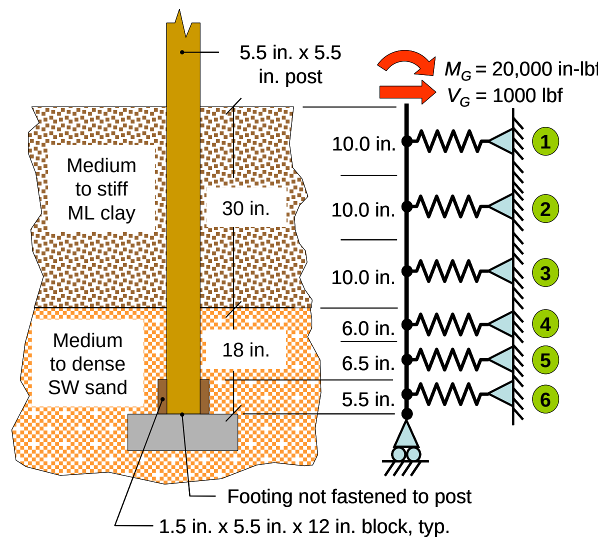


Figure 5-14. Non-constrained post foundation and corresponding spring analog.

Presumptive Soil Properties

From Table 5-2, the medium to stiff ML silt has a Young's modulus of 6160 lbf/in^2, and the medium to dense SW sand has an A_E value of 110 (lbf/in^2)/in. This A_E value is doubled to 220 (lbf/in^2)/inch because the soil represented by the springs is all located above the water table (see Table 5-2 footnote (e)).

The mixture of approximately 2.5 feet of ML silt with approximately 2 feet of SW sand is likely to produce a backfill that would grade out as a silty sand (SM). Determination of the exact designation would require knowledge of the particle size distributions of the ML and SW soils prior to mixing. Hand compaction of this backfill in 6-inch lifts should provide a medium to dense consistency. From Table 5-2, a medium to dense SM soil has an A_E of 55 (lbf/in^2)/in, which is doubled to 110 (lbf/in^2)/inch because the backfill is located entirely above the water table.

E_{SU} and K_H Calculations

Spring number	Thickness of soil layer represented, t	Distance from surface, z	Width of foundation at spring location, b
	inches	inches	inches
1	10	5	5.5
2	10	15	5.5
3	10	25	5.5
4	6	33	5.5
5	6.5	39.25	5.5
6	5.5	45.25	12

Spring number	Increase in Young's Modulus with Depth		Young's Modulus	
	Unexcavated Soil, $A_{E,U}$	Backfill, $A_{E,B}$	Unexcavated Soil, $E_{S,U}$	Backfill, $E_{S,B}$
	lbf/in^2/in	lbf/in^2/in.	lbf/in^2	lbf/in^2
1	-	110	6160	550
2	-	110	6160	1650
3	-	110	6160	2750
4	220	110	7260	3630
5	220	110	8635	4318
6	220	110	9955	4978

Spring number	Backfill thickness, J	Strain Influence Factor, I_s	Effective Young's Modulus, E_{SE}	Horizontal spring stiffness, K_H
	inches	-	lbf/in^2	lbf/in
1	6.25	0.548	935	18700
2	6.25	0.548	2467	49300
3	6.25	0.548	3669	73400
4	6.25	0.548	4691	56300
5	6.25	0.548	5579	72500
6	4.75	0.449	6870	75600

Structural Analysis

The model shown in Figure 5-14 was analyzed using IES Inc.'s VisualAnalysis program (VA, 2013) with the No.2 SP post assigned an E value of 1.2 million lbf/in^2. The predicted displaced shape of the foundation post under the 20,000 lbf-in groundline bending moment and 1000 lbf groundline shear force is shown in figure 5-15. Groundline displacement and rotation were found to be 0.093 inches and 0.4 degrees, respectively.

Like most commercially available structural analysis programs, VisualAnalysis contains a special spring element that makes modeling of soil behavior a very straight forward process (you simply input the node and direction for spring application along with spring stiffness).

In lieu of a special spring element, the resisting force applied to a post by soil can be modeled with a pinned-end element (a.k.a. a truss element) by equating the axial stiffness of the element (AE/L) to spring stiffness K_H (see figure 5-16). For K_H values in lbf/inch, this can be achieved by (1) positioning element nodes so they are exactly an inch apart, (2) setting the element's cross-sectional area equal to exactly one square inch, and (3) setting the element's E value in lbf/in^2 equal to the numeric value of spring stiffness K_H in lbf/inch.

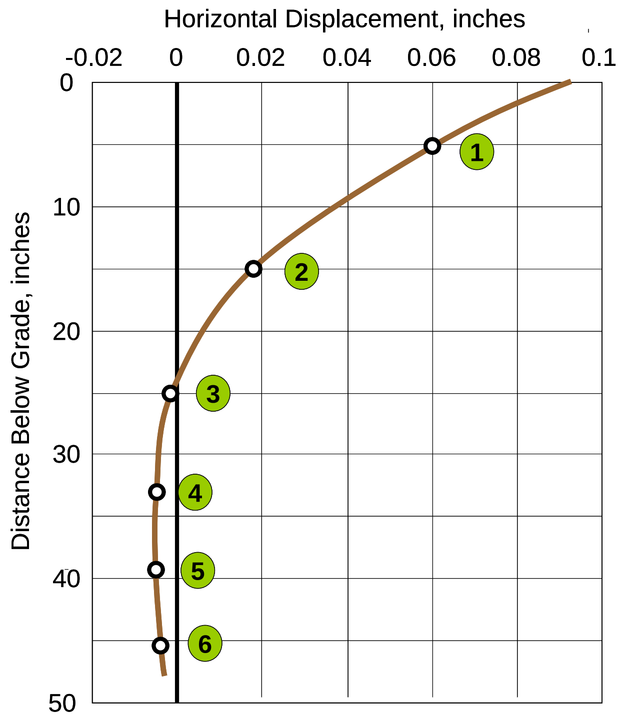


Figure 5-15. Displaced shape of the post foundation model shown in figure 5-14.

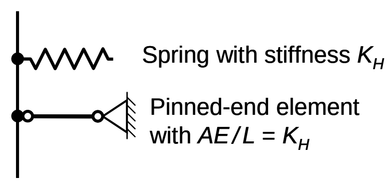


Figure 5-16. Two equivalent ways to model the lateral resisting force of soil.

5.6.6 Equations for Approximating Lateral Foundation Displacements and Soil Pressures

The lateral displacement of the below-grade portion of a post/pier foundation and associated soil pressures induced by a groundline bending moment M_G and groundline shear V_G can be estimated using equations in Sections 5.6.6.1 through 5.6.6.4. During the derivation of these equations, the following simplifying assumptions were made:

1. The below-grade portion of the foundation has an infinite flexural rigidity (EI).
2. Unexcavated soil and backfill is homogeneous for the entire embedment depth.
3. Effective Young's modulus for the soil, E_{SE}, is either constant for all depths below grade or linearly increases with depth below grade.
4. Width of the below-grade portion of the foundation is constant. This generally means that there are no attached collars, uplift anchors or footings that are effective in resisting lateral soil forces.

The first of the preceding simplifying assumptions - that the below-grade portion of the foundation is infinitely stiff - is assumed to hold where soil stiffness is assumed to increase linearly with depth and:

$$d \le 2\{EI/(2A_E)\}^{0.20} \tag{5-5}$$

or, where soil stiffness is assumed constant with depth and:

$$d \le 2\{EI/(2E_{SE})\}^{0.25} \tag{5-6}$$

Where: d is depth of embedment; EI is flexural rigidity of the post/pier foundation; E_{SE} is effective Young's modulus of the soil; and A_E is the linear increase in effective Young's modulus with depth below grade.

Other variables appearing in Sections 5.6.6.1 through 5.6.6.4 include:

- M_G = bending moment applied to foundation at grade (a.k.a. groundline bending moment)
- V_G = shear force applied to foundation at grade (a.k.a. groundline shear force)
- Δ = horizontal displacement of foundation at grade
  = 0 (zero) for surface-constrained foundation
- θ = rotation of the infinitely stiff foundation
- d_R = depth from the ground surface to the pivot point for foundation rotation
  = depth below the ground surface at which foundation does not displace horizontally
  = 0 (zero) for surface-constrained foundation
- p_z = pressure applied to soil by foundation at a depth z

5.6.6.1 Non-constrained posts/piers with linearly increasing E_{SE} (figure 5-17)

The following equations assume that the post/pier foundation is not restrained at grade, and that E_{SE} increases linearly with soil depth, and is numerically equal to $A_E z$.

$$d_R = \frac{d(3V_G d + 4M_G)}{4V_G d + 6M_G}$$

$$\theta = \frac{12V_G d + 18M_G}{d^4 A_E}$$

$$\Delta = \frac{9V_G d + 12M_G}{d^3 A_E}$$

$$p_z = 6z(6M_G z/d + 4V_G z - 3d V_G - 4M_G)/(d^3 b)$$

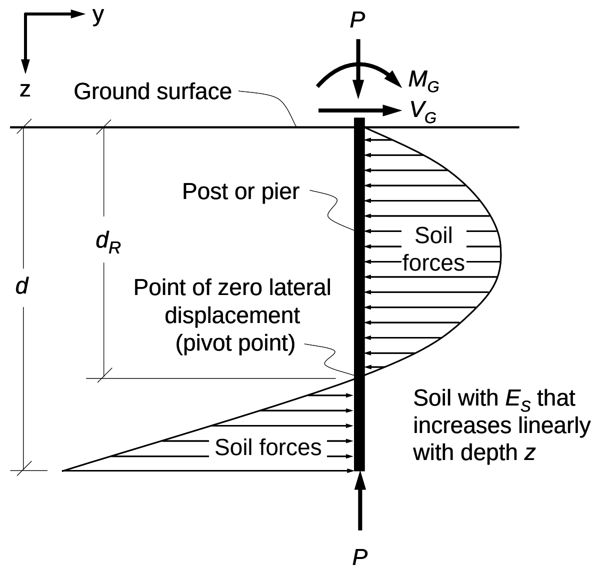


Figure 5-17. Forces acting on a non-constrained post/pier of fixed width *b* when E_{SE} increases linearly with depth.

5.6.6.2 Constrained posts/piers with linearly increasing E_{SE} (figure 5-18)

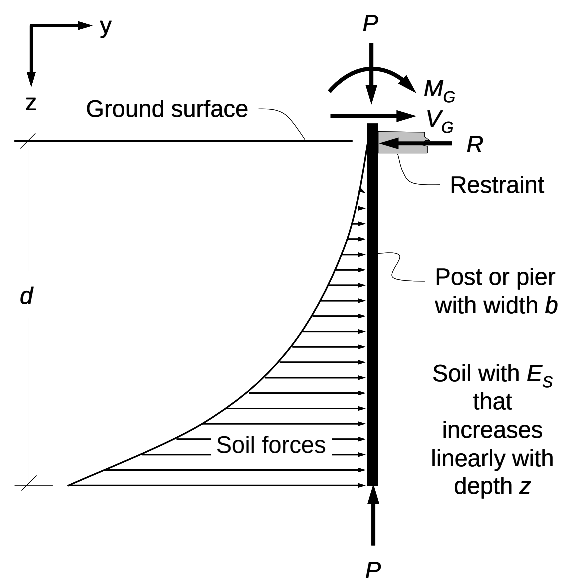


Figure 5-18. Forces acting on a ground surface-constrained post/pier of fixed width *b* when E_{SE} increases linearly with depth.

The following equations assume that the post/pier is restrained at the groundline and that E_{SE} increases linearly with soil depth, and is numerically equal to $A_E z$.

$$\theta = \frac{2M_G}{d^4 A_E}$$

$$p_z = 4z^2 M_G/(d^4 b)$$

5.6.6.3 Non-constrained posts/piers with constant E_{SE} (figure 5-19)

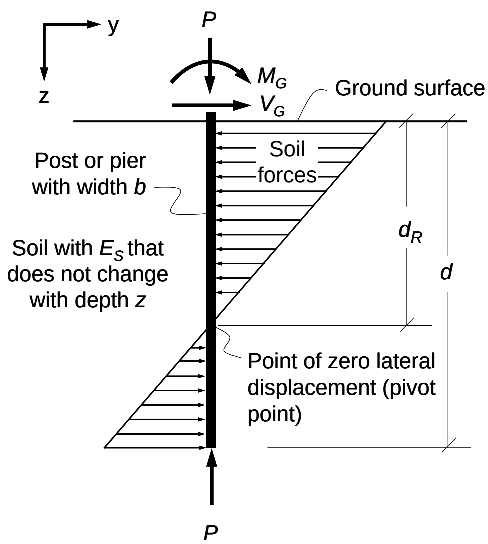


Figure 5-19. Forces acting on a non-constrained post/pier of fixed width *b* when E_{SE} is constant with depth.

The following equations assume that the post/pier foundation is non-constrained and that E_{SE} remains constant with depth.

$$d_R = \frac{d(2V_G d + 3M_G)}{3V_G d + 6M_G}$$

$$\theta = \frac{3V_G d + 6M_G}{d^3 E_{SE}}$$

$$\Delta = \frac{2V_G d + 3M_G}{d^2 E_{SE}}$$

$$p_z = (12M_G z/d + 6V_G z - 4dV_G - 6M_G)/(d^2 b)$$

5.6.6.4 Constrained posts/piers with constant E_{SE} (figure 5-20)

The following equations assume that the post/pier foundation is constrained at grade and E_{SE} remains constant with depth.

$$\theta = \frac{1.5M_G}{d^3 E_{SE}}$$

$$p_z = 3zM_G/(d^3 b)$$

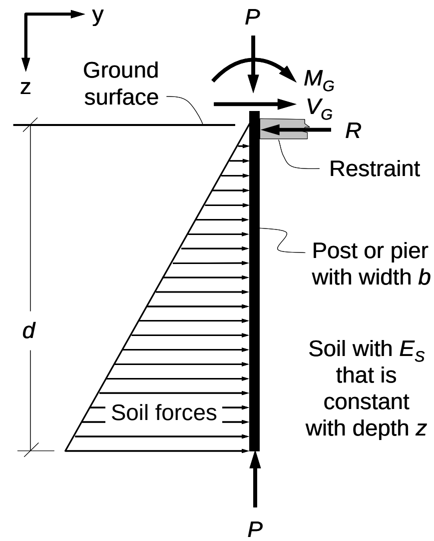


Figure 5-20. Forces acting on a ground surface-constrained post/pier of fixed width *b* when E_{SE} is constant with depth.

5.6.7 Example Analyses

Problem Statement 1

A non-constrained nominal 6- by 6-inch solid-sawn post with a modulus of elasticity of 1.2 million lbf/in^2 rests on a precast footing. The surrounding material (soil and backfill) has an effective Young's modulus that increases 155 lbf/in^2 per inch of embedment depth (i.e., A_E = 155 lbf/in^3). What is the maximum depth the post can be embedded for the equations in Section 5.6.6 to be fully applicable?

Solution

For the equations in Section 5.6.6 to be fully applicable, the inequality expressed by equation 5-5 must be met; that is, embedment depth *d* must be less than or equal to $2\{EI/(2A_E)\}^{0.20}$. Given a moment of inertia *I* of 76.25 in^4 for a nominal 6- by 6-inch post, the quantity $2\{EI/(2A_E)\}^{0.20}$ is equal to a relatively shallow depth of 24.8 inches.

Problem Statement 2

If the post described in problem 1 is embedded 24 inches (i.e., *d* = 24 inches), and a groundline bending moment M_G of 20,000 in-lbf and a groundline shear V_G of 1000 lbf are applied to the foundation, what will be the groundline displacement Δ and foundation rotation θ assuming the soil is not overstressed.

Solution

For an A_E of 155 lbf/in^3, Section 5.6.6.1 equations yield a horizontal displacement Δ of 0.21 inches, and a foundation rotation θ of 0.0126 radians (0.72 degrees).

5.7 Governing Strength Equations

5.7.1 Allowable Stress Design (ASD)

For allowable stress design, the soil surrounding a foundation has adequate strength when the following four inequalities are met:

$$Q_U \geq f_B P_{ASD} \tag{5-7}$$

$$M_U \geq f_L M_{ASD} \tag{5-8}$$

$$V_U \geq f_L V_{ASD} \tag{5-9}$$

$$U \geq f_U (P_{ASD} - g M_F) \tag{5-10}$$

where:

Q_U = Ultimate groundline bearing strength from Section 5.8.1
M_U = Ultimate groundline moment capacity (as limited by soil strength) from Section 5.9
V_U = Ultimate groundline shear capacity (as limited by soil strength) from Section 5.9
U = Ultimate uplift resistance due to soil mass from Section 5.10
f_B = ASD factor of safety for bearing strength assessment from Table 5-3
f_L = ASD factor of safety for lateral strength assessment from Table 5-4 or 5-5
f_U = ASD factor of safety for uplift strength assessment from Table 5-6
P_{ASD} = P_G due to an ASD load combination
M_{ASD} = M_G due to an ASD load combination
V_{ASD} = V_G due to an ASD load combination
P_G = axial force in foundation at the ground surface (at grade)
M_G = groundline bending moment (bending moment in foundation at the ground surface)
V_G = groundline shear force (shear force in foundation at the ground surface)
g = gravitation acceleration constant, 1.0 lbf/lbm (9.81×10^{-3} kN/kg)
M_F = foundation mass

Both equations 5-7 and 5-10 contain P_{ASD}. Equation 5-7 is only applicable when P_{ASD} is pushing down on the foundation at grade. Equation 5-10 is only applicable when P_{ASD} is pulling up on the foundation at grade.

5.7.2 Load and Resistance Factor Design (LRFD)

For load and resistance factor design, the soil surrounding a foundation has adequate strength when the following four inequalities are met:

$$Q_U R_B \geq P_{LRFD} \tag{5-11}$$

$$M_U R_L \geq M_{LRFD} \tag{5-12}$$

$$V_U R_L \geq V_{LRFD} \tag{5-13}$$

$$U R_U \geq P_{LRFD} - g M_F \tag{5-14}$$

where:

Q_U = Ultimate groundline bearing strength from Section 5.8.1
M_U = Ultimate groundline moment capacity (as limited by soil strength) from Section 5.9
V_U = Ultimate groundline shear capacity (as limited by soil strength) from Section 5.9
U = Ultimate uplift resistance due to soil mass from Section 5.10
R_B = LRFD resistance factor for bearing strength assessment from Table 5-3
R_L = LRFD resistance factor for lateral strength assessment from Table 5-4 or 5-5
R_U = LRFD resistance factor for uplift strength assessment from Table 5-6
P_{LRFD} = P_G due to a LRFD load combination
M_{LRFD} = M_G due to a LRFD load combination
V_{LRFD} = V_G due to a LRFD load combination
P_G = axial force in foundation at the ground surface (at grade)
M_G = groundline bending moment (bending moment in foundation at the ground surface)
V_G = groundline shear force (shear force in foundation at the ground surface)
g = gravitation acceleration constant, 1.0 lbf/lbm (9.81×10^{-3} kN/kg)
M_F = foundation mass

Both equations 5-11 and 5-14 contain P_{LRFD}. Equation 5-11 is only applicable when P_{LRFD} is pushing down on the foundation at grade. Equation 5-14 is only applicable when P_{LRFD} is pulling up on the foundation at grade.

5.7.2 Safety and Resistance Factors

Tables 5-3, 5-4, 5-5 and 5-6 contain resistance factors for LRFD design and corresponding safety factors for ASD design. Table 5-3 values apply to bearing strength assessment, Table 5-4 values apply to lateral strength assessment involving the Universal Method of analysis, Table 5-5 values apply to lateral strength assessment involving the Simplified Method of analysis, and Table 5-6 values apply to uplift strength assessment.

For buildings and other structures that represent a low risk to human life in the event of a failure (e.g., ASCE/SEI 7 Category I structures), resistance factors in Tables 5-3 through 5-6 may be increased 25 percent (multiplied by 1.25), and corresponding safety factors may be reduced 20 percent (multiplied by 0.80). In all cases, the adjusted resistance factor is limited to a maximum value of 0.93 and the adjusted safety factor is limited to a minimum value of 1.50.

Bearing, lateral and uplift capacities in cohesionless soils increase exponentially with friction angle, and thus small variances in estimated friction angle have an amplified effect on these capacities as friction angle increases. For this reason, the equations in Tables 5-3 through 5-6 yield a smaller LRFD resistance factor (and conversely, a larger ASD safety factor) for greater soil friction angles ϕ.

Table 5-3. LRFD Resistance Factors and ASD Safety Factors for Bearing Strength Assessment

Soil	Associated Section [(a)]	Method used to determine ultimate bearing capacity q_B	LRFD resistance factor for bearing strength assessment, R_B	ASD safety factor for bearing strength assessment, f_B
Cohesionless (SP, SW, GP. GW, GW-GC, GC, SC, SM, SP-SM, SP-SC, SW-SM, SW-SC)	5.8.3	General bearing capacity equation with ϕ determined from laboratory direct shear or axial compression tests (see Section 5.4.6)	$0.80 - 0.01 \cdot \phi$	$1.4/(0.80 - 0.01 \cdot \phi)$
		General bearing capacity equation with ϕ determined from SPT data in accordance with Section 5.4.6	$0.62 - 0.01 \cdot \phi$	$1.4/(0.62 - 0.01 \cdot \phi)$
		General bearing capacity equation with ϕ determined from CPT data in accordance with Section 5.4.6	$0.71 - 0.01 \cdot \phi$	$1.4/(0.71 - 0.01 \cdot \phi)$
		General bearing capacity equation with presumptive soil properties from Table 5-2	$0.58 - 0.01 \cdot \phi$	$1.4/(0.58 - 0.01 \cdot \phi)$
		General bearing capacity equation with presumptive soil properties from Table 5-2 with soil type verified by construction testing	$0.77 - 0.01 \cdot \phi$	$1.4/(0.77 - 0.01 \cdot \phi)$
	5.8.4	Standard penetration test (SPT)	0.41	3.4
	5.8.5	Cone penetration test (CPT)	0.50	2.8
	5.8.6	Pressuremeter test (PMT)	0.50	2.8
Cohesive (CL,CH, ML, MH)	5.8.3	General bearing capacity equation with undrained shear strength determined from laboratory compression tests (see Section 5.4.5)	0.60	2.3
		General bearing capacity equation with undrained shear strength determined from PBPMT data in accordance with Section 5.4.5	0.60	2.3
		General bearing capacity equation with undrained shear strength determined from CPT data in accordance with Section 5.4.5	0.60	2.3
		General bearing capacity equation with undrained shear strength determined from in-situ vane tests in accordance with Section 5.4.5	0.60	2.3
		General bearing capacity equation with presumptive soil properties from Table 5-2	0.47	3.0
		General bearing capacity equation with presumptive soil properties from Table 5-2 with soil type verified by construction testing	0.60	2.3
	5.8.5	Cone penetration test (CPT)	0.60	2.3
	5.8.6	Pressuremeter test (PMT)	0.60	2.3

[(a)] Section containing the q_B equation to which the resistance/safety factor applies.

Table 5-4. LRFD Resistance Factors and ASD Safety Factors for Lateral Strength Assessment using the Universal Method of Analysis

Soil	Method used to determine ultimate lateral soil resistance, $p_{U,z}$	LRFD resistance factor for lateral strength assessment, R_L	ASD safety factor for lateral strength assessment, f_L
Cohesionless (SP, SW, GP. GW, GW-GC, GC, SC, SM, SP-SM, SP-SC, SW-SM, SW-SC)	Equation from Section 5.9.2.1 with soil friction angle ϕ determined from laboratory direct shear or axial compression tests (see Section 5.4.6)	$0.86 - 0.01 \cdot \phi$	$1.4/(0.86 - 0.01 \cdot \phi)$
	Equation from Section 5.9.2.1 with soil friction angle ϕ determined from SPT data in accordance with Section 5.4.6	$0.66 - 0.01 \cdot \phi$	$1.4/(0.66 - 0.01 \cdot \phi)$
	Equation from Section 5.9.2.1 with soil friction angle ϕ determined from CPT data in accordance with Section 5.4.6	$0.76 - 0.01 \cdot \phi$	$1.4/(0.76 - 0.01 \cdot \phi)$
	Equation from Section 5.9.2.1with presumptive soil friction angle ϕ from Table 5-2	$0.61 - 0.01 \cdot \phi$	$1.4/(0.61 - 0.01 \cdot \phi)$
	Equation from Section 5.9.2.11 with presumptive soil friction angle ϕ from Table 5-2, with soil type verified by construction testing	$0.82 - 0.01 \cdot \phi$	$1.4/(0.82 - 0.01 \cdot \phi)$
	Pressuremeter test (PMT) in accordance with Section 5.9.2.3	0.56	2.5
Cohesive (CL,CH, ML, MH)	Equation from Section 5.9.2.1 with undrained shear strength S_U determined from laboratory compression tests (see Section 5.4.5)	0.68	2.1
	Equation from Section 5.9.2.1 with undrained shear strength S_U determined from PBPMT data in accordance with Section 5.4.5	0.68	2.1
	Equation from Section 5.9.2.1 with undrained shear strength S_U determined from CPT data in accordance with Section 5.4.5	0.68	2.1
	Equation from Section 5.9.2.1 with undrained shear strength S_U determined from in-situ vane tests in accordance with Section 5.4.5	0.68	2.1
	Equation from Section 5.9.2.1 with presumptive undrained shear strength S_U from Table 5-2	0.44	3.2
	Equation from Section 5.9.2.1 with presumptive undrained shear strength S_U from Table 5-2 with soil type verified by construction testing	0.68	2.1
	Pressuremeter test (PMT) in accordance with Section 5.9.2.3	0.68	2.1

Table 5-5. LRFD Resistance Factors and ASD Safety Factors for Lateral Strength Assessment using the Simplified Method of Analysis

Soil	Required property	Method used to determine required soil property	LRFD resistance factor for lateral strength assessment, R_L	ASD safety factor for lateral strength assessment, f_L
Cohesionless (SP, SW, GP. GW, GW-GC, GC, SC, SM, SP-SM, SP-SC, SW-SM, SW-SC)	Soil friction angle ϕ for equations in Section 5.9.3.1, 5.9.3.3, 5.9.3.4 and 5.9.3.6	Laboratory direct shear or axial compression tests (see Section 5.4.6)	$0.83 - 0.01 \cdot \phi$	$1.4/(0.83 - 0.01 \cdot \phi)$
		SPT data in accordance with Section 5.4.6	$0.64 - 0.01 \cdot \phi$	$1.4/(0.64 - 0.01 \cdot \phi)$
		CPT data in accordance with Section 5.4.6	$0.73 - 0.01 \cdot \phi$	$1.4/(0.73 - 0.01 \cdot \phi)$
	Soil friction angle ϕ for equations in Sections 5.9.3.1 and 5.9.3.4	Presumptive value from Table 5-2	$0.60 - 0.01 \cdot \phi$	$1.4/(0.60 - 0.01 \cdot \phi)$
		Presumptive value from Table 5-2 with soil type verified by construction testing	$0.80 - 0.01 \cdot \phi$	$1.4/(0.80 - 0.01 \cdot \phi)$
Cohesive (CL,CH, ML, MH)	Undrained shear strength S_U for equations in Sections 5.9.3.2, 5.9.3.3, 5.9.3.5 and 5.9.3.6	Laboratory compression tests (see Section 5.4.5)	0.64	2.2
		PBPMT data in accordance with Section 5.4.5	0.64	2.2
		CPT data in accordance with Section 5.4.5	0.64	2.2
		In-situ vane tests in accordance with Section 5.4.5	0.64	2.2
	Undrained shear strength S_U for equations in Sections 5.9.3.2 and 5.9.3.5	Presumptive value from Table 5-2	0.42	3.3
		Presumptive value from Table 5-2 with soil type verified by construction testing	0.64	2.2

Table 5-6. LRFD Resistance Factors and ASD Safety Factors for Uplift Strength Assessment

Soil	Required property	Method used to determine required soil property	LRFD resistance factor for uplift strength assessment, R_U [(a)]	ASD safety factor for uplift strength assessment, f_U [(a)]
Cohesionless (SP, SW, GP. GW, GW-GC, GC, SC, SM, SP-SM, SP-SC, SW-SM, SW-SC)	Soil friction angle ϕ for use in the equations of Section 5.10.3	Laboratory direct shear or axial compression tests (see Section 5.4.6)	$1.20 - 0.015 \cdot \phi$	$1.4/(1.20 - 0.015 \cdot \phi)$
		SPT data in accordance with Section 5.4.6	$0.93 - 0.015 \cdot \phi$	$1.4/(0.93 - 0.015 \cdot \phi)$
		CPT data in accordance with Section 5.4.6	$1.07 - 0.015 \cdot \phi$	$1.4/(1.07 - 0.015 \cdot \phi)$
		Presumptive value from Table 5-2	$0.87 - 0.015 \cdot \phi$	$1.4/(0.87 - 0.015 \cdot \phi)$
		Presumptive value from Table 5-2 with soil type verified by construction testing	$1.16 - 0.015 \cdot \phi$	$1.4/(1.16 - 0.015 \cdot \phi)$
Cohesive (CL,CH, ML, MH)	Undrained shear strength S_U for use in the equation of Section 5.10.4	Laboratory compression tests (see Section 5.4.5)	0.70	2.0
		PBPMT data in accordance with Section 5.4.5	0.70	2.0
		CPT data in accordance with Section 5.4.5	0.70	2.0
		In-situ vane tests in accordance with Section 5.4.5	0.70	2.0
		Presumptive value from Table 5-2	0.56	2.5
		Presumptive value from Table 5-2 with soil type verified by construction testing	0.70	2.0

[(a)] In all cases, R_U is limited to a maximum value of 0.93 and F_U is limited to a minimum value of 1.50.

5.8 Bearing Strength Assessment

5.8.1 Ultimate Bearing Strength, Q_U

The ultimate groundline bearing strength of a post or pier foundation is given as:

$$Q_U = (q_B - \gamma\, d_F)\, A \tag{5-15}$$

where:

Q_U = ultimate bearing strength of a post or pier foundation at the ground surface ($z = 0$), lbf
q_B = ultimate soil bearing capacity, lbf/ft^2
γ = moist unit weight of soil, lbf/ft^3
d_F = foundation or footing depth, ft (see figure 8-4)
A = footing bearing area, ft^2

Different methods for calculating ultimate soil bearing capacity q_B are given in Sections 5.8.3, 5.8.4, 5.8.5, and 5.8.6. Equations in these sections assume that the ground surrounding the location of the installed footing is level. If it is not, adjustments to calculated values must be made in accordance with common engineering practice.

Adjustments to q_B are required for cohesiveless soils when the water table is within a distance 1.5 B of the bottom of the footing where B is the breadth of the footing. These adjustment factors are given in Section 5.8.2.

Quantity $\gamma\, d_F$ is the pressure applied to the foundation base (i.e., at a depth, d_F) by the soil overburden. Assuming that the difference is negligible between the density of the soil and the average density of the foundation elements, the ***net*** ultimate bearing capacity can be approximated as the difference between q_B and γd_F as is done in equation 5-15.

5.8.2 Correction Factors for q_B of Cohesionless Soils

Correction factors C_{W1} and C_{W2} are included in equations for cohesionless soils to account for water table depth, d_W relative to foundation depth, d_F. In equation form:

$C_{W1} = 0.5$ when $d_W \le d_F$
$= 1.0$ when $d_W \ge 1.5\, B + d_F$
$= 0.5 + (d_W - d_F)/(3B)$ when $d_F < d_W < 1.5\, B + d_F$

$C_{W2} = 0.5 + 0.5\, d_W / d_F$ when $d_W < d_F$
$= 1.0$ when $d_W \ge d_F$

5.8.3 q_B from the General Bearing Capacity Equation

For saturated clay soils:

$$q_B = S_u\, N_C\, d_C\, s_C + \gamma d_F$$

$$q_B = S_u\,(6.19 + 1.23\, d_F/B) + \gamma d_F \quad \text{for } d_F/B < 2.5$$

$$q_B = S_u\, 9.25 + \gamma d_F \quad \text{for } d_F/B \ge 2.5$$

where:

$N_C = 5.14$ for $\phi = 0$

$s_C = 1.2$ for square and round footings

$d_C = 1 + 0.2\, d_F/B$ for $d_F/B < 2.5$

$d_C = 1.5$ for $d_F/B \ge 2.5$

For cohesionless soils:

$$q_B = \gamma\,(0.5\,B\,C_{w1}\,N_\gamma\,s_\gamma + d_F\,C_{w2}\,N_q\,d_q\,s_q)$$

where:

$N_\gamma = 2\,(N_q + 1)\tan\phi$

$N_q = \exp(\pi\tan\phi)\tan^2(45 + \phi/2)$

$s_\gamma = 0.6$ for square and round footings

$s_q = 1 + \tan\phi$ for square and round footings

$d_q = 1 + 2\tan\phi\,(1-\sin\phi)^2\tan^{-1}(d_F/B)$

Obtain values for C_{W1} and C_{W2} from Section 5.8.2. Values of N_γ, N_q, s_q and d_q for different values of ϕ are given in Table 5-7.

5.8.4 q_B from Standard Penetration Test (SPT) Results

Bearing resistance for foundations in sands can be taken as:

$$q_B = N_1\,C_{SPT}\,B(C_{w1} + C_{w2}\,d_F/B)$$

where: C_{SPT} is a constant equal to 200 lbf/ft^3(31.4 kPa/m); C_{w1} and C_{w2} are given in Section 5.8.2; and N_1 is the SPT blow count, N_{SPT}, normalized with respect to vertical effective stress as given in Section 5.4.4. For calculations of q_B, the SPT blow count, N_{SPT}, shall be obtained within the range of depth from footing base to 1.5B below the footing.

5.8.5 q_B from Cone Penetration Test (CPT) Results

For saturated clay soils:

$$q_B = C_{CPT1} + q_{cr}/3$$

For cohesionless soils:

$$q_B = q_{cr}\,B\,(C_{w1} + C_{w2}\,d_F/B)/C_{CPT2}$$

where: q_{cr} is average cone resistance within a depth B below the bottom of the footing; C_{CPT1} is a constant equal to 11,400 lbf/ft^2(546 kPa); C_{CPT2} is a constant equal to 40 ft (12 m); and C_{w1} and C_{w2} are given in Section 5.8.2.

5.8.6 q_B from Pressuremeter Test (PMT) Results

For all soils:

$$q_B = q_o + C_{PB}\,(p_L - \sigma_{0h})$$

where: q_o is the initial total vertical pressure at the base of the footing; p_L is the average value of limiting pressures obtained from the PMT within a zone of $\pm$1.5 B above and below the footing depth d_F; σ_{0h} is the horizontal total stress at rest from the PMT for the depth where the PMT is performed; and C_{PB} is an empirical bearing capacity coefficient given as:

$C_{PB} = 0.80 + 0.642(d_F/B) - 0.0839(d_F/B)^2$ for sands

$C_{PB} = 0.80 + 0.384(d_F/B) - 0.0572(d_F/B)^2$ for silts

$C_{PB} = 0.80 + 0.223(d_F/B) - 0.0395(d_F/B)^2$ for clays

where: d_F is footing depth; and B is diameter of a round footing or side length of a square footing.

5.8.7 Example Problem

Problem Statement

What is the ultimate groundline bearing strength Q_U for a foundation consisting of a post supported on a footing that is 6 inches thick and 14 inches in diameter if the distance to the top of the footing is 4.5 feet. Surrounding unexcavated soil is classified as a medium to stiff ML soil. This classification is based on observation and not on actual testing. The water table is located two feet below the footing. If the maximum gradeline bearing force due to ASD loadings $P_G = P_{ASD}$ is 3500 lbf, is the foundation adequate for resisting bearing forces?

Solution

Ultimate bearing strength is given by equation 5-15 as:

$$Q_U = (q_B - \gamma\,d_F)\,A$$

In this case d_F is equal to 60 inches (i.e., 54 inches plus the footing thickness of 6 inches) and B is 14 inches. This yields a d_F/B ratio of 4.29. For cohesive soils with a d_F/B ratio of 4.29, q_B is given as:

$$q_B = S_u\,9.25 + \gamma\,d_F$$

Substitution of this equation into the previous equation yields:

$$Q_U = S_u\,9.25\,A = 9965 \text{ lbf}$$

where:

S_U = 7 lbf/in^2 from Table 5-2
A = π(14 in.)^2/4 = 153.9 in.^{2}

Since an ASD loading is involved, the governing strength relationship is given by equation 5-7:

$$Q_U \geq f_B\,P_{ASD}$$

Without soil tests, the ASD factor of safety for bearing strength assessment f_B from Table 5-3 is 3.0. In accordance with Section 5.7.2, this value can be reduced to 2.4 for structures with a low risk to human life; thus:

$$Q_U \geq f_B\,P_{ASD} = 2.4\text{ (3500 lbf)} = 8400 \text{ lbf}$$

Since Q_U is equal to 9965 lbf, this inequality is met and the design is adequate. If it were not, design options include increasing the footing size and/or conducting soil tests to determine actual soil strength. The latter would likely result in a higher S_U value and a lower factor of safety.

Table 5-7. Bearing Capacity Factors as a Function of Soil Friction Angle

Soil friction angle, ϕ	$\tan\phi$	1 - $\sin\phi$	N_γ	N_q	s_q	d_F / B								
						2	3	4	5	6	7	8	10	12
						$\tan^{-1}(d_F / B)$								
						1.11	1.25	1.33	1.37	1.41	1.43	1.45	1.47	1.49
						d_q								
0	0.000	1.000	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1	0.017	0.983	0.07	1.09	1.02	1.04	1.04	1.04	1.05	1.05	1.05	1.05	1.05	1.05
2	0.035	0.965	0.15	1.20	1.03	1.07	1.08	1.09	1.09	1.09	1.09	1.09	1.10	1.10
3	0.052	0.948	0.24	1.31	1.05	1.10	1.12	1.12	1.13	1.13	1.13	1.14	1.14	1.14
4	0.070	0.930	0.34	1.43	1.07	1.13	1.15	1.16	1.17	1.17	1.17	1.18	1.18	1.18
5	0.087	0.913	0.45	1.57	1.09	1.16	1.18	1.19	1.20	1.20	1.21	1.21	1.21	1.22
6	0.105	0.895	0.57	1.72	1.11	1.19	1.21	1.22	1.23	1.24	1.24	1.24	1.25	1.25
7	0.123	0.878	0.71	1.88	1.12	1.21	1.24	1.25	1.26	1.27	1.27	1.27	1.28	1.28
8	0.141	0.861	0.86	2.06	1.14	1.23	1.26	1.28	1.29	1.29	1.30	1.30	1.31	1.31
9	0.158	0.844	1.03	2.25	1.16	1.25	1.28	1.30	1.31	1.32	1.32	1.33	1.33	1.34
10	0.176	0.826	1.22	2.47	1.18	1.27	1.30	1.32	1.33	1.34	1.34	1.35	1.35	1.36
11	0.194	0.809	1.44	2.71	1.19	1.28	1.32	1.34	1.35	1.36	1.36	1.37	1.37	1.38
12	0.213	0.792	1.69	2.97	1.21	1.30	1.33	1.35	1.37	1.37	1.38	1.39	1.39	1.40
13	0.231	0.775	1.97	3.26	1.23	1.31	1.35	1.37	1.38	1.39	1.40	1.40	1.41	1.41
14	0.249	0.758	2.29	3.59	1.25	1.32	1.36	1.38	1.39	1.40	1.41	1.41	1.42	1.43
15	0.268	0.741	2.65	3.94	1.27	1.33	1.37	1.39	1.40	1.41	1.42	1.43	1.43	1.44
16	0.287	0.724	3.06	4.33	1.29	1.33	1.38	1.40	1.41	1.42	1.43	1.44	1.44	1.45
17	0.306	0.708	3.53	4.77	1.31	1.34	1.38	1.41	1.42	1.43	1.44	1.44	1.45	1.46
18	0.325	0.691	4.07	5.26	1.32	1.34	1.39	1.41	1.43	1.44	1.44	1.45	1.46	1.46
19	0.344	0.674	4.68	5.80	1.34	1.35	1.39	1.42	1.43	1.44	1.45	1.45	1.46	1.47
20	0.364	0.658	5.39	6.40	1.36	1.35	1.39	1.42	1.43	1.44	1.45	1.46	1.46	1.47
21	0.384	0.642	6.20	7.07	1.38	1.35	1.39	1.42	1.43	1.44	1.45	1.46	1.46	1.47
22	0.404	0.625	7.13	7.82	1.40	1.35	1.39	1.42	1.43	1.44	1.45	1.46	1.46	1.47
23	0.424	0.609	8.20	8.66	1.42	1.35	1.39	1.42	1.43	1.44	1.45	1.46	1.46	1.47
24	0.445	0.593	9.44	9.60	1.45	1.35	1.39	1.42	1.43	1.44	1.45	1.45	1.46	1.47
25	0.466	0.577	10.87	10.66	1.47	1.34	1.39	1.41	1.43	1.44	1.44	1.45	1.46	1.46
26	0.488	0.562	12.54	11.85	1.49	1.34	1.38	1.41	1.42	1.43	1.44	1.45	1.45	1.46
27	0.510	0.546	14.47	13.20	1.51	1.34	1.38	1.40	1.42	1.43	1.43	1.44	1.45	1.45
28	0.532	0.531	16.71	14.72	1.53	1.33	1.37	1.40	1.41	1.42	1.43	1.43	1.44	1.45
29	0.554	0.515	19.33	16.44	1.55	1.33	1.37	1.39	1.40	1.41	1.42	1.43	1.43	1.44
30	0.577	0.500	22.40	18.40	1.58	1.32	1.36	1.38	1.40	1.41	1.41	1.42	1.42	1.43
31	0.601	0.485	25.99	20.63	1.60	1.31	1.35	1.37	1.39	1.40	1.40	1.41	1.42	1.42
32	0.625	0.470	30.21	23.17	1.62	1.31	1.34	1.37	1.38	1.39	1.39	1.40	1.41	1.41
33	0.649	0.455	35.18	26.09	1.65	1.30	1.34	1.36	1.37	1.38	1.38	1.39	1.40	1.40
34	0.675	0.441	41.06	29.43	1.67	1.29	1.33	1.35	1.36	1.37	1.37	1.38	1.39	1.39
35	0.700	0.426	48.02	33.29	1.70	1.28	1.32	1.34	1.35	1.36	1.36	1.37	1.37	1.38
36	0.727	0.412	56.30	37.74	1.73	1.27	1.31	1.33	1.34	1.35	1.35	1.36	1.36	1.37
37	0.754	0.398	66.18	42.91	1.75	1.26	1.30	1.32	1.33	1.34	1.34	1.35	1.35	1.36
38	0.781	0.384	78.01	48.92	1.78	1.26	1.29	1.31	1.32	1.32	1.33	1.33	1.34	1.34
39	0.810	0.371	92.23	55.94	1.81	1.25	1.28	1.30	1.31	1.31	1.32	1.32	1.33	1.33
40	0.839	0.357	109.39	64.18	1.84	1.24	1.27	1.28	1.29	1.30	1.31	1.31	1.32	1.32
41	0.869	0.344	130.18	73.88	1.87	1.23	1.26	1.27	1.28	1.29	1.29	1.30	1.30	1.31
42	0.900	0.331	155.51	85.35	1.90	1.22	1.25	1.26	1.27	1.28	1.28	1.29	1.29	1.29
43	0.933	0.318	186.48	98.99	1.93	1.21	1.24	1.25	1.26	1.27	1.27	1.27	1.28	1.28
44	0.966	0.305	224.58	115.28	1.97	1.20	1.22	1.24	1.25	1.25	1.26	1.26	1.26	1.27
45	1.000	0.293	271.68	134.84	2.00	1.19	1.21	1.23	1.24	1.24	1.25	1.25	1.25	1.26
46	1.036	0.281	330.25	158.46	2.04	1.18	1.20	1.22	1.22	1.23	1.23	1.24	1.24	1.24
47	1.072	0.269	403.54	187.15	2.07	1.17	1.19	1.21	1.21	1.22	1.22	1.22	1.23	1.23
48	1.111	0.257	495.86	222.24	2.11	1.16	1.18	1.19	1.20	1.21	1.21	1.21	1.22	1.22
49	1.150	0.245	612.97	265.42	2.15	1.15	1.17	1.18	1.19	1.19	1.20	1.20	1.20	1.21
50	1.192	0.234	762.64	318.96	2.19	1.14	1.16	1.17	1.18	1.18	1.19	1.19	1.19	1.19

5.9 Lateral Strength Assessment

5.9.1 Overview

As the groundline shear force V_G and groundline bending moment M_G applied to the top of a post (or pier) foundation are increased, the pressure applied to the foundation by the soil increases. This increase in soil pressure at a particular depth will continue until the ultimate lateral soil resisting pressure p_U at that depth (as calculated in Section 5.9.2) is reached. Once this point is reached, there is no further increase in pressure applied to the foundation by the soil at that depth.

The ultimate strength of a post or pier foundation is reached when *all* soil in contact with the foundation has reached its ultimate lateral soil resisting pressure. The groundline shear force V_G and groundline bending moment M_G when this state is reached are respectively defined as the ultimate groundline shear capacity V_U and the ultimate groundline moment capacity M_U of the foundation as limited by soil strength.

For any foundation, the ultimate groundline moment capacity, M_U, is dependent on the groundline shear force induced in the foundation. Thus there is (in theory) an infinite number of V_U - M_U combinations for each non-constrained foundation design. These combinations can be represented with a V_U-M_U *envelope* on a plot of groundline shear V_G versus groundline bending moment M_G (figure 5-21).

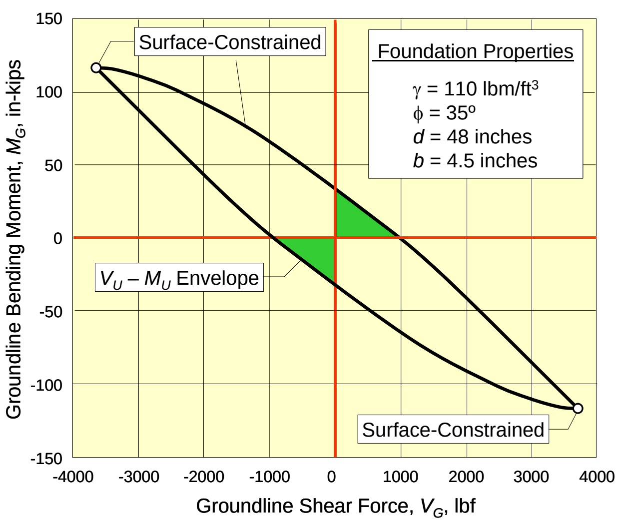


Figure 5-21. V_U - M_U envelope for a post/pier foundation.

Plotted in figure 5-21 is a V_U-M_U envelope for a foundation with a 4.5 inch width and 48 inch depth. Additionally, the foundation is surrounded by a cohesionless soil with a moist unit weight of 110 lbm/ft^3 and soil friction angle of 35 degrees. Any combination of groundline shear V_G and groundline bending moment M_G that falls within the V_U-M_U envelope will not exceed the ultimate capacity of the foundation.

Maintaining a proper sign convention is important. Groundline shear forces and groundline bending moments are given the same sign when they independently rotate the foundation in the same direction (figure 5-22).

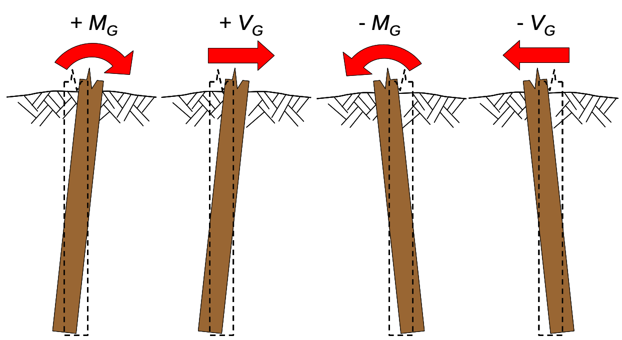


Figure 5-22. Groundline shear forces and groundline bending moments are given the same sign if they independently rotate a foundation in the same direction.

The two shaded regions in figure 5-21 identify loadings in which groundline bending moment and groundline shear have the same sign. Although these regions comprise a relatively small area of the V_U - M_U envelope, the vast majority of loadings on non-constrained foundations are located in these regions.

The two most extreme points on the $V_U - M_U$ envelope represent the $V_U - M_U$ combinations associated with the restraint of the foundation at or just above the ground surface (i.e., constrained or surface-constrained foundation). In this case, the groundline shear force is the force in the foundation at a point just below the surface restraint, and is opposite in sign to the groundline bending moment.

Section 5.9.3 contains equations for calculating M_U for foundations that have a fixed face width and are surrounded by soil that is homogeneous for the entire embedment depth. Application of these equations is relatively straight-forward, and thus is referred to as the Simplified Method of analysis.

Determination of the ultimate lateral strength capacity of foundations that do not meet requirements for application of the Simplified Method, involves modeling soil behavior with discrete springs and is covered in Section 5.9.4. Strength capacity determination utilizing soil springs is referred to as the Universal Method of analysis.

Section 5.9.5 contains an example analysis that showcases methods from Sections 5.9.3 and 5.9.4.

5.9.2 Ultimate Lateral Soil Resistance, p_U

5.9.2.1 p_U Based on Soil Properties

In accordance with ANSI/ASAE EP486.2, the ultimate lateral soil resisting pressure p_U at a given depth z is calculated as:

$$p_{U,z} = 3\sigma'_{v,z} K_P + (2 + z/b)\, c\, K_P^{\,0.5} \quad \text{for } 0 \le z < 4b \qquad (5\text{-}16)$$

$$p_{U,z} = 3\,(\sigma'_{v,z} K_P + 2\, c\, K_P^{\,0.5}) \quad \text{for } z \ge 4b \qquad (5\text{-}17)$$

where:

$p_{U,z}$ = ultimate lateral resistance p_U at depth z
K_P = coefficient of passive earth pressure, dimensionless
= $(1 + \sin\phi)/(1 - \sin\phi)$
ϕ = soil friction angle, degrees
c = soil cohesion at depth z
b = face width of foundation at the groundline
$\sigma'_{v,z}$ = effective vertical stress at depth z
= $\sigma_{v,z} - u_z = \gamma z - u_z$
$\sigma_{v,z}$ = total vertical stress at depth z
= γz
γ = moist unit weight of soil
u_z = pore water pressure at depth z
= γ_w •(distance the water table is above depth z)
γ_w = water unit weight = 62.4 lbf/ft^3 = 0.0361 lbf/in^3

Equations 5-16 and 5-17 equate ultimate lateral soil resisting pressure p_U to three times the Rankine passive pressure. Although basing resisting pressure solely on passive pressure would appear to neglect the active earth-pressure acting on the back of the foundation and side friction, the factor of three by which the passive pressure is increased is based on observed ultimate loads – ultimate loads which were most likely influenced by forces acting on all sides of the foundation system.

Passive pressure due to soil cohesion is assumed to increase from 1/3 its full value at the ground surface to its full value at a depth of 4 b. This partially accounts for the reduced soil containment at the soil surface and less than full mobilization of the soil due to the likelihood of foundation-soil detachment near the surface. The quantity $2cK_P^{\,0.5}$ in equations 5-16 and 5-17 is the Rankine passive pressure due to soil cohesion.

For cohesionless soils, equations 5-16 and 5-17 both reduce to:

$$p_{U,z} = 3\,\sigma'_{v,z} K_P \qquad (5\text{-}18)$$

For cohesive soils, equations 5-16 and 5-17 reduce to:

$$p_{U,z} = 3\, S_U\,(1 + z/(2b)) \quad \text{for } 0 \le z < 4b \qquad (5\text{-}19)$$

$$p_{U,z} = 9\, S_U \quad \text{for } z \ge 4b \qquad (5\text{-}20)$$

where: S_U is undrained soil shear strength at depth z. The value of 9 S_U is approximately equal to three times $2S_UK_P^{\,0.5}$ when ϕ is equal to 32 degrees. S_U is numerically equal to cohesion, c, for a saturated clay soil.

5.9.2.2 p_U for Cohesionless Soils from CPT Tests

At a given depth z, ultimate lateral soil bearing pressure p_u for cohesionless soils can be determined from CPT cone penetration resistance q_{cr} at depth z using the following correlation from Lee et al. (2010).

$$p_{U,z} = (1.959\, p_A^{-0.10}\, q_{cr}^{0.47}) / (\sigma'^{-0.63}_{m,z})$$

where: $p_{U,z}$ is ultimate lateral resistance at depth z; p_A is atmospheric pressure; and $\sigma'_{m,z}$ is mean effective stress at depth z and is given as:

$$\sigma'_{m,z} = (\sigma'_{v,z} + 2\,\sigma'_{0h,z})/3$$

where: $\sigma'_{v,z}$ is effective vertical stress at depth z; and $\sigma'_{0h,z}$ is at rest effective horizontal stress at depth z.

To maintain dimensional homogeneity, input p_A, q_{cr} and $\sigma'_{m,z}$ in identical units. Pressure $p_{U,z}$ will then have the same units as these three input variables.

5.9.2.3 p_U from Pressuremeter Tests

For any given depth, p_u can be determined from a pressuremeter reading in accordance with procedures outlined by Briaud (1992).

5.9.3 M_U and V_U via the Simplified Method

The equations in this Section are only applicable to foundations that have a fixed face width and are surrounded by soil that is homogeneous for the entire embedment depth.

Equations in Section 5.9.3.1, 5.9.3.2 and 5.9.3.3 are for non-constrained foundations, and were set up with M_U as the dependent variable and V_U as an independent variable. To establish a V_U - M_U envelope line for a non-constrained foundation, simply calculate M_U for more than one V_U value as shown in figure 5-23.

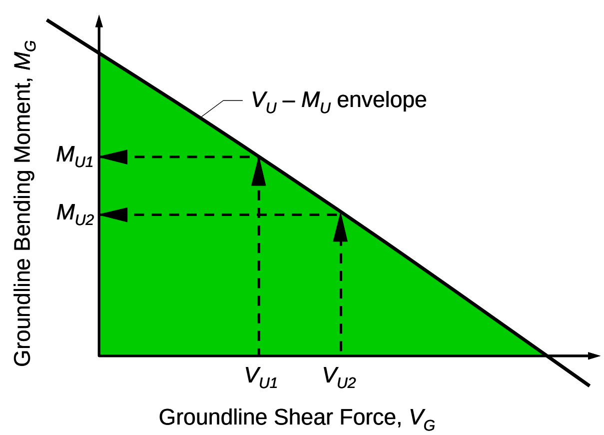


Figure 5-23. Equations for the Simplified Method for non-constrained foundations require selection of a V_U value to determine M_U.

Variables used in this section have been previously defined in Sections 5.7.1, 5.7.2 and 5.9.2.1, with the exception of the following:

d_{RU} = Depth from ground surface to the ultimate pivot point (i.e., the point below grade at which the foundation does not move horizontally under ultimate load).

S_{LU} = Increase in the ultimate lateral force per unit depth applied to a foundation by a cohesionless soil

5.9.3.1 Non-Constrained Foundation in Cohesionless Soils

The ultimate moment M_U that can be applied at the groundline to a post/pier foundation that is not constrained at the groundline and is embedded in cohesionless soil (figure 5-24) is:

$$M_U = S_{LU}\,(d^3 - 2\,d_{RU}^{\,3})\,/\,3 \qquad (5\text{-}21)$$

where:

$d_{RU} = (V_U/S_{LU} + d^2/2)^{0.5} \le d$
$S_{LU} = 3\,b\,K_P\,\gamma$
$K_P = (1 + \sin\phi)/(1 - \sin\phi)$
$V_U = V_{LRFD}/R_L$ for LRFD
$V_U = f_L\,V_{ASD}$ for ASD

If shear V_{LRFD} (or V_{ASD}) and moment M_{LRFD} (or M_{ASD}) rotate the top of the foundation in opposite directions, input a negative value for V_{LRFD} (or V_{ASD}).

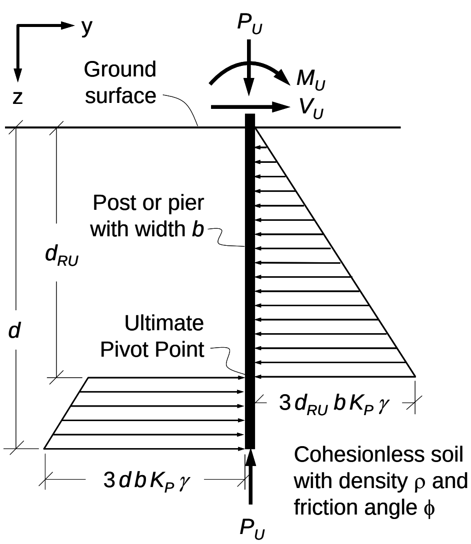
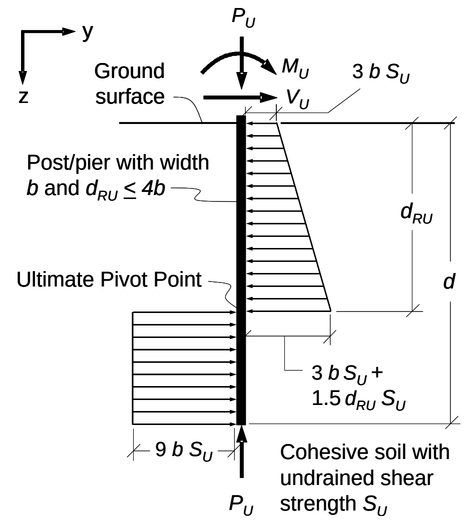


Figure 5-24. Forces acting on a non-constrained foundation of fixed width *b* in cohesionless soil at failure.

5.9.3.2 Non-Constrained Foundation in Cohesive Soils

(a)



(b)

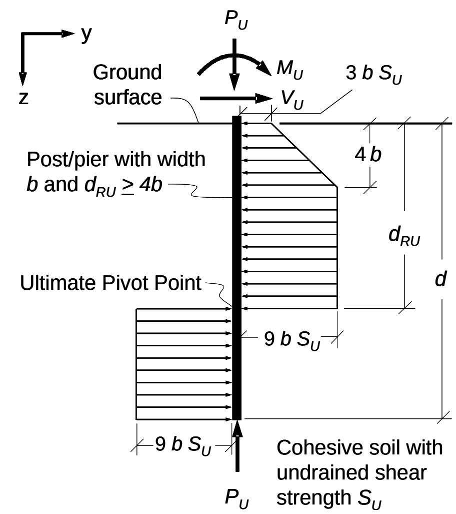


Figure 5-25. Forces acting on a non-constrained foundation of fixed width *b* in cohesive soil at failure (a) when d_{RU} is less than 4 *b*, and (b) when d_{RU} is greater than 4 *b*.

The ultimate moment M_U that can be applied at the groundline to a post/pier foundation that is not constrained at the groundline and is embedded in cohesive soil is:

$$M_U = b\,S_U\,(4.5\,d^2 - 6\,d_{RU}^{\,2} - d_{RU}^{\,3}/(2b)) \qquad (5\text{-}22)$$

where:

$$d_{RU} = [64\,b^2 + 4V_U/(3S_U) + 12\,b\,d]^{1/2} - 8\,b \le d$$

and

$$d_{RU} < 4b$$

The preceding equations apply when d_{RU} is less than 4b and the force distribution shown in figure 5-25(a) applies. If d_{RU} from the preceding equation is greater than 4b (in which case the force distribution shown in figure 5-25(b) applies) then:

$$M_U = 9\, b\, S_U\, (d^{\,2}/2 - d_{RU}^{\;2} + 16\, b^{\,2}/9) \qquad (5\text{-}23)$$

where

$$d_{RU} = V_U / (18\, b\, S_U) + d/2 + 2\, b/3 \le d$$

and

$$d_{RU} \ge 4b$$

In both cases:

$$V_U = V_{LRFD} / R_L \quad \text{for LRFD}$$

$$V_U = f_L\, V_{ASD} \quad \text{for ASD}$$

If shear V_{LRFD} (or V_{ASD}) and moment M_{LRFD} (or M_{ASD}) rotate the top of the foundation in opposite directions, input a negative value for V_{LRFD} (or V_{ASD}).

5.9.3.3 Non-Constrained Foundation in Any Soil

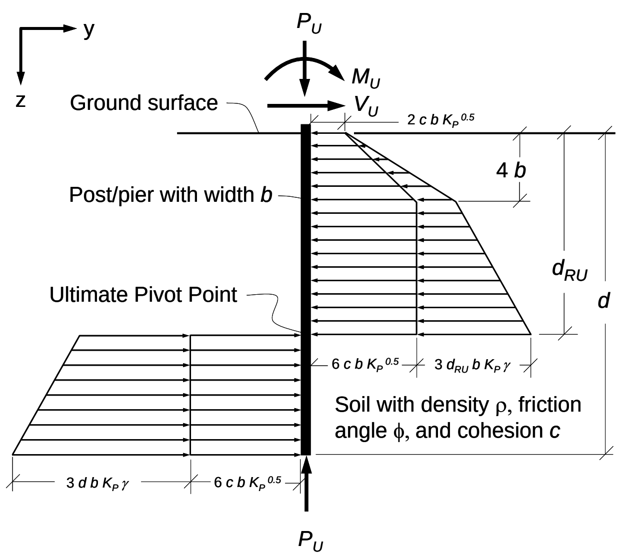


Figure 5-26. Forces acting on a non-constrained foundation of fixed width *b* in a homogenous soil at failure.

The ultimate moment M_U that can be applied at the groundline to a post/pier foundation that is constrained at the groundline and for which d_{RU} is greater than 4b (figure 5-26) is:

$$M_U = S_{LU}\,(d^{\,3} - 2\, d_{RU}^{\;3})/3 + 6\, b\, c\, K_P^{\,0.5}\,(d^{\,2}/2 - d_{RU}^{\;2} + b^{\,2}/4) \qquad (5\text{-}24)$$

where:

$$d_{RU} = [A^2 + V_U / S_{LU} + dA + d^{\,2}/2 + A\,b/2]^{\,0.5} - A \le d$$
$$d_{RU} > 4b$$
$$A = 2c / (K_P^{\,0.5}\,\gamma)$$
$$S_{LU} = 3\, b\, K_P\, \gamma$$
$$K_P = (1 + \sin\phi)/(1 - \sin\phi)$$
$$V_U = V_{LRFD} / R_L \quad \text{for LRFD}$$
$$V_U = f_L\, V_{ASD} \quad \text{for ASD}$$

If shear V_{LRFD} (or V_{ASD}) and moment M_{LRFD} (or M_{ASD}) rotate the top of the foundation in opposite directions, input a negative value for V_{LRFD} (or V_{ASD}).

5.9.3.4 Constrained Foundation in Cohesionless Soils

The ultimate moment M_U that can be applied at the groundline to a post/pier foundation that is constrained at the groundline and is embedded in cohesionless soil (figure 5-27) is:

$$M_U = d^{\,3}\, b\, K_P\, \gamma \qquad (5\text{-}25)$$

$$K_P = (1 + \sin\phi)/(1 - \sin\phi)$$

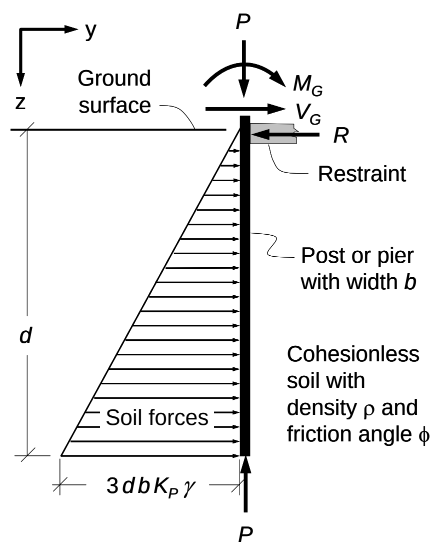


Figure 5-27. Forces acting on a constrained foundation of fixed width *b* in cohesionless soil at failure.

5.9.3.5 Constrained Foundation in Cohesive Soils

The ultimate moment M_U that can be applied at the groundline to a post/pier foundation that is constrained at the groundline and is embedded in cohesive soil (figure 5-28) is:

$$M_U = b\, S_U\,(4.5\, d^2 - 16\, b^2) \qquad \text{for d} \geq 4\text{b} \qquad (5\text{-}26)$$

and

$$M_U = b\, d^2\, S_U\,(3/2 + d/(2b)) \qquad \text{for d} \leq 4\text{b} \qquad (5\text{-}27)$$

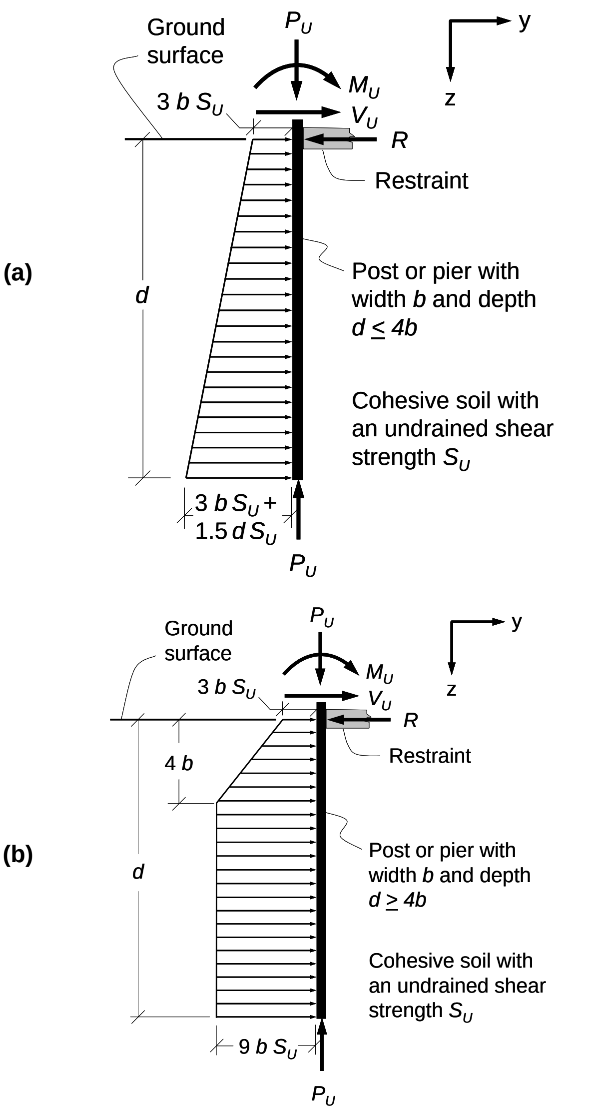


Figure 5-28. Forces acting on a constrained foundation of fixed width *b* in cohesive soil at failure (a) when *d* is less than 4 *b*, and (b) when *d* is greater than 4 *b*.

5.9.3.6 Constrained Foundation in Any Soil

The ultimate moment M_U that can be applied at the groundline to a post/pier foundation that is constrained at the groundline and is embedded in any soil (figure 5-29) is:

$$M_U = d^3 b\, K_P\, \gamma + bcK_P^{0.5}\,(3d^2 - 32b^2/3) \qquad \text{for d} \geq 4\text{b} \qquad (5\text{-}28)$$

and

$$M_U = d^3\, b\, K_P\, \gamma + b\, d^2 c\, K_P^{0.5}\,(1 + d/(3\text{b})) \qquad \text{for d} \leq 4\text{b} \qquad (5\text{-}29)$$

where

$$K_P = (1 + \sin\phi)/(1 - \sin\phi)$$

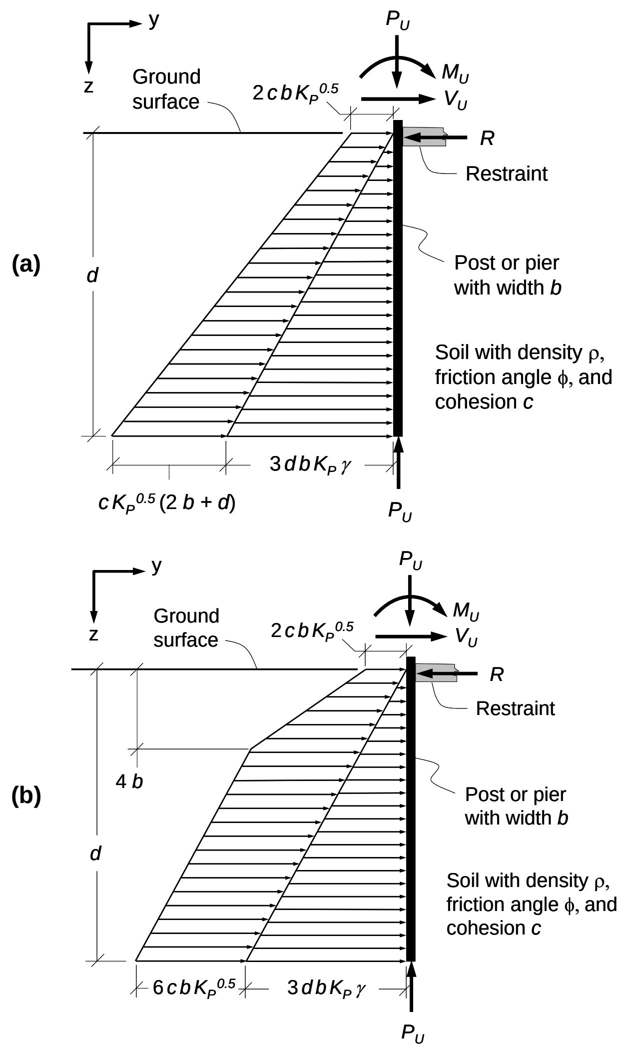


Figure 5-29. Forces acting on a constrained foundation of fixed width *b* in a homogenous soil at failure (a) when *d* is less than 4 *b*, and (b) when *d* is greater than 4 *b*.

5.9.4 M_U and V_U via the Universal Method

The ANSI/ASAE EP486.2 Universal Method of analysis utilizes soil springs with an initial stiffness K_H and ultimate strength F_{ult} as shown in figure 5-13. Of these two spring properties, only F_{ult} is required to establish M_U and V_U.

5.9.4.1 Soil Spring Strength, F_{ult}

The ultimate load that an individual spring can sustain is given as:

$$F_{ult} = p_{U,z}\, t\, b \qquad (5\text{-}30)$$

where:

- F_{ult} = Ultimate load that a spring at depth z can sustain, lbf
- $p_{U,z}$ = Ultimate lateral soil resistance for unexcavated soil at depth z from Section 5.9.2, lbf/in.^{2}
- t = Thickness of the soil layer represented by the soil spring, inches
- b = Face width of post/pier, footing, or collar that is being modeled with the spring, in.
- z = Distance of spring below grade, in.

Although backfill properties will influence spring stiffness K_H, they are not factored into calculations of ultimate spring strength F_{ult}. This is because the soil failure planes associated with the ultimate lateral capacity of the foundation are almost entirely located in the unexcavated soil surrounding the backfill.

5.9.4.2 Conditions at Ultimate Lateral Capacity

Each soil spring is assumed to exhibit linear-elastic behavior until F_{ult} is reached, at which point the spring is assumed to undergo a plastic state of strain with the force in the soil spring remaining at F_{ult}. The lateral strength capacity of a foundation (as limited by soil strength) is reached when *all* springs acting on the foundation have reached their maximum ultimate strength capacity F_{ult}. . In other words, a foundation has reached its lateral strength capacity when there is not a single remaining soil spring that can take additional load.

The groundline shear V_G and groundline bending moment M_G that will result in a plastic state of strain in all soil springs are defined respectively as the ultimate groundline shear capacity V_U and ultimate groundline moment capacity M_U for the foundation.

The key to determining M_U and V_U for any foundation is identifying on which side of the foundation each soil spring is pushing. At loads less than a foundation's ultimate capacity (i.e., prior to the yielding of all soil springs), the direction that many soil springs act is a function of the bending stiffness of the foundation relative to the stiffness of the surrounding soil, and some of these directions can switch as the applied loads increase as shown in figure 5-30.

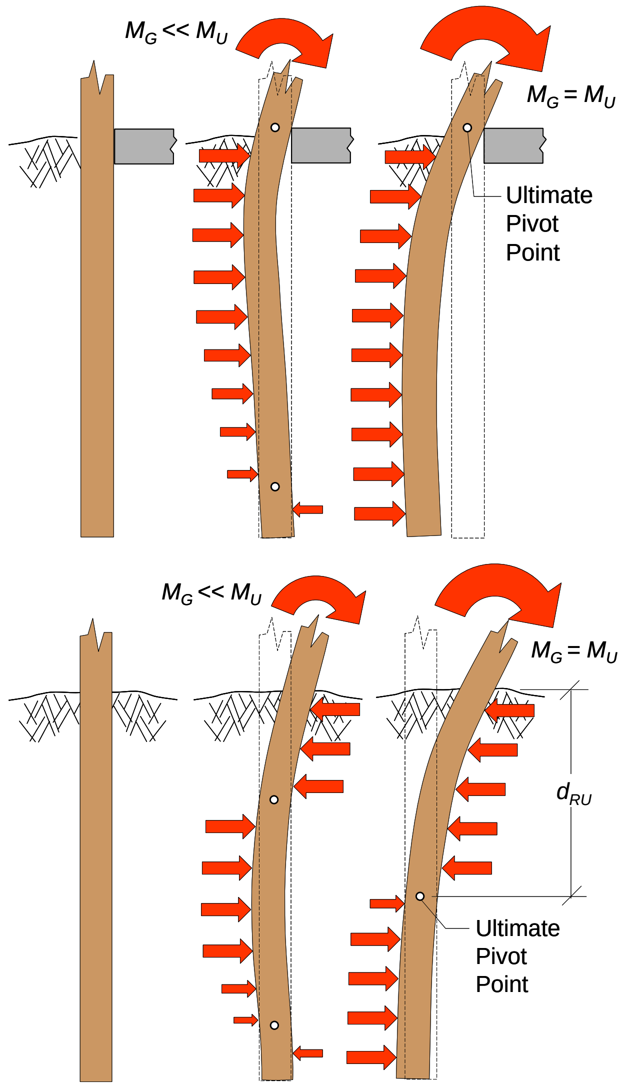


Figure 5-30. Surface-constrained (top) and non-constrained (bottom) post foundations subjected to a groundline bending moment. At ultimate lateral capacity ($M_G = M_U$) there is no more than one pivot point (i.e., the ultimate pivot point) located below grade.

Once all soil springs have yielded (i.e., once the foundation has reached its ultimate capacity), the foundation will pivot about a single point, herein referred to as the *ultimate pivot point* (figure 5-30). The ultimate pivot point is also identified as the *point of zero lateral displacement under ultimate load*. All soil springs located above an ultimate pivot point act in the same direction. Likewise, all springs located below an ultimate pivot point act in the same direction.

It's important to note that the *ultimate* pivot point's location is not a function of the foundation's bending stiffness, nor is it a function of soil spring stiffness K_H

(as previously stated, prior to reaching ultimate capacity, locations of zero lateral displacement in non-constrained posts are a function of the foundation's bending stiffness and soil spring stiffness). This means that M_U and V_U for any foundation can be determined without knowledge of foundation bending properties or soil spring stiffness.

5.9.4.3 M_U and V_U for a Specified Ultimate Pivot Point Location

Each modeling spring represents a soil layer with thickness t. When the ultimate pivot point is located at the interface between two of these soil layers (see figure 5-31) or the ultimate pivot point is located above the soil surface or below the foundation, M_U and V_U can be calculated as:

$$M_U = \sum_{i=1}^{N} z_i \, F_{ult,i} \qquad (5\text{-}31)$$

$$V_U = -\sum_{i=1}^{n} F_{ult,i} \qquad (5\text{-}32)$$

where:

M_U = Ultimate groundline moment capacity of the foundation (as limited by soil strength). Positive when acting clockwise.

V_U = Ultimate shear capacity (as limited by soil strength) of the foundation at a point just below the foundation restraint. Positive when acting to the right.

n = Number of springs used to model the soil surrounding the foundation.

$F_{ult,i}$ = Ultimate strength of spring i. Positive when pushing to the right.

z_i = Absolute distance between groundline and spring i.

Equation 5-31 is obtained by summing forces in the horizontal direction on a free body diagram of the below-grade portion of a foundation. Equation 5-32 is obtained by summing moments about the groundline on the same free body diagram.

Figure 5-32 contains a $V_U – M_U$ envelope obtained by applying equations 5-31 and 5-32 to all 13 ultimate pivot point locations associated with a 12 soil spring model (11 locations between springs plus locations at the groundline and foundation base). To obtain the full $V_U – M_U$ envelope shown in figure 5-32, signs are switched on all thirteen "V_U , M_U" values. This is equivalent to switching the directions of all soil springs at each ultimate pivot point location.

For design purposes, the entire $V_U – M_U$ envelope need not be constructed. Calculating M_U and V_U for three or so ultimate pivot points in the ½ d_f to $^7/_8\, d_f$ range, enables construction of a M_U-V_U envelope line that would cover most loadings associated with a non-constrained foundation. The deeper value of $^7/_8\, d_f$ is associated with foundations that have an attached footing, bottom collar, and/or some other mechanism that results in the base of the foundation having a much greater effective width than the rest of the foundation.

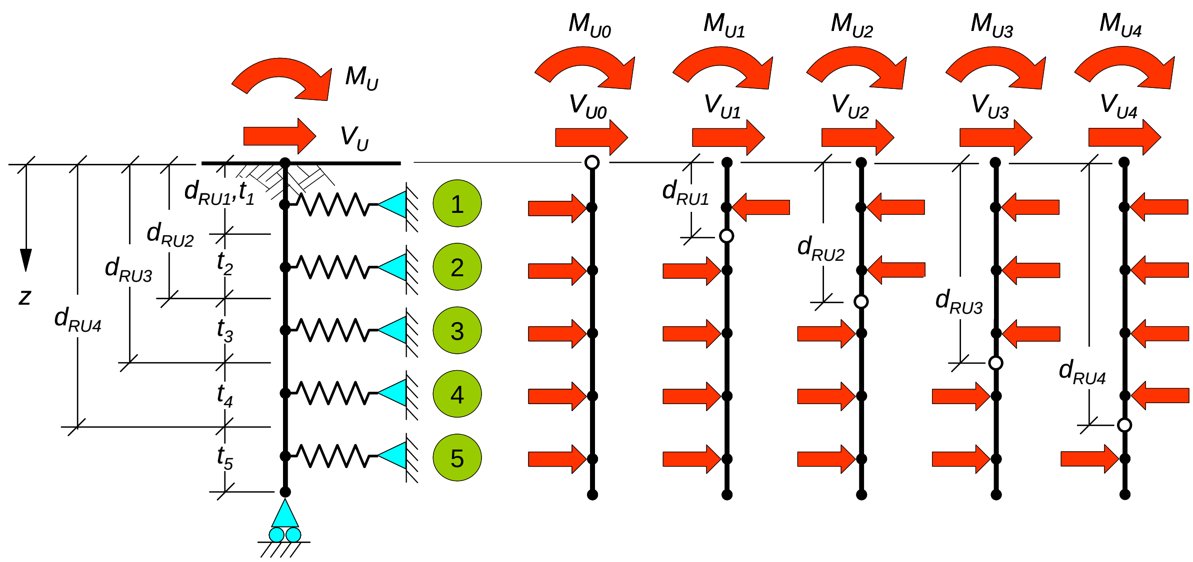


Figure 5-31. When ultimate pivot points are located at the interface between soil layers modeled with different soil springs, Equations 5-30 and 5-31 can be used to calculate M_U and V_U, respectively.

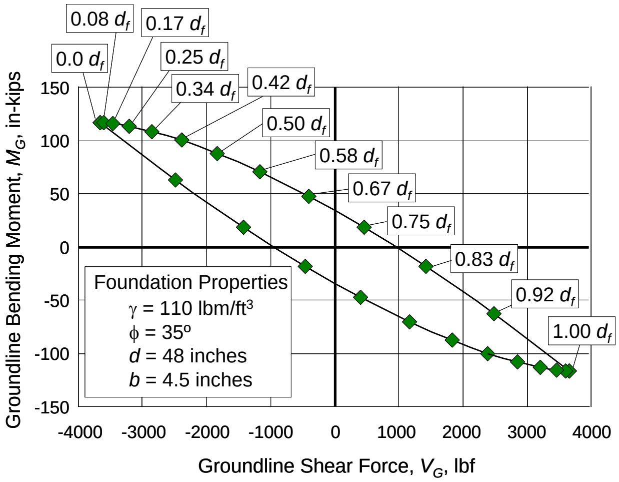


Figure 5-32. V_U - M_U envelope obtained by applying equations 5-31 and 5-32 for 13 different ultimate pivot point locations (12-spring model). "Boxed" values identify ultimate pivot point locations (d_{RU} values).

5.9.4.4 M_U and V_U for a Specified M_G/V_G Value

A foundation is adequate under lateral loads if $M_U \geq M_{ASD}\, f_L$ and $V_U \geq V_{ASD}\, f_L$ for Allowable Stress Design, and $M_U \geq M_{LRFD}/R_L$ and $V_U \geq V_{LRFD}/R_L$ for Load and Resistance Factor Design.

Checking if these inequalities have been met is straight forward once a V_U-M_U envelope plot exists. For example, figure 5-33 shows the results of two different structural analyses involving the same foundation; one ASD and the other LRFD. A quick scan of this plot reveals that the foundation is adequate for the LRFD loading but not for the ASD loading.

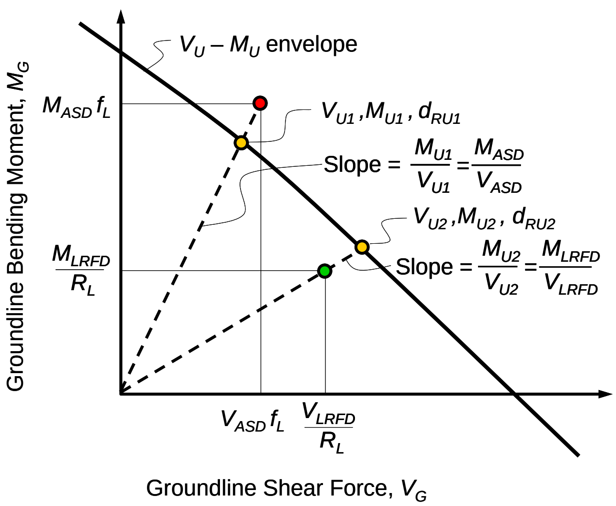


Figure 5-33. Using a V_U - M_U envelope to check the adequacy of a foundation under two different loadings.

In figure 5-33, "V_{U1}, M_{U1}" is the point on the V_U - M_U envelope that is numerically closest to coordinate point "$V_{ASD}\, f_L$, $M_{ASD}\, f_L$". "V_{U1}, M_{U1}" lies on a line drawn through the origin and "$V_{ASD}\, f_L$, $M_{ASD}\, f_L$". Stated differently, the closest "V_U, M_U" point to "$V_{ASD}\, f_L$, $M_{ASD} f_L$" is the one whose M_U/V_U value equals M_{ASD}/V_{ASD}. More generically, the closest "V_U, M_U" point to a particular "V_G, M_G" point is one whose M_U/V_U value equals M_G/V_G. Rearranging yields the equality:

$$M_U = V_U\,(M_G/V_G) \qquad (5\text{-}33)$$

As will be demonstrated in the following paragraph, equation 5-33 makes it possible to determine if a foundation is adequate without having to first establish a V_U - M_U envelope plot like that shown in figure 5-32.

Figure 5-34a shows a nonconstrained post with M_G and V_G applied at the groundline. Figure 5-34b shows V_G located a distance M_G/V_G above the groundline. From a statics perspective, the diagrams in Figures 5-34a and 5-34b are equivalent. As force V_G in Figure 5-34b is increased, soil springs will begin to yield. As a spring yields, it is replaced with an equivalent force as shown in Figure 5-34c. Force V_G can be increased until all but one soil spring has reached its ultimate capacity F_{ult}. The value of V_G when this point is reached is defined as the ultimate groundline shear capacity of the foundation V_U (Figure 5-34d). Once V_U is established, M_U is calculated by direct application of equation 5-33.

The spring that has not reached its ultimate capacity F_{ult} (when V_U is reached) is the spring that represents the soil layer in which the ultimate pivot point is located. For this reason, the spring is referred to as the *pivot* spring. It follows that the pivot spring is simultaneously representing soil forces applied to both sides of the foundation as shown in Figure 5-34e. Because these forces (1) counteract each other, and (2) individually cannot exceed F_{ult}, the pivot spring itself will always have a load less than F_{ult}. The only time this would not be the case is when the ultimate pivot point is located exactly at the interface between soil layers represented by different springs (figure 5-31).

Given that the forces in all soil springs that have yielded are known, the only unknowns in Figure 5-34d are V_U and the force in the pivot spring. Thus, V_U can be calculated by summing moments about the point at which the pivot spring attaches to the foundation, and the force in the pivot spring can be determined by summing moments about the point at which V_U is applied (i.e., at a distance M_G/V_G from the groundline).

It is evident that the procedure for determining V_U (and thus M_U) is very straightforward if one knows which one of the soil springs is the pivot spring. In practice, this can be determined by trial and error. If the wrong spring is selected, the absolute value of the force calculated for that spring will exceed the spring's F_{ult} value.

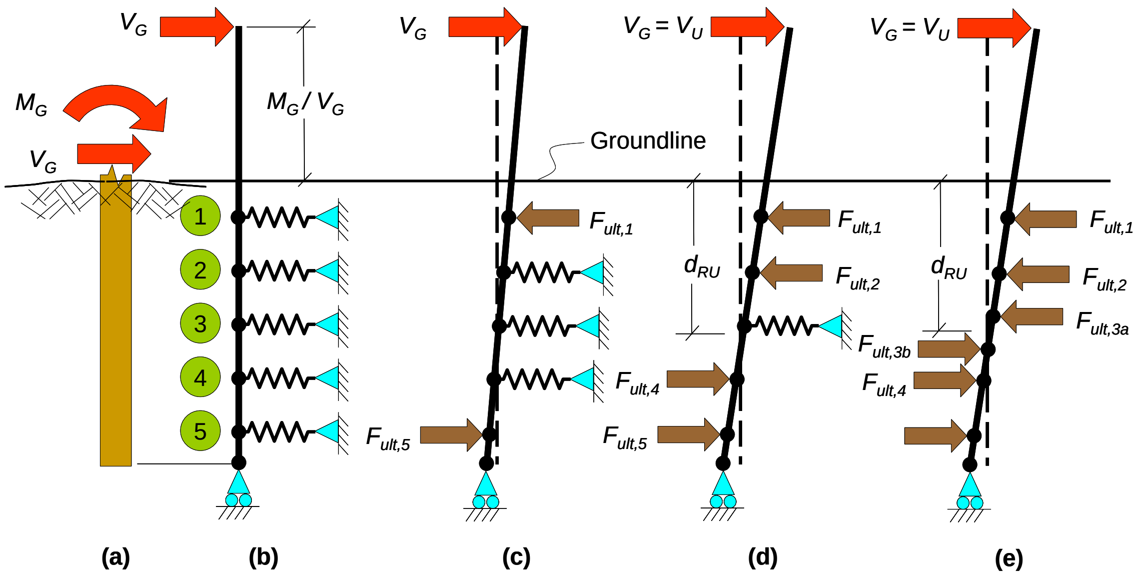


Figure 5-34. (a) Groundline shear V_G and groundline bending moment M_G, (b) equivalent load applied to spring model of foundation, (c) soil springs yield under increased load, (d) ultimate capacity of foundation is reached when all but one soil spring reaches its ultimate strength, (e) spring that doesn't reach its ultimate load is replaced by two opposing forces that represent force applied by soil yielding on both sides of the foundation.

5.9.4.4.1 Example Determination of M_U and V_U for a Specified M_G/V_G Value

A nonconstrained post foundation with a uniform width of 4.5 inches, depth of 48 inches, located in cohesionless soil with a soil friction angle of 35 degrees and moist unit weight of 110 lbf/ft^3, was subjected to a groundline shear V_G of 500 lbf and a groundline bending moment M_G of 10,000 in-lbf (M_G / V_G = 20 inches).

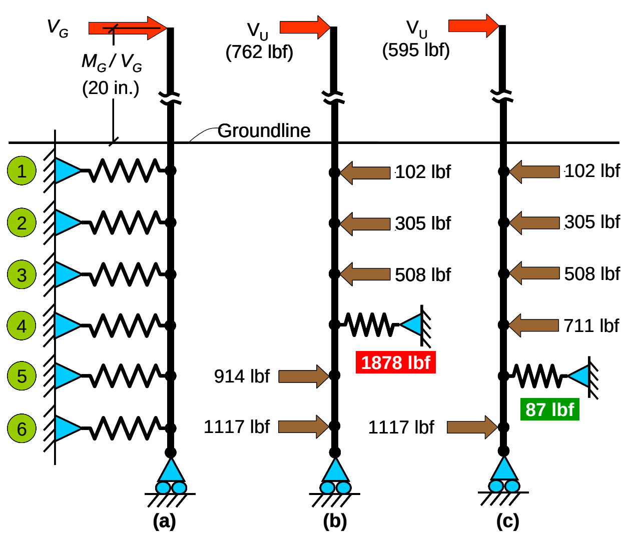


Figure 5-35. (a) Spring model of nonconstrained post foundation, (b) free body diagram with an overloaded spring 4 as pivot spring, and (c) free body diagram with spring 5 as the pivot spring.

In this case, six springs were used to model the soil as shown in figure 5-35 (for accuracy purposes, it is recommended that at least 5 springs be used). Table 5-8 lists the location and ultimate strengths of each spring.

Table 5-8. Spring Forces in a Nonconstrained Post Foundation [a]

Load element	Location, z, inches	F_{ult}, lbf	Pivot Spring 4	Pivot Spring 5	Pivot Spring 6
			Force in load element, lbf		
Spring 1	4	102	-102	-102	-102
Spring 2	12	305	-305	-305	-305
Spring 3	20	508	-508	-508	-508
Spring 4	28	711	-1878 [b]	-711	-711
Spring 5	36	914	914	-87	-914
Spring 6	44	1117	1117	1117	1840 [b]
V_U	-20	NA	762	595	698

[a] b = 4.5 inches, d_f = 48 inches, γ = 110 lbf/ft^3, ϕ = 35
[b] Force exceeds maximum allowable value.

For the trial-and-error analysis, spring 4 was first selected as the pivot spring. This resulted in a V_U value of 762 lbf and a pivot spring force of -1878 lbf as shown in figure 5-35. Because the absolute value of the -1878 lbf force exceeds the F_{ult} for spring 4 of 711 lbf (Table 5-8), spring 4 is not the pivot spring. Consequently, spring 5 was selected as the pivot spring. This resulted in a V_U value of 595 lbf and a pivot spring force of -87 lbf as shown in Figure 5-35. Because the absolute value of -87 lbf does not exceed the F_{ult} for spring 5 of 914 lbf, spring

5 is indeed the pivot spring. For demonstration purposes, spring 6 was also selected as the pivot spring. The results of this analysis are given in the last column of Table 5-8.

Multiplication of the V_U value of 595 lbf by the M_G/V_G ratio of 20 inches yields an M_U of 11,900 in-lbf. Since these values each exceed the V_G and M_G values by 19%, and thus t

5.9.4.4.2 Determining M_U and V_U for a Negative M_G/V_G Value

One variation on the preceding "pivot spring" procedure occurs when M_G and V_G independently rotate the foundation in opposite directions, as shown in Figure 5-36. This produces a negative M_G/V_G ratio. A negative value means that V_G is placed a distance M_G/V_G below the groundline as shown in Figure 5-36. The rest of the analysis is conducted in the same manner, as if V_G was located a distance M_G/V_G above the groundline.

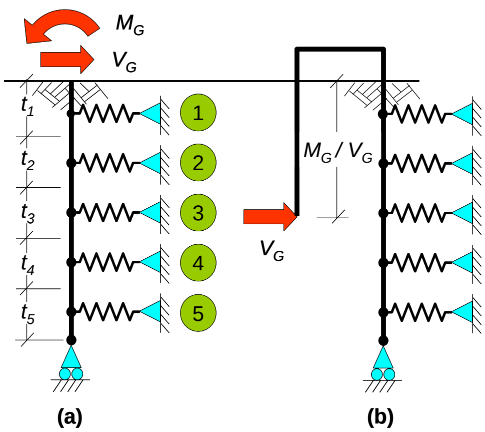


Figure 5-36. (a) Forces V_G and M_G independently rotate the top of the foundation in opposite directions, and (b) a statically equivalent spring model is used for determination of V_U and M_U.

5.9.5 Example Analyses

5.9.5.1 Non-Constrained Foundation in Cohesive Soil – Simplified Method

Problem Statement

A non-constrained foundation consists of a nominal 6-by 6-inch post that extends 48 inches below the soil surface and bears on a footing to which it is not attached. Via testing, the surrounding soil (backfill and unexcavated soil) was verified as a medium to dense silt with higher plasticity (soil type MH). If the groundline shear force V_{ASD} and groundline bending moment M_{ASD} due to the applied ASD structural loads are 800 lbf and 45,000 in-lbf, respectively, is the foundation adequate? V_{ASD} and M_{ASD} rotate the foundation in the same direction.

Solution

Since this is an ASD loading, the governing equations are equations 5-8 and 5-9 which are given as:

$$M_U \geq f_L\, M_{ASD} \qquad \text{and} \qquad V_U \geq f_L\, V_{ASD}$$

The equations in Section 5.9.3.2 (Simplified Method for Constrained Foundation in Cohesive Soils) are applicable to this problem because the foundation has a fixed width and the soil is assumed homogenous for the entire depth of the foundation. When the Simplified Method is used, only the first of the preceding governing equations ($M_U \geq f_L\, M_{ASD}$) needs to be checked. This is because the Simplified Method equation for M_U has been derived such that if M_U is greater than $f_L\, M_{ASD}$, then V_U will automatically exceed $f_L\, V_{ASD}$.

From Section 5.9.3.2 when d_{RU} is less than $4b$:

$$M_U = b\, S_U\, (4.5\, d^{\,2} - 6\, d_{RU}^{\,2} - d_{RU}^{\,3}/(2b)) \geq 0$$

where:

$$d_{RU} = [64\, b^2 + 4V_U/(3S_U) + 12\, b\, d]^{1/2} - 8\, b \leq d$$

When d_{RU} from the preceding equation is greater than $4b$:

$$M_U = 9\, b\, S_U\, (d^{\,2}/2 - d_{RU}^{\,2} + 16\, b^{\,2}/9) \geq 0$$

where:

$$d_{RU} = V_U/(18\, b\, S_U) + d/2 + 2\, b/3 \leq d$$

In both cases $V_U = f_L\, V_{ASD}$ for ASD

From Table 5-2 for a medium to dense MH soil, the wet unit weight γ is 105 lbf/ft^3 (0.06076 lbf/in^3) and the undrained soil shear strength S_U is 7 lbf/in^2. From Table 5-5, f_L = 2.2 which yields a minimum required ultimate groundline soil shear strength V_U of 2.2 x 800 lbf = 1760 lbf. Additionally, b is equal to 5.5 inches and d equals 48 inches. Substituting these variables into the first of the above equations for d_{RU} yields:

$$d_{RU} = [64\, b^2 + 4V_U/(3S_U) + 12\, b\, d]^{1/2} - 8\, b \leq d$$

$$d_{RU} = [64\,(5.5\text{ in})^2 + 4\,(1760\text{ lbf})/(3(7\text{ lbf/in}^2)) + 12(5.5\text{ in})(48\text{ in.})]^{1/2} - 8(5.5\text{ in.}) = 29.75\text{ in.}$$

Since this is greater than $4b$ = 22 inches, d_{RU} must be recalculated as:

$$d_{RU} = V_U/(18\, b\, S_U) + d/2 + 2\, b/3 \leq d$$

$$d_{RU} = (1760\text{ lbf})/(18(5.5\text{ inch})(7\text{ lbf/in}^2)) + (48\text{ in.})/2 + 2(5.5\text{ in.})/3 = 30.21\text{ in.}$$

and M_U is given as:

$$M_U = 9\, b\, S_U\, (d^{\,2}/2 - d_{RU}^{\,2} + 16\, b^{\,2}/9) \geq 0$$

$$M_U = 9(5.5\text{ in.})(7\text{ lbf/in}^2)\,[\,(48\text{ in.})^2/2 - (30.21\text{ in.})^2 + 16\,(5.5)^2/9] = 101{,}650\text{ in-lbf}$$

$$M_U \geq f_L\, M_{ASD} = 2.2\,(45{,}000\text{ in-lbf}) = 99{,}000\text{ in-lbf}$$

Since M_U exceeds 99,000 in-lbf, the foundation is adequate.

5.9.5.2 Constrained Foundation in Cohesionless Soil – Simplified Method

Problem Statement

A surface-constrained foundation consists of a nominal 6- by 6-inch post that extends 48 inches below the soil surface and bears on a footing to which it is not attached. Via testing, the surrounding soil (backfill and unexcavated soil) was identified as a dense, poorly-graded sand (soil type SP). If the groundline bending moment M_{ASD} due to the applied ASD structural loads is 50,000 in-lbf, is the foundation adequate?

Solution

Since this is constrained foundation with ASD loading, the only governing equation is:

$$M_U \geq f_L\, M_{ASD}$$

The following equations from Section 5.9.3.4 (Simplified Method for Constrained Foundation in Cohesionless Soils) are applicable to this problem because the foundation has a fixed width and the soil is assumed homogenous for the entire depth of the foundation.

$$M_U = d^3\, b\, K_P\, \gamma$$

where:

$$K_P = (1 + \sin\phi)/(1 - \sin\phi)$$

From Table 5-2 for a dense, poorly-graded sand, the wet unit weight γ is 120 lbf/ft^3 (0.06944 lbf/in^3) and the drained soil friction angle ϕ' is 35°. From Table 5-5, f_L = 1.4/(0.80 - 0.01·ϕ) = 3.11. With b equal to 5.5 inches and d equal 48 inches:

$$K_P = (1 + \sin\phi)/(1 - \sin\phi) = 3.69$$

$$\begin{aligned} M_U &= d^3\, b\, K_P\, \gamma \\ &= (48\text{ in.})^3(5.5\text{ in.})(3.69)(0.06944\text{ lbf/in}^3) \\ &= 155{,}860\text{ in-lbf} \end{aligned}$$

$$M_U \geq f_L\, M_{ASD} = 3.11(50{,}000\text{ in-lbf}) = 155{,}500\text{ in-lbf}$$

Since M_U exceeds 155,500 in-lbf (just barely), the foundation is adequate.

Notes:

1. The 3.11 safety factor is a relatively high value, and some engineers feel comfortable using a reduced value in this application. ANSI/ASAE EP486.2 allows a 20% reduction in f_L for buildings that represent a low risk to human life in the event of a failure such as an ASCE/SEI 7 Category I building.
2. Using a bottom collar is an effective way to increase M_U of a constrained foundation, and is common where there is a desire to reduce embedment depth.

5.9.5.3 Constrained Foundation in Cohesionless Soil – Universal Method

Problem Statement

To reduce the embedment depth associated with the previous problem from 48 inches to 36 inches, a reduction in the factor of safety of 20% (to 2.5) for a ASCE 7 Category I building is being applied and a cast-in-place concrete collar that fills the 18-inch diameter of the post hole surrounding the foundation is being added. How far above the footing must this concrete collar extend to provide the necessary ultimate groundline bending capacity $M_U \geq f_L\, M_{ASD}$ = 2.5 (50,000 in-lbf) = 125,000 in-lbf?

Solution

Because the collar results in a foundation with a varying thickness, the Universal Method with its soil springs must be used. With an embedment depth of 36 inches, 6 equally-spaced springs are selected to model the soil, each with a ultimate strength given by equation 5-22 as:

$$F_{ult} = p_{U,z}\, t\, b$$

where:

$$\begin{aligned} p_{U,z} &= 3\, K_P\, \sigma'_{v,z} \\ &= 3(3.69)\,(0.06944\text{ lbf/in}^3)\, z = (0.769\text{ lbf/in}^3)\, z \\ K_P &= (1 + \sin\phi)/(1 - \sin\phi) = 3.69 \text{ for } \phi' = 35^\circ \\ \sigma'_{v,z} &= \gamma z \text{ for a homogenous soil located above the water table} \\ &= (0.06944\text{ lbf/in}^3)\, z \end{aligned}$$

t = Thickness of the soil layer represented by the soil spring

b = Face width of post/pier, footing, or collar that is being modeled with the spring

z = Distance of spring below grade

For the initial check, the collar will be assumed to extend 6 inches above the footing, thus providing the follow spring properties and moment resisting values.

No.	z	t	B	$p_{U,z}$	F_{ult}	$F_{ult}{\cdot}z$
	in.	in	in.	lbf/in^2	lbf	in-lbf
1	3	6	5.5	2.3	76	228
2	9	6	5.5	6.9	228	2056
3	15	6	5.5	11.5	381	5710
4	21	6	5.5	16.1	533	11191
5	27	6	5.5	20.8	685	18500
6	33	6	18	25.4	2741	90444
					M_U =	128128

The far right column of the above table contains the resisting moment about the groundline provided by each spring. In accordance with equation 5-31, the summation of these values provides the total ultimate groundline bending capacity M_U of the foundation. Since this exceeds the required value by 3000 in-lbf, the analysis was rerun with a 5.5 inch thick collar. This was quickly accomplished with changes in the depth and associated thickness of the lower two springs.

No.	z in.	t in	b in.	$p_{U,z}$ lbf/in^2	F_{ult} lbf	$F_{ult} \cdot z$ in-lbf
1	3	6	5.5	2.3	76	228
2	9	6	5.5	6.9	228	2056
3	15	6	5.5	11.5	381	5710
4	21	6	5.5	16.1	533	11191
5	27.25	6.5	5.5	21.5	769	20945
6	33.25	5.5	18	26.1	2584	85915
					M_U =	126045

The resulting M_U value still exceeds the required minimum of 125,000 in-lbf. However, reducing the collar thickness another half inch does not work as M_U for the foundation with a 5 inch thick collar is 120,600 in-lbfs.

5.9.5.4 Non-Constrained Foundation in Cohesive Soil – Universal Method

Problem Statement

A non-constrained foundation consisting of a 4.5- by 9.25-inch post is subjected to a groundline shear force V_{LRFD} of 1200 lbf, and a groundline bending moment M_{LRFD} of 80,000 in-lbf. How deep must the post extend into the ground if it is attached to an 8 inches and 16 inch diameter footing? Via testing, the surrounding soil (backfill and unexcavated soil) was verified as a medium to dense silt with higher plasticity (soil type MH). V_{LRFD} and M_{LRFD} rotate the foundation in the same direction.

Solution

Since this is an LRFD loading, the governing equations for this problem are equations 5-12 and 5-13:

$$M_U R_L \geq M_{LRFD} \qquad \text{and} \qquad V_U R_L \geq V_{LRFD}$$

From Table 5-4, the LRFD resistance factor for lateral strength assessment using the Universal Method of analysis R_L is given as 0.68. Thus:

$$M_U \geq M_{LRFD} / R_L = 80{,}000 \text{ in-lbf}/0.68 = 117{,}600 \text{ in-lbf}$$

and

$$V_U \geq V_{LRFD} / R_L = 1200 \text{ lbf}/0.68 = 1768 \text{ lbf}$$

Because the collar results in a foundation with a varying thickness, the Universal Method with its soil springs must be used.

Equations in Section 5.9.3.2 were used to calculate a post embedment depth assuming the foundation was not attached to the footing. For an undrained soil shear strength S_U of 7 lbf/in^2 (from Table 5-2 for a medium to dense cohesive soil), the calculated embedment depth was 55 inches. Based on this value, an overall depth (including the attached footing) of 48 inches was selected. The soil spring model used is shown in figure 5-37.

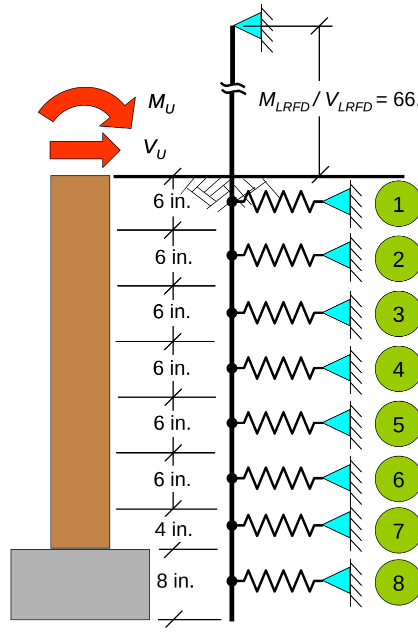


Figure 5-37. Spring placement for non-constrained post with attached footing.

Spring strength was calculated in accordance with equation 5-22 as:

$$F_{ult} = p_{U,z}\, t\, b$$

where:

$p_{U,z}$ = $3\, S_U (1 + z/(2b))$ for $0 \leq z < 4b$
$p_{U,z}$ = $9\, S_U$ for $z \geq 4b$
S_U = 7 lbf/in^2 for a medium to dense cohesive soil (from Table 5-2)
t = Thickness of the soil layer represented by the soil spring
b = 4.5 inches for post
 = 16 inches for footing
z = Distance of spring below grade

Spring Properties

Spring No.	z in.	t in.	b in.	$p_{U,z}$ lbf/in^2	F_{ult} lbf
1	3	6	4.5	28	756
2	9	6	4.5	42	1134
3	15	6	4.5	56	1512
4	21	6	4.5	63	1701
5	27	6	4.5	63	1701
6	33	6	4.5	63	1701
7	38	4	4.5	63	1134
8	44	8	16	63	8064

Following calculation of ultimate spring strengths, the free body diagram shown in figure 5-38(a) was established. In accordance with Section 5.9.4.3, V_U was located a distance M_{LRFD}/V_{LRFD} = 66.67 inches above the groundline. Spring 7 was arbitrarily selected as the pivot spring, and all other springs were replaced with a force equal to their ultimate strength.

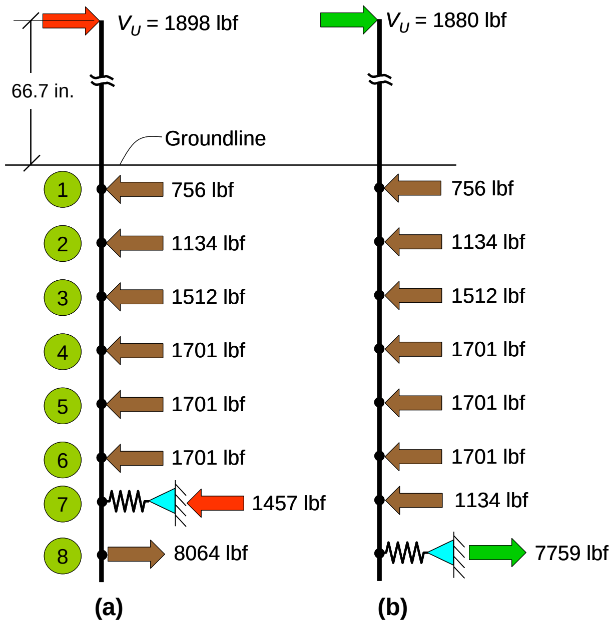


Figure 5-38. Spring placement for non-constrained post with attached footing.

Summing moments about the pivot spring in figure 5-38(a) results in a V_U value of 1898 lbf. With V_U known, a summation of horizontal forces yields a pivot spring (spring 7) force of 1457 lbf. Since this exceeds the maximum force of 1134 lbf allowed in spring 7, the ultimate pivot point is not located in the soil layer represented by spring 7. Based on the direction at which the spring force of 1457 lbf acts, it is apparent that additional force acting to the left is needed. This is only possible if the ultimate pivot point is lower than the soil layer represented with spring 7. Thus, the ultimate pivot point is located in the layer modeled with spring 8.

A subsequent analysis with spring 8 as the pivot spring was conducted (figure 5-38(b)), resulting in a V_U value of 1880 lbf, and a spring 8 force of 7759 lbf. Since the 7759 lbf does not exceed the maximum force of 8064 lbf allowed in spring 8, the ultimate pivot point is indeed located in the soil layer represented by spring 8.

The V_U value of 1880 lbf exceeds the required value of 1768 lbf so the selected overall foundation depth of 48 inches is adequate. A subsequent analysis with an overall foundation depth of 47 inches (post embedment depth of 39 inches) produced a V_U value of 1800 lbf, thereby validating 47 inches as an adequate depth.

5.9.6 Increasing Lateral Strength

Generally, the most cost effective way to increase the lateral strength of a post or pier foundation is to increase its effective depth. Two circumstances where increasing depth may not be cost effective are (1) where hole drilling is difficult because of large rock and/or rock strata, and (2) where an increase in depth will require a post that is measurably more expensive because of the increased overall length requirement.

An alternative to increasing foundation depth is to increase foundation width. This can be accomplished with concrete or CLSM backfill, a concrete collar, an uplift anchorage system, or by laminating dimension lumber to the sides of the embedded portion of the post. With respect to the latter, it is important to note that a single 8-foot piece of dimension lumber, when cut in half and appropriately fastened to both sides of a post, effectively increases the foundation width three full inches.

Attaching a post/pier to the footing upon which it bears will effectively increase the depth of a foundation and the foundation width in the footing region. It is important that such an attachment be properly engineered. Friction between a post/pier and footing can not be relied upon for lateral load transfer.

5.10 Uplift Strength Assessment

5.10.1 General

Foundation uplift strength is provided by the combination of foundation mass M_F and resistance to uplift provided by soil mass U. The governing strength equations introduced in Section 5.7 (equations 5-10 and 5-14) include both of these variables and appear as:

$$U \geq f_U (P_{ASD} - g\, M_F) \qquad \text{for ASD}$$

and

$$U\, R_U \geq P_{LRFD} - g\, M_F \qquad \text{for LRFD}$$

Foundation mass M_F includes all foundation element below grade that are mechanically attached to the post. Thus it may include concrete and CLSM backfill, but does not include soil used as backfill. Foundation mass can also include concrete slabs/paving located at grade when they are mechanically fastened to the foundation.

The resistance to uplift provided by soil mass, U, depends on (1) the size (breadth) of the anchorage system, (2) the depth of the anchorage system, (3) the attachment of the anchorage system, and (4) soil properties.

Anchorage systems are overviewed in Section 5.10.2. Sections 5.10.3 and Sections 5.10.4 contain equations for calculating ultimate uplift resistance due to soil mass, U, for foundations in cohesionless and cohesive soils, respectively.

5.10. 2 Anchorage Systems

An anchorage or uplift resisting system is any system that effectively increases the breath of the lower portion of a foundation. Anchorage systems include attached footings, attached collars, special wood or plastic blocks, steel angles, and any other component(s) that is/are properly attached near the base of the foundation.

The anchorage system must be designed with capacity to adequately handle and transfer load between the soil mass and the post (or pier) foundation. Use the applicable structural design specification(s) to make these determinations. For example, use the ANSI/AWC *National Design Specification (NDS) for Wood Construction* to determine the adequacy of mechanical fasteners used to connect wood uplift blocking to a wood post.

To move the soil mass located above the anchorage systems requires that a failure plane form in the soil. This failure plane extends upward and outward from the edges of the anchorage system. It may or may not reach the ground surface depending on soil properties and the depth d_U and width B_U of the anchorage system.

Without an anchorage system the only resistance to uplift is that provided by friction between the soil and vertical surfaces of the foundation. Friction forces between soil and a shallow post/pier foundation are minimal and unreliable for resisting uplift forces.

5.10.3 Uplift Resistance *U* In Cohesionless Soils

Soil uplift resistance values for foundations in cohesionless soils are based on work by Meyerhof and Adams (1968). The first step in these calculations is determining the *vertical extent of the uplift soil failure surface* for deep foundations, h which is a function of the angle of internal soil friction ϕ, and the anchorage system width B_U. These variables are graphically defined in figure 5-39. The vertical extent of the uplift soil failure surface, h, is given as follows:

For $\phi \leq 20$: $h = 2.5\, B_U$

For $\phi > 20$: $h = B_U\,(5.78 - 0.350\,\phi + 0.00947\,\phi^2)$

where: ϕ is in degrees.

If $h \geq d_U$ the foundation is classified as a *shallow foundation under uplift.* If $h < d_U$ the foundation is a *deep foundation under uplift.* A shallow foundation under uplift is a foundation associated with a failure plane that reaches the ground surface as shown in figure 5-39. Conversely, a deep foundation under uplift is a foundation associated with a failure plane that does not reach the ground surface as shown in figure 5-39.

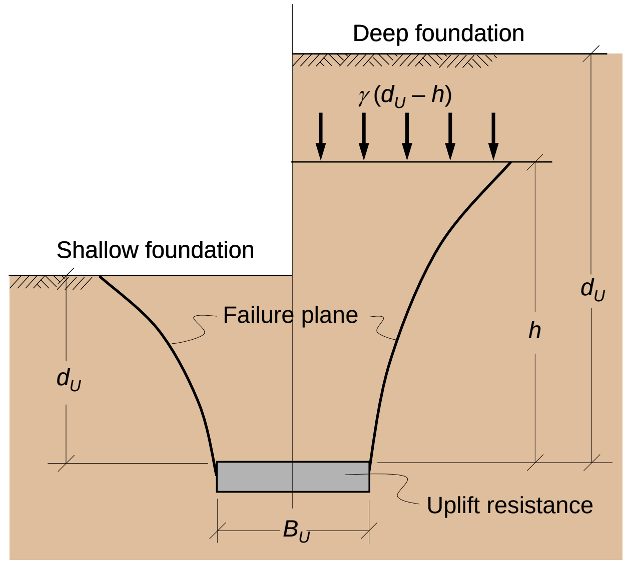


Figure 5-39. Modes of uplift failure for uplift resistance systems at different depths.

For a shallow foundation under uplift ($h \geq d_U$) with a circular anchorage system:

$$U = \gamma\, d_U\,(\pi\, d_U\, s_F\, B_U\, K_U \tan\phi/2 + B_U^2\, \pi/4 - A_p) \qquad (5\text{-}34)$$

For a shallow foundation under uplift ($h \geq d_U$) with a rectangular anchorage system:

$$U = \gamma\, d_U\,[d_U(2 s_F\, B_U + L_U - B_U) K_U \tan\phi + B_U L_U - A_p] \qquad (5\text{-}35)$$

where:

K_U = nominal uplift coefficient of earth pressure on a vertical plane through the edges of the anchorage systems
= 0.95

s_F = Factor that accounts for shape of the failure plane
= $1 + 1.105(10^{-5})\,\phi^{2.815} d_U / B_U$ for ϕ in degrees

A_P = cross-sectional area of post/pier

d_U = distance between soil surface and top of the foundation uplift resisting system

B_U = diameter of a circular uplift resisting system or the smaller of the two dimensions characterizing a rectangular uplift resisting system

L_U = length of a rectangular uplift resisting system with a width B_U

γ = moist unit weight of soil

ϕ = soil friction angle, degrees

For a deep foundation under uplift ($h < d_U$) with a circular anchorage systems:

$$U = \gamma [\pi\, h(d_U - h/2)\, s_F\, B_U\, K_U \tan\phi + d_U\, B_U^2\, \pi/4 - d_U\, A_p] \qquad (5\text{-}36)$$

For a deep foundation under uplift ($h < d_U$) with a rectangular anchorage systems:

$$U = \gamma [h(2d_U - h)(2s_F\, B_U + L_U - B_U)K_U \tan\phi + d_U\, B_U\, L_U - d_U\, A_p] \qquad (5\text{-}37)$$

where:

h = Vertical extent of the uplift soil failure surface
s_F = $1 + 1.105(10^{-5})\, \phi^{2.815} h / B_U$

5.10.4 Uplift Resistance *U* in Cohesive Soils

For foundations with circular anchorage systems surrounded by cohesive soils, uplift resistance is given as:

$$U = \gamma\, d_U (B_U^2\, \pi/4 - A_p) + F_C\, S_u\, B_U^2\, \pi/4$$

For foundations with rectangular anchorage systems surrounded by cohesive soils, uplift resistance is given as:

$$U = \gamma\, d_U (B_U L_U - A_p) + F_C\, S_u\, B_U\, L_U$$

where:

F_C = Breakout factor
= $1.2\, d_U/B_U \le 9$

5.10.5 Backfill Compaction

ANSI/ASAE EP486.2 requires that backfill be compacted to at least 85% of the density of the surrounding soil. Where this compaction requirement is not met, soil uplift resistance *U* shall not exceed the product of the gravitational constant *g* and the mass of backfill material located directly above the anchorage system.

5.10.6 Example Analyses

Problem Statement

Two nominal 2- by 6- inch wood uplift blocks each with a length of 12 inches are bolted to the base of a two-ply nail-laminated pier that itself is fabricated from nominal 2- by 6-inch members (figure 5-40). Attached to the bottom of the uplift blocks and pier is a nominal 2- by 8-inch bearing plate with a length of 12 inches. The foundation system is in a cohesionless soil with a drained soil friction angle of 30 degrees and moist unit weight of 105 lbf/ft^3. The distance between the groundline and the top of the uplift blocks is 41 inches. What is the resistance to uplift provided by soil mass, *U*, for this wood pier anchorage system?

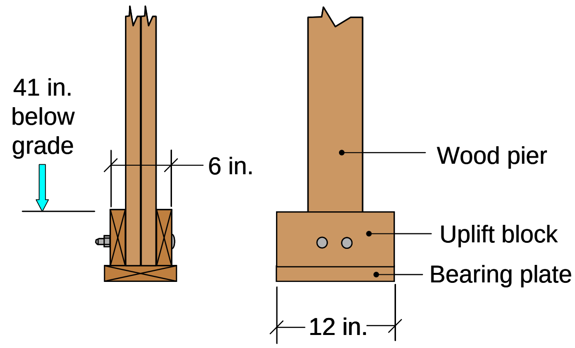


Figure 5-40. Wood pier anchorage system.

Solution

Since the soil is cohesionless, the first step in the analysis is to determine the vertical extent of the uplift soil failure surface, *h*, which is given as:

$$h = B_U (5.78 - 0.350\, \phi + 0.00947\, \phi^2)$$

For a ϕ of 30 and B_U of 6.0 inches (i.e., the smaller of the two dimensions characterizing the uplift resisting system), *h* is 22.8 inches. This is less than the 41 inch distance between the soil surface and the top of the uplift resisting system; therefore the foundation is classified as a *deep* foundation under uplift, and the maximum uplift force that can be applied to the foundation is given by equation 5-37:

$$U = \gamma [h(2d_U - h)(2s_F\, B_U + L_U - B_U)K_U \tan\phi + d_U\, B_U\, L_U - d_U\, A_p]$$

where:

γ = 105 lbf/ft^3 = 0.0608 lbf/in^3
h = 22.8 inches
d_U = distance between soil surface and top of uplift resisting system
= 41 inches
s_F = shape factor for uplift resistance in deep foundations in cohesionless soils
= $1 + 1.105(10^{-5})\, \phi^{2.815}\, h / B_U$
= 1.60
B_U = narrow dimension of rectangular uplift system
= 6.0 inches
L_U = long dimension of rectangular uplift system
= 12.0 inches
K_U = nominal uplift coefficient of earth pressure on a vertical plane
= 0.95
A_P = cross sectional area of pier foundation
= 16.5 in^2

Substitution of these preceding values into equation 5-37 produces an uplift soil resisting force *U* of 1273 lbf

Section 5.11 Frost Heave Considerations

5.11.1 General

Freezing temperatures in the soil result in the formation of ice lenses in the spaces between soil particles. Under the right conditions, these ice lenses will continue to attract water and increase in size. This expansion of ice lenses increases soil volume. If this expansion occurs under a footing, or alongside a foundation element with a rough surface, that portion of the foundation will be forced upward. This action is called frost heave, and can induce large differential movements in a structure. Differential movement can crack building finishes, and induce significant stress in structural connections and components. When ice lenses thaw, soil moisture content increases dramatically. The soil is generally in a saturated state with reduced strength. As soil water drains from the soil, effective soil stresses increase and the foundation will generally settle.

5.11.2 Minimizing Frost Heave

Frost heave can be minimized by building on soils with a low likelihood of freezing, providing good water drainage, and using fine-grained soils with caution.

Footing Location. The best way to avoid foundation frost heave is to minimize the freezing potential of underlying soils. This is accomplished by extending footings below the local frost line or by using a foundation system designed and constructed in accordance with SEI/ASCE 32 (Bohnhoff, 2010a; Bohnhoff, 2010b).

Water Drainage. Proper surface and subsurface drainage can reduce frost heave. Drainage of surface waters from a structure is enhanced by installing rain gutters, adequately sloping the finish grade away from the structure, and raising the building elevation to a level above that of the surrounding area. Subsurface drainage is achieved with the placement of drain tile or coarse granular material below the maximum frost depth, with drainage to an outlet. Such drainage lowers the water table and interrupts the flow of water moving both vertically and horizontally through the soil.

Fine-Grained Soils. Fine-grained soils such as clays and silts are more susceptible to frost heave than sands and gravels because (1) water is drawn up further in the smaller capillaries of fine-grained soils, and (2) there is much more surface area in a unit volume of fine-grained soil, and therefore more surface area for water adsorption. One factor that limits frost heave in fine-grained soils is that water is less mobile (moves slower) as capillaries decrease in size – a factor which explains why frost heave is more of a problem in silts than it is in the more finer-grained clay soils. While it is often recommended to backfill with coarse granular backfill to reduce frost heave, this is not recommended when holes are dug in clay soils. Drilling holes in clay soils and backfilling with a coarse-grained soil turns every post-hole into a sump pit that traps and holds water. This leaves the backfill in a saturated, and thus prolonged low-strength state and very prone to significant frost heave when freezing conditions occur. Consequently, as a general rule, backfill holes in silts and clays with clay soils.

5.11.3 Concrete Floors

If the ground beneath a concrete floor can freeze, the floor should be installed such that its vertical movement is not restricted by embedded posts or by structural elements attached to embedded posts. While concrete shrinkage may break bonds between a floor and surrounding components, more proactive measures will ensure independent vertical behavior. For example, roofing felt or plastic film can be placed against surrounding surfaces prior to placing the floor.

5.11.4 Concrete Backfill

The use of cast-in-place concrete as a backfill material may actually increase the likelihood of frost heave. The rough soil-to-concrete backfill interface provides the potential for significant vertical uplift forces due to frost heave. Also, the placement of concrete in holes that decrease in diameter with depth provide additional risk for frost heave.

Section 5.12 Installation Requirements

This section covers two construction-related factors that can significantly affect structural performance: soil compaction and component placement.

5.12.1 Compaction Under Footings

Compact all disturbed soil at the base of a hole to a level consistent with the soil bearing capacity assumed in design. Soil upon which a precast concrete footing will be placed must be flat and level. A non-flat surface results in uneven soil-to-footing contact, and this increases bending moments and shear stresses within the footing. If the compacted base is not level, the top surface of any precast concrete footing will not be level, resulting in only line or point contact between the footing and post/pier it supports.

5.12.2 Backfill Compaction

Compact all backfill by tamping all soil in layers (a.k.a. lifts) that do not exceed a thickness of 8 inches (0.2 m) so as to achieve lateral stiffness and strength properties consistent with those used in design.

5.12.3 Embedment Depth

Installed depth of a post/pier foundation shall not be less than 90% of the specified depth. A post foundation can be installed deeper than specified without adversely affecting foundation behavior. However, installing a post or pier deeper than specified can leave the top too short to meet specified structural needs. In the case of spliced, laminated wood posts (i.e., posts with preservative-treated lumber spliced to non-treated lumber), deeper embedment may bring the non-treated portion of the post closer to grade, making it more difficult to meet the ANSI/ASAE EP559 requirement that preservative wood treatment extend a minimum of 16 inches above the ground surface.

5.12.4 Footing Placement

The lateral location and plumbness of drilled holes can be adversely affected by: stones and roots struck during drilling, rough/sloping terrain, drilling equipment characteristics, limited site access for drilling equipment, etc. This frequently requires that the base of a hole be manually enlarged to facilitate more accurate footing placement. Unless otherwise permitted by engineering design, a precast concrete footing shall be placed so that the center of the footing is within a distance $b/2$ of the center of the post/pier it supports, where b is the width of the post/pier. Cast-in-place concrete footings shall be placed so that distance from the center of the post/pier to the nearest edge of the footing is not less than half the specified diameter/width of the footing.

5.13 References

5.13.1 Non-Normative References

Bohnhoff, D. R., Hartjes, Z. D., Kammel, D. W., & Ryan, N.P. (2003). In-situ hydration of a dry concrete mix. ASAE Paper No. 034003. St. Joseph, MI.: American Society of Agricultural Engineers.

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5.13.2 Normative References

Structural Design Specifications

ACI 318 *Building code requirements for structural concrete and commentary*

ANSI/AWC NDS *National design specification (NDS) for wood construction* with commentary

ANSI/ASAE EP486.2 *Shallow post and pier foundation design*

ANSI/ASAE EP559 *Design requirements and bending properties for mechanically laminated-wood assemblies*

SEI/ASCE 32 *Design and construction of frost-protected shallow foundations*

Laboratory Soil Testing Standards

ASTM D422 *Standard test method for particle-size analysis of soils*

ASTM D854 *Standard test methods for specific gravity of soil solids by water pycnometer*

ASTM D2166 *Standard test method for unconfined compressive strength of cohesive soil*

ASTM D2216 *Test methods for laboratory determination of water (moisture) content of soil and rock by mass*

ASTM D2487 *Standard practice for classification of soils for engineering purposes (unified soil classification system)*

ASTM D2850 *Standard test method for unconsolidated-undrained triaxial compression test on cohesive soils*

ASTM D3080 *Standard test method for direct shear test of soils under consolidated drained conditions*

ASTM D4318 *Standard test methods for liquid limit, plastic limit, and plasticity index of soils*

ASTM D4643 *Test method for determination of water (moisture) content of soil by microwave oven heating*

ASTM D4767 *Standard test method for consolidated undrained triaxial compression test for cohesive soils*

ASTM D4943 *Standard test method for shrinkage factors of soils by the wax method*

In-Situ Soil Testing Standards

ASTM D1586 *Standard test method for standard penetration test (SPT) and split-barrel sampling of soils*

ASTM D1587 *Standard practice for thin-walled tube sampling of soils for geotechnical purposes*

ASTM D2573 *Standard test method for field vane shear test in cohesive soil*

ASTM D2937 *Standard test method for density of soil in place by the drive-cylinder method*

ASTM D3441 *Standard test method for mechanical cone penetration tests of soil*

ASTM D4719 *Standard test method for prebored pressuremeter testing in soils*

ASTM D5778 *Standard test method for electronic friction cone and piezocone penetration testing of soils*

ASTM D6066 *Standard practice for determining the normalized penetration resistance of sands for evaluation of liquefaction potential*

Preservative-Treated Wood Standard

AWPA U1 *Use category system: User specification for treated wood*

CHAPTER 6

Diaphragm Design

Contents

6.1 Introduction

6.1.1 2-D Frame Analysis

Prior to the 1980's, the common method of analysis for post-frame structures in agricultural, commercial and light industrial applications was to consider the structure as a system of independently-acting, two-dimensional (2-D) post-frames. Although a 2-D frame analysis method works well for designing frames under vertical loadings; it is often too conservative for designing buildings against sidesway. In addition, many 2-D frames offer little or no resistance to loads acting normal to the frames (e.g., wind acting normal to the endwalls).

6.1.2 Diaphragm Action

A considerable portion of the horizontal load applied to many post-frame structures is actually resisted by roof and ceiling diaphragms and shearwalls. As previously stated (Section 4.10), roof and ceiling diaphragms are large plates that are formed when cladding is attached to roof and ceiling framing, respectively. These large plates help redistribute load throughout the structure. This redistribution of load by the diaphragms is called *diaphragm action*. A shearwall is any wall – interior or exterior – with a measurable amount of racking resistance. Most of the load to which a diaphragm is subjected, is transferred to the foundation by shearwalls orientated parallel to the direction of applied load. Figure 6-1 illustrates a situation in which wind load directed at a sidewall, is transferred via the roof diaphragm to the endwalls and one interior wall. Under this loading, the two endwalls and the one interior wall function as shearwalls. When the same wind load is directed toward the endwall, the sidewalls function as shearwalls in transferring the load from the roof diaphragm to the foundation system.

6.1.3 Post-Frame Contributions

Whenever load is applied normal to the sidewall of a structure, any post-frame with measurable racking resistance functions like the interior shearwall in figure 6-1. The amount of load that an individual post-frame will transfer to the foundation is dependent on (1) the in-plane shear stiffness of the diaphragm, and (2) the racking stiffness of the post-frame relative to that of other post-frames and shearwalls. If a diaphragm is constructed in such a way that it is quite stiff in shear, diaphragm action will be enhanced and the diaphragm will transfer load from post-frames with low racking stiffness to shearwalls and post-frames with high racking stiffness. However, if the shear stiffness of the diaphragm is relatively low, load transfer will be minimal and the behavior of the structure will be much more in accordance with the assumption of independently acting post-frames.

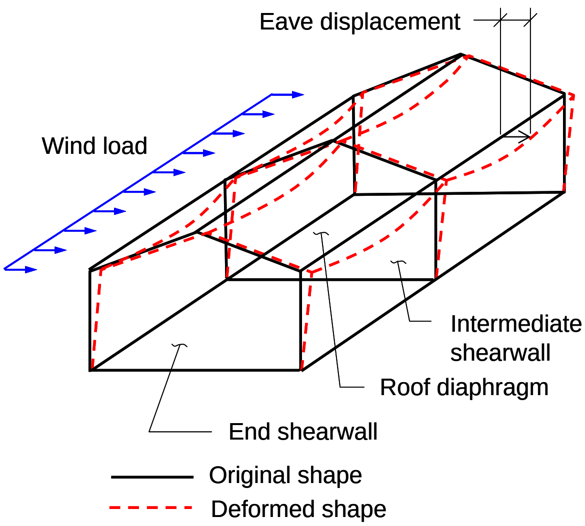


Figure 6-1. Example of diaphragm action in which the roof diaphragm transfers load to three shearwalls – one interior and two exterior walls.

6.1.4 Endwall Loadings

Virtually all post-frame buildings are longer than they are wide. It follows, that diaphragms in such buildings, when viewed from the endwall, appear as narrow, deep plates. For endwall loadings, these narrow, deep diaphragms are generally assumed to have an infinite shear stiffness, which means that every structural element attached to the diaphragm, shifts the same amount when the diaphragm shifts without rotating. For example, under an endwall loading, the roof diaphragm would ensure equal displacement of the top of endwall posts and the top of each sidewall.

6.1.5 Diaphragm Design

When diaphragm action is accounted for in overall building design, the design process is referred to as *diaphragm design*. Diaphragm design is a relatively straight forward process when a diaphragm is (1) assumed to have infinite shear stiffness, and/or (2) only attached to two shearwalls/post-frames (as is generally the case with endwall loadings). When neither of these conditions applies (generally true with loads normal to the sidewall) diaphragm design is more complex.

6.1.6 ASAE EP484.2.

The current diaphragm design procedure is outlined in ANSI/ASAE EP484.2 *Diaphragm Design of Metal-Clad, Wood-Frame Rectangular Buildings*. This procedure, which is outlined in the following sections, can be broken into five steps:

Step 1. Construct a finite element model of the building by breaking the structure into frame, shearwall, and diaphragm elements (Section 6.2)

Step 2. Assign stiffness values to frames and shearwall elements (Section 6.3) and diaphragm elements (Section 6.4).

Step 3. Calculate structural loads (i.e., eave loads) for the model (Section 6.5).

Step 4. Determine the distribution of load to individual elements (Section 6.6).

Step 5. Check to make sure that loads do not exceed allowable values (Section 6.7).

6.2 Structural Model

6.2.1 General

The model developed in this section is only applicable for determining the distribution of loads that are applied parallel to individual post-frames (a.k.a., primary frames). As previously stated, an individual post-frame consists of an individual truss and any attached posts.

6.2.2 Diaphragm Sectioning

The process of modeling a post-frame building for diaphragm design begins with the dividing of individual roof and ceiling diaphragms into sections, herein referred to as *diaphragm sections*. Diaphragm sectioning is a straight-forward process with interior post-frames, interior shearwalls, ridge lines and any other abrupt changes in roof and ceiling slopes servings as lines of demarcation between diaphragm sections.

Figures 6-2a shows a post-frame building with three interior post-frames. Drawing a line along each interior frame and the ridge results in the eight (8) roof diaphragm sections shown in figure 6-2c, and the four ceiling diaphragm sections shown in figure 6-2d.

To avoid confusion when assigning properties to diaphragm sections, it is helpful to identify each diaphragm section with a two-digit identifier. The first digit identifies the bay associated with the section. Bays are generally numbered from left-to-right, as shown in figures 6-2c and 6-2d. The second digit identifies the specific roof or ceiling slope. In figure 6-2, letters have been used to identify these slopes, with letters “a” and “b” representing different roof slopes, and letter “c” used to identify ceiling sections.

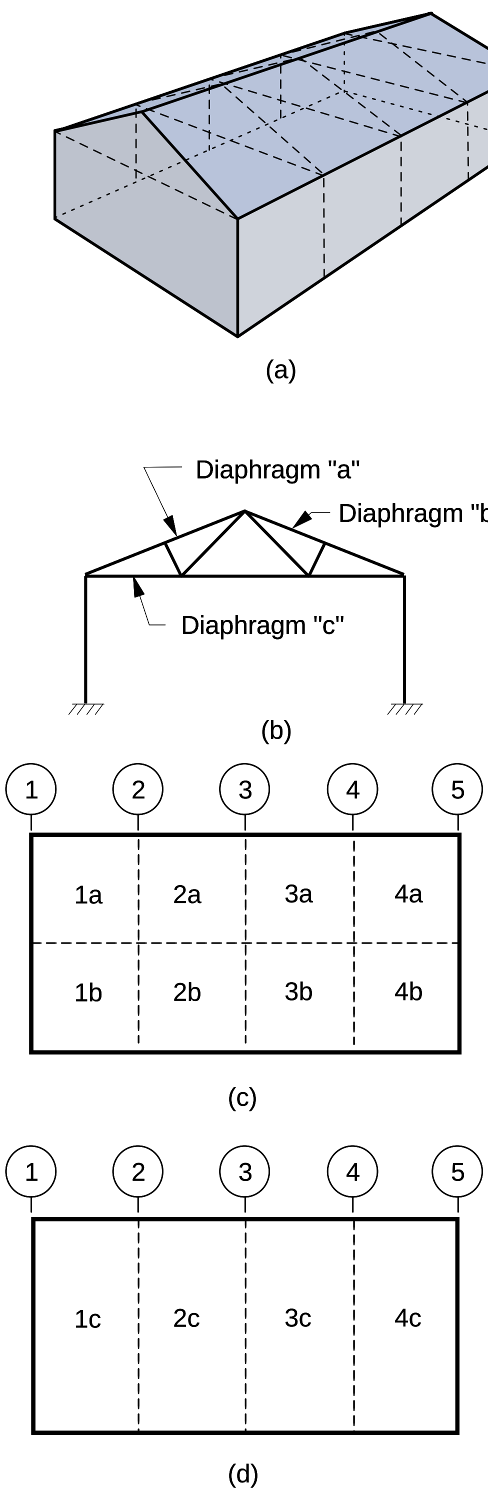


Figure 6-2. (a) Perspective view of a four-bay post-frame building with (b) roof and ceiling diaphragms. Sectioning of (c) roof diaphragms, and (d) ceiling diaphragm.

6.2.3 Discretization

The process of breaking a structure into *elements* for analysis is referred to as *discretization*. For diaphragm design, a structure is broken into frame elements and diaphragm elements. Each post-frame is considered a separate frame element, as is each shearwall orientated in the same direction as the post-frames. The example building shown in figure 6-2 would be modeled with five (5) frame elements. These frame elements have been identified in figures 6-2c and 6-2d with the encircled numbers (as with individual bays, numbering is generally from left-to-right). Each diaphragm element models the diaphragm sections within a single bay. For example, diaphragm sections 1a, 1b, and 1c in figure 6-2 would be represented with a single diaphragm element. It follows that the number of diaphragm elements is equal to the number of building bays, which in turn, is one less than the number of frame elements. Discretization of a four-bay building is shown in figure 6-3a.

6.2.4 Spring Model

To determine the distribution of horizontally applied loads to individual diaphragm and frame elements requires only a single stiffness property for each element. For this reason, diaphragm and frame elements are generally represented with simple springs. As shown in figure 6-3c, frame elements are represented with springs of stiffness, k, and diaphragm elements are represented as springs with stiffness C_h. The element (or spring) connection points (a.k.a. nodes) are taken to represent locations at the eave of each frame/shearwall.

Horizontal components of applied building loads are typically uniformly distributed along the length of the building as shown in figure 6-3a. For modeling purposes, this uniform load is converted into a set of equivalent concentrated loads that are applied at the nodes as shown in figure 6-3b. Because of the location of their application, these forces are referred to as *eave loads*.

6.3 Frame Stiffness, *k*

6.3.1 Definition

To be compatible with a model in which nodes represent points along the eave line (figure 6-3c), frame element stiffness, k, must equal the force required for a unit displacement of the frame at the eave (figure 6-4). In equation form:

$$k = P / \Delta \tag{6-1}$$

where:

k = frame stiffness, lbf/in (N/mm)
P = load applied at eave, lbf (N)
Δ = lateral displacement at eave resulting from applied load P, in (mm)

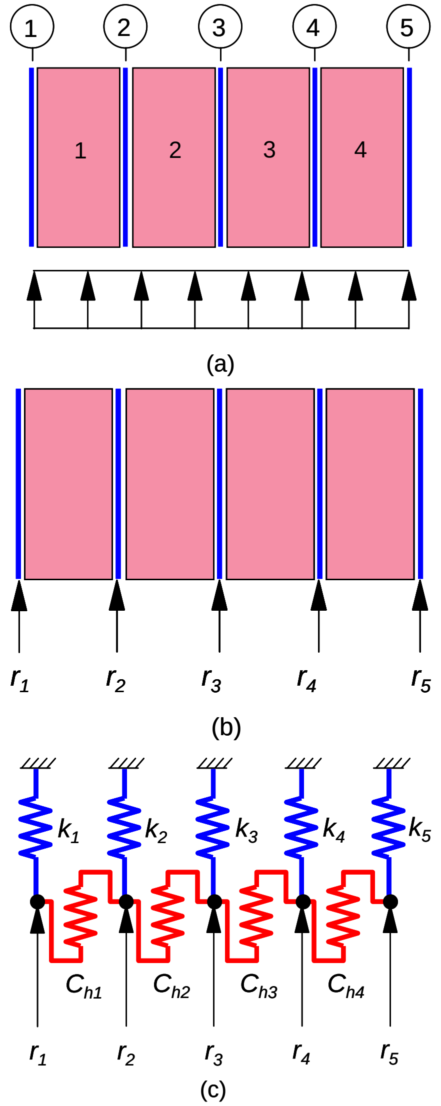


Figure 6.3. (a) Top view of a four-bay building showing individual elements and applied horizontal loads. Encircled numbers identify frame elements, other numbers identify diaphragm elements. (b) Load concentrated at eaves. (c) Corresponding spring model.

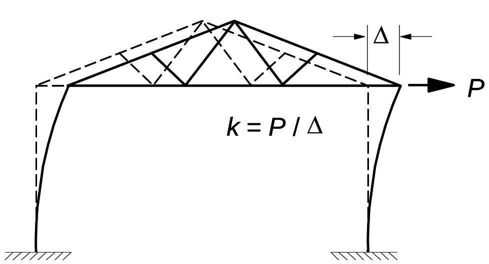


Figure 6-4. Definition of frame stiffness, *k*.

6.3.2 Frame Stiffness via FEA

Frame stiffness is generally obtained by analyzing an entire post-frame with a 2-dimensional finite element analysis (FEA) program. Numerous FEA programs are commercially available for this task. The algorithms used by these programs will produce identical results when a linear, first-order analysis is performed using the same structural analog. When material non-linearities or second order analyses (e.g. P-delta analyses) are conducted, different FEA programs may give slightly different results for the same structural analog.

One of the unique aspects of post-frame buildings for which a modeler must account is the movement associated with the deformation of the soil surrounding embedded posts.

6.3.2.1 Modeling with Soil Springs

The most accurate way to account for movement associated with soil deformation is to use soil springs as presented in Section 5.6.

6.3.2.2 Modeling Without Soil Springs

In the past, engineers generally ignored soil properties, and modeled embedded posts using analogs similar to those shown in figure 6-5. An inherent deficiency of these analogs is that the pin supports used to fix the posts below grade do not allow the posts to naturally displace. More problematic is the fact that the analogs in figures 6-5 produce a reduced post stiffness when depth of embedment, *d*, is increased. In reality, anytime a post is embedded deeper into the ground, the stiffness associated with the post increases.

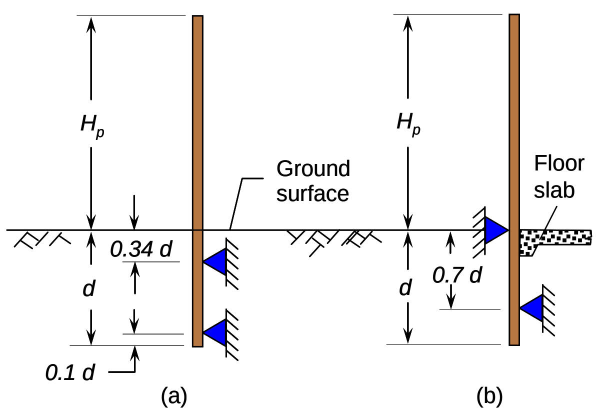


Figure 6.5. Structural analogs once used for modeling (a) non-constrained and (b) surface constrained posts. These analogs are not recommended for use as they provide a reduced post stiffness with an increase in embedment depth.

Individuals who elect to ignore soil behavior in order to simplify the modeling of an embedded post, often fix the post at grade. A better alternative to this practice is to fix the post at a depth below grade that is equal to the post width *w* as shown in figure 6-6. This is because the top few inches of soil do not provide much lateral restraint. Not only will fixing the post below grade produce a more realistic frame stiffness, but the moment at the fixed support will more closely match the maximum bending moment that can be expected to occur in the embedded post below grade.

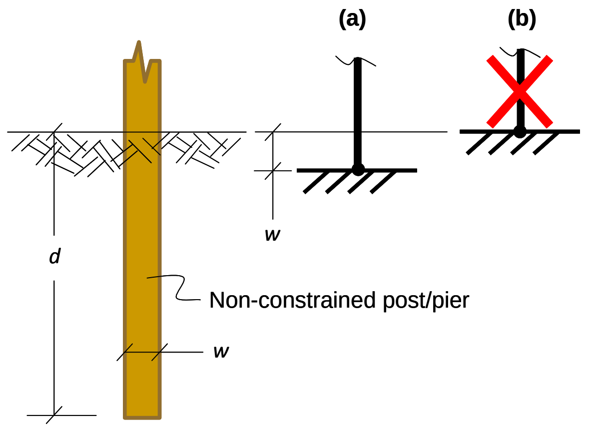


Figure 6-6. When using a fixed base analog to model a non-constrained post, (a) fix the post at a depth below grade equal to the post side width, instead of (b) fixing the post at grade.

6.3.3 Frame Stiffness via Summation of Individual Post Stiffness Values

Frame stiffness can be calculated to varying degrees of accuracy by summing the stiffness of the individual posts within the frame. In equation form:

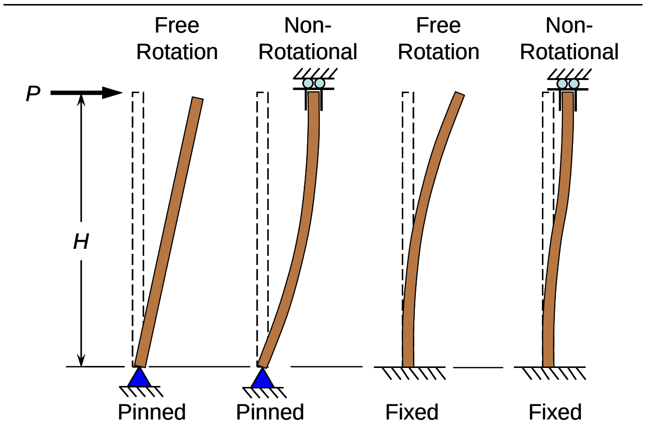
$$k = \sum_{i=1}^{n} k_{P,i} \qquad (6\text{-}2)$$

where:

$k_{P,i}$ = stiffness of post *i*, lbf/in (N/mm)
= $P_{p,i}/\Delta_{p,i}$
$P_{P,i}$ = horizontal force applied to top of post *i*, lbf (N)
$\Delta_{P,i}$ = horizontal displacement of the top of post *i*, due to load $P_{P,i}$, in. (mm)
n = number of posts in the post-frame

Tables 6-1 and 6-2 contain equations for calculating individual post stiffness values for posts with a constant flexural rigidty, *EI*. This requirement excludes spliced mechlam posts as defined in Chapter 8. Equations in Table 6-1 are for posts assumed to be fixed or pinned at their base. Equations in Table 6-2 are for posts embedded in soil.

Table 6-1. Post Stiffness Equations for Posts Not Embedded in Soil



Fixity Case	Post-to-Base Connection	Post Top Restrained from Rotating? (a)	Post Stiffness, k_P (b)
1	Pinned	No	0
2	Pinned	Yes	$3EI/H^3$
3	Fixed	No	$3EI/H^3$
4	Fixed	Yes	$12EI/H^3$

(a) Top of posts are free to translate in the vertical and horizontal directions.
(b) *E* is post modulus of elasticity, *I* is post moment of inertia, and *H* is distance between truss/rafter and base connections.

The Table 6-2 equations assume the embedded portion of the post has an infinite flexural rigidity (*EI*) below grade and a constant flexural rigidity stiffness above grade, and that soil modulus of elasticity E_S increases linearly with depth *z* as

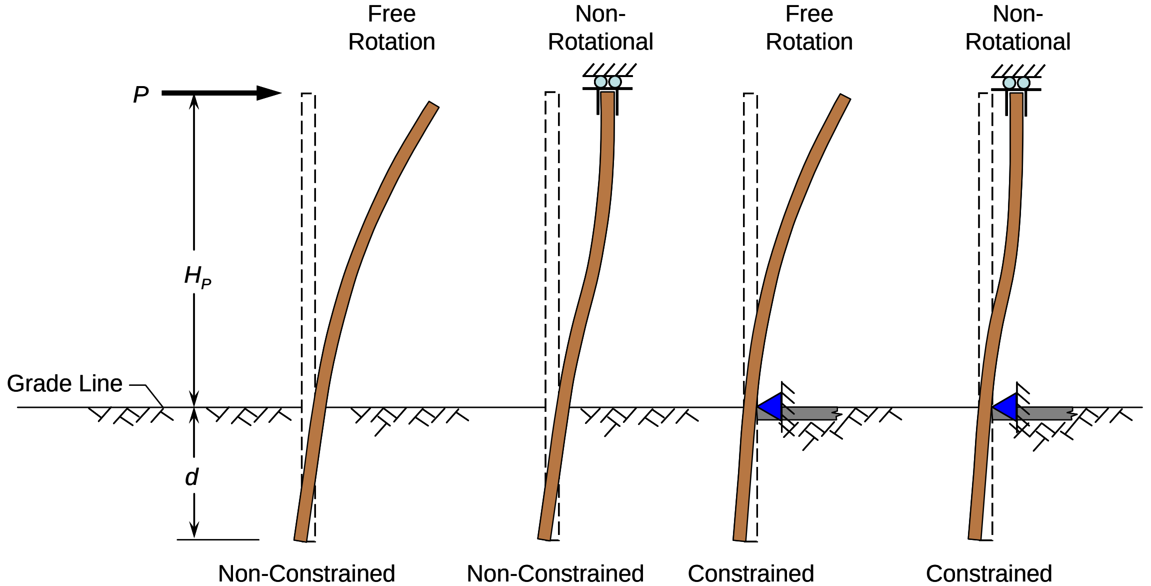
$$E_S = A_E\, z.$$

Calculating frame stiffness by summing individual post stiffness values is relatively accurate when post-to-truss (or post-to-rafter) connections behave as pins and thus can be appropriately modeled without rotational restraint (Fixity Cases 1 and 3 in Table 6-1 and Fixity Cases 5 and 7 in Table 6-2).

As the rigidity of post-to-truss/rafter connections increases, the greater will be the dependency of frame stiffness on the in-plane bending stiffness of the truss/rafter(s).

For there to be full rotational restraint at the top of the post (as is assumed with Fixity Cases 2, 4, 6 and 8), the post-to-truss/rafter connection must be completely rigid AND the truss/rafter must have an infinite bending stiffness. The error introduced by the infinite bending stiffness assumption was investigated by Bohnhoff (1992a) and found to significantly impact some frame stiffness calculations. Specifically, the stiffness of frames in narrower buildings with low eave heights was overestimated by 50%. Conversely, in some taller and wider buildings, the infinitely stiff truss assumption overestimated frame stiffness by less than 10%.

Table 6-2. Post Stiffness Equations for Posts Embedded in Soil [a][b]



Fixity Case	Post Fixity at Groundline	Post Top Restrained from Rotating? [c]	Post Stiffness, k_P [d]
5	Non-Constrained	No	$3\,EI\,[C/\{C+4.5(d/H_P)^2 + 12(d/H_P)+9\}]/H_P{}^3$
6	Non-Constrained	Yes	$12\,EI\,[(C+3)/\{C + (d/H_P)^2(18+6/C) + 24d/H_P + 12\}]/H_P{}^3$
7	Constrained	No	$3\,EI\,[C/(1+C)]/H_P{}^3$
8	Constrained	Yes	$12\,EI\,[(1+3C)/(4+3C)]/H_P{}^3$

(a) Equations assume that the embedded portion of the post has an infinite flexural rigidity (EI) below grade and a constant EI above grade. Soil modulus of elasticity E_S is assumed to increase linearly with depth z as $E_S = A_E\,z$.
(b) From Bohnhoff (1992b)
(c) Top of posts are free to translate in the vertical and horizontal directions.
(d) C = $d^4 A_E H_P/(6\,EI)$
d = distance from grade to the top of a detached footing, or distance from grade to the bottom of an attached footing.
H_P = distance from grade to the post-to-truss/rafter connection.
A_E = increase in Young's modulus for soil per unit increase in depth z below grade.
E = modulus of elasticity for above-grade portion of post.
I = moment of inertia for above-grade portion of post.

6.3.4 Shearwalls

End shearwalls and intermediate shearwalls, like post-frames, are modeled as frame elements (see Section 6.2.3). Consequently, their stiffness, like that for post-frames, is defined as the ratio of a horizontal force, *P*, applied at the eave of the wall, to the resulting horizontal eave displacement, Δ.

The stiffness of shearwalls can be obtained using validated structural models, or from tests of functionally equivalent assemblies. ASAE EP558.1 gives laboratory test procedures that can be used to determine the stiffness of functionally equivalent walls. This topic is also discussed in Section 7.5.

6.3.5 Example Calculations

Problem Statement

Determine the frame stiffness for the post-frame shown in figure 6-7. Posts are nominal 6- by 6-inch No. 2 Southern Pine, that are embedded 3.5 feet. Posts are not attached to the footing upon which they bear. A slab-on-grade restricts inward post movement, but does not restrict movement away from the slab. Truss is pin-connected to the posts. Truss chords are nominal 2- by 6-inch No. 1 SP. Truss webs are nominal 2- by 4-inch No. 2 SP. Both backfill and surrounding soil are best classified as a loose, silty fine-grained sand. The water table is located a couple feet below the footing.

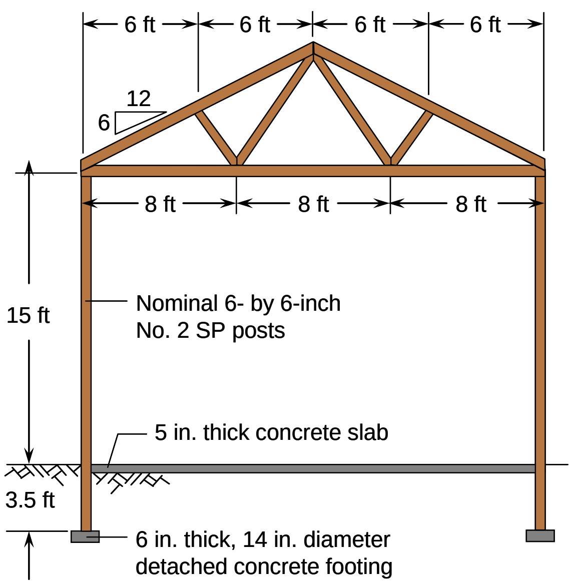


Figure 6-7. Example post-frame.

Solution 1: Modeling with Soil Springs

A model of the below-grade portion of the post frame is shown in figure 6-8. A vertical roller was used to fix the left post at the groundline (i.e., the point where it would make contact with the concrete slab as the post moves inward). Each spring was used to model a 6-inch soil layer, resulting in a total of seven springs per post. The stiffness assigned each spring was calculated as $K_H = 2.0\ t\ A_E\ z$ where A_E was fixed at 880 lbf/in^2/ft. This value corresponds to two time the 440 lbf/in^2/ft value listed in Table 5-2 for "silty or clayey fine to course sand" in a loose state. Doubling of the value from 440 to 880 lbf/in^2/ft is allowed because the soil is located above the water table (see Table 5-2 footnote).

The actual analysis was performed using IES Inc.'s VisualAnalysis program (VA, 2013). A screen capture of the VisualAnalysis model is shown in figure 6-9. Like most commercially available structural analysis programs, VisualAnalysis contains a special spring element that makes modeling of soil behavior a very straight forward process (you simply input the node and direction for spring application along with spring stiffness).

The *E* value of 1.2 million lbf/in^2 applies both above and below the groundline for timber. Where the post is a mechanically-laminated assembly fabricated from dimension lumber, the tabulated reference value for *E* must be multiplied by the NDS wet service factor of 0.90.

With a post *E* value of 1.2 million lbf/in^2 and *I* value of 76.25 in^4, the application of an 1000 lbf force to the eave of the frame produced a horizontal eave displacement of 13.98 inches. This equates to a frame stiffness of 71.5 lbf/in.

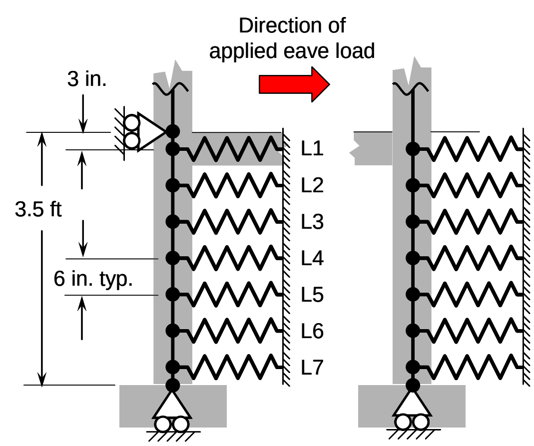


Figure 6-8. Spring analog for the below-grade portion of the post-frame shown in figure 6-7.

Table 6-3. Soil Spring Properties and Modeling Results

Com-poent (a)	Depth Below Grade, z, ft	Spring Stiffness, K_H,(b) lbf/in.	Reaction Force, (c) lbf	Post Move-ment, (d) in.
Slab	0	NA	-5098	0
L1	0.25	2640	74	-0.028
L2	0.75	7920	475	-0.060
L3	1.25	13200	889	-0.067
L4	1.75	18480	1102	-0.060
L5	2.25	23760	1048	-0.044
L6	2.75	29040	739	-0.025
L7	3.25	34320	206	-0.006
			-565(e)	
R1	0.25	2640	-755	0.286
R2	0.75	7920	-1357	0.171
R3	1.25	13200	-1145	0.087
R4	1.75	18480	-521	0.028
R5	2.25	23760	251	-0.011
R6	2.75	29040	1080	-0.037
R7	3.25	34320	2013	-0.059
			-435(f)	

(a) See figure 6-8.
(b) $K_H = 2.0\ t\ A_E\ z$ where A_E is 880 lbf/in^2/ft
(c) Forces due to 1000 lbf eave load. Negative values act in direction opposite to applied eave load.
(d) Post movement at depth z due to 1000 lbf eave load. Negative values are in opposite direction of applied load.
(e) Sum of forces acting on left post.
(f) Sum of forces acting on right post.

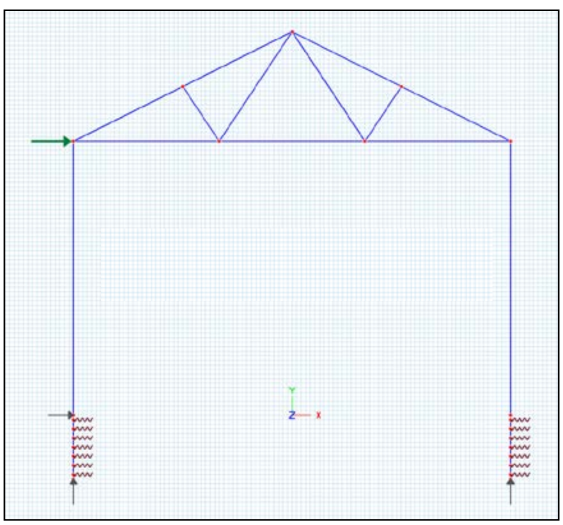


Figure 6-9. Screen capture of VisualAnalysis model. Upper arrow identifies the eave load. Lower arrows show location and direction of fixed displacements.

Two noteworthy items based on the Table 6-3 results. First, the horizontal force that is resisted by the concrete slab is slightly over five times the eave load. Second, the left (or constrained) post resists 56.5% of the applied load, and the right (non-constrained) post the remaining 43.5%. Not surprising, the maximum bending moment is much higher in the left post than in the right (104.7E03 in-lbf versus 76.6E03 in-lbf). Maximum bending moment in the left post occurs at the top of the concrete slab, and that for the right post at the location of spring R1. The significant difference in bending moments points out the importance of accurately modeling different constraint conditions at the post base.

Solution 2: Modeling Without Soil Springs

Two models with fixed supports were analyzed. For the first analysis, both posts were fixed at grade – a model that is not recommended (see figure 6-6(b)) because it *completely* ignores the influence of soil behavior on frame stiffness. For the second analysis, the base of each post was fixed 6 inches below grade – a depth approximately equal to post side width as recommended in Section 6.3.2.2 and shown in figure 6-6(a). Results for both of these analyses are compiled in Table 6-4.

Table 6-4. Results For Models With Fixed Supports

Design Variable	Both Posts Fixed at Grade	Both Posts Fixed Six Inches Below Grade (a)
Eave load, lbf	1000	1000
Eave displacement, inches	10.63	11.29
Frame stiffness, lbf/in.	**94.1**	**88.6**
Shear force in left post, lbf	500.2	519
Shear force in right post, lbf	499.8	481
Maximum bending moment in left post, in-lbf	90.0E03	93.3E03
Maximum bending moment in right post, in-lbf	90.0E03	86.6E03

(a) Left post also fixed from moving horizontally at grade to simulate slab constraint

Both "fixed base" models produce a frame stiffness greater than the 71.5 lbf/inch stiffness associated with the "soil spring" model. Fixing both posts *at grade* resulted in a frame stiffness 31% greater than the soil spring model, and fixing both posts *6-inches below grade* a stiffness 24% greater than the soil spring model.

With both posts fixed at grade, the applied eave load is resisted near equally by the two posts (the slight difference of 0.04% is due to truss deformation). In this case, the horizontal translation of the top of each post could have simply been calculated as $V H^3/(3EI)$ where V is the shear resisted by the post (equal to ½ the applied eave load in this case), H is the vertical distance between the post-to-truss connection and the fixed base, and E and I are the modulus of elasticity and moment of inertia of the post.

Fixing both posts slightly below grade gives a more realistic estimate of frame stiffness, maximum post shear forces and maximum post bending moments than does fixing both posts at grade.

For the specific post-frame being analyzed, the difference in shear forces resisted by the individual posts is a direct measure of the difference between the individual stiffness of a surface-constrained post and that of a non-constrained post. When modeled using soil springs, the post shear forces (and hence maximum bending moments) differed by an amount equal to 13% of the applied eave load. For the model that fixes both posts at a location six inches below grade, this difference in post shear forces is 4%. The fact that 4% is much closer to 0% than 13% implies that fixing the posts six inches below grade, while a step in the right direction - may not be going far enough to capture the affect of soil behavior on frame stiffness.

Solution 3: Frame Stiffness by Summation of Individual Post Stiffness Values

For the post-frame shown in figure 6-7, frame stiffness can be approximated by summing post stiffness values for the surface-constrained (left) post and the non-constrained (right) post. Equations for calculating these values are given in Table 6-2 (Fixity Cases 7 and 5, respectively). In each case d is 42 inches, H_P is 180 inches, A_E is 880 lbf/in^2 per foot of depth (or 73.33 lbf/in^2 per inch of depth), E is 1.2 million lbf/in^2, and I equals 76.25 in^4. These numbers yield post stiffness values of 46.4 lbf/inch and 40.5 lbf/inch for the surface constrained and non-constrained post respectively, for a total frame stiffness of 86.9 lbf/in.

The 86.9 lbf/inch frame stiffness is 20% greater than the frame stiffness of 71.5 lbf/inch obtained with the soil spring model. In this case, the difference is solely due to the fact that the equations in Table 6-2 assume that the embedded portion of a post has an infinite flexural rigidity, EI.

The difference that the rigid post assumption makes in below grade movement of the posts is illustrated in figure 6-11. The displacement values for the curves in figure 6-11 were obtained using the VisualAnalysis (VA) program. VA contains a special "rigid link" element that was used to model those portions of the post assumed to be infinitely rigid. Alternatively, a very large moment of inertia or modulus of elasticity value can be input to VA to produce an element that for all practical purposes behaves as a rigid link. Regardless of the method used, it is important to note that a soil spring model (figure 6-8) will provide the same post stiffness values as the equations in Table 6-2 when (1) the post is pin-connected to the truss/rafter, and (2) the post is assumed to have an infinite flexural rigidity below grade.

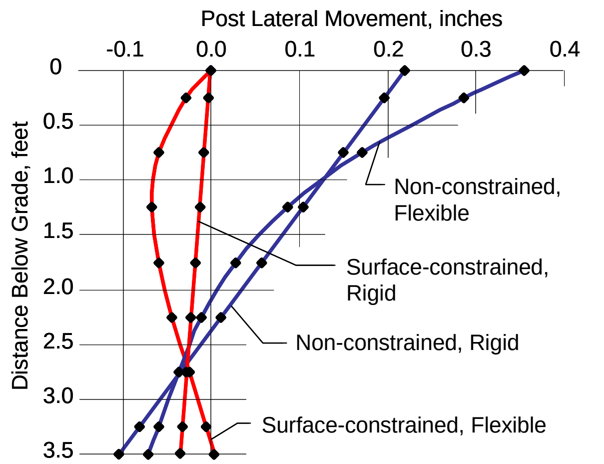


Figure 6-10. Effect of post bending stiffness on the below-grade movement of surface-constrained and non-constrained posts. Not to scale (horizontal displacements significantly exaggerated).

The error introduced by assuming the posts are infinitely rigid below grade is clearly evident in figure 6-10. It is important to note that in both the surface-constrained and non-constrained cases, it is not so much the lateral movement of the posts that influences overall frame stiffness, but the rotation of the posts near grade.

ANSI/ASAE EP 486.2 provides some guidance for use of the "below-grade infinitely rigid post" assumption. When soil modulus of elasticity increases linearly with depth an amount A_E (as was assumed in this case), ANSI/ASAE EP486.2 recommends against assuming a post (with a flexural rigidity EI) has an infinite flexural rigidity below grade if the depth of embedment d exceeds $2[EI/(2A_E)]^{0.20}$. In this case, the quantity $2[EI/(2A_E)]^{0.20}$ equals 28.8 inches, and thus is indeed exceeded by the embedment depth of 42 inches; thereby indicating the posts should not be assumed infinitely rigid below grade. In other words, the designer should probably not rely on Table 6-2 equations to determine the stiffness of the frame in figure 6-7. That said, the equations in Table 6-2 provide a more realistic and more accurate estimate of frame stiffness than obtained by totally ignoring soil stiffness (e.g., modeling without soil springs) as is done when embedded posts are assumed fixed at grade or fixed slightly below grade.

6.4 Diaphragm Stiffness, C_h

6.4.1 Definition

As shown in figure 6-11, the stiffness of a diaphragm element is the horizontal load required to cause a unit shift (in a direction parallel to the trusses/rafters) of the roof/ceiling assembly over a frame spacing (a.k.a. bay width), *s*. This stiffness is commonly referred to as the total horizontal shear stiffness, C_h of the diaphragm for the building bay in question.

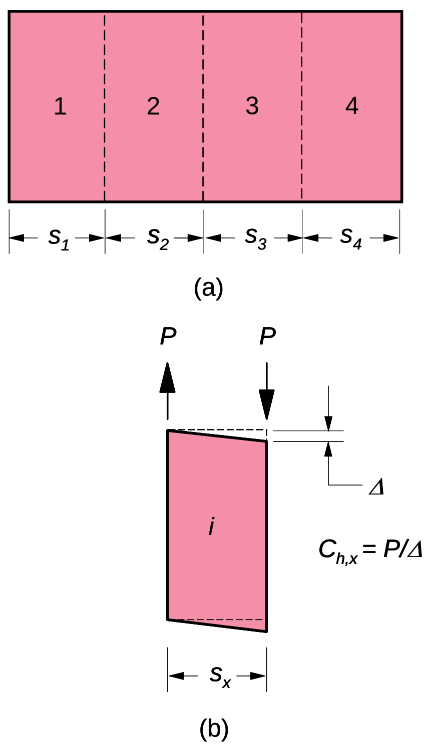


Figure 6-11. (a) Top view of a four-bay building. (b) Definition of total horizontal diaphragm stiffness for building bay *x*, $C_{h,x}$.

6.4.2 Calculation

The total horizontal diaphragm shear stiffness for a building bay is simply equal to the sum of the horizontal shear stiffness values of the diaphragm sections located in the building bay. In equation form:

$$C_{h,x} = \sum_{i=1}^{n} c_{h,i} \qquad (6\text{-}3)$$

where:

$C_{h,x}$ = total horizontal diaphragm shear stiffness for building bay *x*, lbf/in (N/mm)

$c_{h,i}$ = horizontal shear stiffness of diaphragm section *i* in building bay *x* (from Section 6.4.4), lbf/in (N/mm)

n = number of individual diaphragm sections located in building bay *x*

6.4.3 Example Calculation

Problem Statement:

What is the total horizontal diaphragm shear stiffness for each of the four diaphragm elements for the building shown in figure 6-2? Assume that each roof diaphragm section has a horizontal shear stiffness of 6000 lbf/inch and that each ceiling diaphragm section has a horizontal shear stiffness of 4000 lbf/inch.

Solution:

For the building in figure 6-2, each bay is comprised of two roof sections and a ceiling section. Adding the horizontal stiffness of each of these three diaphragm sections yields the total horizontal diaphragm shear stiffness for the bay. For building bay 1:

$$\begin{aligned} C_{h,1} &= c_{h,1a} + c_{h,1b} + c_{h,1c} \\ &= 6000 \text{ lbf/in} + 6000 \text{ lbf/in} + 4000 \text{ lbf/in} \\ &= 16{,}000 \text{ lbf/inch} \end{aligned}$$

Given that all diaphragm in this example are identical, $C_{h,1} = C_{h,2} = C_{h,3} = C_{h,4}$.

6.5 Eave Load, *R*

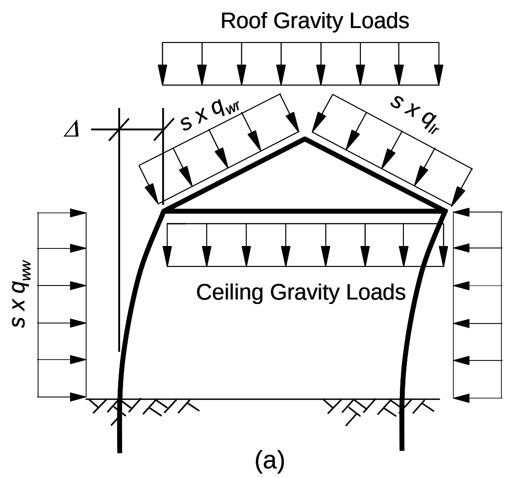
6.5.1 Definition

For diaphragm design, building loads are replaced by an equivalent set of horizontally acting, concentrated (i.e., point) loads. These loads are located at the eave of each frame element (i.e., at each interior post-frame, each end shearwall, and each intermediate shearwall) and therefore are referred to as *eave loads*. Eave loads and applied building loads are equivalent when they horizontally displace the eave an equal amount Δ as shown in figure 6-12.

It is important to note that when eave load *R* from figure 6-12(b) is applied in the opposite direction to the loaded frame in figure 6-12(a), the horizontal displacement of the eave Δ will be numerically equal to zero. In other words, eave load *R* is numerically equal to the force that keeps the eave of a loaded frame from moving horizontally.

6.5.2 Calculation by Plane-Frame Structural Analysis

To determine *R* by plane-frame structural analysis, a horizontal restraint (vertical roller) is placed at the eave line as shown in figure 6-13 and the structural analog is analyzed with all external loads in place. The horizontal reaction at the vertical roller support is numerically equal to the eave load, *R*.



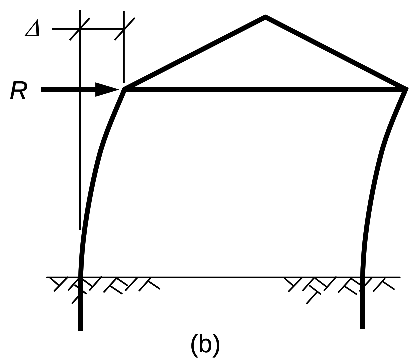


Figure 6-12. Horizontal eave displacement *Δ* due to (a) applied building loads, and (b) eave load, *R*.

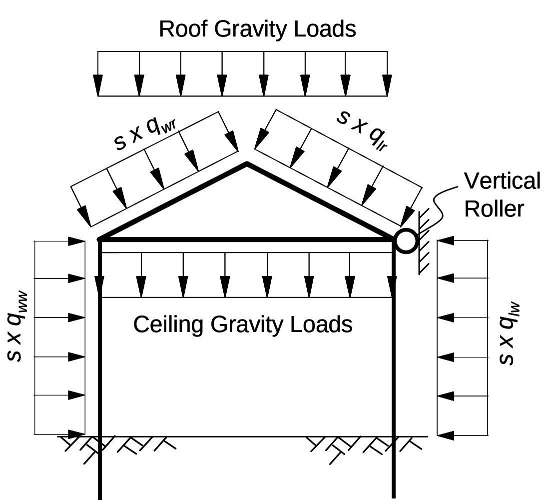


Figure 6-13. Typical structural analog for obtaining eave load, *R*.

Always place the vertical roller at the same location horizontal load *P* was placed when determining frame stiffness (see figure 6-4). The value of *R* is very dependent on the magnitude of forces with horizontal components (i.e., wind and stored materials).

6.5.3 Calculation Using Post Fixity Factors

For a given post-frame, eave load *R* is numerically equal to the total horizontal load applied to the truss (or rafters), plus the horizontal load transferred to the truss (or rafters) by each attached post. When: (1) the only applied loads with horizontal components are due to wind, (2) wind pressure is uniformly distributed on each wall and roof surface, and (3) each post has a fixed flexural rigidity (*EI*), then eave load can be estimated as:

$$R = s\,(H_{wr}\,q_{wr} - H_{lr}\,q_{lr} + H_{ww}\,f_w\,q_{ww} - H_{lw}\,f_l\,q_{lw}) \qquad (6\text{-}4)$$

where:

R = eave load, lbf (N)
s = bay width = frame spacing for interior post-frames and shearwalls, ft (m)
= one-half the frame spacing for endwalls, ft (m)
H_{wr} = windward roof height, ft (m)
H_{lr} = leeward roof height, ft (m)
H_{ww} = windward wall height, ft (m)
H_{lw} = leeward wall height, ft (m)
q_{wr} = windward roof pressure, lbf/ft^2 (N/m^2)
q_{lr} = leeward roof pressure, lbf/ft^2 (N/m^2)
q_{ww} = windward wall pressure, lbf/ft^2 (N/m^2)
q_{lw} = leeward wall pressure, lbf/ft^2 (N/m^2)
f_w = windward post fixity factor (Table 6.5)
f_l = leeward post fixity factor (Table 6.5)

Height and wind pressure variables are graphically defined in Figure 6-14. Inward acting wind pressures have positive signs, outward acting pressures are negative. In buildings with variable frame spacings, set *s* equal to the average of the frame spacings on each side of the frame for which the eave load is being calculated.

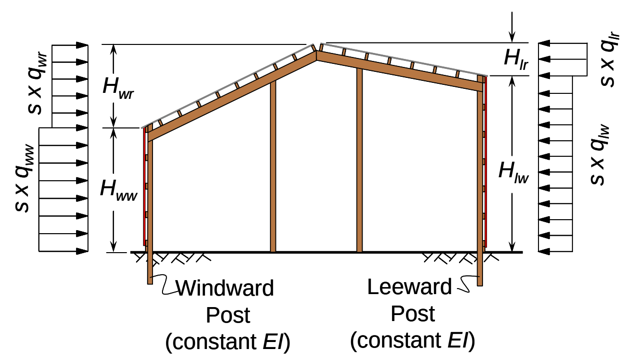


Figure 6-14. Variable definitions for equation 6-4.

For symmetrical base restraint and frame geometry, equation 6-4 reduces to:

$$R = s\,[H_r\,(q_{wr} - q_{lr}) + H_w\,f\,(q_{ww} - q_{lw})] \qquad (6\text{-}5)$$

where:

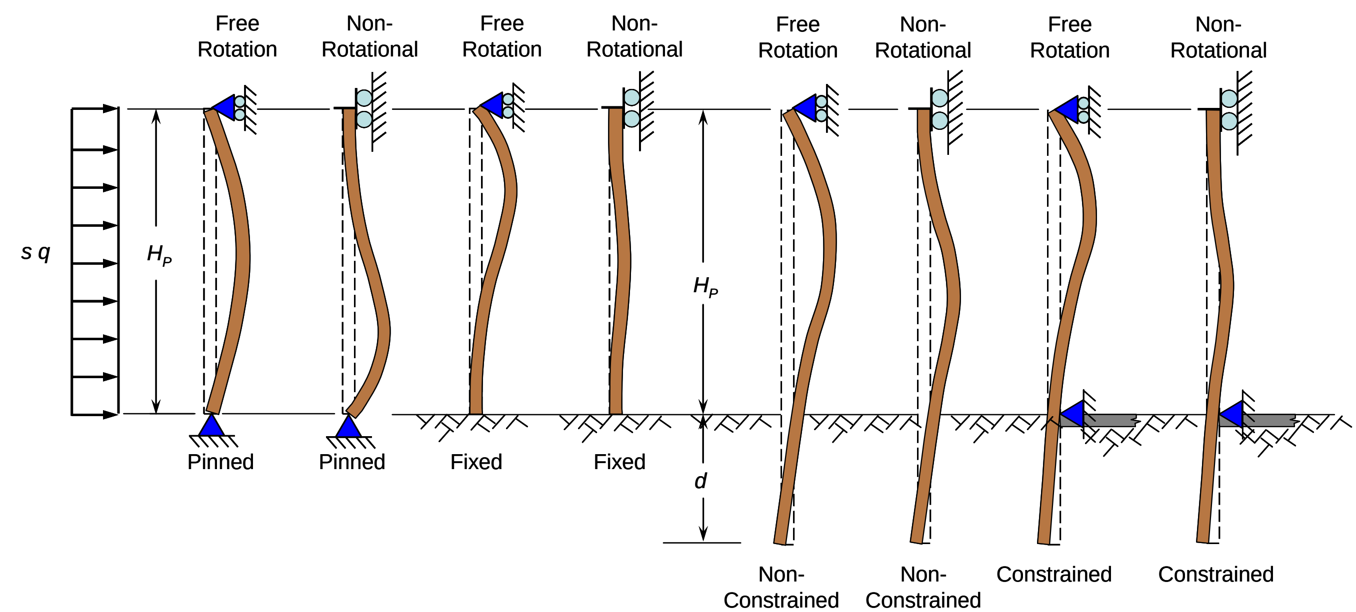
H_r = roof height, ft (m)
H_w = wall height, ft (m)
f = frame-base fixity factor for both leeward and windward posts

A post fixity factor is the fraction of the total load acting on a post that is transferred to the eave by the post. Post fixity factors are a function of load distribution, post properties and post end fixities.

Table 6-5 contains post fixity factors for uniformly loaded posts that have a constant flexural rigidity (EI).

Cases 1 through 4 are for posts assumed to be fixed or pinned at their base. Cases 5 through 8 apply to embedded posts. Equations for embedded posts assume that the embedded portion has an infinite flexural rigidity (EI), and that soil modulus of elasticity E_S increases linearly with depth z as $E_S = A_E z$.

Table 6-5. Post Fixity Factors [(a)(b)]



Fixity Case	Post Fixity at Groundline	Post Top Restrained from Rotating? [(c)]	Post Fixity Factor, f [(d)]
1	Pinned	No	1/2
2	Pinned	Yes	5/8
3	Fixed	No	3/8
4	Fixed	Yes	1/2
5	Non-Constrained	No	$[C/12 + (d/H_P)^2 + 2d/H_P + 1] / [2C/9 + (d/H_P)^2 + 8d/(3H_P) + 2]$
6	Non-Constrained	Yes	$[C/2 + (d/H_P)^2(18+6/C) + 20d/H_P + 7.5] / [C + (d/H_P)^2(18+6/C) + 24d/H_P + 12]$
7	Constrained	No	$(4 + 3C) / (8 + 8C)$
8	Constrained	Yes	$(5 + 3C) / (8 + 6C)$

(a) Equations assume a uniformly distributed load acts over height H_P and that the embedded portion of the post has an infinite flexural rigidity (EI) below grade and a constant flexural rigidity above grade. Soil modulus of elasticity E_S is assumed to increase linearly with depth z as $E_S = A_E z$.

(b) From Bohnhoff (1992b)

(c) Top of posts can freely translate in vertical direction but are fixed from moving horizontally.

(d) $C = d^4 A_E H_P / (6\,EI)$

d = distance from grade to the top of a detached footing, or distance from grade to the bottom of an attached footing.

H_P = distance from grade to the post-to-truss/rafter connection. Also distance over which the uniform load is applied.

A_E = increase in Young's modulus for soil per unit increase in depth z below grade.

E = modulus of elasticity for above-grade portion of post.

I = moment of inertia for above-grade portion of post.

6.5.4 Example Calculations

Problem Statement

The post-frame described in Section 6.3.5 is shown under a combination of wind and snow loads in figure 6-15. These loads include a balanced snow load of 30 lbf/ft^2, dead loads of 5 lbf/ft^2 on both the roof and ceiling, and winds loads of 8, 3, -7, and -5 lbf/ft^2 on the windward wall, windward roof, leeward roof and leeward wall, respectively. For analysis purposes, the roof dead load of 5 lbf/ft^2 was converted to a 5.6 lbf/ft^2 load acting on a horizontal plane. In figure 6-19 this roof load is shown combined with the balanced snow load.

Determine eave load *R* assuming (1) posts are pin-connected to the truss, and then (2) posts are rigidly connected to the truss.

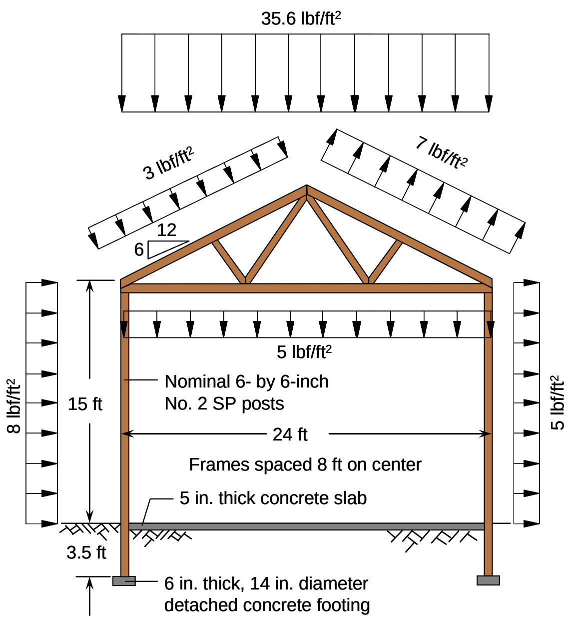


Figure 6-15. Example post-frame with applied loads.

Solution 1: Calculation by Plane-Frame Structural Analysis

The post-frame was modeled as described in Section 6.3.5 using seven springs per post to model soil behavior, and a vertical roller to fix the left post at the groundline (i.e., the point where it would make contact with the concrete slab as the post moves inward). In addition, a vertical roller support was placed at left post-to-truss connection to determine eave load *R*.

The actual analysis was performed using IES Inc.'s VisualAnalysis program (VA, 2013). Screen captures of VisualAnalysis output are shown in figure 6-16 for the pinned post-to-truss model and the fixed post-to-truss model. These models produced eave load values of 1111 lbf and 1249 lbf, respectively.

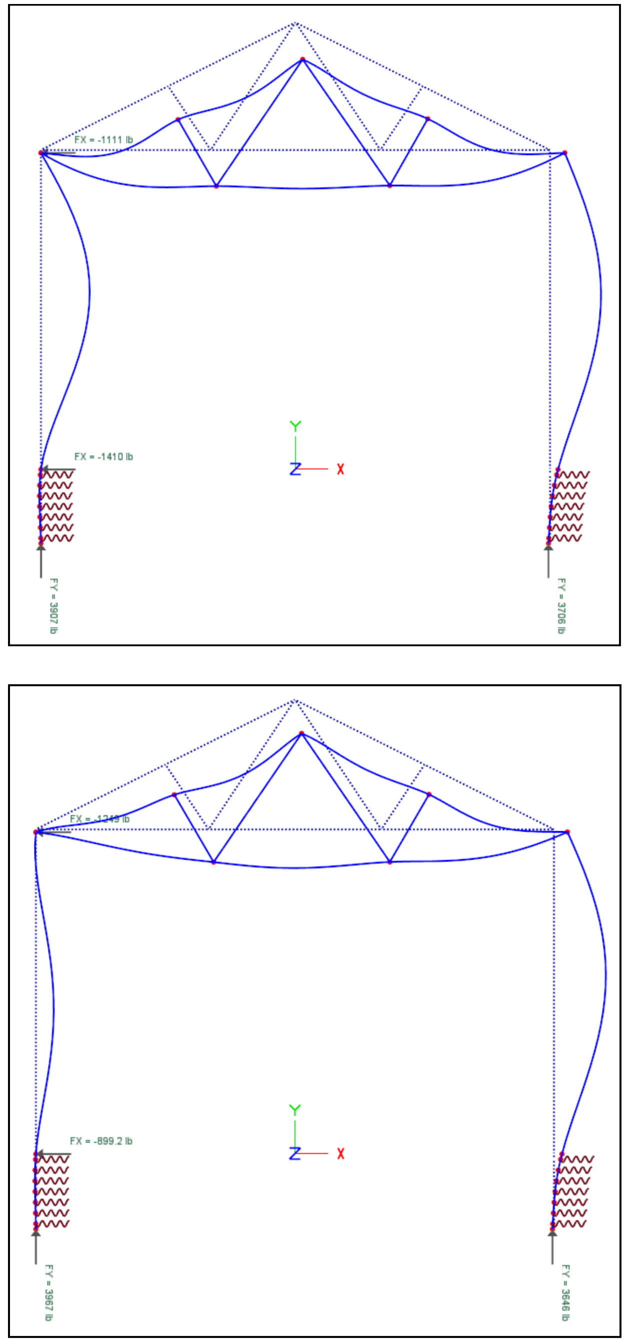


Figure 6-16. Screen captures of VisualAnalysis output showing deformed model for posts pinned to truss (top) and posts rigidly-connected to truss (bottom).

Solution 2: Calculation Using Post Fixity Factors for Embedded Posts

Since leeward and windward roof heights are the same, and leeward and windward wall heights are the same, equation 6-4 can be reduced to:

$$R = s\,[H_r\,(q_{wr} - q_{lr}) + H_w\,(f_w\,q_{ww} - f_l\,q_{lw})] \qquad (6\text{-}6)$$

For this example problem, equation variables are given as:

s = bay width = frame spacing = 8 ft
H_r = roof height = 6 ft
H_w = wall height = 15 ft
q_{wr} = windward roof pressure = 3 lbf/ft^2
q_{lr} = leeward roof pressure = -7 lbf/ft^2
q_{ww} = windward wall pressure = 8 lbf/ft^2
q_{lw} = leeward wall pressure = -5 lbf/ft^2
f_w = windward post fixity factor
= 0.377 for constrained post pinned to truss (Table 6-5 fixity case 7)
= 0.502 for constrained post with top fixed from rotating (Table 6-5 fixity case 8)
f_l = leeward post fixity factor
= 0.402 for non-constrained post pinned to truss (Table 6-7 fixity case 5)
= 0.541 for non-constrained post with top fixed from rotating (Table 6-5 fixity case 6)

Post fixity factors were calculated using the equations in Table 6-5 with C equal to 74.8 and a d/H_P ratio of 0.233.

The proceeding variables produce an eave load R of 1083 lbf when posts are assumed pinned to the trusses, and a value of 1482 lbf when the top of posts are not allowed to rotate.

The 1083 lbf eave load is 2.5% lower than the 1111 lbf value obtained via plane-frame structural analysis with soil springs and posts pin-connected to the truss. This difference can be attributed to the fact that post fixity factors for embedded posts assume an infinitely rigid post below grade which results in more load being attracted to the foundation (and less being shifted to the eave).

The 1482 lbf eave load is 18.6% greater than the 1249 lbf value obtained via plane-frame structural analysis with soil springs and posts rigidly connected to the truss. This significant difference can be attributed to the fact that the post fixity factors assume post tops do not rotate which is typically only the case when (1) posts are rigidly connected to the truss, ***and*** (2) the truss has an infinite bending stiffness. When modeling with soil springs, indivudal truss elements and their connections were modeled; that is, the truss was not assumed to behave as a beam with infinite bending stiffness.

Solution 3: Calculation Using Post Fixity Factors for Fixed-Based Posts

When both posts are assumed to be fixed at grade, equation 6-5 can be used with post fixity factor f equal to 0.375 when posts are assumed to be pin-connected to the truss (Table 6-5 fixity case 3), and f equal to 0.500 when posts tops are not allowed to rotate (Table 6-5 fixity case 4). For this example problem, these fixity factors produce eave loads of 1065 lbf and 1260 lbf, respectively. These values are 4.1% lower and 0.9% higher, respectively, than the values of 1111 lbf and 1249 lbf obtained via plane-frame structural analysis with soil springs.

In general, use of Table 6-5 post fixity factors for Cases 1 through 4 provides a quick and sufficiently accurate estimate of eave load.

6.6 Load Distribution

6.6.1 General

The distribution of horizontal loads to frames, shearwalls, and various diaphragm sections can be determined after stiffness values have been assigned to each frame and diaphragm element, and eave loads have been established.

Load distributions are typically determined using a plane-frame structural analysis program as described in Section 6.6.2, or by using computer program DAFI as described in Section 6.6.3. Neither of these two approaches places restrictions on frame stiffness values, diaphragm stiffness values, or individual eave load values.

Two other methods, one using *mS* and *mD* Tables as described in Section 6.6.4, and the other involving Simple Beam Analogy Equations as provided in Section 6.6.5 can be used when all five of the following conditions exist: (1) all diaphragm elements have the same stiffness C_h, (2) all interior frame elements have the same stiffness, k, (3) both exterior frame elements (i.e., the two elements representing the endwalls) have the same stiffness, k_e, (4) eave load, R, is the same at each interior frame, and (5) the eave load for each exterior frame is equal to one-half that for an interior frame.

6.6.2 Load Distribution via Plane-Frame Structural Analysis

Virtually any finite element or plane-frame structural analysis program can be used to determine the distribution of load between frame and diaphragm elements. The connectivity between, and behavior of these elements are typically represented with a two-dimensional model as described in Section 6.2.4 and illustrated in figure 6-3(c). The figure 6-3(c) illustration, which has been reproduced as figure 6-17(a), has five nodes – one for each interior and endwall frame. These nodes are only allowed to translate in the direction of applied load.

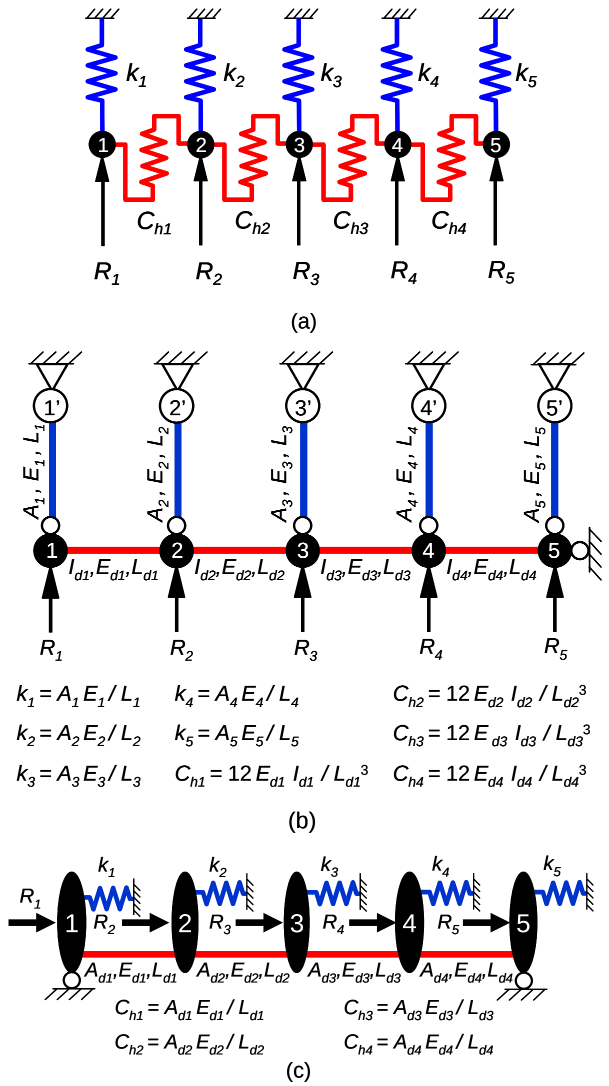


Figure 6-17. (a) Two-dimensional spring model with frame and diaphragm element stiffness values and individual eave loads identified. Black dots represent eave nodes with numbers corresponding to the frame they represent. (b) Alternate 2-D model using beam elements (in red) and truss elements (in blue) to model frame and diaphragm behavior. (c) Alternate 1-D model.

The problem with trying to analyze the figure 6-17(a) model with a plane-frame structural analysis program is the program is unlikely to feature a special spring element that can be used to *completely* model an individual diaphragm (typically a 1-D spring element plus additional nodes and special rigid link elements would be required to model each diaphragm). Two solutions to this problem are to model the system as shown in figures 6-17(b) and 6-17(c).

The model shown in figure 6-17(b) uses *conventional frame elements* to model diaphragms. The modifier "conventional" is used herein to avoid confusion between "frame elements" that are used in this chapter to represent individual post-frames, and the much broader use of "frame elements" as the modeling elements used to represent framing components in finite element analysis programs.

Each of the conventional frame elements in figure 6-17(b) used to model a diaphragm element must be fixed to their respective nodes, **the nodes must also be fixed from rotating** (an often overlooked requirement), one node must be fixed from translating perpendicular to the applied load, and each element must be assigned a modulus of elasticity calculated as:

$$E_d = C_h\, L_d^3/(12\, I_d) \qquad (6\text{-}7)$$

or

$$E_d = C_h\, L_d^3/(b_d\, d_d^3) \qquad (6\text{-}8)$$

where:

E_d = modulus of elasticity assigned to element *x*
C_h = shear stiffness of the diaphragm being modeled with element *x*
L_d = length of element *x*
I_d = moment of inertia of element *x*
b_d = thickness of element *x* when element is rectangular
d_d = depth of element *x* when element is rectangular

Also shown in figure 6-17(b) is the use of conventional frame elements (in place of spring elements) to model the behavior of interior and endwall frames. This requires an additional set of nodes (identified as 1',2', 3', 4' and 5' in figure 6-17(b)). Complete the model by pin-connecting these elements to their respective nodes, and then assign each element a modulus of elasticity calculated as:

$$E = k\, L/A \qquad (6\text{-}9)$$

where:

E = modulus of elasticity assigned to element *x*
k = frame stiffness of the post-frame being modeled with element *x*
L = length of element *x*
A = cross-sectional area of element *x*

When compiling input values for computer analysis, it is important to maintain a consistent set of units in equations 6-7, 6-8 or 6-9. In practice, it is recommended that nodes be spaced one inch apart (which sets the length of each element at one inch), and that the width and depth of all elements be fixed at one inch. When this is done, the value of E_d in lbf/in^2 is numerically equal to the value of C_h in lbf/in. Likewise, the value of E in lbf/in^2 will be numerically equal to the value of k in lbf/in.

Output values of interest from the analysis of the figure 6-17(b) model are: (1) shear forces in the fixed-end elements which equal the shear forces resisted by the diaphragms, (2) axial forces in the pinned-connected elements which equal the forces transferred to the ground by interior and endwall frames, and (3) nodal displacements which equal eave-line displacements of the frames.

Even through it may be less intuitive, the one-dimensional model shown in figure 6-17(c) is typically easier to implement than the figure 6-17(b) model. All nodes, elements and applied loads associated with the figure 6-17(c) model are colinear. To precisely illustrate this would result in an indecipherable drawing. For this reason, the nodes in figure 6-17(c) have been stretched to avoid drawing loads on top of springs on top of elements.

Simple springs are used in the figure 6-17(c) to model interior and endwall frame behavior, and conventional modeling elements are used to model diaphragm behavior. With respect to the later, the modulus of elasticity that should be assigned to each element is given as:

$$E_d = C_h L_d / A_d \qquad (6\text{-}10)$$

where:

E_d = modulus of elasticity assigned to element x

C_h = shear stiffness of the diaphragm being modeled with element x

L_d = length of element x

A_d = cross-sectional area of element x

When analyzing the one-dimensional model in figure 6-17(c), all elements should be fixed to their respective nodes, and at least two of the nodes should be fixed from translating in a direction perpendicular to the applied load (if not, the structure will be unstable). Unlike with the figure 6-17(b) model, the nodes in figure 6-17(c) need not be fixed from rotating. The axial forces induced in the elements of the one-dimensional model will equal the shear forces in the diaphragm being represented by the elements.

6.6.3 Load Distribution via DAFI

To avoid using a plane-frame structural analysis program to determine load distribution due to diaphragm action, Bohnhoff (1992a) developed computer program DAFI (Diaphragm And Frame Interaction). A Microsoft windows-based version of DAFI developed by Ben Bohnhoff can be downloaded at no cost from the NFBA web site: (http://www.nfba.org/Resources/content/dafi.html).

An example use of DAFI is shown in figure 6-18. When DAFI is first opened, the **Default Values** window shown in figure 6-18(a) appears. This window enables users to input a project name, project filename (name under which input data will be saved for later retrieval), number of building bays, and default values for: endwall frame stiffness, interior frame stiffness, diaphragm shear stiffness, and eave load applied to an interior frame. Once default values have been entered, they can be saved by clicking on the button at the bottom of the window. Note that default values from previous examples in this chapter have been used in this demonstration of DAFI.

Clicking on the **Specific Values** tab brings up the window shown in figure 6-18(b). Using the previously input default values, DAFI pre-populates the two tables that comprise this window. Note that the default eave load on each endwall frame is set equal to one-half the default interior frame eave load.

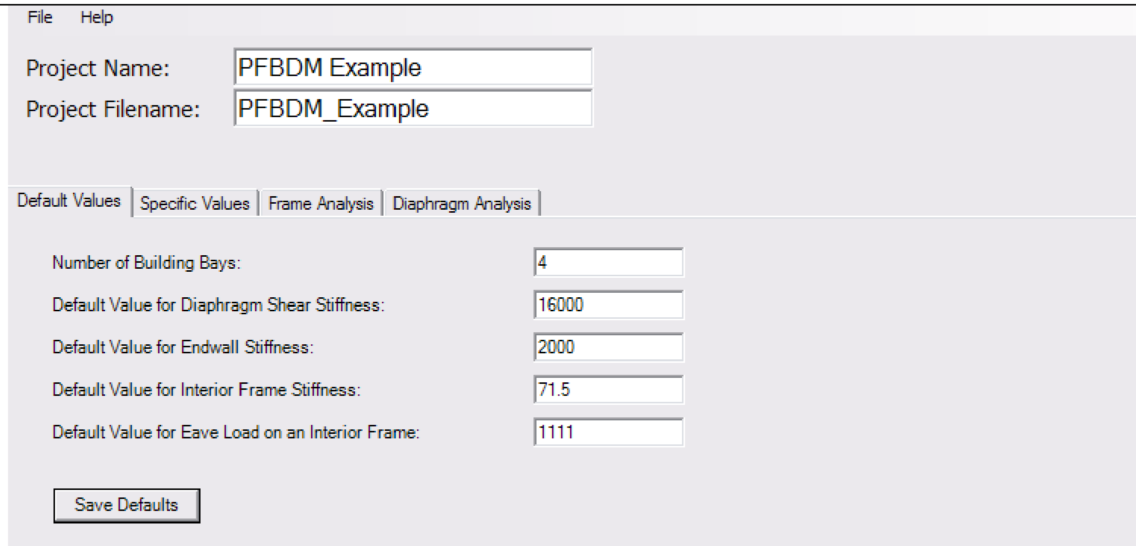
The **Specific Values** window can be used to change any number of stiffness and/or load values from their default values. Simply click on the cell containing the value to be changed and enter a new value.

Analysis results can be obtained by clicking on the **Frame Analysis** tab which brings up the window shown in figure 6-18(c), or the **Diaphragm Analysis** tab which brings up the window shown in figure 6-18(d). Data in either table can be resorted by clicking on the heading of one of the columns. This will sort the data in that column from high-to-low (or with a subsequent click, from low-to-high). This feature enables a user to quickly locate the maximum horizontal displacement, diaphragm shear force, etc.

Shear loads appearing in the DAFI output are horizontal compoents of the in-plane shear force to which the daiphragm is subjected. It is also important to note that the shear load listed for each diaphragm in the DAFI output is essentially an average shear load in the diaphragm. For example, the average shear load listed in figure 6-18(d) for diaphragm 1 is 1542.5 lbf . To calculate the maximum shear load in each diaphragm element, simply add the quantity $R/2$ to the average value. For this example analysis, half the eave load is 555.5 lbf. Adding this to the average shear load in diaphragm 1 yields a maximum shear force in diaphragm 1 of 2098 lbf. As should be expected, the maximum shear force in the end diaphragm is equal to the total eave load resisted by the endwall frame.

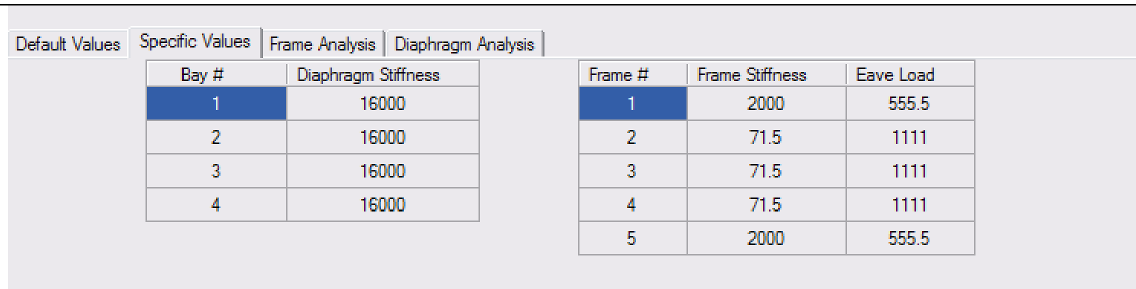
(a)

Default Values window



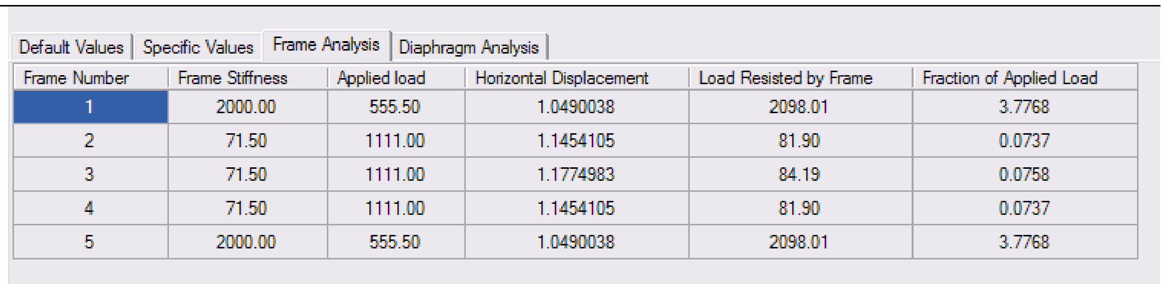
(b)

Specific Values window



(c)

Frame Analysis window



(d)

Diaphragm Analysis window

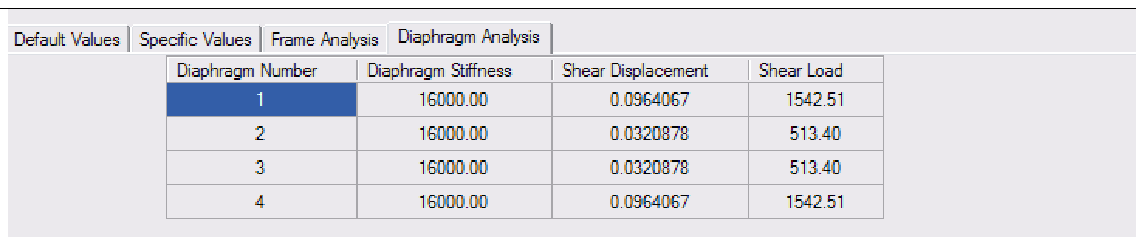


Figure 6-18. Screen captures of the four DAFI windows.

6.6.3.1 DAFI Inner Workings

DAFI sets up and solves the equations that relate eave loads to eave displacements. These equations, which are obtained by summing the forces applied to the eave nodes, can be written as follows (Bohnhoff, 1992):

For the first endwall frame (i = 1):

$$R_1 = \Delta_1 (k_1 + C_{h1}) - \Delta_2 C_{h1} \tag{6-11a}$$

For i = 2 to n – 1:

$$R_i = \Delta_i (k_i + C_{h,i-1} + C_{h,i}) - \Delta_{i+1} C_{h,i} - \Delta_{i-1} C_{h,i-1} \tag{6-11b}$$

For the last frame (i = n):

$$R_n = \Delta_n (k_n + C_{h,n-1}) - \Delta_{n-1} C_{h,n-1} \tag{6-11c}$$

Where:

- i = eave node at which forces are being summed. Eave node i is on frame i
- n = total number of frames both endwall and interior
- Δ_i = displacement of eave node i
- R_i = eave load applied to frame i
- k_i = stiffness of frame i
- $C_{h,i}$ = shear stiffness of diaphragm element i. Diaphragm element i is located between nodes i and $i+1$.

Once structural equations are established, DAFI simultaneously solves them to obtain the unknown eave displacements. The following equations are then used to calculate shear forces in diaphragm elements, and loads resisted by individual interior and exterior frame elements.

$$V_{hi} = C_{hi} (\Delta_{i+1} - \Delta_i) \tag{6-12}$$

$$F_i = \Delta_i k_i \tag{6-13}$$

where:

- V_{hi} = shear transferred by diaphragm element i
- F_i = eave load resisted by frame element i

It is worth noting that the purpose of the models in figures 6-17(b) and 6-17(c) is to essentially trick a plane-frame structural analysis program into developing equations 6-11(a), 6-11(b) and 6-11(c).

6.6.4 *mS* and *mD* Tables

Forces in the most highly loaded diaphragm and frame elements, can be calculated using Tables 6-6 and 6-7 when all five of the following conditions exist: (1) all diaphragm elements have the same stiffness C_h, (2) all interior frame elements have the same stiffness, k, (3) both exterior frame elements (i.e., the two elements representing the endwalls) have the same stiffness, k_e, (4) eave load, R, is the same at each interior frame, and (5) the eave load for each exterior frame is equal to one-half that for an interior frame. These five requirements are generally met in buildings with a fixed bay spacing, endwalls that are virtually identical in construction, and interior frames that don't vary in overall design. When Tables 6-6 and 6-7 are applicable, the analysis tools discussed in Sections 6.6.2 and 6.6.3 are generally not needed.

Input parameters required for Tables 6-6 and 6-7 include: number of frame elements (i.e., the number of interior frames + 2); ratio of diaphragm element to interior frame element stiffness, C_h / k; and ratio of exterior to interior frame element stiffness, k_e / k.

The most highly loaded diaphragm element (in any building that meets the preceding five conditions) is the element located adjacent to the endwalls. The maximum shear force in this diaphragm element, V_h, is equal to the appropriate shear modifier value, mS, from Table 6-6, multiplied by the eave load, R, for an interior frame. In equation form:

$$V_{h,max} = R\ mS \tag{6-14}$$

where:

- $V_{h,max}$ = maximum horizontal shear force in a diaphragm element, lbf (N)
- mS = shear force modifier from Table 6-6
- R = eave load at interior frame, lbf (N)

The value obtained from equation 6-14 is simply equal to one-half of the total horizontal eave load that is not carried by the interior frames.

The most highly loaded *interior* frame element (in any building that meets the preceding five conditions) is the element located closest to the building midlength. Because of diaphragm action, the total horizontal load that this critical frame must resist is reduced from that which it would resist without diaphragm action. The magnitude of this reduction is referred to the *sidesway restraining force* because in reality, it is a restraining force applied to the frame by the roof (and/or ceiling) diaphragms. Numerically, the sidesway restraining force for the critical frame, Q_c, is equal to the product of the eave load R, and the appropriate sidesway restraining force factor, mD from Table 6-7. In equation form:

$$Q_c = R\ mD \tag{6-15}$$

where:

- Q_c = sidesway restraining force for the critical frame, lbf (N)
- mD = sidesway restraining force factor from Table 6-7
- R = eave load at interior frame, lbf (N)

Table 6-6. Shear Force Modifier (mS)

k_e / k	C_h / k	Number of frames (endwalls are counted as frames)													
		3	4	5	6	7	8	9	10	11	12	13	14	15	16
5	5	0.88	1.14	1.33	1.45	1.53	1.59	1.62	1.65	1.66	1.67	1.68	1.68	1.68	1.68
5	10	0.89	1.19	1.42	1.59	1.72	1.82	1.89	1.94	1.98	2.00	2.02	2.04	2.05	2.06
5	20	0.90	1.22	1.48	1.68	1.85	1.98	2.08	2.16	2.23	2.29	2.33	2.36	2.39	2.41
5	50	0.91	1.24	1.51	1.74	1.93	2.10	2.23	2.35	2.45	2.53	2.60	2.67	2.72	2.77
5	100	0.91	1.24	1.53	1.76	1.97	2.14	2.29	2.42	2.53	2.63	2.72	2.80	2.87	2.93
5	200	0.91	1.25	1.53	1.77	1.98	2.16	2.32	2.46	2.58	2.69	2.79	2.87	2.95	3.02
5	500	0.91	1.25	1.54	1.78	1.99	2.18	2.34	2.48	2.61	2.73	2.83	2.92	3.01	3.08
5	1000	0.91	1.25	1.54	1.78	2.00	2.18	2.35	2.49	2.62	2.74	2.84	2.94	3.02	3.10
5	10000	0.91	1.25	1.54	1.79	2.00	2.19	2.35	2.50	2.63	2.75	2.86	2.95	3.04	3.12
10	5	0.91	1.23	1.46	1.62	1.73	1.81	1.86	1.89	1.91	1.92	1.93	1.93	1.94	1.94
10	10	0.93	1.29	1.58	1.81	1.99	2.13	2.23	2.31	2.36	2.40	2.44	2.46	2.48	2.49
10	20	0.94	1.33	1.66	1.94	2.17	2.36	2.52	2.66	2.76	2.85	2.92	2.98	3.03	3.06
10	50	0.95	1.35	1.70	2.02	2.30	2.55	2.76	2.96	3.12	3.27	3.40	3.51	3.61	3.70
10	100	0.95	1.36	1.72	2.05	2.35	2.62	2.86	3.08	3.27	3.45	3.61	3.76	3.89	4.01
10	200	0.95	1.36	1.73	2.07	2.37	2.65	2.91	3.14	3.36	3.56	3.74	3.90	4.06	4.20
10	500	0.95	1.36	1.74	2.08	2.39	2.68	2.94	3.19	3.41	3.62	3.82	4.00	4.17	4.32
10	1000	0.95	1.36	1.74	2.08	2.40	2.68	2.95	3.20	3.43	3.64	3.84	4.03	4.20	4.37
10	10000	0.95	1.36	1.74	2.08	2.40	2.69	2.96	3.21	3.45	3.66	3.87	4.06	4.24	4.41
20	5	0.93	1.28	1.54	1.73	1.85	1.94	2.00	2.03	2.06	2.07	2.09	2.09	2.10	2.10
20	10	0.95	1.35	1.68	1.95	2.16	2.33	2.45	2.55	2.62	2.67	2.71	2.74	2.76	2.78
20	20	0.96	1.39	1.76	2.09	2.38	2.62	2.83	3.00	3.14	3.25	3.35	3.43	3.49	3.54
20	50	0.97	1.41	1.82	2.20	2.54	2.85	3.14	3.39	3.62	3.83	4.01	4.17	4.32	4.44
20	100	0.97	1.42	1.84	2.23	2.60	2.95	3.26	3.56	3.83	4.09	4.32	4.54	4.74	4.92
20	200	0.97	1.42	1.85	2.25	2.63	2.99	3.33	3.65	3.95	4.24	4.50	4.75	4.99	5.21
20	500	0.98	1.43	1.86	2.27	2.65	3.02	3.38	3.71	4.03	4.33	4.62	4.90	5.16	5.41
20	1000	0.98	1.43	1.86	2.27	2.66	3.03	3.39	3.73	4.06	4.37	4.66	4.95	5.22	5.48
20	10000	0.98	1.43	1.86	2.27	2.67	3.04	3.40	3.75	4.08	4.40	4.70	5.00	5.28	5.55
50	5	0.95	1.31	1.59	1.79	1.93	2.03	2.09	2.14	2.16	2.18	2.19	2.20	2.20	2.21
50	10	0.97	1.38	1.74	2.04	2.28	2.46	2.61	2.72	2.80	2.86	2.91	2.94	2.97	2.99
50	20	0.98	1.43	1.83	2.20	2.52	2.80	3.04	3.25	3.41	3.55	3.67	3.77	3.84	3.91
50	50	0.99	1.45	1.90	2.32	2.71	3.08	3.42	3.73	4.01	4.26	4.50	4.70	4.89	5.06
50	100	0.99	1.46	1.92	2.36	2.78	3.18	3.57	3.93	4.27	4.60	4.90	5.18	5.45	5.69
50	200	0.99	1.47	1.93	2.38	2.82	3.24	3.65	4.04	4.42	4.79	5.14	5.47	5.79	6.09
50	500	0.99	1.47	1.94	2.40	2.84	3.28	3.70	4.12	4.52	4.91	5.29	5.66	6.02	6.37
50	1000	0.99	1.47	1.94	2.40	2.85	3.29	3.72	4.14	4.55	4.96	5.35	5.73	6.11	6.47
50	10000	0.99	1.47	1.94	2.40	2.86	3.30	3.74	4.16	4.58	5.00	5.40	5.80	6.19	6.57
100	5	0.95	1.32	1.61	1.82	1.96	2.06	2.13	2.17	2.20	2.22	2.23	2.24	2.24	2.25
100	10	0.97	1.40	1.76	2.07	2.32	2.51	2.67	2.78	2.87	2.93	2.98	3.02	3.05	3.06
100	20	0.98	1.44	1.86	2.24	2.58	2.87	3.12	3.34	3.52	3.67	3.79	3.89	3.98	4.05
100	50	0.99	1.47	1.92	2.36	2.77	3.16	3.52	3.85	4.16	4.43	4.69	4.91	5.12	5.30
100	100	0.99	1.48	1.95	2.40	2.85	3.27	3.68	4.07	4.44	4.79	5.13	5.44	5.73	6.01
100	200	0.99	1.48	1.96	2.43	2.89	3.33	3.77	4.19	4.61	5.00	5.39	5.76	6.12	6.46
100	500	1.00	1.48	1.97	2.44	2.91	3.37	3.83	4.27	4.71	5.14	5.56	5.98	6.38	6.78
100	1000	1.00	1.48	1.97	2.45	2.92	3.39	3.85	4.30	4.75	5.19	5.62	6.05	6.48	6.89
100	10000	1.00	1.49	1.97	2.45	2.93	3.40	3.86	4.32	4.78	5.23	5.68	6.12	6.56	7.00
1000	5	0.95	1.33	1.63	1.84	1.99	2.09	2.16	2.20	2.23	2.25	2.27	2.27	2.28	2.28
1000	10	0.98	1.41	1.78	2.10	2.36	2.56	2.72	2.84	2.93	3.00	3.05	3.09	3.12	3.14
1000	20	0.99	1.45	1.88	2.28	2.63	2.93	3.20	3.43	3.62	3.78	3.91	4.02	4.11	4.18
1000	50	1.00	1.48	1.95	2.40	2.83	3.24	3.62	3.97	4.30	4.60	4.87	5.12	5.34	5.54
1000	100	1.00	1.49	1.97	2.45	2.91	3.36	3.79	4.21	4.61	4.99	5.35	5.69	6.02	6.32
1000	200	1.00	1.49	1.99	2.47	2.95	3.42	3.89	4.34	4.78	5.22	5.64	6.05	6.44	6.83
1000	500	1.00	1.50	1.99	2.49	2.98	3.46	3.95	4.42	4.90	5.37	5.83	6.29	6.74	7.18
1000	1000	1.00	1.50	2.00	2.49	2.98	3.48	3.97	4.45	4.94	5.42	5.90	6.37	6.85	7.31
1000	10000	1.00	1.50	2.00	2.50	2.99	3.49	3.98	4.48	4.97	5.47	5.96	6.45	6.94	7.43
10000	5	0.96	1.33	1.63	1.84	1.99	2.09	2.16	2.21	2.24	2.26	2.27	2.28	2.28	2.29
10000	10	0.98	1.41	1.79	2.10	2.36	2.57	2.72	2.85	2.94	3.01	3.06	3.10	3.12	3.14
10000	20	0.99	1.45	1.89	2.28	2.63	2.94	3.21	3.43	3.63	3.79	3.92	4.03	4.12	4.19
10000	50	1.00	1.48	1.95	2.40	2.84	3.25	3.63	3.98	4.31	4.61	4.89	5.14	5.36	5.57
10000	100	1.00	1.49	1.98	2.45	2.92	3.37	3.80	4.22	4.62	5.01	5.37	5.72	6.05	6.35
10000	200	1.00	1.50	1.99	2.48	2.96	3.43	3.90	4.35	4.80	5.24	5.66	6.08	6.48	6.87
10000	500	1.00	1.50	2.00	2.49	2.98	3.47	3.96	4.44	4.92	5.39	5.86	6.32	6.78	7.23
10000	1000	1.00	1.50	2.00	2.50	2.99	3.49	3.98	4.47	4.96	5.44	5.93	6.41	6.88	7.36
10000	10000	1.00	1.50	2.00	2.50	3.00	3.50	4.00	4.50	4.99	5.49	5.99	6.49	6.98	7.48

Table 6-6. Shear Force Modifier (mS), cont.

k_e / k	C_h / k	Number of frames (endwalls are counted as frames)													
		17	18	19	20	21	22	23	24	25	26	27	28	29	30
5	5	1.69	1.69	1.69	1.69	1.69	1.69	1.69	1.69	1.69	1.69	1.69	1.69	1.69	1.69
5	10	2.06	2.07	2.07	2.07	2.07	2.08	2.08	2.08	2.08	2.08	2.08	2.08	2.08	2.08
5	20	2.43	2.44	2.46	2.46	2.47	2.48	2.48	2.49	2.49	2.49	2.49	2.49	2.50	2.50
5	50	2.81	2.84	2.87	2.89	2.92	2.94	2.95	2.97	2.98	2.99	3.00	3.01	3.01	3.02
5	100	2.98	3.03	3.07	3.11	3.14	3.18	3.20	3.23	3.25	3.27	3.29	3.30	3.32	3.33
5	200	3.09	3.14	3.19	3.24	3.28	3.32	3.36	3.39	3.42	3.45	3.48	3.50	3.52	3.54
5	500	3.15	3.22	3.28	3.33	3.38	3.43	3.47	3.51	3.55	3.58	3.61	3.64	3.67	3.70
5	1000	3.18	3.24	3.30	3.36	3.41	3.46	3.51	3.55	3.59	3.63	3.66	3.70	3.73	3.75
5	10000	3.20	3.27	3.33	3.39	3.45	3.50	3.54	3.59	3.63	3.67	3.71	3.74	3.78	3.81
10	5	1.94	1.94	1.94	1.94	1.94	1.94	1.94	1.94	1.94	1.94	1.94	1.94	1.94	1.94
10	10	2.50	2.50	2.51	2.51	2.51	2.52	2.52	2.52	2.52	2.52	2.52	2.52	2.52	2.52
10	20	3.09	3.12	3.14	3.15	3.16	3.17	3.18	3.19	3.19	3.20	3.20	3.20	3.21	3.21
10	50	3.77	3.84	3.89	3.94	3.99	4.02	4.06	4.09	4.11	4.13	4.15	4.17	4.18	4.19
10	100	4.12	4.21	4.30	4.38	4.45	4.52	4.58	4.63	4.68	4.72	4.76	4.80	4.83	4.86
10	200	4.33	4.45	4.56	4.66	4.76	4.84	4.92	5.00	5.07	5.13	5.19	5.25	5.30	5.35
10	500	4.47	4.61	4.74	4.86	4.97	5.08	5.18	5.27	5.36	5.44	5.52	5.60	5.67	5.73
10	1000	4.52	4.66	4.80	4.93	5.05	5.16	5.27	5.37	5.47	5.56	5.65	5.73	5.81	5.88
10	10000	4.57	4.72	4.86	4.99	5.12	5.24	5.36	5.47	5.57	5.67	5.76	5.86	5.94	6.03
20	5	2.10	2.10	2.10	2.10	2.10	2.10	2.10	2.10	2.10	2.10	2.10	2.10	2.10	2.10
20	10	2.79	2.80	2.80	2.81	2.81	2.81	2.82	2.82	2.82	2.82	2.82	2.82	2.82	2.82
20	20	3.58	3.62	3.64	3.66	3.68	3.69	3.71	3.71	3.72	3.73	3.73	3.74	3.74	3.74
20	50	4.56	4.65	4.74	4.82	4.88	4.94	4.99	5.03	5.07	5.11	5.14	5.16	5.18	5.20
20	100	5.08	5.24	5.38	5.51	5.62	5.73	5.83	5.91	5.99	6.07	6.13	6.20	6.25	6.30
20	200	5.42	5.61	5.80	5.97	6.13	6.28	6.42	6.55	6.67	6.79	6.90	7.00	7.09	7.18
20	500	5.65	5.88	6.09	6.30	6.50	6.69	6.87	7.04	7.20	7.36	7.51	7.65	7.78	7.91
20	1000	5.73	5.97	6.20	6.42	6.64	6.84	7.03	7.22	7.40	7.58	7.74	7.90	8.06	8.21
20	10000	5.81	6.06	6.30	6.54	6.77	6.98	7.20	7.40	7.60	7.79	7.97	8.15	8.33	8.50
50	5	2.21	2.21	2.21	2.21	2.21	2.21	2.21	2.21	2.21	2.21	2.21	2.21	2.21	2.21
50	10	3.00	3.01	3.02	3.02	3.03	3.03	3.03	3.03	3.03	3.04	3.04	3.04	3.04	3.04
50	20	3.96	4.00	4.03	4.06	4.08	4.10	4.11	4.12	4.13	4.14	4.14	4.15	4.15	4.16
50	50	5.20	5.33	5.45	5.55	5.64	5.72	5.79	5.85	5.90	5.95	5.99	6.03	6.06	6.08
50	100	5.92	6.13	6.33	6.51	6.67	6.83	6.97	7.10	7.21	7.32	7.42	7.51	7.59	7.67
50	200	6.39	6.66	6.93	7.18	7.41	7.64	7.85	8.05	8.24	8.42	8.59	8.75	8.90	9.04
50	500	6.71	7.04	7.36	7.67	7.97	8.26	8.54	8.81	9.07	9.32	9.57	9.80	10.03	10.25
50	1000	6.83	7.18	7.52	7.85	8.18	8.50	8.80	9.10	9.40	9.68	9.96	10.23	10.50	10.75
50	10000	6.94	7.31	7.68	8.03	8.38	8.72	9.06	9.39	9.72	10.04	10.35	10.66	10.97	11.27
100	5	2.25	2.25	2.25	2.25	2.25	2.25	2.25	2.25	2.25	2.25	2.25	2.25	2.25	2.25
100	10	3.08	3.09	3.10	3.10	3.11	3.11	3.11	3.11	3.11	3.12	3.12	3.12	3.12	3.12
100	20	4.10	4.14	4.18	4.21	4.23	4.25	4.27	4.28	4.29	4.30	4.30	4.31	4.31	4.31
100	50	5.46	5.61	5.74	5.85	5.95	6.04	6.12	6.19	6.24	6.30	6.34	6.38	6.42	6.45
100	100	6.26	6.50	6.72	6.93	7.12	7.29	7.45	7.60	7.74	7.86	7.98	8.08	8.18	8.27
100	200	6.79	7.10	7.41	7.69	7.97	8.23	8.48	8.72	8.94	9.15	9.35	9.54	9.72	9.89
100	500	7.16	7.54	7.91	8.27	8.62	8.96	9.29	9.62	9.93	10.24	10.53	10.82	11.10	11.37
100	1000	7.30	7.70	8.10	8.49	8.87	9.24	9.61	9.97	10.33	10.67	11.01	11.35	11.68	12.00
100	10000	7.43	7.85	8.28	8.69	9.11	9.51	9.92	10.32	10.72	11.11	11.50	11.88	12.27	12.64
1000	5	2.28	2.29	2.29	2.29	2.29	2.29	2.29	2.29	2.29	2.29	2.29	2.29	2.29	2.29
1000	10	3.15	3.16	3.17	3.18	3.18	3.18	3.19	3.19	3.19	3.19	3.19	3.19	3.19	3.19
1000	20	4.24	4.29	4.32	4.36	4.38	4.40	4.42	4.43	4.44	4.45	4.46	4.46	4.47	4.47
1000	50	5.72	5.88	6.02	6.15	6.26	6.36	6.44	6.52	6.59	6.65	6.70	6.74	6.78	6.81
1000	100	6.61	6.87	7.12	7.35	7.57	7.77	7.95	8.12	8.28	8.43	8.56	8.68	8.79	8.89
1000	200	7.20	7.56	7.90	8.23	8.55	8.85	9.14	9.41	9.68	9.93	10.17	10.39	10.61	10.81
1000	500	7.62	8.05	8.48	8.89	9.30	9.70	10.10	10.48	10.86	11.22	11.58	11.93	12.27	12.61
1000	1000	7.78	8.24	8.69	9.15	9.59	10.04	10.47	10.91	11.33	11.75	12.17	12.58	12.99	13.39
1000	10000	7.92	8.41	8.90	9.39	9.87	10.36	10.84	11.33	11.81	12.29	12.77	13.25	13.73	14.20
10000	5	2.29	2.29	2.29	2.29	2.29	2.29	2.29	2.29	2.29	2.29	2.29	2.29	2.29	2.29
10000	10	3.16	3.17	3.18	3.19	3.19	3.19	3.19	3.20	3.20	3.20	3.20	3.20	3.20	3.20
10000	20	4.25	4.30	4.34	4.37	4.40	4.42	4.43	4.45	4.46	4.46	4.47	4.48	4.48	4.48
10000	50	5.75	5.91	6.05	6.18	6.29	6.39	6.48	6.56	6.62	6.68	6.73	6.78	6.82	6.85
10000	100	6.64	6.91	7.17	7.40	7.62	7.82	8.01	8.18	8.34	8.49	8.62	8.74	8.86	8.96
10000	200	7.24	7.60	7.95	8.29	8.61	8.92	9.21	9.49	9.76	10.01	10.26	10.49	10.71	10.91
10000	500	7.67	8.11	8.54	8.96	9.38	9.78	10.18	10.57	10.96	11.33	11.70	12.06	12.41	12.75
10000	1000	7.83	8.30	8.76	9.22	9.67	10.12	10.57	11.01	11.44	11.88	12.30	12.72	13.14	13.55
10000	10000	7.98	8.47	8.97	9.46	9.96	10.45	10.94	11.44	11.93	12.42	12.91	13.40	13.89	14.38

Table 6-7. Sidesway Restraining Force Modifier (mD)

k_e / k	C_h / k	Number of frames (endwalls counted as frames)													
		3	4	5	6	7	8	9	10	11	12	13	14	15	16
5	5	0.75	0.64	0.52	0.43	0.34	0.28	0.22	0.18	0.14	0.12	0.09	0.08	0.06	0.05
5	10	0.78	0.69	0.59	0.52	0.44	0.39	0.33	0.28	0.24	0.21	0.18	0.15	0.13	0.11
5	20	0.80	0.72	0.64	0.58	0.51	0.46	0.41	0.37	0.33	0.30	0.26	0.24	0.21	0.19
5	50	0.81	0.74	0.67	0.62	0.56	0.52	0.48	0.44	0.41	0.38	0.35	0.32	0.30	0.28
5	100	0.81	0.74	0.68	0.63	0.58	0.54	0.50	0.47	0.44	0.41	0.38	0.36	0.34	0.32
5	200	0.82	0.75	0.69	0.64	0.59	0.55	0.52	0.48	0.46	0.43	0.41	0.38	0.36	0.35
5	500	0.82	0.75	0.69	0.64	0.60	0.56	0.52	0.49	0.47	0.44	0.42	0.40	0.38	0.36
5	1000	0.82	0.75	0.69	0.64	0.60	0.56	0.53	0.50	0.47	0.45	0.42	0.40	0.39	0.37
5	10000	0.82	0.75	0.69	0.64	0.60	0.56	0.53	0.50	0.47	0.45	0.43	0.41	0.39	0.37
10	5	0.83	0.73	0.60	0.51	0.41	0.34	0.27	0.22	0.17	0.14	0.11	0.09	0.07	0.06
10	10	0.86	0.79	0.70	0.63	0.54	0.48	0.41	0.36	0.30	0.26	0.22	0.19	0.16	0.14
10	20	0.88	0.83	0.76	0.70	0.64	0.58	0.52	0.48	0.43	0.39	0.35	0.31	0.28	0.25
10	50	0.90	0.85	0.80	0.75	0.71	0.66	0.62	0.58	0.55	0.51	0.48	0.45	0.42	0.39
10	100	0.90	0.86	0.81	0.77	0.73	0.70	0.66	0.63	0.60	0.57	0.54	0.51	0.49	0.46
10	200	0.90	0.86	0.82	0.78	0.75	0.71	0.68	0.65	0.63	0.60	0.57	0.55	0.53	0.51
10	500	0.90	0.86	0.82	0.79	0.75	0.72	0.70	0.67	0.64	0.62	0.60	0.58	0.56	0.54
10	1000	0.90	0.86	0.83	0.79	0.76	0.73	0.70	0.67	0.65	0.63	0.61	0.59	0.57	0.55
10	10000	0.91	0.86	0.83	0.79	0.76	0.73	0.70	0.68	0.66	0.63	0.61	0.59	0.58	0.56
20	5	0.87	0.78	0.65	0.56	0.45	0.38	0.30	0.25	0.19	0.16	0.13	0.10	0.08	0.07
20	10	0.91	0.85	0.76	0.69	0.60	0.54	0.46	0.41	0.35	0.30	0.26	0.22	0.19	0.16
20	20	0.93	0.89	0.83	0.78	0.72	0.66	0.60	0.55	0.50	0.46	0.41	0.37	0.33	0.30
20	50	0.94	0.91	0.87	0.84	0.80	0.76	0.72	0.69	0.65	0.62	0.58	0.55	0.51	0.48
20	100	0.95	0.92	0.89	0.86	0.83	0.80	0.77	0.75	0.72	0.69	0.66	0.64	0.61	0.58
20	200	0.95	0.92	0.90	0.87	0.85	0.83	0.80	0.78	0.76	0.73	0.71	0.69	0.67	0.65
20	500	0.95	0.93	0.90	0.88	0.86	0.84	0.82	0.80	0.78	0.76	0.74	0.72	0.71	0.69
20	1000	0.95	0.93	0.91	0.88	0.86	0.84	0.82	0.81	0.79	0.77	0.75	0.74	0.72	0.71
20	10000	0.95	0.93	0.91	0.89	0.87	0.85	0.83	0.81	0.80	0.78	0.76	0.75	0.73	0.72
50	5	0.89	0.81	0.68	0.59	0.48	0.40	0.32	0.26	0.21	0.17	0.13	0.11	0.09	0.07
50	10	0.93	0.88	0.80	0.73	0.65	0.58	0.50	0.44	0.38	0.33	0.28	0.24	0.21	0.18
50	20	0.96	0.93	0.88	0.83	0.77	0.72	0.66	0.61	0.55	0.51	0.46	0.41	0.37	0.34
50	50	0.97	0.95	0.93	0.90	0.87	0.84	0.80	0.77	0.73	0.70	0.66	0.63	0.59	0.56
50	100	0.98	0.96	0.94	0.93	0.90	0.88	0.86	0.84	0.81	0.79	0.76	0.74	0.71	0.69
50	200	0.98	0.97	0.95	0.94	0.92	0.91	0.89	0.88	0.86	0.84	0.82	0.81	0.79	0.77
50	500	0.98	0.97	0.96	0.95	0.94	0.92	0.91	0.90	0.89	0.88	0.86	0.85	0.84	0.83
50	1000	0.98	0.97	0.96	0.95	0.94	0.93	0.92	0.91	0.90	0.89	0.88	0.87	0.86	0.85
50	10000	0.98	0.97	0.96	0.95	0.94	0.93	0.93	0.92	0.91	0.90	0.89	0.88	0.87	0.87
100	5	0.90	0.82	0.69	0.60	0.48	0.41	0.32	0.27	0.21	0.17	0.14	0.11	0.09	0.07
100	10	0.94	0.90	0.82	0.75	0.66	0.59	0.51	0.45	0.39	0.34	0.29	0.25	0.21	0.18
100	20	0.97	0.94	0.89	0.85	0.79	0.74	0.68	0.63	0.57	0.52	0.47	0.43	0.39	0.35
100	50	0.98	0.97	0.94	0.92	0.89	0.86	0.83	0.80	0.76	0.73	0.69	0.66	0.62	0.59
100	100	0.99	0.98	0.96	0.95	0.93	0.91	0.89	0.87	0.85	0.83	0.80	0.78	0.75	0.73
100	200	0.99	0.98	0.97	0.96	0.95	0.94	0.93	0.91	0.90	0.88	0.87	0.85	0.84	0.82
100	500	0.99	0.98	0.98	0.97	0.96	0.96	0.95	0.94	0.93	0.92	0.91	0.90	0.89	0.88
100	1000	0.99	0.98	0.98	0.97	0.97	0.96	0.95	0.95	0.94	0.93	0.93	0.92	0.91	0.91
100	10000	0.99	0.99	0.98	0.98	0.97	0.97	0.96	0.96	0.95	0.95	0.94	0.94	0.93	0.93
1000	5	0.91	0.83	0.70	0.61	0.49	0.41	0.33	0.27	0.22	0.18	0.14	0.11	0.09	0.07
1000	10	0.95	0.91	0.83	0.76	0.67	0.60	0.52	0.46	0.40	0.35	0.30	0.26	0.22	0.19
1000	20	0.98	0.95	0.91	0.87	0.81	0.76	0.70	0.65	0.59	0.54	0.49	0.45	0.40	0.36
1000	50	0.99	0.98	0.96	0.94	0.91	0.89	0.86	0.83	0.79	0.76	0.72	0.69	0.65	0.62
1000	100	0.99	0.99	0.98	0.97	0.95	0.94	0.92	0.90	0.88	0.86	0.84	0.82	0.79	0.77
1000	200	1.00	0.99	0.99	0.98	0.98	0.97	0.96	0.95	0.94	0.93	0.91	0.90	0.88	0.87
1000	500	1.00	1.00	0.99	0.99	0.99	0.99	0.98	0.98	0.97	0.97	0.96	0.95	0.95	0.94
1000	1000	1.00	1.00	1.00	1.00	0.99	0.99	0.99	0.99	0.98	0.98	0.98	0.97	0.97	0.97
1000	10000	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	0.99	0.99	0.99	0.99	0.99
10000	5	0.91	0.83	0.70	0.61	0.49	0.42	0.33	0.27	0.22	0.18	0.14	0.11	0.09	0.07
10000	10	0.95	0.91	0.83	0.76	0.68	0.61	0.53	0.46	0.40	0.35	0.30	0.26	0.22	0.19
10000	20	0.98	0.95	0.91	0.87	0.81	0.76	0.70	0.65	0.59	0.54	0.49	0.45	0.40	0.37
10000	50	0.99	0.98	0.96	0.94	0.92	0.89	0.86	0.83	0.79	0.76	0.72	0.69	0.65	0.62
10000	100	1.00	0.99	0.98	0.97	0.96	0.94	0.93	0.91	0.89	0.87	0.84	0.82	0.80	0.77
10000	200	1.00	1.00	0.99	0.99	0.98	0.97	0.96	0.95	0.94	0.93	0.92	0.90	0.89	0.87
10000	500	1.00	1.00	1.00	0.99	0.99	0.99	0.98	0.98	0.98	0.97	0.96	0.96	0.95	0.95
10000	1000	1.00	1.00	1.00	1.00	1.00	0.99	0.99	0.99	0.99	0.99	0.98	0.98	0.98	0.97
10000	10000	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Table 6-7. Sidesway Restraining Force Modifier (mD), cont.

k_e / k	C_h / k	Number of frames (endwalls counted as frames)													
		17	18	19	20	21	22	23	24	25	26	27	28	29	30
5	5	0.04	0.03	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.00	0.00	0.00	0.00
5	10	0.09	0.08	0.07	0.06	0.05	0.04	0.04	0.03	0.03	0.02	0.02	0.02	0.01	0.01
5	20	0.17	0.15	0.13	0.12	0.11	0.10	0.09	0.08	0.07	0.06	0.06	0.05	0.04	0.04
5	50	0.26	0.24	0.22	0.21	0.19	0.18	0.17	0.16	0.14	0.13	0.12	0.12	0.11	0.10
5	100	0.30	0.29	0.27	0.26	0.24	0.23	0.22	0.20	0.19	0.18	0.17	0.17	0.16	0.15
5	200	0.33	0.31	0.30	0.29	0.27	0.26	0.25	0.24	0.23	0.22	0.21	0.20	0.19	0.19
5	500	0.35	0.33	0.32	0.31	0.29	0.28	0.27	0.26	0.25	0.25	0.24	0.23	0.22	0.21
5	1000	0.35	0.34	0.33	0.31	0.30	0.29	0.28	0.27	0.26	0.25	0.25	0.24	0.23	0.23
5	10000	0.36	0.35	0.33	0.32	0.31	0.30	0.29	0.28	0.27	0.26	0.26	0.25	0.24	0.24
10	5	0.05	0.04	0.03	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.00	0.00	0.00
10	10	0.12	0.10	0.09	0.08	0.06	0.06	0.05	0.04	0.03	0.03	0.03	0.02	0.02	0.02
10	20	0.23	0.20	0.18	0.16	0.15	0.13	0.12	0.11	0.09	0.08	0.08	0.07	0.06	0.05
10	50	0.36	0.34	0.32	0.30	0.28	0.26	0.24	0.23	0.21	0.20	0.18	0.17	0.16	0.15
10	100	0.44	0.42	0.40	0.38	0.36	0.34	0.33	0.31	0.29	0.28	0.27	0.25	0.24	0.23
10	200	0.49	0.47	0.45	0.43	0.42	0.40	0.39	0.37	0.36	0.34	0.33	0.32	0.31	0.30
10	500	0.52	0.50	0.49	0.47	0.46	0.44	0.43	0.42	0.40	0.39	0.38	0.37	0.36	0.35
10	1000	0.53	0.52	0.50	0.49	0.47	0.46	0.45	0.43	0.42	0.41	0.40	0.39	0.38	0.37
10	10000	0.54	0.53	0.51	0.50	0.49	0.47	0.46	0.45	0.44	0.43	0.42	0.41	0.40	0.39
20	5	0.05	0.04	0.03	0.03	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.00
20	10	0.14	0.12	0.10	0.09	0.07	0.06	0.05	0.05	0.04	0.03	0.03	0.03	0.02	0.02
20	20	0.27	0.24	0.22	0.20	0.17	0.16	0.14	0.13	0.11	0.10	0.09	0.08	0.07	0.06
20	50	0.45	0.42	0.40	0.37	0.35	0.33	0.30	0.28	0.27	0.25	0.23	0.22	0.20	0.19
20	100	0.56	0.53	0.51	0.49	0.47	0.45	0.43	0.41	0.39	0.37	0.35	0.34	0.32	0.31
20	200	0.63	0.61	0.59	0.57	0.55	0.53	0.52	0.50	0.48	0.47	0.45	0.44	0.42	0.41
20	500	0.67	0.66	0.64	0.63	0.61	0.60	0.59	0.57	0.56	0.55	0.53	0.52	0.51	0.50
20	1000	0.69	0.68	0.66	0.65	0.64	0.62	0.61	0.60	0.59	0.58	0.57	0.55	0.54	0.53
20	10000	0.71	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62	0.61	0.60	0.59	0.58	0.57
50	5	0.06	0.05	0.04	0.03	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.00	0.00
50	10	0.15	0.13	0.11	0.10	0.08	0.07	0.06	0.05	0.04	0.04	0.03	0.03	0.02	0.02
50	20	0.30	0.27	0.24	0.22	0.20	0.18	0.16	0.14	0.13	0.11	0.10	0.09	0.08	0.07
50	50	0.52	0.49	0.46	0.44	0.41	0.38	0.36	0.34	0.31	0.29	0.27	0.26	0.24	0.22
50	100	0.66	0.64	0.61	0.59	0.56	0.54	0.52	0.50	0.47	0.45	0.43	0.41	0.40	0.38
50	200	0.75	0.73	0.71	0.69	0.68	0.66	0.64	0.62	0.60	0.59	0.57	0.55	0.54	0.52
50	500	0.81	0.80	0.79	0.78	0.76	0.75	0.74	0.73	0.71	0.70	0.69	0.68	0.67	0.65
50	1000	0.84	0.83	0.82	0.81	0.80	0.79	0.78	0.77	0.76	0.75	0.74	0.73	0.72	0.71
50	10000	0.86	0.85	0.84	0.84	0.83	0.82	0.81	0.81	0.80	0.79	0.79	0.78	0.77	0.77
100	5	0.06	0.05	0.04	0.03	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.00	0.00
100	10	0.16	0.13	0.11	0.10	0.08	0.07	0.06	0.05	0.04	0.04	0.03	0.03	0.02	0.02
100	20	0.31	0.28	0.25	0.23	0.20	0.18	0.16	0.15	0.13	0.12	0.11	0.09	0.08	0.08
100	50	0.55	0.52	0.49	0.46	0.43	0.41	0.38	0.36	0.33	0.31	0.29	0.27	0.25	0.24
100	100	0.70	0.68	0.65	0.63	0.60	0.58	0.56	0.53	0.51	0.49	0.47	0.45	0.43	0.41
100	200	0.80	0.78	0.77	0.75	0.73	0.71	0.69	0.68	0.66	0.64	0.62	0.61	0.59	0.57
100	500	0.87	0.86	0.85	0.84	0.83	0.82	0.81	0.80	0.79	0.77	0.76	0.75	0.74	0.73
100	1000	0.90	0.89	0.88	0.88	0.87	0.86	0.85	0.84	0.84	0.83	0.82	0.81	0.80	0.80
100	10000	0.92	0.92	0.91	0.91	0.90	0.90	0.90	0.89	0.89	0.88	0.88	0.87	0.87	0.86
1000	5	0.06	0.05	0.04	0.03	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.00	0.00
1000	10	0.16	0.14	0.12	0.10	0.09	0.07	0.06	0.05	0.05	0.04	0.03	0.03	0.02	0.02
1000	20	0.33	0.29	0.26	0.24	0.21	0.19	0.17	0.15	0.14	0.12	0.11	0.10	0.09	0.08
1000	50	0.58	0.55	0.52	0.49	0.46	0.43	0.40	0.38	0.35	0.33	0.31	0.29	0.27	0.25
1000	100	0.74	0.72	0.69	0.67	0.64	0.62	0.60	0.57	0.55	0.53	0.50	0.48	0.46	0.44
1000	200	0.85	0.84	0.82	0.80	0.79	0.77	0.75	0.74	0.72	0.70	0.68	0.66	0.65	0.63
1000	500	0.93	0.93	0.92	0.91	0.90	0.89	0.88	0.87	0.86	0.85	0.84	0.83	0.82	0.81
1000	1000	0.96	0.96	0.95	0.95	0.94	0.94	0.93	0.93	0.92	0.92	0.91	0.90	0.90	0.89
1000	10000	0.99	0.99	0.99	0.99	0.99	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
10000	5	0.06	0.05	0.04	0.03	0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.00	0.00
10000	10	0.16	0.14	0.12	0.10	0.09	0.07	0.06	0.05	0.05	0.04	0.03	0.03	0.02	0.02
10000	20	0.33	0.30	0.26	0.24	0.21	0.19	0.17	0.15	0.14	0.12	0.11	0.10	0.09	0.08
10000	50	0.58	0.55	0.52	0.49	0.46	0.43	0.40	0.38	0.36	0.33	0.31	0.29	0.27	0.25
10000	100	0.75	0.72	0.70	0.67	0.65	0.62	0.60	0.58	0.55	0.53	0.51	0.49	0.47	0.45
10000	200	0.86	0.84	0.83	0.81	0.79	0.78	0.76	0.74	0.72	0.71	0.69	0.67	0.65	0.64
10000	500	0.94	0.93	0.92	0.92	0.91	0.90	0.89	0.88	0.87	0.86	0.85	0.84	0.83	0.82
10000	1000	0.97	0.96	0.96	0.96	0.95	0.95	0.94	0.94	0.93	0.93	0.92	0.91	0.91	0.90
10000	10000	1.00	1.00	1.00	1.00	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99

6.6.4.1 Example Application of *mS* and *mD* Tables

Problem Statement

For a four bay building with k = 71.5 lbf/inch, k_e =2000 lbf/inch, C_h = 16,000 lbf/inch, and R = 1111 lbf, use the *mS* and *mD* Tables to determine: (1) the maximum diaphragm element shear force, and (2) the sidesway restraining force for the middle (i.e., critical) post frame.

Solution

k_e / k = 2000 lbf/in. / 71.5 lbf/ in. = 28.0

C_h / k = 16,000 lbf/in. / 71.5 lbf/in. = 224

By interpolation from Table 6-6, *mS* is equal to 1.87. By interpolation from Table 6-7, *mD* is equal to 0.91.

$V_{h,max} = R\ mS$ = 1111 lbf (1.87) = 2076 lbf

$Q_c = R\ mD$ = 1111 lbf (0.91) = 1011 lbf

The input values for this problem are identical to those used in the example DAFI analysis shown in figure 6-18. As previously described, the maximum horizontal diaphragm shear force, $V_{h,max}$, based on the DAFI output is 2098 lbf. The sidesway restraining force for the critical frame, Q_c, based on the DAFI output is 1027 lbf. The Q_c value is obtained by subtracting the load of 84.19 lbf resisted by frame 3 (i.e., the critical frame) from the load of 1111 lbf applied to frame 3.

Comparing the $V_{h,max}$ and Q_c values of 2098 lbf and 1027 lbf obtained using DAFI, to the $V_{h,max}$ and Q_c values of 2076 lbf and 1011 lbf obtained using the *mS* and *mD* Tables, illustrates the slight error introduced with interpolation of the lower precision numbers in the *mS* and *mD* Tables.

6.6.5 Simple Beam Analogy Equations

McGuire (1998) presented the concept of modeling the diaphragm as a simple beam with an applied load inversely proportional to deflection. This analogy resulted in the following equations for calculating diaphragm shear forces and lateral displacements for the special case when: (1) all diaphragm elements have the same stiffness C_h, (2) all interior frame elements have the same stiffness, k, (3) both exterior frame elements (i.e., the two elements representing the endwalls) have the same stiffness, k_e, and (4) eave load, R, is the same at each interior frame.

$$V_h = C_h \alpha s[A \sinh(\alpha x) + B \cosh(\alpha x)] \tag{6-16}$$

$$V_{h,max} = C_h \alpha s B \tag{6-17}$$

$$y = A \cosh(\alpha x) + B \sinh(\alpha x) + R/k \tag{6-18}$$

$$y_{max} = A \cosh(\alpha L/2) + B \sinh(\alpha L/2) + R/k \tag{6-19}$$

$$y_e = R / [\ k\ (1 - D)] \tag{6-20}$$

$$Q_c = R - y_{max}\, k \tag{6-21}$$

where:

R = eave load, lbf (N)
s = frame spacing, in. (mm)
L = distance between endwalls, in. (mm)
C_h = horizontal shear stiffness for a width s of the diaphragm, lbf/in. (N/mm)
k = stiffness of interior frames, lbf/in. (N/mm)
k_e = stiffness of endwall frames (or shearwalls), lbf/in. (N/mm)
x = distance from endwall, in. (mm)
y = lateral displacement of diaphragm at a distance x from the endwall, in. (mm)
y_e = lateral displacement of the endwall, in. (mm)
y_{max} = maximum eave displacement, in. (mm)
= lateral displacement of diaphragm at a distance $x = L/2$ from the endwall
V_h = diaphragm shear force, lbf (N)
$V_{h,max}$ = maximum diaphragm shear force, lbf (N)
= diaphragm shear force at x= 0 or $x = L$
Q_c = sidesway restraining force for the critical frame, lbf (N)
sinh = hyperbolic sine
cosh = hyperbolic cosine

$$\alpha = \frac{(k / C_h)^{1/2}}{s}$$

$$A = y_e - R/k$$

$$B = \frac{A\ (\ 1 - \cosh(\alpha L))}{\sinh(\alpha L)}$$

$$D = \frac{k_e \sinh(\alpha L)}{\alpha C_h\, s\ (1 - \cosh(\alpha L))}$$

Entering simple beam analogy equations into a spreadsheet program provides for quick and precise calculations, and thus is recommended over calculations requiring interpolation of *mS* and *mD* Table values.

6.6.5.1 Example Application of Simple Beam Analogy Equations

Problem Statement

For a four bay building with s = 120 inches, k = 71.5 lbf/inch, k_e =2000 lbf/inch, C_h = 16,000 lbf/inch, and R = 1111 lbf, use the simple beam analogy equations to determine: (1) lateral displacement of the endwall, (2) maximum eave displacement, (3) maximum diaphragm element shear force, and (4) sidesway restraining force for the critical post frame.

Solution

Properties		
R	=	1111 lbf
s	=	120 in.
L	=	480 in.
k_e	=	2000 lbf/in.
k	=	71.5 lbf/in.
C_h	=	16,000 lbf/in.

Intermediate Calculations		
α	=	0.00055707 in.$^{-1}$
$\cosh(\alpha L)$	=	1.03496
$\sinh(\alpha L)$	=	0.27059
$\cosh(\alpha L/2)$	=	1.00895
$\sinh(\alpha L/2)$	=	0.13410
D	=	-14.069
y_e	=	1.0311 inches
A	=	-14.507 inches
B	=	1.9281 inches

Calculated Displacements and Forces		
y_e	=	1.031 inches
y_{max}	=	1.160 inches
$V_{h,max}$	=	2062 lbf
Q_c	=	1028 lbf

6.6.6 In-Plane Shear Force in a Diaphragm Section, V_p

The analysis tools/methods discussed in sections 6.6.2 through 6.6.5 provide horizontal components of diaphragm *element* in-plane forces. In most cases, each element is comprised of two or more diaphragm sections. The in-plane shear force in each of these diaphragm sections is calculated as:

$$V_{p,i} = (c_{h,i} / C_{h,x})\, V_{h,x} / (\cos \theta_i) \quad (6\text{-}22)$$

where:

$V_{p,i}$ = in-plane shear force in diaphragm section i, lbf (N)
$V_{h,x}$ = horizontal shear force in diaphragm element x, (from Sections 6.6.2 through 6.6.5), lbf (N)
$c_{h,i}$ = horizontal shear stiffness of diaphragm section i, lbf/in. (N/mm)
$C_{h,x}$ = horizontal shear stiffness of diaphragm element x
θ_i = slope of diaphragm section i

6.6.6.1 Example Calculation

Problem Statement

Diaphragm element 1 in figure 6-3 consists of three diaphragm sections as shown in figure 6-2: roof sections 1a and 1b each with a horizontal shear stiffness of 6000 lbf/inch, and ceiling section 1c with a horizontal shear stiffness of 4000 lbf/inch. When diaphragm element 1 is subjected to a horizontal shear force C_{h1} of 2100 lbf, what in-plane shear forces are induced in diaphragm sections 1a, 1b and 1c? Note that both roof sections are sloped at 26.6 degrees (6-in-12 slope) and the ceiling section is horizontal.

Solution

C_{h1} = $c_{h,1a} + c_{h,1b} + c_{h,1c}$
= 6000 lbf/in + 6000 lbf/in + 4000 lbf/in
= 16000 lbf/in

$V_{p,i}$ = $(c_{h,i} / C_{h1})\, V_{h1} / (\cos \theta_i)$

$V_{p,1a}$ = (6000/16000) (2100 lbf) / 0.894 = 881 lbf

$V_{p,1b}$ = (6000/16000) (2100 lbf) / 0.894 = 881 lbf

$V_{p,1c}$ = (4000/16000) (2100 lbf) / 1.00 = 525 lbf

6.6.7 Forces Applied to Frames by Individual Diaphragms

The horizontal movement of most building frames is resisted by roof/ceiling diaphragms. The total horizontal resisting force applied to an individual frame by the roof/ceiling diaphragms attached to both sides of the frame is defined as the sidesway restraining force, Q.

To accurately model a frame with resisting forces applied by the roof and ceiling diaphragms, requires that the sidesway restraining force, Q, first be divided up between the individual diaphragm (e.g., diaphragms a, b, and c in figure 6-2b). This is accomplished using the following equation:

$$Q_i = Q\,(c_{h,i} / C_h) \quad (6\text{-}23)$$

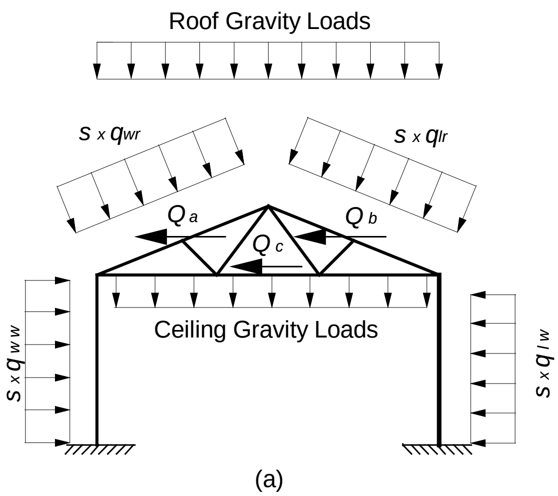
where:

Q_i = sidesway resisting force due to diaphragm i, lbf (N)
Q = total sidesway resisting force acting on the frame, lbf (N)
= Q_c for the critical frame
C_h = horizontal shear stiffness for a width s of the roof/ceiling assembly, lbf/in. (N/mm)
$c_{h,i}$ = horizontal shear stiffness of diaphragm i with width s, lbf/in. (N/mm)

The total sidesway resisting force acting on a frame is not output by DAFI directly, but can be obtained by subtracting the load resisted by a frame (which is output by DAFI) from the eave load, *R*, applied to the frame.

Since diaphragm construction typically doesn't change from one side of a frame to the other side of the frame, C_h and $c_{h,i}$ values associated with either of the two diaphragm elements (that are adjacent to the frame) can be used in equation 6-23.

Horizontal restraining forces calculated for the three diaphragms in figure 6-2(b), are graphically illustrated in figure 6-19(a). For post-frame component stress analysis, these restraining forces should be applied as in-plane forces as shown in figure 6-19(b). In-plane forces are calculated from the horizontal forces as follows:



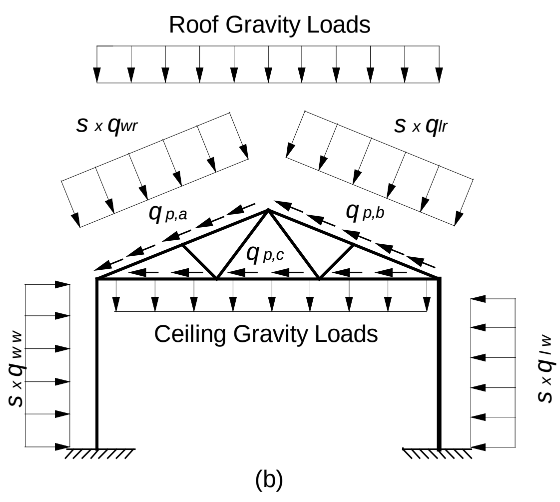


Figure 6-19. (a) Frame with diaphragm resisting forces. (b) Resisting forces applied as uniformly distributed in-plane loads for frame component stress analysis.

$$Q_{p,i} = Q_i / (\cos \theta_i) \tag{6-24}$$

or

$$q_{p,i} = Q_i / (d_i \cos \theta_i) \tag{6-25}$$

where:

Q_{pi} = in-plane force applied to frame by diaphragm *i*, lbf (N)
Q_i = sidesway resisting force due to diaphragm *i*, lbf (N)
θ_i = slope of diaphragm *i*
$q_{p,i}$ = in-plane force applied to the frame per unit length of diaphragm *i*, lbf/ft (N/m)
d_i = slope length of diaphragm *i*, ft (m)

6.7 Component Design

6.7.1 General

All building components must be checked to ensure that actual loads do not exceed allowable design values. In this section, special attention is given to components that are involved in load transfer by diaphragm action.

6.7.2 Diaphragms

The maximum shear in a diaphragm section, $V_{p,i}$, cannot exceed the allowable shear strength of the section, $v_{a,i}$, multiplied by the diaphragm length.

$$V_{p,i} \le v_{a,i}\, d_i \tag{6-26}$$

where:

$V_{p,i}$ = in-plane shear force in diaphragm section *i* from equation 6-22, lbf (N)
$v_{a,i}$ = allowable in-plane shear strength of diaphragm *i* (see Section 7.3.3), lbf/ft (N/m)
d_i = slope length of diaphragm *i*, ft (m)

6.7.3 Diaphragm Chords

In addition to shear forces, a roof/ceiling diaphragm assembly must also resist bending moment. The magnitude of this bending moment is dependent on a number of factors. For design, this bending moment is assumed to be no greater than:

$$M_d = V_{h,max}\, L / 4 \tag{6-27}$$

where:

M_d = diaphragm bending moment, lbf-ft (N m)
$V_{h,max}$ = maximum total shear in roof/ceiling diaphragm assembly, lbf (N)
L = distance between shearwalls, ft (m)

Equation 6-27 treats the roof/ceiling assembly as a uniformly loaded beam that is *simply* supported by two shearwalls spaced a distance *L* apart. Each shearwall is assumed to be subjected to a force that is equal to the maximum total shear in the roof/ceiling assembly, $V_{h,max}$.

The maximum total shear in the roof/ceiling assembly, $V_{h,max}$ can be obtained via plane-frame structural analysis (Section 6.6.2), DAFI output (Section 6.6.3), or if applicable, equations 6-14 or 6-17. The uniform load on the roof/ceiling assembly (w in figure 6-20a) is set equal to $2V_{h,max}/L$. This quantity is multiplied by $L^2/8$ to obtain M_d.

The bending moment applied to a roof/ceiling diaphragm assembly is resisted by axial forces (a.k.a. chord forces) in members oriented perpendicular to trusses/rafters. This includes roof purlins and analogous framing members in the ceiling diaphragm. For bending moment calculations, these members are referred to as diaphragm chords (figure 6-20a). Any connection in the chords, either between intermediate chord members or where they are connected to the endwalls, must be designed to resist the calculated axial force.

If the roof/ceiling assembly behaves as a *single* beam in resisting bending moment, the maximum chord force (which is located in the edge chords) can be calculated as:

$$P_e = M_d\,\alpha / b \tag{6-28}$$

where:

P_e = axial force in edge chord, lbf (N)
M_d = diaphragm bending moment from equation 6-27, lbf-ft (N m)
α = reduction factor dependent on chord force distribution
b = horizontal distance between edge chords, ft (m)

The axial force in an edge chord is dependent on chord force distribution as indicated by the presence of α in equation 6-28. The current ANSI/ASAE EP484 diaphragm design procedure assumes that edge chords act alone in resisting bending moment (figure 6-20b). For this case, α is numerically equal to one (1). This is a conservative approach. Alternatively, many engineers assume a linear distribution of chord forces as shown in figure 6-20c. When a linear distribution is assumed, the reduction factor α is a function of chord location. If there are an even number of chords and they are evenly spaced, then α is given as:

$$\alpha = \frac{(n-1)^2}{\sum_{i=1}^{n/2} (n - 2i + 1)^2} \tag{6-29}$$

If there are an odd number of chords and they are evenly spaced, then α is given as:

$$\alpha = \frac{(n-1)^2}{\sum_{i=1}^{(n-1)/2} (n - 2i + 1)^2} \tag{6-30}$$

where:

α = reduction factor when chords are evenly spaced and chord forces are linearly distributed as shown in figure 6-20c
n = number of chord rows, including the two rows of edge chords

Equations 6-29 and 6-30 were used to calculate the values given in Table 6-8.

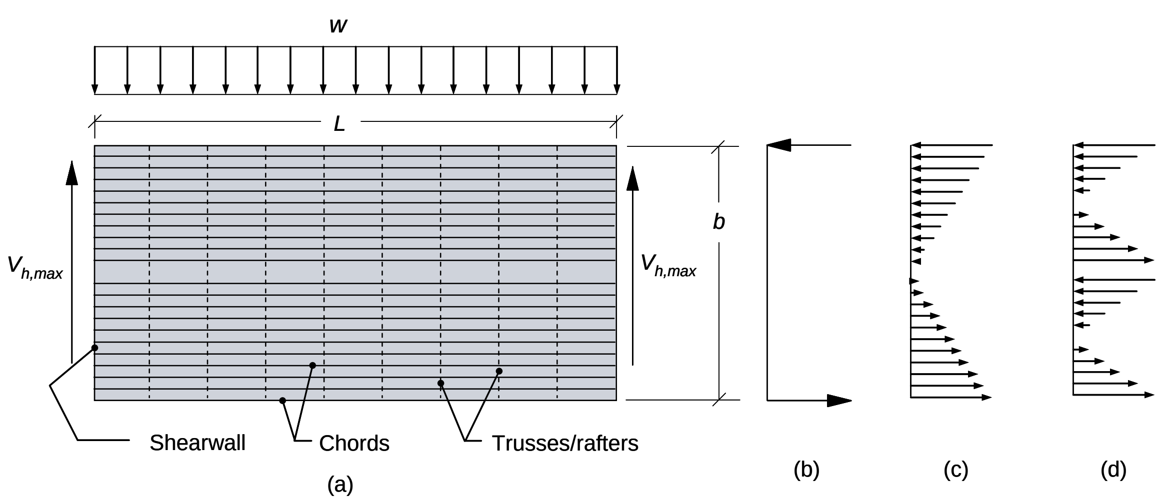


Figure 6-20. (a) Plan view of a diaphragm under a uniform load, *w*. Chord force distributions when (b) moment resisted by edge chords only, (b) chord force distribution is linear, and (c) chord force distribution is linear, but diaphragm halves assumed to act independently in resisting moment.

Table 6-8 Reduction Factor, α, for Axial Force in Edge Chords

n*	α	n*	α
2	1.000	22	0.249
3	1.000	23	0.239
4	0.900	24	0.230
5	0.800	25	0.222
6	0.714	26	0.214
7	0.643	27	0.206
8	0.583	28	0.200
9	0.533	29	0.193
10	0.491	30	0.187
11	0.455	31	0.181
12	0.423	32	0.176
13	0.396	33	0.171
14	0.371	34	0.166
15	0.350	35	0.162
16	0.335	36	0.158
17	0.314	37	0.154
18	0.298	38	0.150
19	0.284	39	0.146
20	0.271	40	0.143
21	0.260	41	0.139

* n is the number of chord rows, including the two rows of edge chords

If a linear distribution of chord force is assumed (figure 6-20c), and interior chords are evenly spaced, the load in an interior chord, P_i, is given as:

$$P_i = 2\, P_e\, x_i / b \tag{6-31}$$

where:

P_i = axial force for chord in row i, lbf (N)
P_e = axial force in edge chord from equation 6-30, lbf (N)
b = horizontal distance between edge chords, ft (m)
x_i = horizontal distance from center of diaphragm to chord row i.

6.7.3.1 Additional Considerations Regarding Chord Forces

The axial force induced in an individual chord by applied building loads is a function of many complex, interacting design variables. For this reason, designers have had to rely on simplifying assumptions in order to approximate chord forces.

The assumption inherent in the development of equation 6-27 is that the roof/ceiling assembly acts as a large deep beam that is simply supported by two end shearwalls, and that half the applied load induced in the diaphragm goes to each of these shearwalls. The moment calculated using equation 6-27 is the moment midway between the two shearwalls. At all other locations, the moment is lower, and thus use of equation 6-27 to calculate chord forces at others locations would in theory be conservative.

Equation 6-27 neglects the resistance to in-plane bending contributed by sidewalls. Sidewalls help resist (and thereby reduce) in-plane bending moments in two ways. First, they brace endwalls and other shearwalls, which limits rotation of the diaphragm at these shearwalls. Second, they resist a change in eave length (and hence changes in eave chord forces) by virtue of their own in-plane shear stiffness.

Because of the influence of sidewalls, the distribution of in-plane bending moment will not follow that for a typical simple-supported beam (i.e., zero moment at the supports, and maximum moment at midspan). For this reason, Pollock and others (1996) recommend modeling the roof/ceiling assembly as a deep beam with fixed supports. In this case, the maximum bending moment in the diaphragm is at the shearwalls and is given as follows.

$$M_d = V_{h,max}\, L / 6 \tag{6-32}$$

where:

M_d = diaphragm bending moment, lbf-ft (N m)
$V_{h,max}$ = maximum total shear in roof/ceiling diaphragm assembly, lbf (N)
L = distance between shearwalls, ft (m)

Equations 6-27 and 6-32 both assume that the diaphragm is located between shearwalls with an identical frame stiffness k_e and thus each is subjected to an identical shear force $V_{h,\,max}$ as shown in figure 6-20a. More often than not, a diaphragm is located between shearwalls with differing frame stiffness values. In the extreme case, one shearwall attracts all the load, potentially subjecting the diaphragm at that shearwall to a maximum bending moment given as:

$$M_d = V_{h,max}\, L / 2 \tag{6-33}$$

Equation 6-33 would be applicable to a building of length L in which one of the endwalls has a frame stiffness similar to that of the interior frames. Typically in such structures, the sidewalls (i.e., the walls perpendicular to the applied load) will play a major roll in resisting the moment induced in the diaphragm. Whether or not the diaphragm is subjected to the maximum moment given by equation 6-33 depends on how the sidewalls are connected to the diaphragm and the in-plane shear stiffness of the sidewalls.

It is important to realize that even for a known uniformly applied load, the exact distribution of bending moment along the length of a diaphragm is a complex function of diaphragm geometry (e.g. length-to-width ratio), diaphragm properties, the geometry and properties of the endwalls and sidewalls to which the diaphragm is connected, and the connections between all these elements. It will also vary with load level and changes in load direction due to the inherent nonlinear behavior of components and connections.

Because of uncertainty surrounding variation in in-plane bending moment with building length, engineers will often assign the maximum calculated in-plane bending moment M_d to every location along the length of the building. To balance the conservativeness of this assignment, these engineers will use equation 6-32 instead of equation 6-27 to calculate M_d.

Another major assumption that a designer must make involves the distribution of chord forces across a building. Three different chord force distributions are shown in figure 6-20b, 6-20c, and 6-20d. Based on full-scale building tests (Niu and Gebremedhin, 1997; Bohnhoff and others, 2003) and computer modeling (Wright and Manbeck, 1993; Williams, 1999; and Bohnhoff and others, 1999), the actual distribution of chord forces appears to be a complex function of numerous variables. In general, it is recommended that designers assume the chord force distribution shown in figure 6-20c unless there is a major separation between two diaphragms in which case a distribution similar to that shown in figure 6-20d may be more appropriate.

6.7.3.2 Example Calculations

Problem Statement

A building with a width of 24 feet and length of 32 feet has two similar endwalls, a gable roof with slope of 6-in-12, and nine rows of purlins per slope (i.e., a purlin spacing of 20 inches). If the maximum total horizontal shear force $V_{h,max}$ in the roof diaphragm is 2025 lbf, what is the maximum chord force in the roof diaphragm.

Solution

Using equation 6-32 to calculate diaphragm bending moment:

$$M_d = V_{h,max}\, L / 6$$

$$= 2025 \text{ lbf } (32 \text{ ft})/6 = 10800 \text{ ft-lbf}$$

If a linear distribution of chord forces is assumed

$$P_e = M_d\, \alpha / b$$

$$= 10800 \text{ ft-lbf } (0.298)/24 \text{ ft} = 134 \text{ lbf}$$

If bending moment is assumed to be resisted only by edge purlins then:

$$P_e = 10800 \text{ ft-lbf } (1.00)/24 \text{ ft} = 450 \text{ lbf}$$

6.7.4 Shearwalls

End and intermediate shearwalls must have sufficient strength to transmit forces from roof and ceiling diaphragms to the foundation system. In equation form:

$$v_a \geq V_s / (W - D_T) \qquad (6\text{-}34)$$

where:

v_a = allowable shear capacity of shearwall, lbf/ft (N/m)
V_s = force induced in shearwall, lbf (N)
W = building width, ft (m)
D_T = total width of door and window openings in the shearwall, ft (m)

The allowable shear capacity of end and intermediate shearwalls, v_a, is obtained from validated structural models, or from tests as outlined in ASAE EP558 (see Section 7.5). The total force in the shear wall, V_s, is obtained via plane-frame structural analysis (Section 6.6.2), DAFI output (Section 6.6.3), or if applicable, equations 6-14 or 6-17 (note that V_s is equal to $V_{h,max}$ in buildings for which equations 6-14 and 6-17 are applicable).

The total width of door and window openings, D_T, generally varies with height as shown in figure 6-21. At locations where D_T is the greatest (section b-b in figure 6-21) additional reinforcing may be required to ensure that the allowable shear stress is not exceeded.

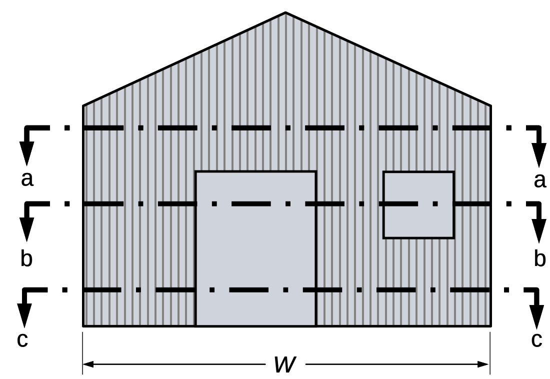


Figure 6-21. Shearwall showing variations in opening width, D_T, with height.

The structural framing over a door or window opening will act as a drag strut transferring shear across the opening. The header over the opening shall be designed to carry the force in tension and/or compression across the opening.

Shearwall strength can easily be increased when the applied load exceeds shearwall capacity. For example, the density of stitch screws can be increased and additional fasteners can be added in panel flats (on both sides of each major rib is the most effective). If only one side of the wall has been sheathed, add wood paneling or metal cladding to the other side. Metal diagonal braces can also be added beneath any wood paneling or corrugated metal siding.

6.7.5 Shearwall Connections

Connections that fasten (1) roof and ceiling diaphragms to a shearwall, and (2) shearwalls to the foundation system, must be designed to carry the appropriate amount of shear load. The design of these connections may be proved by tests of a typical connection detail or by an appropriate calculation method.

At end shearwalls it is not uncommon to use the truss top chord to transfer load from roof cladding to endwall cladding. Sidewall steel is fastened directly to the truss chord, as is the roof steel when purlins are inset. In buildings with top-running purlins, roof cladding can not be fastened directly to the truss. In such cases, blocking equal in depth to the purlins is placed between the purlins and fastened to the truss. Roof cladding is then attached directly to this blocking.

6.7.6 Shearwall Overturning

Diaphragm loading produces overturning moment in shearwalls. This moment induces vertical forces in shearwall-to-foundation connections that must be added to vertical forces resulting from tributary loads. In the case of embedded posts, increases in uplift forces may require an increase in embedment depth, and increases in downward force may require an increase in footing size (see Chapter 5).

6.8 Rigid Roof Design

6.8.1 General

When diaphragm stiffness is considerably greater than the stiffness of interior post frames, the designer may want to assume that the diaphragm and shearwalls are infinitely stiff. Under this assumption, 100% of the applied eave load, R, is transferred by the diaphragm to shearwalls, and none of the applied eave load is resisted by the frames. Because all eave load is assumed to be transferred to shearwalls, no special analysis tools or design tables are required to determine load distribution between diaphragms and post-frames. This simplifies the entire diaphragm design process. This simplified procedure is referred to as *rigid roof design* (Bender and others, 1991).

6.8.2 Calculation

When (1) the shearwalls and roof/ceiling diaphragm assembly are assumed to be infinitely rigid, (2) the only applied loads with horizontal components are due to wind, and (3) wind pressure is uniformly distributed on each wall and roof surface, then the maximum shear force in the diaphragm assembly is given as:

$$V_{h,max} = L\,(h_{wr}\,q_{wr} - h_{lr}\,q_{lr} + h_{ww}\,f_w\,q_{ww} - h_{lw}\,f_l\,q_{lw})\,/\,2 \qquad (6\text{-}35)$$

where:

$V_{h,max}$ = maximum diaphragm element shear force, lbf (N)
L = building length, ft (m)
h_{wr} = windward roof height, ft (m)
h_{lr} = leeward roof height, ft (m)
h_{ww} = windward wall height, ft (m)
h_{lw} = leeward wall height, ft (m)
q_{wr} = design windward roof pressure, lbf/ft^2 (N/m^2)
q_{lr} = design leeward roof pressure, lbf/ft^2 (N/m^2)
q_{ww} = design windward wall pressure, lbf/ft^2 (N/m^2)
q_{lw} = design leeward wall pressure lbf/ft^2 (N/m^2)
f_w = frame-base fixity factor, windward post
f_l = frame-base fixity factor, leeward post

Inward acting wind pressures have positive signs, outward acting pressures are negative (figure 6-14). As previously noted, frame-base fixity factors, f_w and f_l, determine how much of the total wall load is transferred to the eave, and how much is transferred directly to the ground. The greater the resistance to rotation at the base of a wall, the more load will be attracted directly to the base of the wall.

For symmetrical base restraint and frame geometry, equation 6-35 reduces to:

$$V_{h,max} = L\,[h_r\,(q_{wr} - q_{lr}) + h_w\,f\,(q_{ww} - q_{lw})]\,/\,2 \qquad (6\text{-}36)$$

where:

h_r = roof height, ft (m)
h_w = wall height, ft (m)
f = frame-base fixity factor for both leeward and windward posts

6.8.3 Application

The $V_{h,max}$ value calculated using equation 6-35 (or 6-36) is *always* a conservative estimate of the actual maximum shear force (due to wind) in a diaphragm assembly. This estimate becomes increasingly conservative as the amount of load resisted by interior post-frames increases. Equations 6-35 and 6-36 are most accurate when diaphragm stiffness is considerably greater than interior post-frame stiffness. This tends to be the case in buildings that are relatively wide and/or high, and in buildings where individual posts offer no resistance to rotation (i.e., the posts are more-or less pin-connected at both the floor and eave lines).

Output from a DAFI analysis of a building with relatively high diaphragm and shearwall stiffness values is presented in figure 6-18. This output shows less than 6% of the total horizontal eave load being resisted by the interior frames.

Although rigid roof design expedites calculation of maximum diaphragm shear forces, the design procedure does not provide estimates of sidesway restraining force for interior post-frame design.

6.9 References

6.9.1 Non-Normative References

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6.9.2 Normative References

ANSI/ASAE EP486.2 *Shallow post and pier foundation design*

ANSI/ASAE EP484.2 *Diaphragm design of metal-clad wood-frame rectangular buildings*

ASAE EP558.1 *Load tests for metal-clad wood-frame diaphragms*

CHAPTER 7

Metal-Clad Wood-Frame Diaphragm Properties

Contents

7.1 Introduction

7.1.1 General

One of the first steps in diaphragm design is to establish in-plane shear strength and stiffness values for each identified diaphragm section. In most post-frame buildings, these diaphragm sections consist of corrugated metal panels that have been screwed or nailed to wood framing. Behavior of these metal-clad wood-frame (MCWF) diaphragms is complex, and consequently, has been the subject of considerable research during the past 40 years. In addition to improving overall design, this research has led to improved methods for predicting metal-clad wood-frame diaphragm strength and stiffness.

7.1.2 Predicting Diaphragm Behavior

There are essentially three procedures for predicting the strength and stiffness of a building diaphragm. First, an exact replica of the building diaphragm (a.k.a. a full-size diaphragm) can be built and tested to failure. Second, a smaller, representative section of the building diaphragm can be built and laboratory tested. The strength and stiffness of this test assembly are then extrapolated to obtain strength and stiffness values for the building diaphragm. Lastly, diaphragm behavior can be predicted using finite element analysis software. The latter requires that the strength and stiffness properties of individual component (e.g., wood framing, mechanical connections, cladding) be known.

Of the three procedures for predicting metal-clad wood-frame diaphragm properties, only the second one - extrapolation of diaphragm test assembly data - is commonly used. This is because testing full-size diaphragms is simply not practical (a new test would have to be conducted every time overall dimensions changed), and finite element analysis of MCWF diaphragms is, for practical purposes, still in a developmental stage. The later can be attributed to the fact that the large number of variables affecting diaphragm structural properties, as well as the nonlinear behavior of some variables, has thus far precluded the development of a quick and reasonably accurate closed-form approximation of diaphragm strength and stiffness.

7.1.3 ASAE EP558 and EP484

Construction specifications and testing procedures for diaphragm test assemblies are given in ASAE EP558 *Load Tests for Metal-Clad Wood-Frame Diaphragms* (ASABE, 2013). EP558 also gives equations for calculating diaphragm test assembly strength and stiffness. These calculations along with construction specifications and testing procedures from EP558 are outlined in Section 7.3 *Diaphragm Assembly Tests*. For additional details and further explanation of testing procedures, readers are referred to the ASAE EP558 Commentary (ASABE, 2013).

ANSI/ASAE EP484 *Diaphragm Design of Metal-Clad, Wood-Frame Rectangular Buildings* (ASABE, 2012) contains the equations for extrapolating diaphragm test assembly properties for use in building design. These calculations are presented in Section 7.4 *Building Diaphragm Properties*.

7.2 Design Variables

7.2.1 General

Many variables affect the shear stiffness and strength of a structural diaphragm, including: overall geometry, cladding characteristics, wood properties, fastener type and location, and blocking. A short description of each of these variables follows.

7.2.2. Geometry

Geometric variables include: spacing between secondary framing members (e.g. purlins), spacing between primary framing members (e.g., trusses/rafters), and overall dimensions. With respect to overall dimensions, diaphragm *depth* is measured parallel to primary frames, diaphragm *length* is measured perpendicular to primary frames. In most structures, the overall length of a roof diaphragm is equal to the length of the building.

7.2.3 Cladding

Cladding type (e.g., wood, metal, fiberglass, etc.) is a significant design variable. Coverage (and examples) in this design manual is limited to corrugated metal cladding. Important design characteristics of this type of cladding include: base metal (e.g., steel, aluminum), base metal thickness, panel profile, and individual sheet width and length.

7.2.4 Wood Framing

The species, moisture content and specific gravity of wood used in the framing system will not only affect the structural properties of the wood members, but also the shear stiffness and strength of mechanical connections

between wood members and between wood members and cladding.

7.2.5 Mechanical Connections

Type (screw or nail), size, and relative location of mechanical fasteners used to join components significantly impact diaphragm properties. Fasteners are primarily defined by what they connect. Major categories include purlin-to-rafter, sheet-to-purlin, and sheet-to-sheet (see figure 7-1). Sheet-to-sheet fasteners are more commonly referred to as stitch or seam fasteners. Removing stitch fasteners can dramatically reduce the shear strength and stiffness of a diaphragm. Sheet-to-purlin fasteners are also defined by their location (i.e., end, edge, and field). A sheet-to-purlin fastener may be located in a rib or in the flat of a corrugated metal panel. Locating fasteners in the flat generally produces stronger and stiffer diaphragms. The nonlinear nature of fastener performance is one of the more complex variables affecting diaphragm stiffness.

7.2.6 Blocking

When secondary framing members are installed above primary framing (e.g. top running purlins) or below primary framing (e.g. bottom-running ceiling framing), cladding can only be fastened directly to the secondary framing (see figure 7-1). In such cases, blocking is often placed between the cladding and primary framing to increase shear transfer between the components. This is commonly done at locations where diaphragms and shearwalls intersect.

7.3 Diaphragm Test Assemblies

7.3.1 Construction

With the exception of overall length and width, a diaphragm test assembly is required to be identical to the diaphragm in the building being designed. Specifically, frame members must be of identical size, spacing, species and grade; metal cladding must be identical in composition, profile and thickness; and fastener type and location must be the same. ASAE EP558 has established minimum sizes for diaphragm test assemblies to ensure that there is not *too* great a difference between the size of a diaphragm test assembly and the actual building diaphragm.

7.3.2 Test Configurations

ASAE EP558 allows for two different testing configurations: a simple beam test (figure 7-2) and a cantilever test (figures 7-3 and 7-4). In figures 7-2 and 7-4, variable "*a*" represents the spacing between rafters/trusses (a.k.a. the frame spacing). This spacing should be equal to, or a multiple of, the frame spacing in the building being designed.

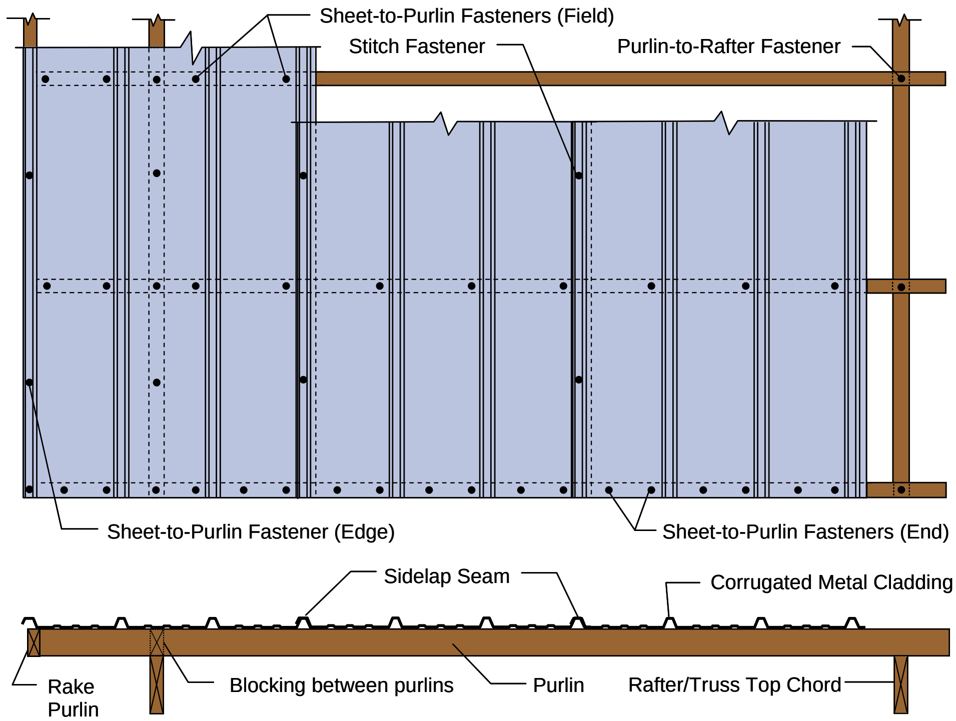
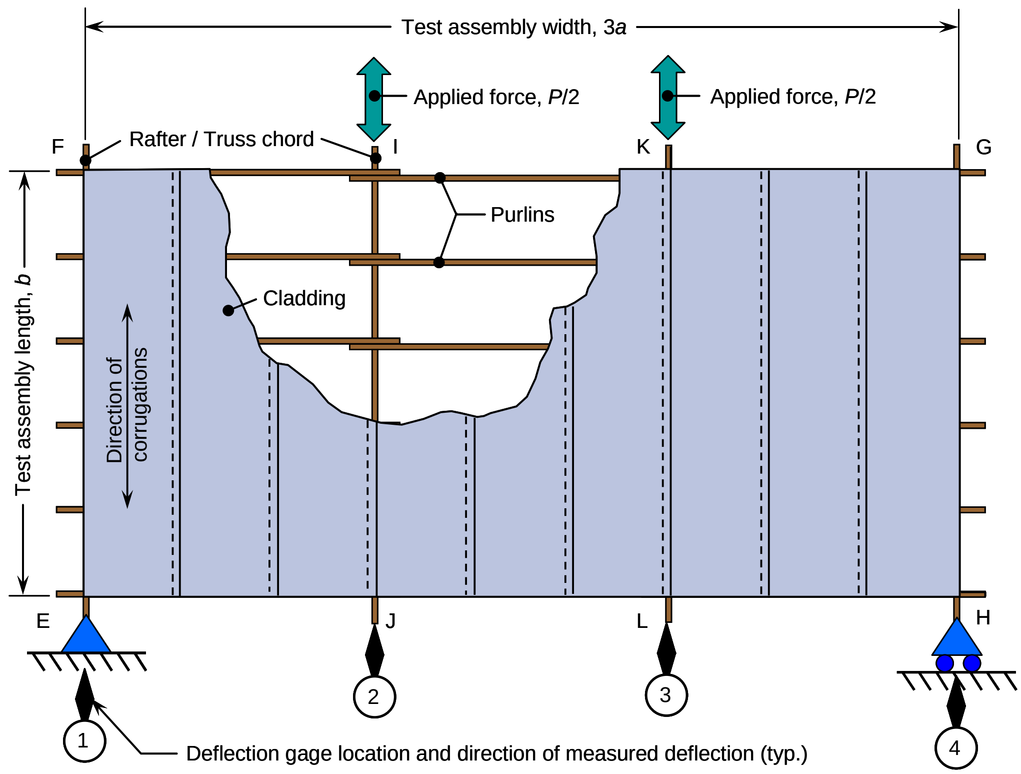
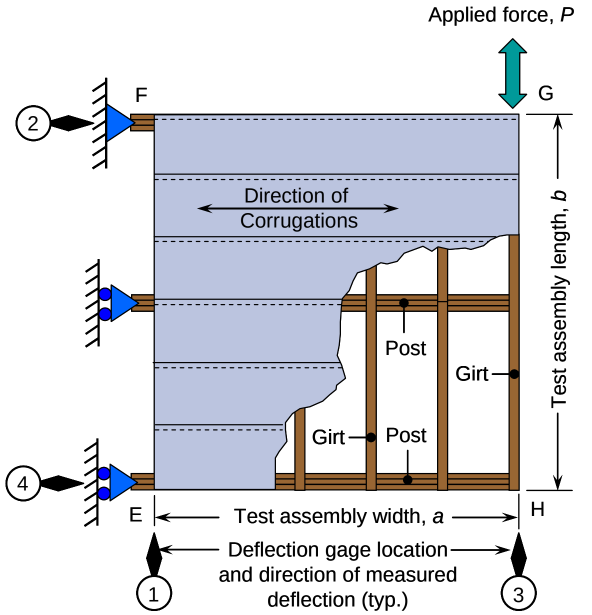


Figure 7-1. Components of a metal-clad wood-frame roof diaphragm.



Notes:
1. The applied forces may alternately be applied at points J and L
2. Locate gages 1, 2, 3 and 4 on the rafters/ truss chords

Figure 7-2. Simple beam test configuration for roof and ceiling diaphragm test assemblies.



Notes:
1. Force P may be alternately applied at point H
2. Locate gages 1, 2 and 4 on the posts
3. Locate gage 3 on the girt

Figure 7-3. Cantilever test configuration for shear wall test assemblies.

7.3.3 Shear Strength

The allowable design shear strength, of a diaphragm test assembly is equal to 40% of the ultimate strength of the assembly. In equation form:

Cantilever test:

$$v_a = 0.40\ P_u / b \qquad (7\text{-}1)$$

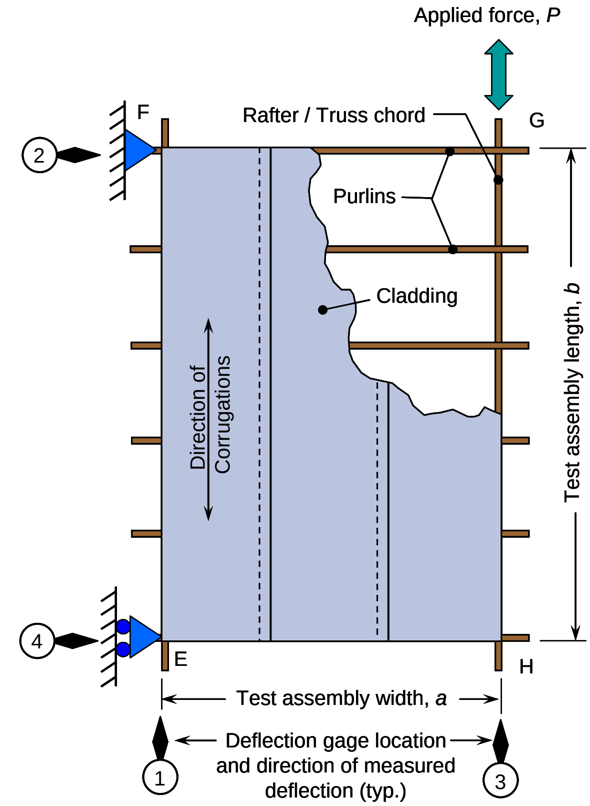
Simple beam test:

$$v_a = 0.40\ P_u / (2b) \qquad (7\text{-}2)$$

where:

v_a = allowable design shear strength, lbf/ft (N/m)
P_u = ultimate strength, lbf (N)
= total applied load at failure
b = assembly length, ft (m) (see figures 7-2, 7-3 and 7-4)

If one or more of the test assembly failures were initiated by lumber breakage or by failure of the fastenings in the wood, then the allowable design shear stress must be adjusted to account for test duration. To adjust from a total elapsed testing time of 10 minutes to a normal ASD load duration of ten years, divide v_a by a factor of 1.6. When this reduction is not applied (as would be the case when test assembly failure is not initiated by wood failure), the NDS load duration factor C_D (or time effect factor λ), can not be used to increase the allowable design shear strength during building design.



Notes:
1. Force P may be alternately applied at point H
2. Locate gages 2 and 4 on the edge purlins
3. Locate gages 1 and 3 on the rafter / truss chord

Figure 7-4. Cantilever test configuration for roof and diaphragm test assemblies.

7.3.4 Shear Stiffness

The procedure for determining the effective shear modulus of a test assembly begins with calculation of the adjusted load-point deflection, D_T. This value takes into account rigid body rotation/translation during assembly test and is calculated as follows:

Cantilever test:

$$D_T = D_3 - D_1 - (a/b)\,(D_2 + D_4) \qquad (7\text{-}3)$$

Simple beam test:

$$D_T = (D_2 + D_3 - D_1 - D_4) / 2 \qquad (7\text{-}4)$$

where:

D_T = adjusted load point deflection, in. (mm)
D_1, D_2, D_3, and D_4 = deflection measurements, in. (mm) (see figures 7-2, 7-3 and 7-4)
a = assembly width, ft (m)

b = assembly length, ft (m)

The effective in-plane shear stiffness, c, for a diaphragm test assembly is defined as the ratio of applied load to adjusted load point deflection at 40% of ultimate load. For a cantilever test:

$$c = 0.4\, P_u / D_{T,d} \tag{7-5}$$

For a simple beam test:

$$c = 0.2\, P_u / D_{T,d} \tag{7-6}$$

where:

c = effective in-plane shear stiffness for the diaphragm test assembly, lbf/in. (N/mm)
$D_{T,d}$ = adjusted load-point deflection, D_T, at 0.4 P_u, in. (mm)

The in-plane shear stiffness for the diaphragm test assembly, c, is converted to an effective shear modulus for the test assembly, G, as:

$$G = c\,(a/b) \tag{7-7}$$

where:

G = effective shear modulus of the test assembly, lbf/in (N/mm)

7.4 Building Diaphragm Properties

7.4.1 General

As described in Chapter 6, each building diaphragm is sectioned for analysis. Each of these sections must be assigned a horizontal stiffness value, c_h, and an allowable load, v_a.

7.4.2 Shear Strength

The allowable design shear strength of a building diaphragm is equal to that calculated for the diaphragm test assembly. Consequently, to calculate the *total* in-plane shear load that a building diaphragm can sustain, simply multiply the allowable design shear strength, v_a, by the slope length of the building diaphragm.

7.4.3 In-Plane Shear Stiffness

The in-plane shear stiffness, c_p, of a building diaphragm section is calculated from the effective shear modulus, G, of the diaphragm test assembly using the following equation:

$$c_p = \frac{G\, b_s}{s} \tag{7-8}$$

or

$$c_p = \frac{G\, b_h}{s \cos(\theta)} \tag{7-9}$$

where:

G = effective shear stiffness of test assembly, lbf/in (N/mm)
b_S = slope length of building diaphragm section being modeled, ft (m)
s = width of the building diaphragm section being modeled, ft (m)
b_h = horizontal span length of building diaphragm section, ft (m)
θ = slope of the building diaphragm section, degrees

Implicit in equation 7-8 is the assumption that the total shear stiffness of a building diaphragm is a linear function of length.

7.4.4 Horizontal Shear Stiffness

The horizontal shear stiffness, c_h, of a building diaphragm section is related to its in-plane shear stiffness as follows:

$$c_h = c_p \cos^2(\theta) \tag{7-10}$$

or

$$c_h = G\, b_h \cos(\theta) / s \tag{7-11}$$

7.5 Building Shearwall Properties

7.5.1 General

The same procedure used to determine the strength and stiffness of building diaphragms is used to determine the strength and stiffness of building shearwalls. That is, representative test assemblies are loaded to failure, to determine their shear strength and stiffness. These properties are then linearly extrapolated to obtain strength and stiffness values for the building shearwall(s).

7.5.2 Shearwall Test Assemblies

ASAE EP558 also contains guidelines for construction and testing of shearwall test assemblies. With the exception of overall length and width, a shearwall test assembly is required to be identical to the shearwall in the building being designed. Specifically, frame members must be of identical size, spacing, species and grade; cladding must be identical; and fastener type and location must be the same.

7.6 Tabulated Data

7.6.1 Sources

Testing replicate samples of diaphragm test assemblies can get expensive. For this reason, a designer may choose not to conduct his/her own diaphragm tests, relying instead on designs that have been previously tested by others. Information on many tested designs is available in the public domain. Cladding manufacturers may have additional test information on assemblies that feature their own products.

7.6.2 Example Tabulated Data

Table 7-1 contains design details and engineering properties for roof diaphragm tests assemblies. Table 7-2 contains design details and engineering properties for shear wall tests assemblies. The information in these tables represents a small percentage of available data.

Table 7-1. MCWF Roof Diaphragm Test Assembly Data

Test Assembly Number	1	2	3	4
Test Configuration	Cantilever	Cantilever	Cantilever	Cantilever
Cladding				
Manufacturer/Trade Name	Wick Agri Panel	Wick Agri Panel	Wick Agri Panel	Midwest Manufacturing.
Base Metal Thickness Gauge	28	28	29	29
Major Rib Spacing, inches	12	12	12	12
Major Rib Height, inches	0.75	0.75	0.75	1.0
Major Rib Base Width, inches	1.25	1.25	1.25	2.5
Major Rib Top Width, inches	0.375	0.375	0.375	0.5
Yield Strength, ksi	50	50	80	80
Overall Design				
Width, feet	9	9	9	6
Length, *b* , feet	12	12	12	12
Purlin Spacing, feet	2	2	2	2
Rafter Spacing, feet	9	9	9	6
Purlin Location	Top running	Top running	Top running	Top running
Purlin Orientation	On edge	On edge	On edge	On edge
Number of Internal Seams	2	2	2	2
Wood Properties				
Purlin Size	2- by 4-inch	2- by 4-inch	2- by 4-inch	2- by 4-inch
Purlin Species and Grade	No.1 & 2 SPF	No.1 & 2 SPF	No.1 & 2 SPF	No.2 SYP
Rafter Species and Grade	No. 1 SYP	No. 1 SYP	No. 1 SYP	No. 1 SYP
Stitch Fastener				
Type	None	Screw	Screw	EZ Seal Nail
Length, inches		1.0	1.0	2.5
Diameter		#10	#10	8d
On Center Spacing, inches		24	24	24
Sheet-to-Purlin Fasteners				
Type	Screw	Screw	Screw	EZ Seal Nail
Length, inches	1.0	1.0	1.0	2.5
Diameter	#10	#10	#10	8d
Location in Field	In Flat	In Flat	In Flat	Major Rib
Location on End	In Flat	In Flat	In Flat	In Flat
Avg. On-Center Spacing in Field, in.	12	12	12	12
Avg. On-Center Spacing on End, in.	6	6	6	12
Purlin-to-Rafter Fastener	60d Threaded Hardened Nail	60d Threaded Hardened Nail	60d Threaded Hardened Nail	60d Threaded Hardened Nail
Engineering Properties				
Ultimate Strength, P_u, lbf.	2140	3390	3220	1930
Allowable Shear Strength, v_a, lbf/ft	71	113	107	64
Effective In-Plane Stiffness, *c* ,lbf/in	1625	2720	2720	1590
Effective Shear Modulus, *G*, lbf/in	1220	2040	2040	795
Reference	Anderson, 1989	Anderson, 1989	Anderson, 1989	Wee & Anderson, 1990

Table 7-1. cont., MCWF Roof Diaphragm Test Assembly Data

Test Assembly Number	5	6	7	8
Test Configuration	Cantilever	Cantilever	Cantilever	Cantilever
Cladding				
Manufacturer/Trade Name	Midwest Manufacturing	Grandrib 3	Grandrib 3	Walters STR-28
Base Metal Thickness Gauge	29	29	29	28
Major Rib Spacing, inches	12	12	12	12
Major Rib Height, inches	1.0	0.75	0.75	0.94
Major Rib Base Width, inches	2.5	1.75	1.75	
Major Rib Top Width, inches	0.5	0.5	0.5	
Yield Strength, ksi	80	80	80	80
Overall Design				
Width, feet	6	9	9	9
Length, *b* , feet	12	12	12	16
Purlin Spacing, feet	2	2	2	2
Rafter Spacing, feet	6	9	9	9
Purlin Location	Top running	Top running	Top running	Top running
Purlin Orientation	On edge	On edge	On edge	On edge
Number of Internal Seams	2	2	2	2
Wood Properties				
Purlin Size	2- by 4-inch	2- by 4-inch	2- by 4-inch	2- by 4-inch
Purlin Species and Grade	No.2 SYP	No.2 DFL	No.2 SPF	No.2 SYP
Rafter Species and Grade	No. 1 SYP	No. 2 DFL	No. 2 SPF	1950f1.7E SYP
Stitch Fastener				
Type	EZ Seal Nail	None	None	Screw
Length, inches	2.5			1.5
Diameter	8d			#10
On Center Spacing, inches	24			24
Sheet-to-Purlin Fasteners				
Type	Screw	Screw	Screw	Screw
Length, inches	0.75	1.0	1.0	1.5
Diameter	#12	#10	#10	#10
Location in Field	In Flat	In Flat	In Flat	In Flat
Location on End	In Flat	In Flat	In Flat	In Flat
Avg. On-Center Spacing in Field, in.	6	12	12	12 and 18
Avg. On-Center Spacing on End, in.	6	6	6	12
Purlin-to-Rafter Fastener	60d Threaded Hardened Nail	1-60d Spike + 2-10d Toenails	1-60d Spike + 2-10d Toenails	60d Threaded Hardened Nail
Engineering Properties				
Ultimate Strength, P_u, lbf.	3995	3300	2775	4884
Allowable Shear Strength, v_a, lbf/ft	133	110	93	122
Effective In-Plane Stiffness, c ,lbf/in	2980	2920	2950	3890
Effective Shear Modulus, G, lbf/in	1490	2190	2210	2190
Reference	Wee & Anderson, 1990	Lukens & Bundy, 1987	Lukens & Bundy, 1987	Bohnhoff and others, 1991

Table 7-1. cont., MCWF Roof Diaphragm Test Assembly Data

Test Assembly Number	9	10	11	12
Test Configuration	Simple Beam			
Cladding				
Type	Regular Leg	Extended Leg	Regular Leg	Extended Leg
Base Metal Thickness Gauge	29			
Major Rib Spacing, inches	9			
Major Rib Height, inches	0.62			
Major Rib Base Width, inches	1.75			
Major Rib Top Width, inches	0.75			
Yield Strength, ksi	80			
Overall Design				
Width, feet	36			
Length, *b* , feet	12			
Purlin Spacing, feet	2			
Rafter Spacing	Pair of rafters every 12 feet (each pair spaced 6 in. apart)			
Purlin Location	Top running and lapped		Inset	
Purlin length, ft	13.2 and 12.0		11.25	
Purlin Attachment	To special blocking nailed between each pair of rafters		To joist hanger attached to rafters	
Purlin Orientation	On edge			
Number of Internal Seams	11			
Wood Properties				
Purlin Size	2- by 6-inch			
Purlin Species and Grade	No.2 DFL and 1650f DFL			
Rafter Species and Grade	No. 2 DFL			
Stitch Fastener*				
Type	None	Screw*	None	Screw*
Length, inches		1.5		1.5
Diameter		#10		#10
On Center Spacing, inches		24		24
Sheet-to-Purlin Fasteners				
Type	Screw			
Length, inches	1.5			
Diameter	#10			
Location in Field	In Flat			
Location on End	In Flat			
Avg. On-Center Spacing in Field, in.	9			
Avg. On–Center Spacing on End, in.	9			
Engineering Properties				
Ultimate Strength, P_u, lbf.	6950	7850	6400	6950
Allowable Shear Strength, v_a, lbf/ft	116	131	107	116
Effective In-Plane Stiffness, c ,lbf/in	4700	7500	3700	4400
Effective Shear Modulus, G, lbf/in	4700	7500	3700	4400
Reference	NFBA, 1996			

* Because of the extended leg, screws installed in the flat at overlapping seams function as stitch fasteners.

Table 7-1. cont., MCWF Roof Diaphragm Test Assembly Data

Test Assembly Number	13	14	15
Test Configuration	Simple Beam	Simple Beam	Simple Beam
Cladding			
Manufacturer/Trade Name	Metal Sales Pro Panel II	Metal Sales Pro Panel II	McElroy Metal Max Rib
Base Metal Thickness Gauge	30	30	29
Major Rib Spacing, inches	9.0	9.0	9.0
Major Rib Height, inches			0.75
Major Rib Base Width, inches			1.75
Major Rib Top Width, inches			
Yield Strength, ksi	104	104	80
Overall Design			
Width, feet	24	24	24
Length, *b* , feet	12	12	12
Purlin Spacing, feet	2.33	2.33	2
Rafter Spacing, feet	Pair of rafters every 12 feet (each pair spaced 6 in. apart)	Pair of rafters every 12 feet (each pair spaced 6 in. apart)	8
Purlin Location	Top running	Top running	Top running
Purlin Orientation	On edge	On edge	NA
Number of Internal Seams	8	8	7
Wood Properties			
Purlin Size	2- by 6-inch	2- by 6-inch	Mac-Girt steel hat section: 1.5 in. tall, 3.2 in. wide, 18 ga.
Purlin Species and Grade	1650f 1.5E SPF	1650f 1.5E SPF	
Rafter Species and Grade	1650f 1.5E SPF	1650f 1.5E SPF	2250f 1.9E SP
Stitch Fastener			
Type	Screw	None	None
Length, inches	0.625		
Diameter	#12		
On Center Spacing, inches	9		
Sheet-to-Purlin Fasteners			
Type	Screw	Screw	Screw
Length, inches	1.5	1.5	1.0
Diameter	#10	#10 in field #14 in ends	#14
Location in Field	In Flat	In Flat	In Flat
Location on End	In Flat	In Flat	In Flat
Avg. On-Center Spacing in Field, in.	9	9	18 (3 screws/sheet)
Avg. On-Center Spacing on End, in.	4.5	4.5	9 (4 screws/sheet)
Purlin-to-Rafter Fastener			Two - #12 x 1.6 in. screws/joint
Engineering Properties			
Ultimate Strength, P_u, lbf.	9600	6600	8645
Allowable Shear Strength, v_a, lbf/ft	160	110	144
Effective In-Plane Stiffness, *c* ,lbf/in	7680	7100	10700
Effective Shear Modulus, *G*, lbf/in	7680	7100	7130
Reference	Townsend, 1992	Townsend, 1992	Myers, 1994

Table 7-1. cont., MCWF Roof Diaphragm Test Assembly Data

Test Assembly Number	16	17	18	19
Test Configuration	Simple Beam			
Cladding				
Manufacturer/Trade Name	Fabral Grandrib 3			
Base Metal Thickness Gauge	29			
Major Rib Spacing, inches	9			
Major Rib Height, inches	0.75			
Major Rib Base Width, inches				
Major Rib Top Width, inches				
Yield Strength (measured), ksi	83			
Overall Design				
Width, feet	24			
Length, b , feet	12			
Purlin Spacing, feet	2			
Rafter Spacing, feet	8 (center bay), 7.88 (outer two bays)			
Purlin Location	Purlins-on-edge with blocking between all purlins. Each block connected with two 60d hardened ring shank nails			Fully recessed in Simpson ST PFD24 and PF24A hangers
Number of Internal Seams	7			
Wood Properties				
Purlin Size	2- by 4-inch			
Purlin Species and Grade	SPF 1650 Fb-1.5E			
Rafter Species and Grade	Doug Fir (N) Select Structural			
Stitch Fastener				
Type	Screw	Screw	Screw	Screw
Length, inches	1.5	0.75	1.5	0.75
Diameter	#12	#12	#12	#12
On Center Spacing, inches	24	8	24	8
Sheet-to-Purlin Fasteners				
Type	Screw			
Length, inches	1.0			
Diameter	#10			
Location in Field	In Flat			
Location on End	In Flat			
Avg. On-Center Spacing in Field, in.	9			
Avg. On-Center Spacing on End, in.	4.5			
Purlin-to-Rafter Fastener	One 60d hardened ring-shank nail			
Tension Chord Reinforcement	Eight 3 in. x 0.131 in. smooth shank nails per lap	Twelve 3 in. x 0.131 in. smooth shank nails per lap	Eight 3 in. x 0.131 in. smooth shank nails per lap	Simpson CS 16, CS14 and CS20 straps with numerous 8d nails
Engineering Properties				
Ultimate Strength, P_u, lbf.	7100	13000	7800	12800
Allowable Shear Strength, v_a, lbf/ft	120	215	130	210
Effective In-Plane Stiffness, c ,lbf/in	9300	16200	9690	9500
Effective Shear Modulus, G, lbf/in	6200	11000	6400	6300
Reference	CMEC, 2013			

Table 7-2. MCWF Shearwall Test Assembly Data

Test Assembly Number	1	2	3	4	5
Test Configuration	Cantilever				
Cladding					
Manufacturer/Trade Name	Fabral Grandrib 3				
Base Metal Thickness Gauge	29				
Major Rib Spacing, inches	9				
Major Rib Height, inches	0.75				
Major Rib Base Width, inches					
Major Rib Top Width, inches					
Yield Strength, ksi	72 (measured, 0.2% offset)				
Overall Design					
Width (wall height), *a*, feet	12				
Length, *b*, feet	16				
Girt/Splash Plank Spacing, feet	3	2			3
Post Spacing, feet	8				
Girt/Splash Plank Type	Exterior				
Post Type	3-ply, spliced, nail-laminated				
Blocking	None		Between girts on outer posts	None	Between girts on outer posts
Number of Internal Seams	5				
Wood Properties					
Girts	2- by 4-inch, 1650 Fb-1.5E Spruce-Pine-Fir				
Splash Plank	2- by 8-inch, No.2, PPT (Pressure Preservative Treated), Hem Fir				
Blocking	NA		Same as girts	NA	Same as girts
Posts	Base: 2x6 in., No.2 PPT Hem Fir. Top: 2x6 in., SS Doug Fir-Larch				
Stitch Fastener					
Type	None		Screw		
Length, inches	NA		0.75		
Diameter	NA		#12		
On Center Spacing, inches	NA		8	24	18
Sheet-to-Girt Fasteners					
Type	Screw				
Length, inches	1.0				
Diameter	#10				
Avg. On-Center Spacing in Field, in.	7.2	9			
Avg. On-Center Spacing on End, in.	4.5				
Girt-to-Post Fasteners	Three 3.5 in. x 0.162 in. diameter ring shank nails (galvanized in PPT lumber) in each girt end and in each post-girt lap. Same for splash plank.				
Blocking-to-Post Fasteners	NA		Four 3.5 in. x 0.162 in. dia. ring shank nails per block		
Engineering Properties					
Ultimate Strength, P_u, lbf.	4020	5810	9650	5700	5880
Allowable Shear Strength, v_a, lbf/ft	100	145	240	140	145
Effective In-Plane Stiffness, *c* ,lbf/in	7400	9500	19000	10100	14500
Effective Shear Modulus, *G*, lbf/in	5500	7100	14000	7600	11000
Reference	CMEC (2012)				

7.7 Example Calculations

Problem Statement

A designer wishes to find c_h and v_a for roof diaphragm sections in a gable-roofed building with roof slopes of 4-in-12. Distance between eaves is 36 feet, and post-frame spacing, s, is 10 feet.

A cantilever test of a representative diaphragm test assembly with a width, a, of 10 feet and a length, b, of 12 feet, yields an ultimate strength, P_u of 3900 lbf and an effective in-plane stiffness, c, of 4000 lbf/in. The test assembly failure was not wood related, therefore the ultimate strength was not adjusted for load duration.

Solution

Equation 7-1: v_a (test assembly) = 0.40 P_u / b

v_a = 0.40 (3900 lbf) /12 ft = 130 lbf/ft

Equation 7-7: $G = c\ (a/b)$

G = (4000 lbf/in) (10 ft/12 ft) = 3333 lbf/in.

Equation 7-11: $c_h = G\ b_h \cos(\theta) / s$

c_h = (3333 lbf/in) (36 ft / 2) (cos 18.4°) / 10 ft

c_h = 5690 lbf/in.

The horizontal stiffness, c_h of 5690 lbf/in represents a single diaphragm section that runs from eave to ridge and has a width of 10 feet.

v_a (diaphragm) = v_a (test diaphragm)

v_a (diaphragm) = 130 lbf/ft

7.8 References

7.8.1 Non-Normative References

Anderson, G.A. (1989) Effect of fasteners on the stiffness and strength of timber-framed metal-clad roof sections. ASAE Paper No. MCR89-501. ASAE, St. Joseph, MI. www.asabe.org.

Bohnhoff, D. R., Anderson, G. A. & Boor, P.A. (1991). Influence of insulation on the behavior of steel-clad wood frame diaphragms. *Applied Engineering in Agriculture* 7(6):748-754.

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Myers, N.C. (1994). McElroy Metal Post-Frame Roof Diaphragm Test. Test Report 94-418. Progressive Engineering, Inc., Goshen, IN.

NFBA. (1996). 1996 diaphragm test. National Frame Building Association (NFBA), Chicago, IL. www.nfba.org.

Townsend, M. (1992). Alumax test report: diaphragm loading on roofs and end wall sections. Alumax Building Products, Perris, CA.

Wee, C.L. & Anderson, G.A. (1990). Strength and stiffness of metal clad roof section. ASAE Paper No. 904029. ASABE, St. Joseph, MI. www.asabe.org.

7.8.2 Normative References

ANSI/ASAE EP484.2 *Diaphragm design of metal-clad, wood-frame rectangular buildings.*

ASAE EP558 *Load tests for metal-clad wood-frame diaphragms*

CHAPTER 8

Post Design

Contents

8.1 Introduction

Wood posts are the identifying characteristic of a post-frame building system, and thus post-frame building design is largely centered around post design.

A designer has many different post types from which to choose. These different types were introduced in Chapter 1. Section 8.2 repeats and expounds on these definitions.

Section 8.2 is followed by six sections intended to assist designers in post type selection. These six sections address relative cost, preservative treatment, corrosion potential, bending characteristics, structural framing requirements and options, and thermal requirements. Given that selection criteria can change throughout a building, it is not uncommon for more than one post type to be featured in the same building.

Section 8.9 covers determination of post forces; Section 8.10 contains reference design values for different post types; Section 8.11 addresses adjustment factors for reference design values; Section 8.12 overviews controlling design equations; and Section 8.13 provides example post design calculations.

8.2 Post Definitions

Posts are defined by basic type, direction of applied bending load, base support, and where they are positioned within a building.

8.2.1 Basic Post Types

Basic wood post types include solid-sawn, structural composite lumber, glue-laminated, mechanically-laminated and poles. These post types were defined in Section 1.2.3 as:

- **Solid-sawn post:** Post comprised of a single piece of sawn lumber.
- **Structural composite lumber post (SCL post):** Post comprised of a single piece of structural composite lumber. Structural composite lumber (SCL) includes, but is not limited to: parallel strand lumber (PSL), laminated veneer lumber (LVL), and laminated strand lumber (LSL).

- **Glued-laminated post (or glulam post):** Post consisting of suitably selected sawn lumber laminations joined with a structural adhesive.
- **Mechanically-laminated post (or mechlam post):** Post consisting of suitably selected sawn lumber laminations or structural composite lumber (SCL) laminations joined with nails, screws, bolts, and/or other mechanical fasteners.
- **Pole:** A round, naturally tapered, unsawn, wood post. Poles are sometimes slabbed to aid in attaching other framing members to the pole.

Mechlam posts are further categorized by the type of mechanical fastener used to join individual wood layers. **Nail-laminated** (**or nail-lam**) posts are mechlams that only use nails, and **screw-laminated** (**or screw-lam**) posts are mechlams that only use screws. Although not covered in Chapter 1, a mechanically laminated post can also be formed by bolting together individual wood layers.

Individual layers in glulam and mechlam posts often contain end joints. Such end joints can have a significant impact on assembly behavior and thus require special design consideration. An **unspliced** post is one in which all laminations behave as unspliced members. A lamination behaves as an unspliced member when it does not contain any end joints, or when the end joints it contains are *certified structural glued end joints*. A **spliced** post is one in which individual laminations are fabricated by end-joining shorter wood members without the use of certified structural glued end joints. These end joints are generally either unreinforced butt joints, mechanically-reinforced butt joints, or they are glued scarf joints and glued finger joints that do not meet the strength criteria to be designated as certified structural glued end joints.

8.2.2 Direction of Loading

Post bending strength must frequently be calculated for bending about one or both post axes. Depending upon which axis a post is being bent for the calculation at hand, one of the following definitions will apply.

Plank Loading: An LVL post with a bending load applied perpendicular to individual plies (figure 8-1(a)), or an LSL or PSL post with a bending load applied perpendicular to the wide faces of strands.

Beam Loading: An LVL post with a bending load applied parallel to individual plies (figure 8-1(b)), or an LSL or PSL post with a bending load applied parallel the wide faces of strands.

Horizontally-Laminated Post: A mechlam or glulam post primarily designed to resist bending loads applied perpendicular (or normal) to the interlayer planes (figure 8-1(c)).

Vertically-Laminated Post: A mechlam or glulam post primarily designed to resist bending loads applied parallel to the planes of contact between the individual layers (figure 8-1(d)).

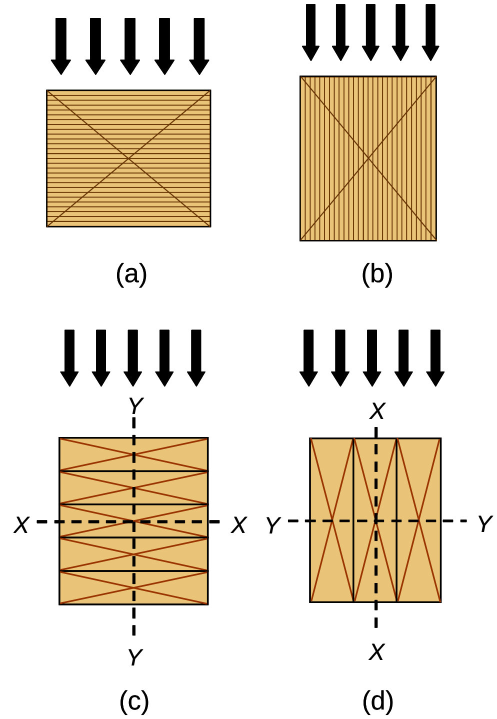


Figure 8-1. (a) plank loading, (b) beam loading, (c) horizontally-laminated, (d) vertically laminated.

8.2.3 Base Support

Wood posts anchored in the soil are defined as **embedded** posts, whereas those supported by concrete piers, slabs and walls are defined as **non-embedded** posts. Embedded posts are further classified as constrained and non-constrained. A **constrained** embedded post is one that is restrained from significant horizontal movement at or near the ground surface, typically by a concrete slab, whereas a **non-constrained** embedded post has no such lateral restraint at or near the ground surface.

8.2.4 Post Location

As defined in Section 1.2.3, a post is frequently defined by its location within a structural frame as either an **endwall**, **sidewall**, **corner**, **jamb** or **interior** post. Post location generally dictates boundary conditions relating to lateral support and post end fixity.

8.3 Relative Cost

Proper cost comparison of posts requires that material, fabrication, storage, handling, transportation, and erection costs be considered.

8.3.1 Material Costs

With respect to *material* costs, the most expensive posts are SCL posts and posts laminated using SCL materials. The relative high cost of SCL can be attributed to the cost of SCL manufacturing facilities, as well as the limited number of such facilities which increase the cost of transporting material to and from the facilities.

The total material cost associated with glulam and mechlam posts fabricated from dimension lumber is almost always less than that of solid-sawn posts, even when the cost of adhesive, fasteners and joint reinforcing is included. This is because laminated posts are pieced together from smaller, shorter, less expensive dimension lumber. As laminated post length increases, material cost per foot of post stays relatively constant. Conversely, solid-sawn posts become increasingly more expensive (on a unit length basis) in lengths over 16 ft (4.9 m). Additional savings in material costs, specifically in wood preservative, may be realized if spliced posts are used when only one post end requires treatment.

8.3.2 Fabrication Costs

Whether or not it is feasible to use glulam and mechlam posts is largely dependent on labor and equipment costs associated with post fabrication. Glulam post assembly involves planing operations to prepare surfaces for gluing and then to remove excess glue. Equipment for clamping layers together is essential in glulam post fabrication and is often used in the fabrication of mechlam posts. If metal plate connectors will be used to reinforce end joints in mechlam posts, equipment with the capacity to embed the plates is also required. The initial cost of equipment is highly dependent on the degree to which the assembly process will be automated. Machines specifically built to manufacture nail-laminated posts are in use.

8.3.3 Miscellaneous Costs

Costs associated with storage, handling, transportation, and erection influence post selection to a lesser degree. Maintaining a large inventory of SCL or solid-sawn posts in a variety of lengths adds to overhead costs. Thus builders who predominately use laminated posts are able to reduce their inventory of posts.

8.4 Preservative Treatment

8.4.1 Requirements

Any post or portion thereof that is in ground contact or in freshwater should be pressure preservative-treated in accordance with AWPA U1 Use Category UC4B or better. ANSI/ASAE EP559 requires that for mechlam posts, Use Category UC4B or better treatment extend a minimum of 16 inches (40 cm) above the ground or waterline. Posts that are located above ground, but are exposed to all weather cycles, including prolonged wetting, should be treated in accordance with AWPA Use Category UC4A ore better.

Retention levels for Use Categories UC4A and UC4B are provided in Section 6 of AWPA U1. Section 6 categorizes retention levels by commodity. AWPA U1 *Commodity Specification A* covers solid-sawn posts and post fabricated from dimension lumber; AWPA U1 *Commodity Specification F* covers posts comprised of structural composite lumber.

AWPA U1 retention levels are considered minimums and are tabulated in units of lbm/ft^3 and kg/m^3. In addition to Use Category, minimum retention levels are a function of preservative type. In some cases, wood species and component size can also influence the minimum required retention level.

As an example of the difference between Use Categories UC4A and UC4B, the minimum required retention level for CCA-treated dimension lumber of any wood species is 0.40 lbm/ft^3 for Use Category UC4A, and 0.60 lbm/ft^3 for Use Category UC4B.

Because use of certain treatments in certain SCL materials is associated with a reduction in design strengths, consult manufacturer's literature for appropriate treatment adjustment factors.

8.4.2 Uniformity of Treatment

Uniformity of treatment is dependent on wood species, presence of heartwood, and incising. Although incising is associated with a reduction in strength, it is commonly used prior to treatment of non-Southern Pine species. With or without incising, preservative penetration depth is limited in all species. For this reason, it is not uncommon for the center of a large, solid-sawn post to be void of preservative (figure 8-2(b)). This becomes problematic when the untreated center is exposed by drilling, sawing, or the formation of a primary check. Since thinner lumber will be more uniformly treated through its entire cross-section, use of laminated posts fabricated from layers of preservative treated lumber (figure 8-2(a)) minimizes the situation illustrated in figure 8-2(b). For this reason, individual laminations of a treated glulam post are generally preservative-treated before they are glued together.

8.4.3 Treating Option for Spliced Posts

Laminated posts in which each layer consists of treated wood that has been end-jointed to non-treated wood can be used where only one post end requires preservative treatment (frequently the case with embedded posts).

Although this practice saves preservative treatment, the cost savings of using less preservative treated wood must be weighed against: (1) the additional manufacturing costs associated with splicing, and (2) the reduction in post strength and stiffness that may be required depending on the type of end joints used.

(a) (b)

Figure 8-2. (a) Cross section of a glulam post with preservative that penetrates 100% of each piece of dimension lumber, and (b) cross section of a nominal 6- by 6-inch member with poor envelope penetration of preservative.

8.5 Corrosion Potential

The environment in which a post is placed will dictate the extent to which corrosion potential will influence material selection.

8.5.1 Fasteners Located Above Grade

ANSI/ASAE EP559 requires that mechanical fasteners used above grade to join waterborne preservative-treated lumber be of AISI type 304 or 316 stainless steel, silicon bronze, or copper, or contain a coating applied in accordance with the treated wood or fastener manufacturer recommendations for AWPA Use Category UC4A treatment levels for sawn lumber products. In the absence of manufacturer's recommendations, a minimum ASTM A153 Class D zinc coating (average 1.00 ounce per square foot) or equivalent, must be used.

8.5.2 Fasteners Located Below Grade

ANSI/ASAE EP 559 also requires AISI type 304 or 316 stainless steel mechanical fasteners below grade to assure compatibility of deformation between treated laminates.

8.5.3 Highly Corrosive Environments

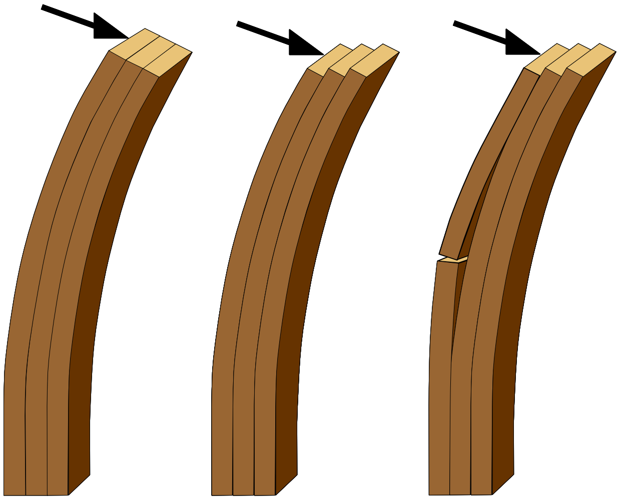
In environments that are especially corrosive, it is often best to avoid use of mechlam posts. This may include for example: moist, unvented manure storage areas; salt storage structures; and certain water treatment facilities.

8.6 Bending Characteristics

8.6.1 Uni- Versus Biaxial Bending Strength

Post selection is largely dictated by post bending properties. In some cases, post bending strength and stiffness about a single axis is important. For other applications, post bending strength about both axes is important.

Solid-sawn, SCL and glulam posts that are square will have similar bending properties about both axes. This is not the case for mechlam posts, and thus mechlam posts are typically not an optimal choice where an application requires similar bending strength about both posts axes. This is frequently the case with a post that is not laterally supported in any direction between its base and its top.



(a) (b) (c)

Figure 8-3. Posts bent about axis X-X showing: (a) interlayer rigidity of a horizontally glued laminated post, (b) interlayer slip of a horizontally mechanically laminated post, (c) delamination of a horizontally nail-laminated post

8.6.2 Effect of Interlayer Slip

The significantly different bending properties that a mechlam post can exhibit under strong and weak axis bending (when compared to a glulam post) are attributable to interlayer slip. If an unspliced mechlam is subjected to bending about axis Y-Y (figure 8-1(d)), there is negligible slip between layers. If the same unspliced post is subject to bending about axis X-X (figure 8-1(c)), there often is considerable slip between layers (figure 8-3(b)). In some mechlam posts, this slip can be so large that for all practical purposes, the individual layers act independently to resist the applied loads. For this reason, mechanically laminated posts are oriented and designed to resist the highest bending moments in bending about axis Y-Y. When this is done, the posts are classified as vertically laminated assemblies.

8.6.3 Effect of End Joint Type

The bending strength of a spliced mechlam post about the Y-Y axis is highly dependent on the type of end joints used in the assembly. When end joints are structural glued end joints, the post behaves and can be treated as an unspliced post (i.e., a post that does not contain end joints) as shown in figure 8-4(a). At the other end of the spectrum are spliced mechlams with unreinforced butt joints. Such joints increase interlayer slip and significantly reduce post bending strength in the vicinity of the joints (figure 8-4(b)). In fact, the bending strength about axis Y-Y in the splice region of a mechlam post with unreinforced butt joints will typically be less than one-half the bending strength about axis Y-Y in the unspliced regions of the same post. This reduction due to splicing is even worse if the splice region is not laterally supported when the post is bent about the Y-Y axis. Without lateral support, lateral torsional buckling will increase the likelihood of delamination (figure 8-3(c)), especially in nail-laminated assemblies.

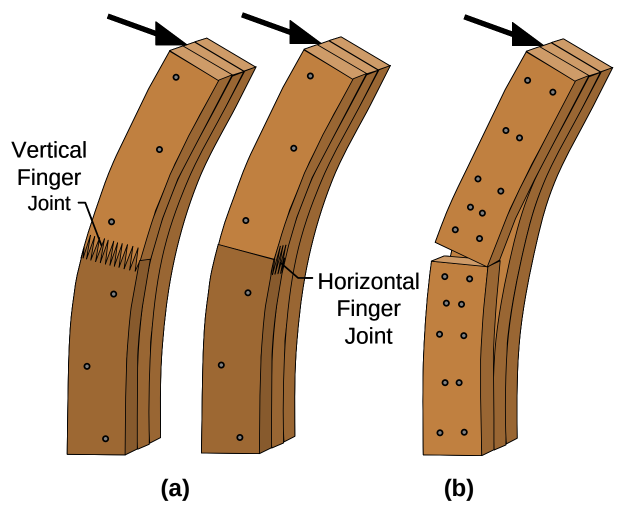


Figure 8-4. Mechanically-laminated posts bent about axis Y-Y showing: (a) negligible interlayer slip at a glued finger joint, and (b) significant interlayer slip and a higher required fastener density at an unreinforced butt joint.

8.6.3.1 Butt-Joint Reinforcement

In between unspliced mechlams and spliced mechlams with unreinforced butt joints, are spliced mechlams with butt-joint reinforcement. The main objective when designing butt joint reinforcement is to obtain a significant increase in bending strength about axis Y-Y without substantially increasing assembly cost. Because of this, the cost of adding reinforcement must always be considered along with the relative increase in post strength. Reinforcing in most laminated posts consist of 16 to 24 gauge: steel sheets, nail plates, or metal plate connectors. This type of joint reinforcing would be considered "light". Heavier reinforcement is generally not cost effective.

8.6.4 Relative Bending Strength

If solid-sawn, SCL, glulam, unspliced mechlam, and spliced mechlam posts of equal size were fabricated using material from the same log and then compared on the basis of bending strength about their strongest axis, the rating (from strongest to weakest) would be:

1. SCL and glulam
2. Unspliced mechlam and spliced mechlam with certified structural glued end joints
3. Solid-sawn
4. Spliced mechlam without certified structural glued end joints

If the same posts were compared on the basis of bending about their weak axis, the rating (from strongest to weakest) would be:

1. SCL and glulam
2. Solid-sawn
3. Unspliced mechlam and spliced mechlam with certified structural glued end joints
4. Spliced mechlam without certified structural glued end joints

The main advantage that SCL, mechlam and glulam posts have over solid-sawn posts is that they have more uniform strength and stiffness properties. This is because laminating spreads out natural and seasoning defects and, consequently, they are not concentrated in a particular area to the extent that they are in solid-sawn posts. This characteristic translates into greater reliability and therefore higher design stresses are justified for SCL and glulam posts.

8.6.5 Face Plates

Where bending strength of a mechlam about the X-X axis is important, or where a mechlam requires greater stability for bending about the Y-Y axis, face plates can be added to the post as shown in figure 8-5. In many cases this is a less expensive option than switching to a solid-sawn, SCL or glulam post.

In most cases, face plates must extend the full length of the post (Bohnhoff and Gadani, 2002) to be effective. Face plates must also be properly attached. Typical attachment consists of 12d to 16d nails driven into every mechlam layer on a one- to two-foot spacing.

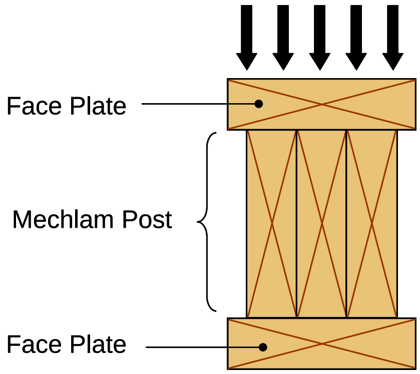


Figure 8-5. Face plates added to a mechlam post to increase bending strength about the X-X axis and lateral stability for bending about the Y-Y axis.

8.6.6 I-Posts

An excellent post for applications where bi-axial bending strength is important, is an I-section fabricated by attaching dimension lumber flanges to a laminated strand lumber (LSL) web with screws and polyurethane adhesive (figure 8-6). Testing by Holstein and Bohnhoff (2013) has shown that the combination of screws and polyurethane adhesive produces an assembly that exhibits near complete composite action. Use of an LSL web helps insure a straight, non-twisted post, and provides a flat and dry, oil-free surface for adhesive bonding. Screws serve three purposes; they provide interlayer shear strength, flanges tight to the web, and hold the assembly together while the adhesive cures, and thus enable on-site fabrication of the assemblies. In exterior walls, I-shaped sections provide more room for insulation, and are associated with reduced thermal bridging. When placed in an interior space, the area between the flanges provides a space for electrical, plumbing and ductwork runs.

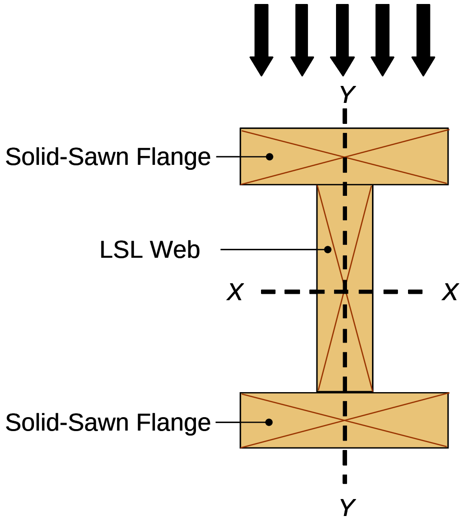


Figure 8-6. I-section post.

8.6.7 Steel Columns

Where tall interior columns without lateral support are required or desired (e.g., interior roof supports for some dairy freestall barns), many designers have used round or square steel tubes in lieu of wood posts. In some cases the decision is based on wood versus steel prices. In other cases the decision is based on overall weight/size.

It is not uncommon for steel columns and wood posts to be used in the same post-frame.

8.7 Structural Framing Requirements and Options

Structural framing requirements and options that influence post selection include such variables as total post length, post end fixity, laterally unsupported length, and girt and bracing attachment.

8.7.1 Total Post Length

Total post length influences post cost and often limits post type options. Solid-sawn posts and dimension lumber become increasingly expensive on a board-foot basis as overall length increases. Additionally, solid-sawn posts and dimension lumber are typically unavailable in lengths over 24 feet. Probably the greatest advantage of splicing and laminating lumber (i.e. fabricating mechlams and glulams with end joints) is that continuous members of any length can be built. This has enabled the construction of buildings requiring sidewall posts upward of 30 ft and endwall posts of even greater length.

Total post length is largely dictated by eave height and post end fixity. Post end fixity refers to the manner in which the bottom of the post is held in place and how the top of the post is attached to a truss or girder. Embedding a post typically adds 4 to 5 feet to overall post length. Extending a post to the top chord of a truss or girder is commonly done to add stability and rigidity to a post-to-truss/girder connection, thereby increasing total post length.

8.7.2 Mechlam-to-Truss/Rafter Attachment

Mechlam posts provide more, and in many cases, better truss/rafter attachment options. First and foremost, a truss/rafter can be sandwiched between the outer layers of a mechlam post. This is accomplished by leaving out a portion of the layer upon which the truss/rafter will bear. If the post is embedded, the exact height at which the truss/rafter will bear on the post can not be determined until the post is fixed in place. Once the post has been fixed in place, the location of the bottom of the truss/rafter is marked on the post, thereby enabling an accurate determination of the length of the missing piece upon which the truss will bear. This piece only functions as blocking and should not be more than one to two feet in length. This process is illustrated in figure 8-7 for a three-layer post used to support the end of a truss. The same method is frequently used with double trusses and four-layer mechlams. Gaps are reduced by setting the truss/rafter on the block, using C-clamps to draw the outside layers tight to the truss/rafter and block, and then fastening the truss/rafter and block in place.

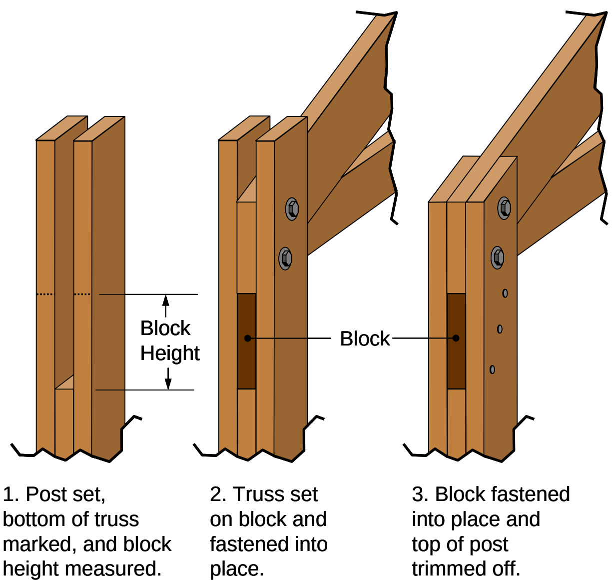


Figure 8-7. Assembly procedure that enables a truss/rafter to be sandwiched between the outer layers of a mechlam post whose height can not be adjusted because it has been embedded in the soil.

The primary advantage of sandwiching a truss/rafter between outer posts layers (instead of resting it on an outside layer or on attached blocking) is that the fasteners used to attach the truss/rafter to the post can be placed in double shear. Relative to other attachment methods, sandwiching a truss/rafter between outer post layers tends to make it more difficult to rotate the truss/rafter off the post, and it reduces weak-axis bending moments induced in the post by vertically- and eccentrically-applied truss loads.

A disadvantage of sandwiching a truss between outer post layers is that bolt holes can not be drilled in the truss prior to placement. This has become less of an issue with the increased use of self-drilling screws for post-to-truss connections. Predrilling of bolt holes in trusses is an option anytime a truss/rafter is placed (1) on the outer layer of a mechlam post, (2) in a notch cut into an SCL, glulam or solid-sawn post, or (3) on a bearing block attached to the side of the post.

8.7.3 Onsite Splicing

An option with mechlam posts is that they can be entirely or partially fabricated at the job site. In applications where spliced posts are being embedded, the lower (treated) portion of the post can be fabricated and set in place before the top half is assembled. This has two advantages. First, it is often easier to set only the lower part of the post. Second, the length of the bottom portion of each post can be adjusted prior to fabrication to account for differing elevations of the embedded portions (figure 8-8). This procedure eliminates the need for additional adjustments for truss attachment after the top of the post has been erected (e.g., blocking as illustrated in figure 8-7 is no longer needed).

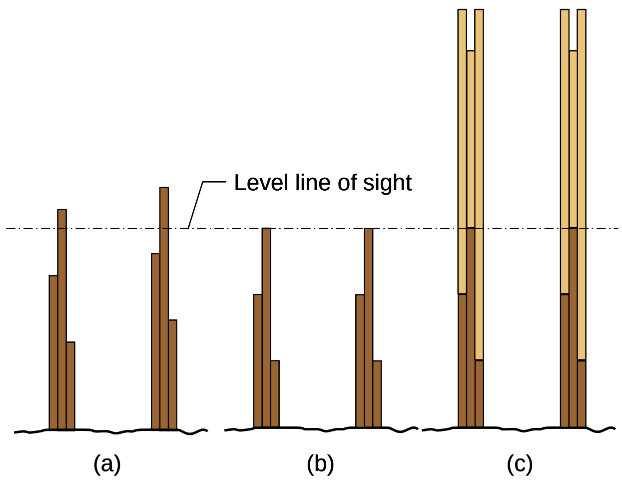


Figure 8-8. (a) Treated portions of 3-layer spliced posts are embedded in the soil. (b) Top of treated portions cut so that tops at same elevation. (c) Untreated post portions spliced to treated portions.

Figure 8-9 illustrates an alternative application of the concept shown in figure 8-8. In this case, the untreated upper portion of the post is an I-section. The embedded "stub post" is a 3-layer preservative-treated mechlam. In this design, the web of the I-section rests on the shorter, middle ply of the stub post or on a short block that is placed on the middle ply to increase the height of the I-section. The outer plies of the stub posts are fastened to both the web and flanges of the I-section.

8.7.4 Lateral Support

Since lateral support dictates effective buckling length and hence load capacity, the ability to provide lateral support often controls post selection. When a post is used in an application where it is free standing (e.g., not located within a wall) and is not laterally supported in any manner, it usually requires similar bending strength about both axes. Use of lateral support may be restricted where it interferes with building use (e.g. forklift traffic, pallet racks), interior finishes, and/or mechanical systems.

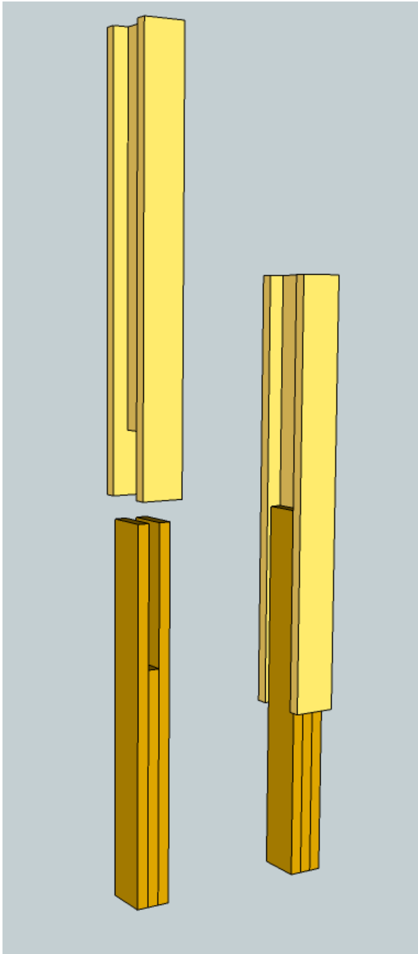


Figure 8-9. Use of a thee-layer preservative treated stub post to support a wood I-section.

8.7.5 Girt Attachment

Any post type can be used in a building that only requires exterior girts. If interior girts are added to facilitate an interior finish, then a post with constant (not tapered) cross-section is generally desired. In applications where both exterior and interior sheathing materials will be attached to inset girts, it is desirable for inset girt depth and post depth to match. For this reason, it should be noted that the depth of many vertically-laminated glulam posts is slightly less than that of the dimension lumber from which they were fabricated because of planing operations conducted after the pieces have been glued together.

8.7.6 Tongued Columns

A number of older post-frame buildings were constructed using vertically mechanically laminated posts featuring nominal 2- by 8-inch *inner* layers with nominal 2- by 6-inch *outer* layers, or with nominal 2- by 10-inch *inner* layers with nominal 2- by 8-inch *outer* layers. This was primarily done to facilitate girt attachment and post spacing during construction. Girts were cut so that they would fit between the tongues of adjacent posts as shown in figure 8-10. Such posts, are commonly referred to as tongued columns.

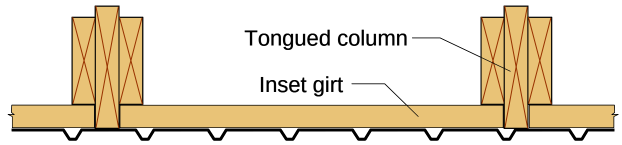


Figure 8-10. Use of a tongued column to facilitate use of an inset girt. Tongued columns are not recommended as they use wood inefficiently.

Tongued columns do not use wood very efficiently and are not recommended. In fact, if you have a mechlam post fabricated from three 2- by 6-inch pieces of lumber and you replace the middle layer with a 2- by 8-inch member of the same grade and species of wood, you will likely obtain a post with a lower overall bending strength. This is because the 2- by 8-inch member is stiffer than the 2- by 6-inch members and thus will fracture before the 2- by 6-inch members are loaded to capacity. Once the 2- by 8-inch member fractures, you are effectively left with a 2-layer post that is more likely to have a lower bending strength that a post fabricated from three 2- by 6-inch pieces.

8.7.7 Quality Control in Manufacturing

There is a need for quality control during fabrication of laminated posts. Inconsistencies in the placement of fasteners in mechanically laminated posts, or poor surface preparation and glued joints in glued-laminated assemblies, can substantially reduce post strength. Quality control of mechlams is more challenging when fabrication is done at the job site. Glulam posts must be fabricated in an AITC or EWP-APA certified facility.

Manufacturing requirements for mechlam posts are provided in ANSI/ASAE EP599. Requirements for the manufacture and quality control of glulam posts are contained in ANSI/AITC A190.1 *Standard for Wood Products - Structural Glued Laminated Timber.*

8.8 Thermal Considerations

A thermal bridge is any component in an insulated assembly that (1) has a higher thermal conductivity than the insulation, and (2) effectively short circuits the insulation. A more efficient thermal envelope is obtained by selecting a post that minimizes air infiltration and/or thermal bridging. Air infiltration through posts is typically only a concern with mechlam posts. This concern is minimized by placing caulk or a construction adhesive between post layers.

It is advantageous to minimize post width where minimization of heat transfer is a primary objective.

From purely a minimization of heat transfer perspective, an I-section assembly (figures 8-6 and 8-9) is the best post option.

8.9 Post Analysis

Post analysis refers to determination of the bending moments, shear forces and axial forces induced in a post by the applied structural loads. This is accomplished with the aid of (1) a plane-frame structural analysis program, or (2) direct application of equations of static equilibrium. These two methods are overviewed in Sections 8.9.1 and 8.9.2, respectively.

8.9.1 Post Forces by Plane-Frame Structural Analysis

The simplest and most direct way to determine the forces induced in posts by applied structural loads is to conduct a plane-frame structural analysis. The frame analog used for this analysis is very similar to that used to determine eave load, *R* (see Section 6.5.2 and figure 6-17). The only difference is that the vertical roller support used to determine *R* is replaced by the total sidesway restraining force, *Q* (figure 8-11(a)). Alternatively, the vertical roller support can be replaced by a series of distributed forces applied to the frame such that they align with the plane of each diaphragm (figure 8-11(b)).

Restraining force *Q* accounts for the resistance to movement provided by diaphragm action, and thus equals zero (0) for a frame that does not receive diaphragm support. Various methods for determining *Q* are covered in Section 6.6.6.

Of the two options presented in figure 8-11, application of the single force *Q* is easier to implement, but would be considered less accurate than spreading the load out as shown in figure 8-11(b). In general, the biggest relative difference between the two analogs in figure 8-11 would be in predictions of post axial forces.

Force *Q* can be converted to a series of distributed in-plane forces ($q_{p,i}$ values) with the following equation which is based on equations 6-23, 6-24 and 6-25.

$$q_{p,i} = Q\,(c_{h,i} / C_h) / b_i \qquad (8\text{-}1)$$

where:

$q_{p,i}$ = in-plane force applied to the frame per unit length of diaphragm section *i*, lbf/ft (N/m)

Q = total sidesway resisting force acting on the frame, lbf (N)

$c_{h,i}$ = horizontal shear stiffness of diaphragm section *i* with width *s*, lbf/in. (N/mm)

C_h = horizontal shear stiffness for a width *s* of the roof/ceiling assembly, lbf/in. (N/mm)

b_i = horizontal span (distance measured parallel to the frame) of diaphragm section *i*, ft (m)

Forces are generally only determined in posts that are part of the critical post-frame (i.e., the post-frame with the greatest horizontal eave displacement). This frame should be evaluated for each applicable load combination from ANSI/ASCE 7-10 (Section 3.5).

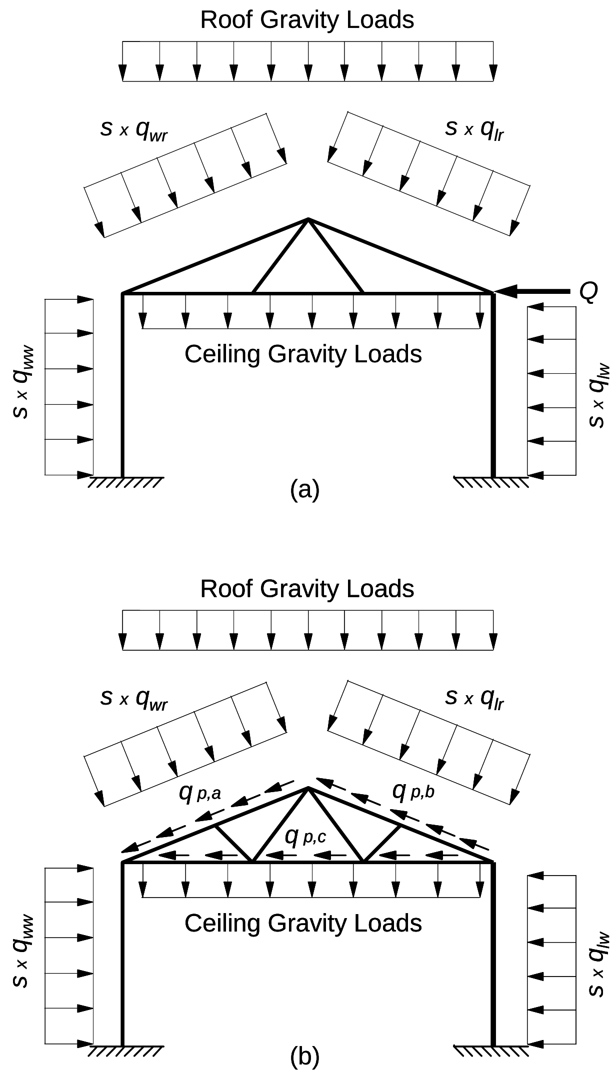


Figure 8-11. Example structural analogs for determination of post forces using (a) a single sidesway restraining force, and (b) uniformly distributed in-plane forces to model the resisting influence of diaphragms.

8.9.2 Post Forces by Equations of Static Equilibrium

Determination of post forces via application of static equilibrium equations is a two step process:

Step 1: Determine post shear forces and post bending moments for all posts in the critical frame. This step requires knowledge of (1) the load(s) applied to the side of each post, and (2) the horizontal eave displacement (from diaphragm analysis) of the critical post-frame.

Step 2: Determine axial post forces. This step requires knowledge of (1) all loads acting on the critical post-frame, and (2) shear forces and bending moments (from Step 1) in each post of the critical frame.

8.9.2.1 Step 1: Determination of Shear Force and Bending Moment

Table 8-1 contains equations for calculating shear forces and bending moments at any above-grade location in a post. Equations are provided for the eight post fixity cases illustrated in figure 8-12. The last four of these cases are applicable to posts embedded in soil whose modulus of elasticity increases linearly with depth. Equations apply only to posts with a constant flexural rigidity *EI*. The embedded post equations assume that the below-grade portion of the posts has infinite flexural rigidity *EI*.

In addition to post and soil properties, application of the Table 8-1 equations requires knowledge of the wind pressure *w* tributary to the post (generally equal to zero for posts that are not part of an exterior wall), frame spacing *s*, and the horizontal eave displacement Δ of the critical post-frame. In all cases, the wind load is assumed to be uniformly distributed over the entire height *H*.

Figure 8-13 illustrates positive sign conventions for the variables defined and used in Table 8-1. The important rule of thumb to follow is that things moving in the same direction have the same sign. Thus if wind pressure is acting in the same direction as the horizontal displacement of the top of the post, then both *w* and Δ must be input with the same sign.

Calculation of shear forces and bending moments for embedded posts requires that the rotation of the post at grade θ_b first be calculated. For non-constrained embedded posts, this rotation calculation must be followed with a calculation of the horizontal movement of the post at grade Δ_b. All calculations associated with embedded posts require calculation of the dimensionless variable *C* which is a measure of soil stiffness relative to post bending stiffness. When calculating *C* it is important to use a consistent set of units so the resulting value is indeed dimensionless.

No truss/rafter is capable of completely preventing rotation of the top of a post. For this reason, Table 8-1 equations for fixity cases 2, 4, 6 and 8 will produce results that may significantly differ from those obtained from a plane-frame structural analysis (PFSA) of the entire post-frame. Conversely, when posts are assumed to be pin-connected to a truss/rafter (fixity cases 1, 3, 5 and 7), Table 8-1 equations will produce values nearly identical to those obtained from a PFSA of the entire post-frame. The one exception is that for PFSA output to match values obtained using Table 8-1 equations for fixity cases 5 and 7, the embedded portion of the posts would have to be modeled with completely rigid links. The error introduced by assuming an infinitely rigid post below grade was demonstrated and discussed in Section 5.3.5.

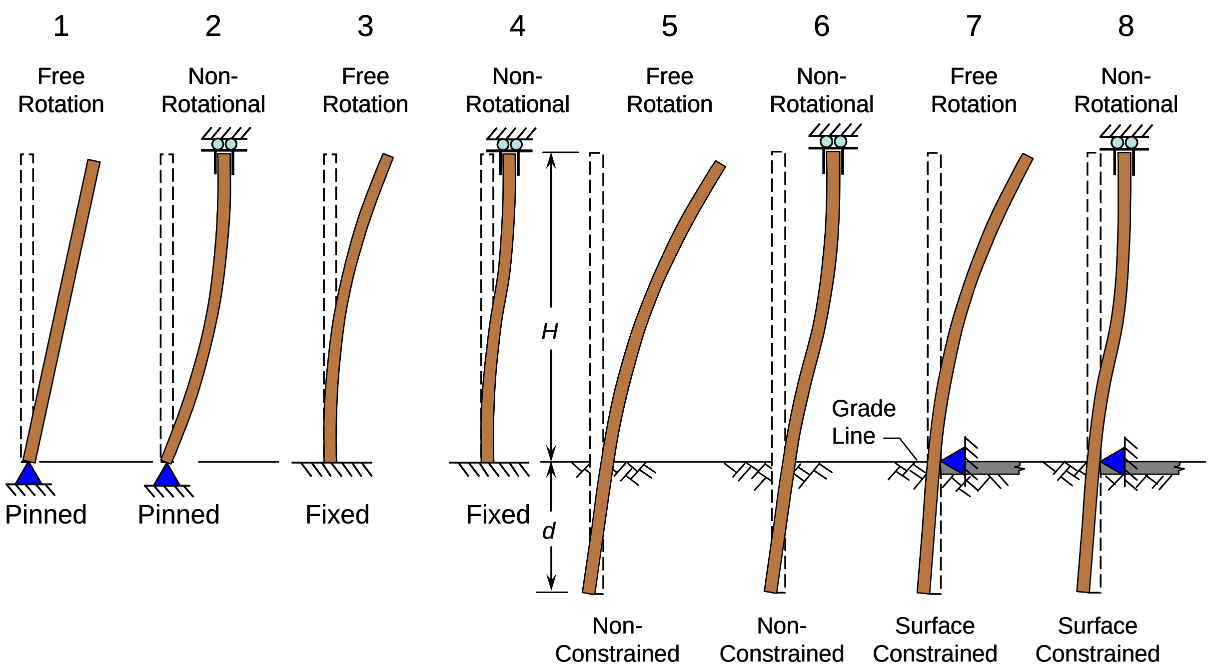


Figure 8-12. Post end fixity conditions covered in Table 8-1.

Table 8-1. Equations for Post Shear Forces and Bending Moments[a][b]

Fixity Case [c]	Base Connection [c]	Post Top Restrained from Rotating[c]	Applicable Equations [d]
1	Pinned at $y = 0$	No	$V_y = sw(H/2-y)$ $M_y = swy(H-y)/2$
2	Pinned at $y = 0$	Yes	$V_y = 3EI\Delta/H^3 + sw(3H/8 - y)$ $M_y = 3EI\Delta y/H^3 + swy(3H/8 - y/2)$
3	Fixed at $y = 0$	No	$V_y = 3EI\Delta/H^3 + sw(5H/8 - y)$ $M_y = 3EI\Delta(y - H)/H^3 - sw(H^2 - 5Hy + 4y^2)/8$
4	Fixed at $y = 0$	Yes	$V_y = 12EI\Delta/H^3 + sw(H/2 - y)$ $M_y = 6EI\Delta(2y - H)/H^3 - sw(H^2/6 - Hy + y^2)/2$
5	Embedded, Non-Constrained	No	$\theta_b = \dfrac{(9+6d/H)\Delta/H + swH(6H^2 + 20dH - 12d^2/C)/(16EI)}{C + 4.5(d/H)^2 + 12d/H + 9}$ $\Delta_b = [\theta_b(3d + 4H) + swH^3 d/(2EIC)]/(4 + 6H/d)$ $V_y = 3EI(\Delta - \Delta_b - \theta_b H)/H^3 + sw(5H/8 - y)$ $M_y = 3EI(y - H)(\Delta - \Delta_b - \theta_b H)/H^3 - sw(H^2 - 5Hy + 4y^2)/8$
6	Embedded, Non-Constrained	Yes	$\theta_b = \dfrac{(18 + 24d/H)\Delta/H + swH(H^2/4 + dH - d^2/C)/(EI)}{C + (d/H)^2(18 + 6/C) + 24d/H + 12}$ $\Delta_b = [\theta_b(3d + d/C + 2H) + swH^3 d/(6EIC)]/(4 + 3H/d)$ $V_y = 6EI(2\Delta - 2\Delta_b - \theta_b H)/H^3 + sw(H/2 - y)$ $M_y = 6EI[(\Delta - \Delta_b)(2y/H-1) + \theta_b(2H/3-y)]/H^2 - sw(H^2/6-Hy+y^2)/2$
7	Embedded, Surface-Constrained	No	$\theta_b = [swH^3/(24EI) + \Delta/H]/(1+C)$ $V_y = 3EI(\Delta - \theta_b H)/H^3 + sw(5H/8 - y)$ $M_y = 3EI(y - H)(\Delta - \theta_b H)/H^3 - sw(H^2 - 5Hy + 4y^2)/8$
8	Embedded, Surface-Constrained	Yes	$\theta_b = [swH^3/(36EI) + 2\Delta/H]/(4/3+C)$ $V_y = 6EI(2\Delta - \theta_b H)/H^3 + sw(H/2 - y)$ $M_y = 2EI(6y\Delta/H - 3\Delta + 2\theta_b H - 3y\theta_b)/H^2 - sw(H^2/6 - Hy + y^2)/2$

(a) Equations for embedded posts assume post has an infinite flexural rigidity (EI) below grade and a constant EI above grade. Soil modulus of elasticity E_S is assumed to increase linearly with depth z as $E_S = A_E z$.
(b) From Bohnhoff (1992)
(c) See figure 8-12 for graphical depiction of the eight fixity cases.
(d) Definitions (see figure 8-13 for sign convention).

- V_y = shear at height y, ($0 \le y \le H$).
- M_y = moment at height y, ($0 \le y \le H$).
- θ_b = rotation at grade (i.e., groundline).
- y = distance above grade.
- H = distance from grade to the post-to-truss/rafter connection.
- d = distance from grade to the top of a detached footing, or distance from grade to the bottom of an attached footing.
- Δ_b = horizontal displacement of post at grade.
- Δ = horizontal displacement of post at $y = H$ (assumed equal to eave displacement) from post-frame structural analysis (Section 6.6.2 or 6.6.3).
- s = frame spacing.
- w = uniform wind pressure tributary to the side of the post. Positive if in same direction as Δ.
- E = modulus of elasticity for above-grade portion of post.
- I = moment of inertia for above-grade portion of post.
- C = $d^4 A_E H/(6\,EI)$
- A_E = increase in Young's modulus for soil per unit increase in depth z below grade.

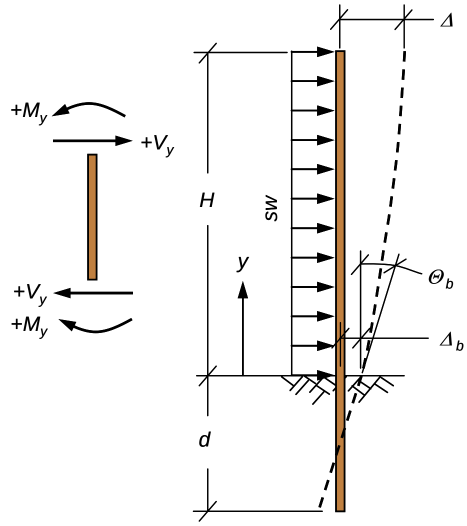


Figure 8-13. Positive sign convention for variables defined and used in Table 8-1.

8.9.2.1 Step 2: Determination of Axial Force

Where there are no more than two posts per frame, free body diagrams can be used to accurately determine post axial forces. When drawing these diagrams, posts are typically "cut" at grade (figure 8-14(a)) or at the post-to-truss connection (figure 8-14(b)). Shear forces and bending moment determined using the Table 8-1 equations are applied to the cut ends, and the two unknown axial forces are determined by (1) summing moments around one of the cut ends, and then (2) summing forces in the vertical direction.

When a post-frame is comprised of more than two posts, there must be an additional equation of static equilibrium for each additional post in order to determine all post axial forces. For example, it is possible to determine the three unknown axial forces for the three posts in Figure 8-15(a) because the hinge between the two rafters provides an additional equation (i.e., the moment in the structure at the hinge is numerically equal to zero). Note that by drawing a free body diagram of the structure to the right of the hinge (figure 8-15(b)), and summing moments about the hinge, the axial force in the right post can be determined. Once this is done, the axial forces in the other two posts can be determined by returning to the free body diagram of the entire structure (figure 8-15(a)) and summing moments around the base of the left post to obtain the axial force in the center post, and then summing forces in the vertical direction to obtain the axial force in the left post.

Classical methods of structural analysis exist for approximating post axial forces in frames that are statically indeterminate. These methods depend on simplifying assumptions which tend to be frame dependent. With the advent of plane-frame structural

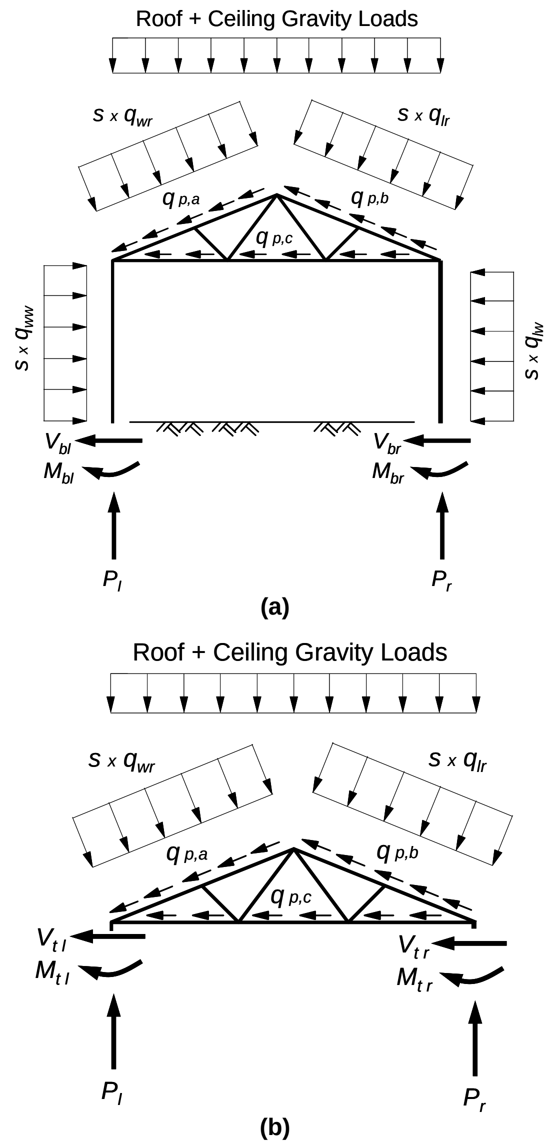


Figure 8-14. Free body diagrams (FBDs) used to determine axial post forces.

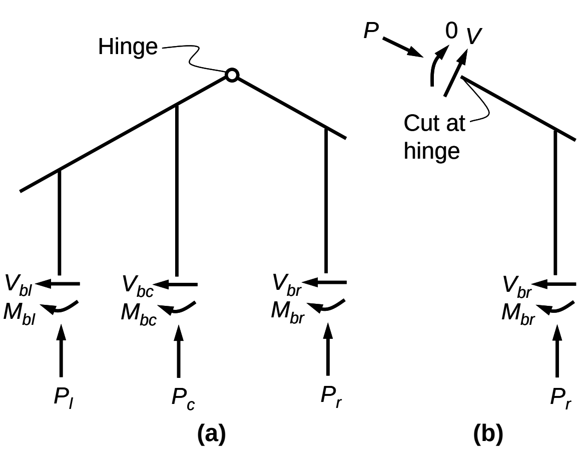


Figure 8-15. (a) Post-frame comprised of three posts and two rafters hinged to each other (b) FBD of the right rafter and right post only.

analysis programs, use of these classical method of structural analysis is virtually unheard of. In other words, use a plane-frame structural analysis program as outlined in Section 8.9.1 to determine forces in post-frames comprised of more than two posts.

8.10 Reference Design Values

Compiled in this section are reference design values for commonly used solid-sawn posts, structural composite lumber (SCL) posts, glulam posts, round timber poles, and mechlam posts.

Reference design values and the applicable adjustment factors in Section 8.11 are included in this manual primarily for comparison purposes (i.e., to enable designers to quickly ascertain strength differences between post types, species and grades) . For a complete compilation of reference design values, see the *National Design Specifications* (NDS) *for Wood Construction* (ANSI/AWC, 2012).

When a post is fabricated using individual wood laminates, veneers or strands, the statistical probability of having a strength-reducing characteristic (such as a knot) running through the entire cross section is greatly diminished. Consequently, such built-up posts have more uniform strength and stiffness properties than solid-sawn posts. This increased reliability results in higher allowable design values.

8.10.1 Solid-Sawn Posts

Reference design values for solid-sawn posts common to post-frame building construction are given in Table 8-2. These design values are applicable for members that fall into the size category of "posts and timbers" which includes rectangular lumber 5 inches or more in nominal thickness with a width that is not more than 2 inches greater than it thickness. The species combination Douglas Fir-Larch includes both Douglas Fir and Western Larch; Northern Pine includes Jack Pine, Norway (Red) Pine and Pitch Pine; and Southern Pine includes Loblolly Pine, Longleaf Pine, Shortleaf Pine and Slash Pine.

Table 8-2. Reference Design Values for Solid-Sawn Posts (a)(b)(c)

Species (or Species Combination) and Grade	Bending (lbf/in^2)	Tension Parallel to Grain (lbf/in^2)	Shear Parallel to Grain (lbf/in^2)	Compression Perpendicular to Grain (lbf/in^2)	Compression Parallel to Grain (lbf/in^2)	Modulus of Elasticity (10^3 lbf/in^2) Mean	Modulus of Elasticity (10^3 lbf/in^2) For Stability Calcs.
	F_b	F_t	F_v	$F_{c\perp}$	F_c	E	E_{min}
Douglas Fir-Larch (Specific Gravity = 0.50)							
Sel Str	1500	1000	170	625	1150	1600	580
No. 1	1200	825	170	625	1000	1600	580
No. 2	750	475	170	625	700	1300	470
Northern Pine (Specific Gravity = 0.42)							
Sel Str	1150	800	135	435	900	1300	470
No. 1	950	650	135	435	800	1300	470
No. 2	500	375	135	435	375	1000	370
Ponderosa Pine (Specific Gravity = 0.43)							
Sel Str	1000	675	130	535	800	1100	400
No. 1	825	550	130	535	700	1100	400
No. 2	475	325	130	535	325	900	330
Southern Pine (Specific Gravity = 0.55)							
Sel Str	1500	1000	165	375	950	1500	550
No. 1	1350	900	165	375	825	1500	550
No. 2	850	550	165	375	525	1200	440

(a) From the National Design Specifications (NDS) for Wood Construction. Values are for visually graded timber in the size classification of "Posts and Timbers".
(b) Reference design values are for normal (10 yr) load duration and dry service conditions, and must be adjusted by factors specified in Section 8.11 where applicable.
(c) For rectangular, solid-sawn wood members whose minimum dimension is 5 nominal inches or greater.

8.10.2 Structural Composite Lumber Posts

A structural composite lumber (SCL) post was previously defined as a post comprised of a *single* piece of structural composite lumber. Posts that are comprised of *multiple* pieces of SCL that have been mechanically-fastened together fall under the category of mechlam posts and must be designed as such.

Shown in figures 8-16, 8-17 and 8-18 are three types of structural composite lumber: laminated veneer lumber (LVL), parallel strand lumber (PSL) and laminated stand lumber (LSL).

Tables 8-3 and 8-4 contain dimensions and reference design values, respectively, for structural composite lumber manufactured by three major United States companies. The dimensions given in Table 8-3 are sizes that appear in product literature. Reference design values are from the ICC-ES Evaluation Reports listed in Table 8-3. These reports are available at http://www.icc-es.org/Evaluation_Reports/index.shtml.

Figure 8-16. Laminated veneer lumber (LVL).

Figure 8-17. Parallam PSL.

Figure 8-18. Timberstrand LSL.

Table 8-3 dimensions are actual dry dimensions. Not all sizes listed in Table 8-3 are commonly stocked, and sizes other than those listed can generally be special ordered. Regardless of type, SCL is typically available in lengths up to (and in some case exceeding) 60 foot.

Reference design values in Table 8-4 were established in accordance with ASTM D5456, *Standard Specification for Evaluation of Structural Composite Lumber Products*.

8.10.3 Glulam Posts

Glulam posts used in post-frame buildings typically consist of three or four laminations of identical size, species and grade. They are nearly square in shape having a cross-sectional aspect ratio typically less than 1.5. Because they are nearly square, and because the same grade and species of lumber is used throughout, such assemblies are efficient at resisting axial loads as well as bending forces applied to either face. In almost all cases, the assemblies are oriented (and hence designed) so the larger bending loads are applied parallel to interlayer planes as shown in figure 8-19(a).

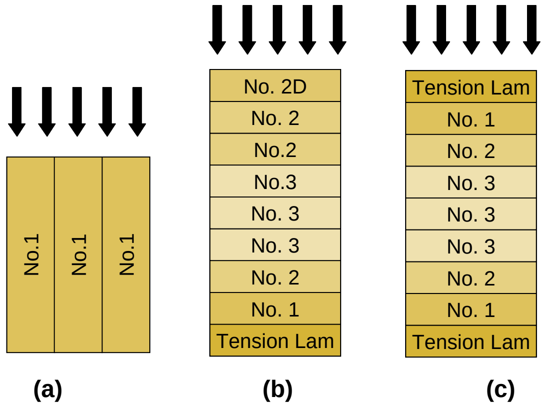


Figure 8-19. Glulam assemblies (a) vertical lamination (b) horizontal lamination with an unbalanced layup, and (c) horizontal lamination with a balanced layup.

In contrast to the relatively square glulams used as posts, deep glulam beams (e.g. door headers, girders) are generally designed to only handle bending loads about one axis. These assemblies are designed as horizontally laminated components with higher grade lumber in outer laminates where bending stresses are higher (figures 8-19(b) and 8-19(c)).

Reference design values for glulams fabricated with a single species and grade of softwood lumber are compiled in Table 8-5. The values in this table are for glulams primarily loaded parallel to the wide face of laminations, or stressed primarily in axial tension or compression. Table 8-5 values and layups and associated design values for horizontally-laminated glulams are compiled in AITC 117 (AITC, 2004) and in the NDS (ANSI/AWC, 2012).

Fabrication procedures for glulams must conform to ANSI/AITC A190.1 (AITC, 2007) which covers physical construction issues as well as quality control, testing and marking procedures. The rigorous requirements outlined in this standard essentially eliminate the possibility of on-site fabrication of glulams.

Several companies now manufacture and market glulams specifically for use in post-frame buildings. These posts are intended for soil embedment, with pressure preservative treated wood on one end, and non-treated wood on the other. Fabrication of such posts requires special resins and procedures for joining and laminating treated wood to non-treated wood.

Table 8-3. Dimensions of Selected Commercially-Available Structural Composite Lumber

SCL Type	Manufacturer	Product Name	IIC-ES Evaluation Report No.	Grade	Available Thicknesses, inches	Available Depths, inches
LSL	Weyerhaeuser iLevel	TimberStrand	1387	1.55E	1-¾, 3-½	9-¼, 9-½, 11-¼, 11-7/8, 14, 16
				1.3E	3-½	3-½, 4-3/8. 5-½, 7-¼, 8-5/8, 9-¼, 11-¼
	Louisiana-Pacific	SolidStart	2403	All	1-½, 1-¾, 3-½	3-½, 4-3/8, 5-½, 7-¼, 9-¼, 9-½, 11-¼, 11-7/8, 14, 16, 18, 20, 22, 24
LVL	Boise Cascade	Versa-Lam	1040	1.4E 1800Fb	1-¼	5, 9-½,11-7/8, 14, 16
				1.7E 2400Fb	1-5/16, 1-½	3-½, 5-½, 7-¼, 9-¼, 9-½, 11-¼, 11-7/8, 14, 18
				2.0E 2800Fb	1-¾, 2-5/8	5-½, 7-¼, 9-¼, 9-½, 11-¼, 11-7/8, 14, 16, 18
				2.0E 3100Fb	1-¾	3-½, 5-½, 7-¼, 9-¼, 9-½, 11-¼, 11-7/8, 14, 16, 18, 24
				2.0E 3100Fb	3-½	5-½, 7-¼, 9-¼, 9-½, 11-¼, 11-7/8, 14, 16, 18, 20
				2.0E 3100Fb	5-¼	5-¼, 5-½, 7-¼, 9-¼, 9-½, 11-¼, 11-7/8, 14, 16, 18, 20, 24
				2.0E 3100Fb	7	9-¼, 9-½, 11-¼, 11-7/8, 14, 16, 18, 20, 24
				1.7E 2650Fb	1-½	3-½,5-¼,7
				1.7E 2650Fb	3-½	3-½,5-¼,7
				1.7E 2650Fb	5-¼	5-¼,7
				1.7E 2650Fb	7	7
	Louisiana-Pacific	SolidStart	2403	1.5E, 1.9E, and 2.0E	1-½, 1-¾, 3-½	3-½, 5-½, 7-¼, 9-¼, 9-½, 11-7/8, 11-¼, 14, 16, 18, 20, 22, 23-7/8
				2.0E	3-½, 5-¼, 7	3-½, 5-½, 7-¼, 9-¼, 9-½, 11-7/8, 11-¼, 14, 16, 18, 20, 22, 23-7/8
	Weyerhaeuser iLevel	Microlam	1387	1.9E	1-¾	5-½, 7-¼, 9-¼, 9-½, 11-¼, 11-7/8, 14, 16, 18, 20
PSL	Weyerhaeuser iLevel	Parallam	1387	1.8E	3-½	3-½. 5-¼, 7
				1.8E	5-¼	5-¼, 7
				1.8E	7	7
				2.0E	3-½, 5-¼, 7	9-¼, 9-½, 11-¼, 11-7/8, 14, 16, 18

Table 8-4. Reference Design Values for Structural Composite Lumber (SCL) [(a)]

SCL Type	Product Name	Grade	Orien-tation[(b)]	Bending (lbf/in^2)	Tension parallel to grain (lbf/in^2)	Shear parallel to grain (lbf/in^2)	Compression (lbf/in^2) Perpen-dicular to grain	Compression (lbf/in^2) Parallel to grain	Modulus of elasticity (10^3 lbf/in^2) Mean	Modulus of elasticity (10^3 lbf/in^2) For Beam and Column Stability	Shear Modulus (10^3 lbf/in^2)	Specific Gravity
				F_b	F_t	F_v	$F_{c\perp}$	F_c	E	E_{min}	G	SG
LSL	SolidStart	1730Fb-1.35E	Beam	1730	1300	410	750	1650	1350	686	84.4	0.50
	SolidStart	1730Fb-1.35E	Plank	1910	1300	155	440	1650	1350	686	84.4	0.50
	SolidStart	2360Fb-1.55E	Beam	2360	1750	410	750	2175	1550	788	96.9	0.50
	SolidStart	2360Fb-1.55E	Plank	2620	1750	155	440	2175	1550	788	96.9	0.50
	SolidStart	2500Fb-1.75E	Beam	2500	2100	410	950	2450	1750	880	96.9	0.50
	SolidStart	2500Fb-1.75E	Plank	2800	2100	155	440	2450	1750	880	96.9	0.50
	Timberstrand	1.3E	Beam	1700	1075	400	680	1400	1300	661	81.3	0.50
	Timberstrand	1.3E	Plank	1900	1075	150	435	1400	1300	661	81.3	0.50
	Timberstrand	1.55E	Beam	2325	1600	400	800	2050	1550	788	96.9	0.50
	Timberstrand	1.55E	Plank	2615	1600	150	485	2050	1550	788	96.9	0.50
LVL	Microlam	1.9E	Beam	2600	1555	285	750	2510	1900	966	118.8	0.50
	Microlam	1.9E	Plank	3075	1555	190	480	2510	1900	966	118.8	0.50
	SolidStart	2250Fb-1.5E	Beam	2250	1350	285	750	2350	1500	778	93.8	0.46
	SolidStart	2250Fb-1.5E	Plank	2200	1350	140	450	2350	1400	726	87.5	0.46
	SolidStart	2950Fb-2.0E	Beam	2950	1800	290	750	3200	2000	1037	125.0	0.46
	SolidStart	2950Fb-2.0E	Plank	2910	1800	140	450	3200	2000	1037	125.0	0.46
	Versa-Lam	1.7E 2650Fb	Beam	2650	1650	285	750	3000	1700	881	106.3	0.50
	Versa-Lam	1.7E 2650Fb	Plank	2400	1650	190	450	3000	1700	881	106.3	0.50
	Versa-Lam	2.0E 3100Fb	Beam	3100	2150	285	750	3000	2000	1037	125.0	0.50
	Versa-Lam	2.0E 3100Fb	Plank	3100	2150	190	450	3000	2000	1037	125.0	0.50
	Versa-Lam	1.4E 1800Fb	Beam	1800	1250	225	525	2500	1400	726	87.5	0.50
	Versa-Lam	1.4E 1800Fb	Plank	1800	1250	150	450	2500	1400	726	87.5	0.50
	Versa-Lam	1.7E 2400Fb	Beam	2400	1650	285	750	3000	1700	881	106.3	0.50
	Versa-Lam	1.7E 2400Fb	Plank	2400	1650	190	450	3000	1700	881	106.3	0.50
	Versa-Lam	2.0E 2800Fb	Beam	2800	2150	285	750	3000	2000	1037	125.0	0.50
	Versa-Lam	2.0E 2800Fb	Plank	2800	2150	190	450	3000	2000	1037	125.0	0.50
PSL	Parallam	1.8E	Beam	2500	1755	230	600	2500	1800	915	112.5	0.50
	Parallam	1.8E	Plank	2400	1755	190	425	2500	1800	915	112.5	0.50
	Parallam	2.0E	Beam	2900	2025	290	750	2900	2000	1016	125.0	0.50
	Parallam	2.0E	Plank	2900	2025	210	525	2900	2000	1016	125.0	0.50

(a) Reference design values are for normal (10 yr) load duration and dry service conditions, and must be adjusted by factors specified in Section 8.11 where applicable.

(b) Beams are members loaded parallel to the wide faces of strands (see figure 8-1b). Planks are members loaded perpendicular to the wide faces of strands (see figure 8-1a).

Table 8-5. Reference Design Values for Structural Glued Laminated Timber [a][b]

ID Number	Grade [c]	All Loadings			Axially Loaded		
		Modulus of Elasticity		Compression Perpendicular to Grain (lbf/in^2)	Tension Parallel to Grain	Compression Parallel to Grain	
		Mean (10^3 lbf/in^2)	For Beam and Column Stability (10^3 lbf/in^2)		2 or More Laminations (lbf/in^2)	4 or More Laminations (lbf/in^2)	2 or 3 Laminations (lbf/in^2)
		E	E_{min}	$F_{c\perp}$	F_t	F_c	F_c
Douglas Fir – Visually Graded Laminations							
1	L3	1500	780	560	900	1550	1200
2	L2	1600	830	560	1250	1950	1600
3	L2D	1900	980	650	1450	2300	1850
4	L1CL	1900	980	590	1400	2100	1900
5	L1D	2000	1040	650	1600	2400	2100
Hem Fir – Visually Graded Laminations							
14	L3	1300	670	375	800	1100	980
15	L2	1400	730	375	1050	1350	1300
16	L1	1600	830	375	1200	1500	1450
17	L1D	1700	880	500	1400	1750	1700
Southern Pine – Visually Graded Laminations							
47	N2M14	1400	730	650	1200	1900	1150
48	N2D14	1700	880	740	1400	2200	1350
49	N1M16	1700	880	650	1350	2100	1450
50	N1D14	1900	980	740	1550	2300	1700
Douglas Fir – E-Rated Laminations							
27	1.9E-2	1800	930	560	900	1750	1200
28	2.1E-2	2000	1040	650	1100	2000	1400
29	2.3E-2	2200	1140	650	1250	2250	1550
30	1.9E-6	1800	930	560	1550	2100	1700
31	2.1E-6	2000	1040	650	1800	2400	1950
32	2.3E-6	2200	1140	650	1800	2400	2200
62	2.2E-2	2100	1090	650	1150	1850	1500
63	2.2E-6	2100	1090	650	1950	2300	2000
Hem Fir – E-Rated Laminations							
33	1.6E-2	1500	780	375	800	1050	950
34	1.9E-2	1800	930	500	900	1500	1200
35	2.1E-2	2000	1040	500	1100	1550	1400
36	1.6E-4	1500	780	375	1200	1450	1350
37	1.9E-6	1800	930	500	1550	1950	1700
38	2.1E-6	2000	1040	500	1800	2400	1950
Southern Pine – E-Rated Laminations							
54	2.1E-2	2000	1040	740	1100	2300	1400
55	2.3E-2	2200	1140	740	1250	2400	1550
56	1.9E-6	1800	930	650	1550	1850	1700
57	2.1E-6	2000	1040	740	1800	2300	1950
58	2.3E-6	2200	1140	740	1800	2400	2200

(a) For: (1) assemblies primarily loaded parallel to the wide face of laminations (i.e., vertically-laminated assemblies, assemblies bent about axis Y-Y), (2) assemblies stressed primarily in axial tension or compression, and (3) two- and three-layer assemblies bent about axis X-X.

(b) Reference design values are for normal (10 yr) load duration and dry service conditions, and must be adjusted by factors specified in Section 8.11 where applicable.

(c) See AITC 117-2004 Annex C (AITC, 2004) for *AITC Grading Handbook for Laminating Lumber*.

Table 8-5. (Cont.) Reference Design Values for Structural Glued Laminated Timber

ID Number	Grade	**Bending About Y-Y Axis** Loaded Parallel to Wide Faces of Laminations				**Bending About X-X Axis** Loaded Perpendicular to Wide Faces of Laminations	
		Bending, 4 or More Laminations (lbf/in^2)	Bending, 3 Laminations (lbf/in^2)	Bending, 2 Laminations (lbf/in^2)	Shear Parallel to Grain [(d)(e)(f)] (lbf/in^2)	Bending, 2 Laminations to 15 in. Deep [(g)] (lbf/in^2)	Shear Parallel to Grain [(c)] (lbf/in^2)
		F_{by}	F_{by}	F_{by}	F_{vy}	F_{bx}	F_{vx}
Douglas Fir – Visually Graded Laminations							
1	L3	1450	1250	1000	230	1250	265
2	L2	1800	1600	1300	230	1700	265
3	L2D	2100	1850	1550	230	2000	265
4	L1CL	2200	2000	1650	230	1900	265
5	L1D	2400	2100	1800	230	2200	265
Hem Fir – Visually Graded Laminations							
14	L3	1200	1050	850	190	1100	215
15	L2	1500	1350	1100	190	1450	215
16	L1	1750	1550	1300	190	1600	215
17	L1D	2000	1850	1550	190	1900	215
Southern Pine – Visually Graded Laminations							
47	N2M14	1750	1550	1300	260	1400	300
48	N2D14	2000	1800	1500	260	1600	300
49	N1M16	1950	1750	1500	260	1800	300
50	N1D14	2300	2100	1750	260	2100	300
Douglas Fir – E-Rated Laminations							
27	1.9E-2	1450	1250	1000	230	1250	265
28	2.1E-2	1650	1450	1150	230	1500	265
29	2.3E-2	1900	1650	1350	230	1700	265
30	1.9E-6	2400	2400	2100	230	1800	265
31	2.1E-6	2400	2400	2400	230	2100	265
32	2.3E-6	2400	2400	2400	230	2400	265
62	2.2E-2	1800	1550	1250	230	1800	265
63	2.2E-6	2400	2400	2400	230	2200	265
Hem Fir – E-Rated Laminations							
33	1.6E-2	1200	1050	850	190	1100	215
34	1.9E-2	1450	1250	1000	190	1300	215
35	2.1E-2	1650	1450	1150	190	1850	215
36	1.6E-4	2100	1900	1700	190	1400	215
37	1.9E-6	2400	2400	2100	190	1800	215
38	2.1E-6	2400	2400	2400	190	2100	215
Southern Pine – E-Rated Laminations							
54	2.1E-2	1650	1450	1150	260	1500	300
55	2.3E-2	1900	1650	1350	260	1700	300
56	1.9E-6	2400	2400	2100	260	1800	300
57	2.1E-6	2400	2400	2400	260	2100	300
58	2.3E-6	2400	2400	2400	260	2400	300

(d) F_{vy} shall be multiplied by a factor of 0.84 or 0.95 for members with 2 or 3 laminations, respectively.

(e) Multiply F_{vy} by 0.4 for members with 5, 7 or 9 laminations manufactured from multiple piece laminations (across width) that are not edge bonded. Multiply F_{vy} by 0.5 for all other glulams comprised of multiple piece laminations with unbonded edge joints. This reduction shall be cumulative with the adjustments in footnotes (d) and (f).

(f) Multiply F_{vx} and F_{vy} by 0.72 for non-prismatic members, notched members, and all members subject to impact or cyclic loading. Use the reduced design value for glulam connections that transfer shear by mechanical fasteners.

(g) For members greater than 15 inches deep and without special laminations, multiply F_{bx} by 0.88. This factor shall not be applied cumulatively with the adjustment in footnote (g).

8.10.4 Round Timber Poles

The modern post-frame building evolved from pole buildings – structures in which the main support members were obtained by simply peeling (figure 8-20) and preservative treating logs. Although use of naturally round and tapered construction poles in post-frame buildings is relatively infrequent in the United States today, reference design values for them are given in Table 6B of the NDS (AWC, 2012). NDS Table 6B is included here as Table 8-6.

The values in Table 8-6 were established in accordance with ASTM D3200 *Establishing Recommended Design Stresses for Round Timber Construction Poles* (ASTM, 2012c). ASTM D3200 is a short document as it simply adopts (by reference) specifications for round timber piles as established in ASTM D25 *Specification for Round Timber Piles* (ASTM, 2012a), and ASTM D2899 *Practice for Establishing Allowable Stresses for Round Timber Piles* (ASTM, 2012b).

The values in Table 8-6 can be applied as long as the quality, length, size, sapwood, cutting, trimming, peeling (shaving), straightness, twist of grain, knot, holes, scars, check, shake and split requirements outlined in ASTM D25 and the taper requirements tabulated in ASTM D3200 are met.

Figure 8-20. Logs prior to peeling (top) and during peeling (bottom) at Frontier Builders Inc., Land O' Lakes, WI.

Table 8-6. Reference Design Values for Round Timber Construction Poles [(a)(b)]

Species	Bending (lbf/in^2)	Shear Parallel to Grain (lbf/in^2)	Compression Perpendicular to Grain (lbf/in^2)	Compression Parallel to Grain (lbf/in^2)	Modulus of Elasticity (10^3 lbf/in^2) Mean	Modulus of Elasticity (10^3 lbf/in^2) For Stability Calcs.	Specific Gravity
	F_b	F_v	$F_{c\perp}$	F_c	E	E_{min}	G
Douglas Fir	2050	160	490	1300	1700	690	0.50
Lodgepole Pine	1275	125	265	825	1100	430	0.42
Ponderosa Pine	1200	175	295	775	1000	400	0.43
Red Pine	1350	125	270	850	1300	520	0.42
Southern Pine	1950	160	440	1250	1500	600	0.55
Western Hemlock	1550	165	275	1050	1300	560	0.47
Western Larch	1900	170	405	1250	1500	660	0.49
Western Red Cedar	1250	140	260	875	1000	360	0.34

(a) Naturally round and tapered members graded per ASTM D3200. Tabulated values are from Table 6B of the National Design Specifications (NDS) for Wood Construction.

(b) Reference design values are for normal (10 yr) load duration and dry service conditions, and must be adjusted by factors specified in Section 8.11 where applicable.

8.10.5 Mechlam Posts

Mechlam posts are fabricated from either dimension lumber or structural composite lumber. Reference design values for structural composite lumber were presented in Table 8-4. Reference design values for selected grade and species of dimension lumber are compiled in Table 8-7 for visually grades, and in Tables 8-8 and 8-9 for machine graded lumber. The latter includes both machine stressed rated (MSR) lumber and machine evaluated lumber (MEL).

Mechlam posts, especially spliced mechlam posts, are complex structural assemblies, with many unique design provisions as outlined in ANSI/ASAE EP559 (ASABE, 2010). These provisions are presented in Chapter 9.

8.11 Adjustment Factors

Reference design values tabulated in Section 8.10 apply for a very specific set of conditions and thus must be adjusted when the designed post is used under different conditions.

Tables 8-10 and 8-11 contain adjustment factors applicable to reference design values for each post type covered in Section 8.10. Table 8-10 contains adjustments for ASD and Table 8-11 those for LFRD.

Only adjustment factors that are applicable to common post-frame building posts are listed in Tables 8-10 and 8-11. For example, the tables do not contain adjustment factors that are applicable to only curved glulam beams.

Unless noted, modification factors are cumulative; for example, the ASD adjusted design shear stress for dimension lumber is $F_v' = F_v\, C_D\, C_M\, C_t\, C_i$.

Adjustment factors are discussed further in the referenced section number listed in row 3 of Tables 8-10 and 8-11.

Table 8-7. Reference Design Values for Visually Graded Dimension Lumber (a)(b)(c)

Grade	Nominal Width	Bending (lbf/in^2)	Tension parallel to grain (lbf/in^2)	Shear parallel to grain (lbf/in^2)	Compression: Perpendicular to grain (lbf/in^2)	Compression: Parallel to grain (lbf/in^2)	Modulus of elasticity: Mean (10^3 lbf/in^2)	Modulus of elasticity: 5th pct divided by 1.66 (10^3 lbf/in^2)
		F_b	F_t	F_v	$F_{c\perp}$	F_c	E	E_{min}
Douglas Fir-Larch (Specific Gravity = 0.50)								
Select Structural	2 inches	1500	1000	180	625	1700	1900	690
No. 1	and	1000	675	180	625	1500	1700	620
No. 2	wider	0900	575	180	625	1350	1600	580
Hem-Fir (Specific Gravity = 0.43)								
Select Structural	2 inches	1400	925	150	405	1500	1600	580
No. 1	and	975	625	150	405	1350	1500	550
No. 2	wider	850	525	150	405	1300	1300	470
Southern Pine (Specific Gravity = 0.55)								
Select Structural	5 to 6	2100	1450	175	565	1800	1800	660
No. 1	inches	1350	875	175	565	1550	1600	580
No. 2	wide	1000	600	175	565	1400	1400	510
Select Structural	8 inches wide	2300	1350	175	565	1700	1800	660
No. 1		1950	800	175	565	1500	1600	580
No. 2		925	550	175	565	1350	1400	510
Select Structural	10	1700	1150	175	565	1650	1800	660
No. 1	inches	1050	700	175	565	1450	1600	580
No. 2	wide	800	475	175	565	1300	1400	510
Select Structural	12	1600	1100	175	565	1650	1800	660
No. 1	inches	1000	650	175	565	1400	1600	580
No. 2	wide	750	450	175	565	1250	1400	510

(a) From the National Design Specifications (NDS) for Wood Construction (ANSI/AWC, 2012)
(b) Reference design values are for normal (10 yr) load duration and dry service conditions, and must be adjusted by factors specified in Section 8.11 where applicable.
(c) Dimension lumber includes rectangular, solid-sawn wood members whose minimum dimension is greater or equal to 2 nominal inches but less than 5 nominal inches.

Table 8-8. Reference Design Values for Machine Graded Lumber (a)(b)(c)

Grade name	Bending (lbf/in^2)	Tension parallel to grain (lbf/in^2)	Comp. parallel to grain (lbf/in^2)	Modulus of elasticity: Mean (10^3 lbf/in^2)	Modulus of elasticity: 5th pct divided by 1.66 (10^3 lbf/in^2)	Grading Rules Agency
	F_b	F_t	F_c	E	E_{min}	
Machine Stress Rated (MSR)						
1350f-1.3E	1350	750	1600	1300	660	NLGA, SPIB, WCLIB, WWPA
1450f-1.3E	1450	800	1625	1300	660	NLGA, WCLIB, WWPA
1650f-1.5E	1650	1020	1700	1500	760	NLGA, SPIB, WCLIB, WWPA
1800f-1.6E	1800	1175	1750	1600	810	NLGA, SPIB, WCLIB, WWPA
1800f-1.8E	1800	1200	1750	1800	910	WCLIB, WWPA
1950f-1.7E	1950	1375	1800	1700	860	NLGA, SPIB, WWPA
2100f-1.8E	2100	1575	1875	1800	910	NLGA, SPIB, WCLIB, WWPA
2400f-2.0E	2400	1925	1975	2000	1020	NLGA, SPIB, WCLIB, WWPA
2550f-2.1E	2550	2050	2025	2100	1070	NLGA, SPIB, WWPA
2700f-2.2E	2700	2150	2100	2200	1120	NLGA, SPIB, WCLIB, WWPA
2850f-2.3E	2850	2300	2150	2300	1170	NLGA, SPIB, WWPA
Machine Evaluated Lumber (MEL)						
M-10	1400	800	1600	1200	560	NLGA, SPIB
M-11	1550	850	1675	1500	700	NLGA, SPIB
M-14	1800	1000	1750	1700	790	NLGA, SPIB
M-19	2000	1300	1825	1600	750	NLGA, SPIB
M-21	2300	1400	1950	1900	890	NLGA, SPIB
M-23	2400	1900	1975	1800	840	NLGA, SPIB
M-24	2700	1800	2100	1900	890	NLGA, SPIB

(a) From the National Design Specifications (NDS) for Wood Construction (ANSI/AWC, 2012) .
(b) Reference design values are for normal (10 yr) load duration and dry service conditions, and must be adjusted by factors specified in Section 8.11 where applicable.
(c) Design values for shear and for compression perpendicular to grain for mechanically graded lumber are species dependent and shall be obtained from Table 8-9.

Table 8-9. Reference Design Values for Machine Graded Lumber(a)(b)(c)

Species	Modulus of elasticity (10^3 lbf/in^2)	Specific gravity	Shear parallel to grain (lbf/in^2)	Compression perpendicular to grain (lbf/in^2)	Grading rule agency
	E	G	F_v	$F_{c\perp}$	
Douglas Fir-Larch	1000 or higher	0.50	180	625	WCLIB, WWPA
Hem-Fir	1000 or higher	0.43	150	405	WCLIB, WWPA
Southern Pine	1000 and higher	0.55	175	565	SPIB
	1800 and higher	0.57	190	805	SPIB

(a) From the National Design Specifications (NDS) for Wood Construction (ANSI/AWC, 2012).
(b) Reference design values are for normal (10 yr) load duration and dry service conditions, and must be adjusted by factors specified in Section 8.11 where applicable.
(c) Values are for MSR and MEL lumber. For species or species groups not shown above, the G, F_v and $F_{c\perp}$, values for visually graded lumber may be used. Higher G values may be claimed when (a) specifically assigned by the rules writing agency or (b) when qualified by test.

Table 8-10. Applicable Adjustment Factors for ASD of Post

	Symbol →		C_D	C_M	C_t	C_i	C_{fu}	C_F	C_V	C_r	C_L	C_b	C_P	C_{ct}	C_{cs}
	Description →		Load Duration Factor	Wet Service Factor	Temperature Factor	Incising Factor	Flat Use Factor	Size Factor	Volume Factor	Repetitive Member Factor	Beam Stability Factor	Bearing Area Factor	Column Stability Factor	Condition Treatment Factor	Critical Section Factor
	Section 8.11.X→		1	2	3	4	5	6	7	8	9	10	11	12	13
Solid-Sawn Posts (Post & Timber Size Category)	$F_b' = F_b$	x	C_D	-	C_t	-	-	C_F		-	C_L	-	-	-	-
	$F_t' = F_t$	x	C_D	-	C_t	-	-	-		-	-	-	-	-	-
	$F_v' = F_v$	x	C_D	-	C_t	-	-	-		-	-	-	-	-	-
	$F_{c\perp}' = F_{c\perp}$	x	-	C_M	C_t	-	-	-		-	-	C_b	-	-	-
	$F_c' = F_c$	x	C_D	C_M	C_t	-	-	-		-	-	-	C_P	-	-
	$E' = E$	x	-	-	C_t	-	-			-	-	-	-	-	-
	$E_{min}' = E_{min}$	x	-	-	C_t	-	-			-	-	-	-	-	-
Vertically-Laminated Glulam Posts	$F_b' = F_b$	x	C_D	C_M	C_t	-	-	-	C_V	-	C_L	-	-	-	-
	$F_t' = F_t$	x	C_D	C_M	C_t	-	-	-	-	-	-	-	-	-	-
	$F_v' = F_v$	x	C_D	C_M	C_t	-	-	-	-	-	-	-	-	-	-
	$F_{c\perp}' = F_{c\perp}$	x	-	C_M	C_t	-	-	-	-	-	-	C_b	-	-	-
	$F_c' = F_c$	x	C_D	C_M	C_t	-	-	-	-	-	-	-	C_P	-	-
	$E' = E$	x	-	C_M	C_t	-	-	-	-	-	-	-	-	-	-
	$E_{min}' = E_{min}$	x	-	C_M	C_t	-	-	-	-	-	-	-	-	-	-
Structural Composite Lumber Posts	$F_b' = F_b$	x	C_D	C_M	C_t	-	-	-	C_V	-	C_L	-	-	-	-
	$F_t' = F_t$	x	C_D	C_M	C_t	-	-	-	C_V	-	-	-	-	-	-
	$F_v' = F_v$	x	C_D	C_M	C_t	-	-	-	-	-	-	-	-	-	-
	$F_{c\perp}' = F_{c\perp}$	x	-	C_M	C_t	-	-	-	-	-	-	C_b	-	-	-
	$F_c' = F_c$	x	C_D	C_M	C_t	-	-	-	-	-	-	-	C_P	-	-
	$E' = E$	x	-	C_M	C_t	-	-	-	-	-	-	-	-	-	-
	$E_{min}' = E_{min}$	x	-	C_M	C_t	-	-	-	-	-	-	-	-	-	-
Round Construction Poles	$F_b' = F_b$	x	C_D	-	C_t	-	-	C_F	-	-	-	-	-	C_{ct}	-
	$F_v' = F_v$	x	C_D	-	C_t	-	-	-	-	-	-	-	-	C_{ct}	-
	$F_{c\perp}' = F_{c\perp}$	x	-	-	C_t	-	-	-	-	-	-	C_b	-	C_{ct}	-
	$F_c' = F_c$	x	C_D	-	C_t	-	-	-	-	-	-	-	C_P	C_{ct}	C_{cs}
	$E' = E$	x	-	-	C_t	-	-	-	-	-	-	-	-	-	-
	$E_{min}' = E_{min}$	x	-	-	C_t	-	-	-	-	-	-	-	-	-	-
Unspliced Mechanically-Laminated Posts	$F_b' = F_b$	x	C_D	C_M	C_t	C_i	C_{fu}	C_F	C_V	C_r	C_L	-	-	-	-
	$F_t' = F_t$	x	C_D	C_M	C_t	C_i	-	C_F	C_V	-	-	-	-	-	-
	$F_v' = F_v$	x	C_D	C_M	C_t	C_i	-	-		-	-	-	-	-	-
	$F_{c\perp}' = F_{c\perp}$	x	-	C_M	C_t	C_i	-	-		-	-	C_b	-	-	-
	$F_c' = F_c$	x	C_D	C_M	C_t	C_i	-	C_F		-	-	-	C_P	-	-
	$E' = E$	x	-	C_M	C_t	C_i	-			-	-	-	-	-	-
	$E_{min}' = E_{min}$	x	-	C_M	C_t	C_i	-			-	-	-	-	-	-

Table 8-11. Applicable Adjustment Factors for LRFD of Posts

		Symbol ➔	λ	C_M	C_t	C_i	C_{fu}	C_F	C_V	C_r	C_L	C_b	C_P	C_{ct}	C_{cs}	K_F	ϕ
		Description ➔	Time Effect Factor	Wet Service Factor	Temperature Factor	Incising Factor	Flat Use Factor	Size Factor	Volume Factor	Repetitive Member Factor	Beam Stability Factor	Bearing Area Factor	Column Stability Factor	Condition Treatment Factor	Critical Section Factor	Format Conversion Factor	Resistance Factor
		Section 8.11.X➔	1	2	3	4	5	6	7	8	9	10	11	12	13	-	-
Solid-Sawn Posts (Post & Timber Size Category)	$F_b' = F_b$	x	λ	-	C_t	-	-	C_F		-	C_L	-	-	-	-	2.54	0.85
	$F_t' = F_t$	x	λ	-	C_t	-	-	-		-	-	-	-	-	-	2.70	0.80
	$F_v' = F_v$	x	λ	-	C_t	-	-	-		-	-	-	-	-	-	2.88	0.75
	$F_{c\perp}' = F_{c\perp}$	x	-	C_M	C_t	-	-	-		-	-	C_b	-	-	-	1.67	0.90
	$F_c' = F_c$	x	λ	C_M	C_t	-	-	-		-	-	-	C_P	-	-	2.40	0.90
	$E' = E$	x	-	-	C_t	-	-			-	-	-	-	-	-	-	-
	$E_{min}' = E_{min}$	x	-	-	C_t	-	-			-	-	-	-	-	-	1.76	0.85
Vertically-Laminated Glulam Posts	$F_b' = F_b$	x	λ	C_M	C_t	-	-	-	C_V	-	C_L	-	-	-	-	2.54	0.85
	$F_t' = F_t$	x	λ	C_M	C_t	-	-	-	-	-	-	-	-	-	-	2.70	0.80
	$F_v' = F_v$	x	λ	C_M	C_t	-	-	-	-	-	-	-	-	-	-	2.88	0.75
	$F_{c\perp}' = F_{c\perp}$	x	-	C_M	C_t	-	-	-	-	-	-	C_b	-	-	-	1.67	0.90
	$F_c' = F_c$	x	λ	C_M	C_t	-	-	-	-	-	-	-	C_P	-	-	2.40	0.90
	$E' = E$	x	-	C_M	C_t	-	-	-	-	-	-	-	-	-	-	-	-
	$E_{min}' = E_{min}$	x	-	C_M	C_t	-	-	-	-	-	-	-	-	-	-	1.76	0.85
Structural Composite Lumber Posts	$F_b' = F_b$	x	λ	C_M	C_t	-	-	-	C_V	-	C_L	-	-	-	-	2.54	0.85
	$F_t' = F_t$	x	λ	C_M	C_t	-	-	-	C_V	-	-	-	-	-	-	2.70	0.80
	$F_v' = F_v$	x	λ	C_M	C_t	-	-	-	-	-	-	-	-	-	-	2.88	0.75
	$F_{c\perp}' = F_{c\perp}$	x	-	C_M	C_t	-	-	-	-	-	-	C_b	-	-	-	1.67	0.90
	$F_c' = F_c$	x	λ	C_M	C_t	-	-	-	-	-	-	-	C_P	-	-	2.40	0.90
	$E' = E$	x	-	C_M	C_t	-	-	-	-	-	-	-	-	-	-	-	-
	$E_{min}' = E_{min}$	x	-	C_M	C_t	-	-	-	-	-	-	-	-	-	-	1.76	0.85
Round Construction Poles	$F_b' = F_b$	x	λ	-	C_t	-	-	C_F	-	-	-	-	-	C_{ct}	-	2.54	0.90
	$F_v' = F_v$	x	λ	-	C_t	-	-	-	-	-	-	-	-	C_{ct}	-	2.88	0.75
	$F_{c\perp}' = F_{c\perp}$	x	-	-	C_t	-	-	-	-	-	-	C_b	-	C_{ct}	-	1.67	0.90
	$F_c' = F_c$	x	λ	-	C_t	-	-	-	-	-	-	-	C_P	C_{ct}	C_{cs}	2.40	0.90
	$E' = E$	x	-	-	C_t	-	-	-	-	-	-	-	-	-	-	-	-
	$E_{min}' = E_{min}$	x	-	-	C_t	-	-	-	-	-	-	-	-	-	-	1.76	0.85
Unspliced Mechanically-Laminated Posts	$F_b' = F_b$	x	λ	C_M	C_t	C_i	C_{fu}	C_F	C_V	C_r	C_L	-	-	-	-	2.54	0.85
	$F_t' = F_t$	x	λ	C_M	C_t	C_i	-	C_F	C_V	-	-	-	-	-	-	2.70	0.80
	$F_v' = F_v$	x	λ	C_M	C_t	C_i	-	-		-	-	-	-	-	-	2.88	0.75
	$F_{c\perp}' = F_{c\perp}$	x	-	C_M	C_t	C_i	-	-		-	-	C_b	-	-	-	1.67	0.90
	$F_c' = F_c$	x	λ	C_M	C_t	C_i	-	C_F		-	-	-	C_P	-	-	2.40	0.90
	$E' = E$	x	-	C_M	C_t	C_i	-			-	-	-	-	-	-	-	-
	$E_{min}' = E_{min}$	x	-	C_M	C_t	C_i	-			-	-	-	-	-	-	1.76	0.85

8.11.1 Load Duration and Time Effect Factors, C_D and λ

Wood has the property of carrying significantly greater maximum loads for short durations of loading. In ASD, this relative increase in strength is accounted for with the load duration factor (Table 8-12). In LRFD, this increase is accounted for with the time effect factor (Table 8-13)

Table 8-12 Load Duration Factor, C_D [(a)]

Load Duration	C_D	Typical Design Loads
Permanent	0.9	Dead load
Ten year	1.0	Occupancy live load
Two months	1.15	Snow load
Seven days	1.25	Construction load
Ten minutes	1.60	Wind/earthquake load
Impact [(a)]	2.0	Impact load

(a) Load duration factors greater than 1.60 shall not apply to connections or structural members pressure-treated with waterborne preservatives or fire retardant chemicals

Table 8-13 Time Effect Factors, λ [(a)]

LRFD Load Combination [(b)]	Time Effect Factor, λ
$1.4 \cdot D$	0.6
$1.2 \cdot D + 1.6\ H + 0.5 \cdot (L_r$ or S or $R)$	0.6
$1.2 \cdot D + 1.6 \cdot L + 0.5 \cdot (L_r$ or S or $R)$	**(c)**
$1.2 \cdot D + 1.6 \cdot (L_r$ or S or $R) + (x \cdot L$ or $0.5 \cdot W)$	0.8
$1.2 \cdot D + 1.0 \cdot W + x \cdot L + 0.5 \cdot (L_r$ or S or $R)$	1.0
$1.2 \cdot D + 1.0 \cdot E + x \cdot L + 0.2 \cdot S$	1.0
$0.9 \cdot D + 1.0 \cdot W$	1.0
$0.9 \cdot D + 1.0 \cdot E$	1.0

(a) Time effect factors, λ, greater than 1.0 shall not apply to connections or to structural members pressure-treated with water-borne preservatives or fire retardant chemicals.
(b) x = 1.0 for garages, areas of public occupancy, and values of L greater than 100 lbf/ft^2. When L is less than or equal to 100 lbf/ft^2, set x equal to 0.5.
(c) 0.7 when L is from storage; 0.8 when L is from occupancy; 1.25 when L is from impact.

When induced stresses are due to a combination of loads, the ASD load duration factor associated with the shortest duration load in the combination should be applied for that load combination since the combined load will only be acting as long as the load with the shortest duration is present.

The time effect factor λ differs from the load duration factor C_D in that it is linked to specific load combinations via the reliability of the various loads in the combination. The time effect factor also differs from the load duration factor in that the baseline load duration for λ is 10 minutes, whereas that for C_D is 10 years.

8.11.2 Wet Service Factor, C_M

Wood strength and stiffness decrease as wood moisture content increases. Since reference design values are applicable to seasoned (dry) wood, they must be reduced in accordance with Table 8-14 when exposed to a high moisture environment for an extended time period. Specific wet service factors are not tabulated for structural composite lumber as such factors may be influenced by adhesives and fabrication techniques used in product manufacture - adhesives and manufacturing techniques which have not been standardized for these products. Tabulated reference design values for round construction poles are applicable to both wet and dry service conditions, and thus C_M does not apply to poles.

Table 8-14 Wet Service Factors, C_M

Material (In Service Moisture Content)	Reference Design Property	C_M
Dimension Lumber (greater than 19%)	F_b (when $F_b \bullet C_F \leq$ 1.150 kips/in^2)	1.00
	F_b (when $F_b \bullet C_F >$ 1.150 kips/in^2)	0.85
	F_t	1.00
	F_c (when $F_c \bullet C_F \leq$ 0.750 kips/in^2)	1.00
	F_c (when $F_c \bullet C_F >$ 0.750 kips/in^2)	0.80
	F_v	0.97
	$F_{c\perp}$	0.67
	E, E_{min}	0.90
Solid Sawn Timber (greater than 19%)	F_b, F_t, F_v, E, E_{min}	1.00
	F_c	0.91
	$F_{c\perp}$	0.67
Glulams (16% or greater)	F_b	0.80
	F_t	0.80
	F_v	0.875
	$F_{c\perp}$	0.53
	F_c	0.73
	E, E_{min}	0.833
Structural Composite Lumber (16% or greater)	See manufacturer recommendations	

8.11.3 Temperature Factor, C_t

As wood is cooled below normal temperatures, its strength increases; when heated, its strength decreases. This temperature effect is dependent on wood moisture content. Up to 150°F, the effect of temperature is assumed by design codes to be reversible. Prolonged heating to temperatures above 150°F can cause permanent loss of strength. For structural members and connections exposed to temperatures less that 150°F, reference design values are multiplied by the factors in Table 8-15. No adjustment is needed unless temperatures exceed 100°F for extended periods of time. For example, the adjustments are not required in applications where diurnal temperatures occasionally exceed 100°F.

Table 8-15. Temperature Factors, C_t

Reference Design Value	End-Use Moisture Content	C_t (a)	
		$100<T\leq 125$	$125<T\leq 150$
F_t, E, E_{min},	Dry or Wet	0.9	0.9
F_b, F_v, F_c, $F_{c\perp}$,	Dry	0.8	0.7
	Wet	0.7	0.5

(a) T = sustained temperature, °F

8.11.4 Incising Factor, C_i

Reference design values for dimension lumber shall be multiplied by the appropriate incising factor, C_i, from Table 8-16 when the lumber is incised to increase penetration of treatments with incisions cut parallel to grain with a maximum depth of 0.40 inches, a maximum length of 0.375 inches, and a maximum density of 1100 per square foot. Incising factors shall be determined by test for incising patterns exceeding these limits. The incising factor does not apply to solid-sawn posts, glulams, round poles or structural composite lumber.

Table 8-16. Incising Factors for Dimension Lumber, C_i

Reference Design Value	C_i
E, E_{min}	0.95
F_b, F_t, F_c, F_v	0.80
$F_{c\perp}$	1.00

8.11.5 Flat Use Factor, C_{fu}

With respect to post design, the flat use factor only applies to mechlam posts fabricated from dimension lumber. In such posts, the factors in Table 8-17 are used to increase the bending strength of individual laminates when they are being bent about the post's X-X axis (see figure 8-1(c)).

Table 8-17. Flat Use Factors for Nominal Two-Inch Thick Dimension Lumber, C_{fu}

Nominal Width, inches	C_{fu}
6	1.15
8	1.15
10 and wider	1.20

While there is a flat use adjustment factor for glulams, it does not apply to the reference design values for bending in Table 8-5, as the effect that orientation has on bending strength has already been effectively incorporated into the tabulated reference design values in Table 8-5.

8.11.6 Size Factor, C_F

The relative size of a wood member has an effect on its unit strength. This general behavior is known as the size effect. There are two adjustment factors used to adjust for member size: the size factor and the volume factor. The size factor applies to visually-graded dimension lumber and timber. The volume factor is used for glulam members and structural composite lumber.

Reference design stresses for bending, tension, and compression parallel-to-grain for all ***visually*** graded dimension lumber ***except Southern Pine between 2 and 4-inchs in nominal thickness*** are multiplied by the size factors in Table 8-18. These factors are from equations appearing in ASTM D1990 and are based on the in-grade testing program. The reason for exempting Southern Pine between 2 and 4-inches in nominal thickness from the adjustment in Table 8-18 is because the tabulated reference design values appearing in the NDS for Southern Pine in these thicknesses already include a size adjustment. This explains why the values for Southern Pine appearing in Table 8-7 are a function of width whereas those appearing in Table 8-7 for other species are not a function of width.

Table 8-18. Size Factors, C_F, For Nominal Two-Inch Thick Dimension Lumber (a)

Grade	Nominal Width, inches	F_b (b) and F_t	F_c
Select Structural, No.1 & Btr., No.1, No. 2, No. 3	4 and less	1.5	1.15
	5	1.4	1.1
	6	1.3	1.1
	8	1.2	1.05
	10	1.1	1.0
	12	1.0	1.0
	14 and wider	0.9	0.9

(a) For nominally two-inch thick Southern Pine, the value of C_F is 1.0 for all properties and all sizes 12 inches in width and less. For mechanically graded dimension lumber (Table 8-8), the value of C_F is 1.0 for all properties and all sizes.

(b) For bending about the strong axis only.

For visually graded timbers (reference design values in Table 8-2) whose actual depth, d, exceeds 12 nominal inches, multiply F_b by C_F where C_F is given as: $C_F = (12/d)^{1/9}$. C_F is equal to 1.0 for all other visually graded timber properties.

8.11.7 Volume Factor, C_V

The volume factor C_V is used to account for the size effect in structural glued-laminated timber and structural composite lumber. Historically, the size effect in glulams was accounted for by multiplying the reference design value for bending stress by the same size factor as used for visually grade timber ($C_F = (12/d)^{1/9}$). This changed after research showed the glulam size effect to be a function of all three dimensions and not just depth. The end result of this research was the volume factor, which is used to adjust F_{bx} (i.e., the reference bending design value for bending about axis X-X) when the glulam is loaded perpendicular to the wide face of the laminations. In equation form, the volume factor given as:

$$C_V = (21/L)^{1/x} \bullet (12/d)^{1/x} \bullet (5.125/b)^{1/x} \leq 1.0$$

where:

- L = length of bending member between points of zero moment, feet
- d = depth of bending member, inches
- b = width (breadth) of bending member. For multiple piece width layups, b = width of widest piece in the layup, inches
- x = 20 for Southern Pine
- x = 10 for all other species

The C_V equation is used to adjust F_{bx} whenever the glulam is other than 12 inches in depth, 5.125 inches in breadth and/or the distance between points of zero bending moment is not 21 feet. The different "x" values account for the volume factor being less pronounced in glulams fabricated from Southern Pine than glulams fabricated from other species. The volume factor for glulams is not to be applied simultaneously with the beam stability factor, C_L. Therefore apply only the lesser of the two factors.

Reference design values for bending (F_b) and tension (F_t) for structural composite lumber must be multiplied by the volume factor, C_V, in Tables 8-19 and 8-20, respectively. These values are from the ICC-ES Evaluations Reports listed in Table 8-3 for the products. In these reports, the volume factor for bending is often listed as a size factor and the volume factor for tension is listed as a length factor. Use herein of the term volume factor is consistent with the NDS and ASTM D5456. When C_V for bending is less than 1.0, the factor shall not apply simultaneously with the beam stability factor C_L, and therefore the lesser of these adjustment factors shall apply. However, when C_V for bending is greater than 1.0, the factor shall apply simultaneously with the beam stability factor C_L.

Table 8-19. Volume Factor for SCL Bending

SCL Type	Product Name	Orien-tation	C_V for Bending[a]	
			$d > 3.5$ inches	$d \leq 3.5$ inches
LSL	SolidStart	Beam	$(12/d)^{0.143}$	1.19
		Plank	1.00	1.00
	Timberstrand	Beam	$(12/d)^{0.092}$	1.12
		Plank	1.00	1.00
LVL	Microlam	Beam	$(12/d)^{0.136}$	1.18
		Plank	1.00	1.00
	SolidStart	Beam	$(12/d)^{0.143}$	1.15
		Plank	1.00	1.00
	Versa-Lam	Beam	$(12/d)^{0.111}$	1.15
		Plank	1.00	1.00
PSL	Parallam	Beam	$(12/d)^{0.111}$	1.15
		Plank	$(12/d)^{0.111}$	1.15

(a) d is depth in inches. Depth is the dimension of the member in the same direction of (i.e. parallel to) the applied load.

Table 8-20. Volume Factor for SCL Tension

SCL Type	Product Name	C_V for Tension[a]
LSL	SolidStart	$(3/L)^{0.092}$ for $L > 3$ ft 1.00 for $L \leq 3$ ft
	Timberstrand	1.00
LVL	Microlam	1.00
	SolidStart	$(3/L)^{0.111}$ for $L > 3$ ft 1.00 for $L \leq 3$ ft
	Versa-Lam	$(4/L)^{0.125}$ for $L > 4$ 1.00 for $L \leq 4$ ft
PSL	Parallam	1.00

(a) L is member length in feet

8.11.8 Repetitive Member Factor, C_r

The reference bending design value for a given group of structural framing members (e.g., lumber of the same species, grade and size) is based on the weakest members in the group. These same weak members tend to be less stiff and thus deflect more than stronger members in the same group when both the weak and strong members are subjected to the same load.

When both a flexible and a stiff member are forced to deflect the same amount by a load-distributing element, the stiffer member will resist more load. This is advantageous given the probability that the stiffer member is more likely to be the stronger member. It follows that the total amount of load that such a "load-sharing system" will be able to support without failure is more than would be predicted based on the strength of the weaker members in the group. To account for this increase in overall strength due to load-sharing, the reference bending design value is multiplied by the repetitive member factor, C_r.

The magnitude of the repetitive member factor depends on the variation in bending stiffness and bending strength of the components within a group. The variation in bending stiffness controls the magnitude of load that a load distributing element transfers between the individual components. The less variation in bending stiffness, the lower the magnitude of load that is transferred, and the lower will be the repetitive member factor. The less variation in bending strength between individual members, the less there is to be gained from load-sharing, and thus the lower will be the repetitive member factor as well.

The ultimate in load-sharing occurs between layers in vertically-laminated assemblies (figure 8-1(d)) especially when individual layers contain numerous strength-reducing knots. For glulams, this load sharing is already incorporated into the reference bending design strength F_{by} and thus no further increase for repetitive member use is allowed. For mechanically-laminated assemblies, multiply F_b for bending about the Y-Y axis by the applicable repetitive member factor from Table 8-21.

Table 8-21 Repetitive Member Factors for Mechlams Bent About Axis Y-Y [a]

Laminate Material	Number of laminations	
	3	4
Visually graded dimension lumber	1.35	1.40
Mechanically graded dimension lumber	1.25	1.30
Structural composite lumber (SCL)	1.04	1.04

(a) For mechlams with minimum interlayer shear capacity as specified in Table 8-22.

The factors in Table 8-21 are from ANSI/ASAE EP559 *Design Requirements and Bending Properties for Mechanically-Laminated Wood Assemblies*. These values apply when the actual thickness of each lamination is between 1.5 and 2.0 inches (38 and 51 mm), all laminations have the same depth (face width), faces of adjacent laminations are in contact and joined with connections that meet the requirement in Table 8-22 and 8-23, all laminations are of the same material (e.g., grade and species of lumber), and the centroid of each lamination is located on the centroidal axis of the assembly (axis Y-Y in figure 8-1(d)), that is, no laminations are offset.

As long as the interlayer shear capacities in Table 8-22 are met, the mechanical connections joining the individual layers in a mechanically-laminated wood assembly function as an effective load distributing element between the layers, and thus no other "external" load distributing element is needed to apply the factors in Table 8-21. The capacities in Table 8-22 are expressed in units of force per unit length of the assembly.

Table 8-22. Minimum Required Interlayer Shear Capacities for Mechlams Bent About Axis Y-Y (Figure 7-1)[a]

Actual face width of laminations, inches	Minimum required interlayer shear capacity per interface per unit length of assembly, lbf/in.	
	Allowable Stress Design (ASD)	Load and Resistance Factor (LRFD) Design
5.5	12	16
7.25	15	20
9.25	19	26
11.25	24	32

(a) For unspliced mechlams, mechlams with either common glued end joints and/or certified structural glued end joints, and unspliced regions of mechlams with butt joints.

Table 8-23. Minimum Fastener Spacings for Mechlams Bent About Axis Y-Y (Figure 7-1)

Dimension	Nail/screw diameters
Edge distance	10
End distance	15
Spacing (pitch) between fasteners in a row	20
Spacing (gage) between rows of fasteners	
- in-line	10
- staggered	5

8.11.9 Beam Stability Factor, C_L

Anytime a member is subjected to bending forces, a portion of the member is in compression, and there is a tendency for this portion of the member to buckle laterally. This buckling results in both a lateral displacement and twist of the member as shown in figure 8-21 and thus is referred to as lateral-torsional buckling. The tendency for this buckling to occur increases as (1) member thickness decreases, (2) member depth increases, and/or (3) the distance between points of lateral support increases.

To ensure that lateral torsional buckling does not take place, the reference bending design strength is reduced by multiplying it by the beam stability factor, C_L (this effectively reduces the amount of moment that can be induced in the member where lateral torsional buckling is likely). Where it is highly unlikely that a member will laterally displace under load, C_L is equal to 1.00. This would include situations where member depth, does not exceed effective member thickness, as well as situations where the compressive edge of the member is supported throughout its length to prevent lateral displacement.

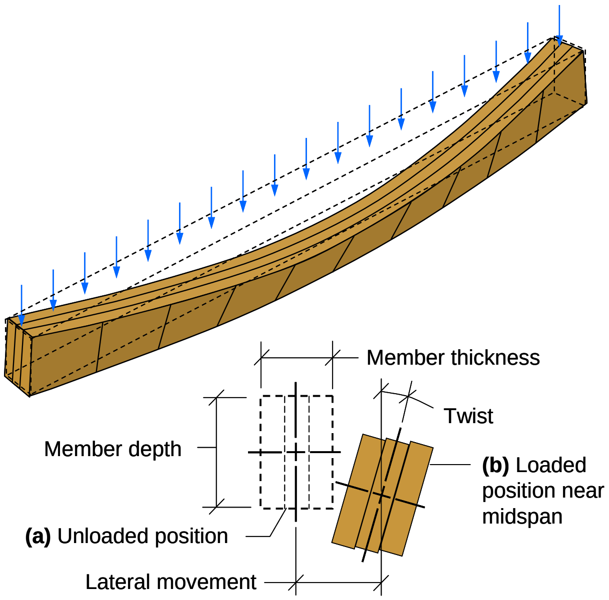


Figure 8-21. Perspective view (top) and cross-sectional view (bottom) of lateral-torsional buckling of a mechlam assembly loaded in bending about its Y-Y axis.

The first step in calculating C_L is to establish the slenderness ratio, R_B, which is given as:

$$R_B = (L_e \cdot d / b_e^2)^{1/2} \leq 50 \qquad (8\text{-}2)$$

where:

R_B	=	Slenderness ratio, dimensionless
b_e	=	Effective member thickness, inches (mm)
	=	b for all posts except mechlam posts
	=	0.60 b for mechlam posts
b	=	Actual member thickness, inches (mm)
d	=	Actual member depth, inches (mm)
L_e	=	Effective span length, inches (mm)

ANSI/ASAE EP559 requires *effective* member thickness b_e to be set equal to 60% of actual total assembly thickness. This reduced effective thickness accounts for the interlayer slip (i.e. lack of complete composite action) shown in figure 8-21 that occurs as the laminates work to resist lateral movement.

The effective span length L_e in equation 8-2 is a function of the laterally unsupported length L_u which is the actual distance between supports that prevent lateral displacement under the applied bending load. In figure 8-22, L_u is equal to the girt spacing when the post is being bent out of the plane of the wall (i.e., being bent about mechlam post axis Y-Y) and the girts themselves are prevented from shifting horizontally.

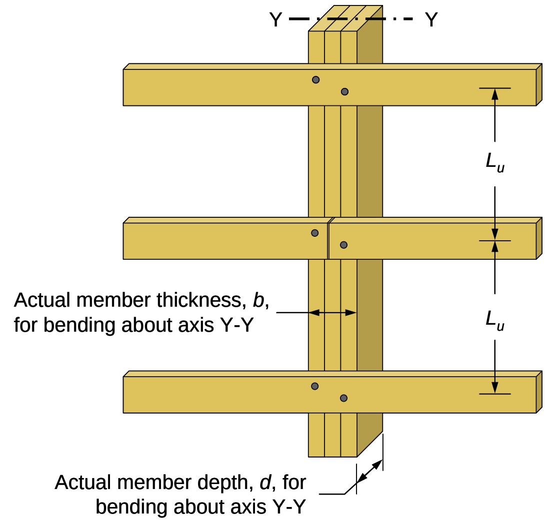


Figure 8-22. Variable definitions for post bending about its Y-Y axis.

Conservative estimates of L_e are provided by the following equations.

$$L_e = 2.06 \cdot L_u \qquad \text{for } L_u/d < 7 \qquad (8\text{-}3)$$

$$L_e = 1.63 \cdot L_u + 3 \cdot d \qquad \text{for } 7 \leq L_u/d \leq 14.3 \qquad (8\text{-}4)$$

$$L_e = 1.84 \cdot L_u \qquad \text{for } L_u/d \geq 14.3 \qquad (8\text{-}5)$$

For posts that are both loaded (in bending) and laterally supported by girts or other horizontal members that are uniformly spaced, the effective span length can be obtained using the equations in Table 8-24. For in-depth information on calculation of L_e see *Designing for Lateral-Torsional Stability in Wood Members* (AWC, 2003).

Table 8-24. Effective Span Length for Posts With Bending Loads Applied by Girts [(a)]

Number of Uniformly-Spaced Girts Between Top and Bottom of Post [(b)]	L_e [(c)]
1	1.11 L_u
2	1.68 L_u
3	1.54 L_u
4	1.68 L_u
5	1.73 L_u
6	1.78 L_u
7 or more	1.84 L_u

(a) Girts assumed to provide adequate lateral support at their point of attachment, and all loads applied to the side of the post are assumed to occur at these points of attachment.
(b) Do not count girts at very top and bottom of post.
(c) Girts uniformly spaced an amount L_u.

In accordance with equation 8-2, R_B can not exceed 50. If it does, either member depth must be decreased, member thickness increased, and/or the distance between points of lateral support decreased.

Once the slenderness ratio has been determined, the beam stability factor can be calculated as:

$$C_L = \frac{1+\alpha_b}{2\cdot c} - \left[\frac{(1+\alpha_b)^2}{4\cdot c^2} - \frac{\alpha_b}{c}\right]^{1/2} \qquad (8\text{-}6)$$

where:

$$\alpha_b = \frac{F_{bE}}{F_b*} = \frac{1.20\cdot E_{min}'}{F_b*\cdot R_B^2}$$

c = 0.95

F_b* = F_b multiplied by all applicable adjustment factors except C_{fu}, C_V and C_L, lbf/in^2

F_{bE} = Critical buckling design stress, lbf/in^2 = $1.20\cdot E_{min}'/R_B^2$

E_{min}' = Adjusted minimum modulus of elasticity, , lbf/in^2

R_B = Slenderness ratio from equation 8-2

8.11.10 Bearing Area Factor, C_b

The reference strength values for compression perpendicular-to-grain, $F_{c\perp}$, apply to bearings of any length at the ends of a member, and to all bearings 6 inches or more in length at any other location. For bearings less than 6 inches in length and not nearer than 3 inches to the end of the member $F_{c\perp}$ can be multiplied by the following bearing area factor, C_b:

$$C_b = (L_b + 0.375) / L_b \qquad (8\text{-}7)$$

where L_b is the bearing length measured parallel-to-grain in inches. For round bearing areas such as washers, bearing length L_b, is taken as the diameter of the bearing area.

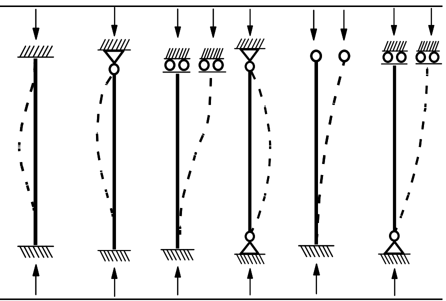
8.11.11 Column Stability Factor, C_P

The column stability factor is used to reduce F_c values so that buckling (instability) failures do not occur. When a compression member is supported throughout its length to prevent lateral displacement in all directions, C_P = 1.0.

The first step in calculating C_P is to calculate the slenderness ratio, L_e/d, for the column. L_e is the effective column length of a compression member. It is the distance between two points along the member length at which the member is assumed to buckle in the shape of a sine wave.

To calculate effective column length L_e multiply the actual column length (or the length of the column between lateral supports), L_u, by the appropriate effective buckling length factor, K_e from Table 8-25. Where a post has multiple inflection points due to multiple lateral supports as shown in figure 8-23, the effective buckling length between each set of supports may need to be calculated unless it is obvious which one is the largest, and hence will control buckling in the direction in question.

Table 8-25. Effective Length Factors for Wood Column Design



Theoretical K_e value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design K_e when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.4

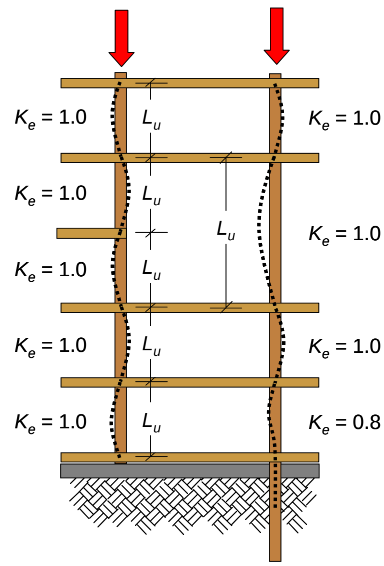


Figure 8-23. Effective buckling length $L_e = K_e\,L_u$ will vary along a post as L_u and K_e change. The maximum value will control buckling for the direction in question.

For all posts the slenderness ratio, L_e/d, is taken as the larger of the ratios $K_e\cdot L_1/d_1$ and $K_e\cdot L_2/d_2$ as defined in figure 8-24. The slenderness ratio for solid columns, L_e/d, is not allowed to exceed 50, except during construction when L_e/d is not allowed to exceed 75.

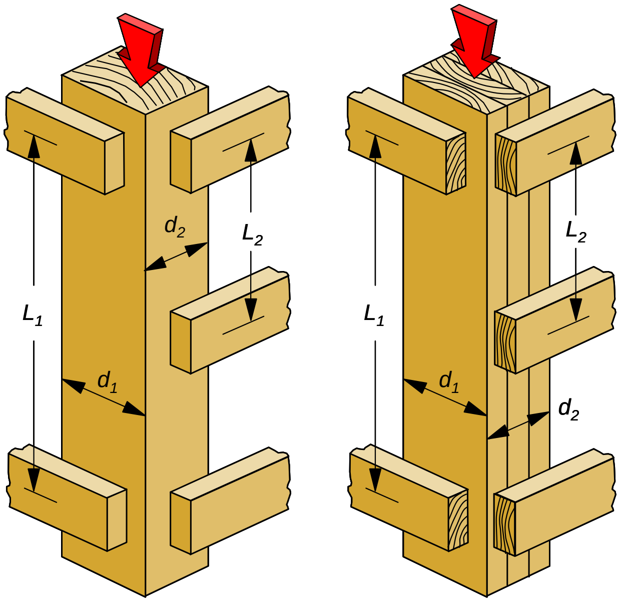


Figure 8-24. Solid post (left) and mechlam post (right) under an axial compressive load. For mechlam posts L_1 and L_2 are the distances between points of lateral support in a direction parallel and perpendicular to interlayer planes, respectively, d_1 is the face width of an individual lamination and d_2 is the thickness of the assembly.

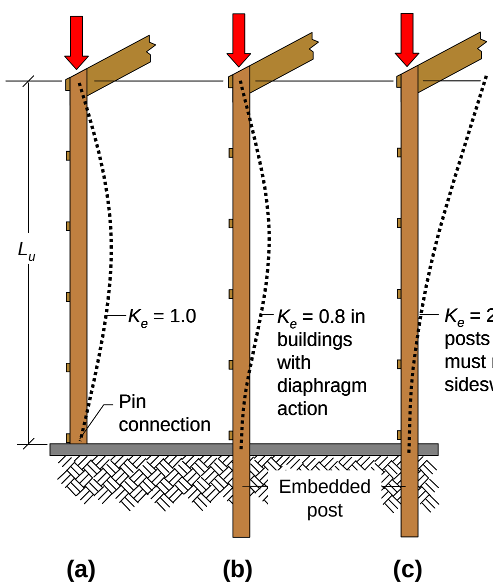


Figure 8-25. K_e for out-of-wall-plane buckling associated with common post support conditions. In all three of the above cases, the post is assumed pin-connected to the truss/rafter.

The ability of an individual post-frame to sidesway determines the effective length of the posts that are part of the post-frame. In buildings that rely on diaphragm action, sideway is prevented for all practical purposes, and the effective length factor K_e is assumed to never exceed 1.0 (see figures 8-25(a) and 8-25(b)). Conversely, in buildings that do not rely on diaphragm action, sidesway is largely dependent on the flexural stiffness of the posts, and this sidesway can be significant. In such buildings, if the post is embedded and pin-connected to the truss/rafter, K_e is taken as 2.1 (Table 8-25) as shown in figure 8-25(c).

Properly embedded posts provide significant resistance to base rotation, especially when restrained at grade by a concrete slab. With respect to determination of effective length, it is general practice to model all embedded posts with fixed supports, and all non-embedded posts with pin supports.

Once the slenderness ratio has been determined, the column stability factor for all posts *except* mechlams is calculated as:

$$C_P = \frac{1+\alpha_c}{2\cdot c} - \left[\frac{(1+\alpha_c)^2}{4\cdot c^2} - \frac{\alpha_c}{c}\right]^{1/2} \qquad (8\text{-}8)$$

where:

$$\alpha_c = \frac{F_{cE}}{F_c^*} = \frac{0.822\cdot E_{min}'}{F_c^*\cdot(L_e/d)^2}$$

c = 0.80 for solid sawn posts
= 0.85 for round poles
= 0.90 for glued laminated posts and posts fabricated from a single piece of structural composite lumber

F_c^* = F_c multiplied by all applicable adjustment factors except C_P, lbf/in^2

F_{cE} = Critical buckling design stress, lbf/in^2
= $0.822\cdot E_{min}'/(L_e/d)^2$

E_{min}' = Adjusted minimum modulus of elasticity, ksi

L_E/d = Slenderness ratio for column
= Maximum of $K_e\, L_1/d_1$ and $K_e\, L_2/d_2$

For mechlam posts, column stability factor C_P is calculated using the procedure outlined in Section 15.3.2 of the NDS as:

$$C_P = K_f\left[\frac{1+\alpha_c}{2\cdot c} - \left[\frac{(1+\alpha_c)^2}{4\cdot c^2} - \frac{\alpha_c}{c}\right]^{1/2}\right] \qquad (8\text{-}9)$$

where:

α_c = F_{cE}/F_c^*

c = 0.80 for dimension lumber laminates
= 0.90 for SCL laminates

$F_c{}^*$ = reference compression design value parallel-to-grain for a single laminate multiplied by all applicable adjustment factors except C_P (see equation 21 definitions)
F_{cE} = $0.822 \cdot E_{min}'/(L_e/d)^2$
E_{min}' = Adjusted minimum modulus of elasticity for a single laminate
K_f = 0.60 for nail- and screw-laminated assemblies where L_e/d is equal to $K_e L_2/d_2$
= 0.75 for bolt-laminated assemblies where L_e/d is equal to $K_e L_2/d_2$
= 1.0 for mechlams where L_e/d is equal to $K_e L_1/d_1$
L_e/d = Slenderness ratio for the assembly
= Maximum of $K_e L_1/d_1$ and $K_e L_2/d_2$

where:

K_e = Buckling length coefficient from Table 8-25
L_1 = Distance between supports running parallel to interlayer planes (see figures 8-23 and 8-24)
L_2 = Distance between supports running normal to interlayer planes (see figures 8-23 and 8-24)
d_1 = Face width of an individual laminate (see figures 8-23 and 8-24)
d_2 = Laminated assembly thickness (see figure 8-24)

The K_f factors of 0.60 and 0.75 require that nailing/screwing and bolting are performed in accordance with the provisions of NDS 15.3.3 and 15.3.4, respectively. In lieu of meeting the nailing/screwing requirements in NDS 15.3.3, the more conservative provisions in Table 8-23 can be followed.

8.11.12 Condition Treatment Factor, C_{ct}

Reference design values for round construction poles are based on air-dried conditioning. If either kiln-drying, steam-conditioning, or boultonizing are used prior to treatment, then the reference design values must be reduced. This is accomplished by multiplying the design values by a C_{ct} of 0.90 for kiln-drying, 0.95 for boulton drying, 0.80 for normal steaming, and 0.74 for marine steaming.

8.11.13 Critical Section Factor, C_{cs}

Reference compression design values parallel-to-grain for round construction poles, F_c, are based on the strength at the tip of the pole. To increase this value at other locations along the post, F_c can be multiplied by the critical section factor C_{cs} calculated as:

$$C_{cs} = 1.0 + 0.004\, L_c \qquad (8\text{-}10)$$

where L_c is the distance in feet from the tip of the pile to the point at which F_c is being determined. This increase for location cannot exceed 10% ($C_{cs} \leq 1.10$).

8.12 Controlling Design Equations

Of the major NDS equations for checking component strength, there are typically only three that are utilized when sizing post-frame building posts. These include equation 8-11 for shear, equation 8-12 for axial compression without bending, and equation 8-13 for axial compression with uniaxial bending.

$$f_v \leq F_v' \qquad (8\text{-}11)$$

$$f_c \leq F_c' \qquad (8\text{-}12)$$

$$(f_c/F_c')^2 + f_b/\{F_b'[1-(f_c/F_{cE})]\} \leq 1.0 \qquad (8\text{-}13)$$

where:

f_v = actual shear stress
= $1.5\, V/(bd)$ for rectangular members
f_c = actual compressive stress
= $P/(bd)$ for rectangular members
f_b = actual bending stress
= $6M/(bd^2)$ for rectangular members
F_v' = adjusted shear design value
F_c' = adjusted compressive design value parallel-to-grain
F_b' = adjusted bending design value
F_{cE} = critical buckling design value for compression
= $0.822\, E_{min}'/(L_e/d)^2$
d = post dimension measured parallel to plane of bending (see figure 8-26)
b = post dimension measure perpendicular to plane of bending (see figure 8-26)
L_e = effective buckling length
= $K_e L_u$ where L_u is depicted in figure 8-26

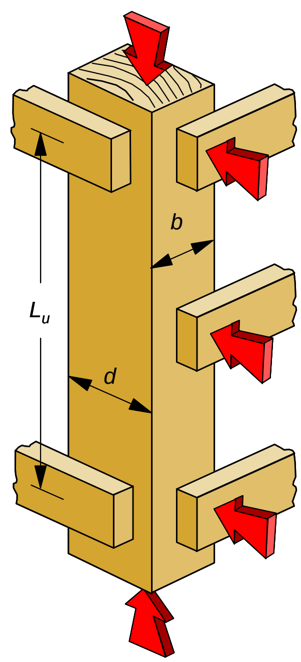


Figure 8-26. Variable description for axial compression with uniaxial bending.

NDS equations for axial tension, axial tension combined with bending, and bi-axial bending with compression are not given here as they are either not applicable to, and/or they will not control post-frame building post design.

Equation 8-13 is the governing equations for *uniaxial* bending and compression. Seldom is a post in a post-frame building subjected to high bending forces about both axes. This is because posts that are subjected to measureable bending forces are typically well supported in the plane normal to the direction that they are being bent.

For design of a post without any bending, equations 8-11 and 8-12 govern the design. Design of a post with bending moments is governed by equations 7-11 and 7-13. Typically the only time that bending moments are not induced in a post is when it is both pin-connected on each end and not part of a wall.

Although equations 8-11, 8-12, and 8-13 must be met along the entire length of the post, they are typically only checked at the locations of maximum positive and maximum negative bending moment.

8.13 Example Calculations

8.13.1 Critical Frame Analysis

Problem Statement

Determine the forces in both posts of the critical frame of a 4 bay building with interior post-frames described and loaded as shown in figure 8-27. The posts are nominal 6- by 6-inch No. 2 Southern Pine, that are embedded 3.5 feet. Posts are not attached to the footing upon which they bear. A slab-on-grade restricts inward post movement, but does not restrict movement away from the slab. Trusses are pin-connected to the posts. Truss chords are nominal 2- by 6-inch No. 1 SP. Truss webs are nominal 2- by 4-inch No. 2 SP. Both backfill and surrounding soil are best classified as a loose, silty fine-grained sand. The water table is located several feet below the footing. Each roof slope has a horizontal diaphragm stiffness of 6000 lbf/inch, the ceiling has a horizontal diaphragm stiffness of 4000 lbf/in, and both endwalls have a frame stiffness of 2000 lbf/inch.

Solution 1: Post Forces by Plane-Frame Structural Analysis

The post-frame shown in figure 8-27 was previous analyzed in Chapter 6. Calculated interior frame stiffness k and eave load R were 71.5 lbf/inch and 1111 lbf, respectively. DAFI output (figure 6-18) shows that the frame with greatest eave displacement (i.e., the critical frame) is the frame at the center of the building. This frame has a horizontal eave displacement of 1.18 inches, and resists 84 lbf of the applied eave load of

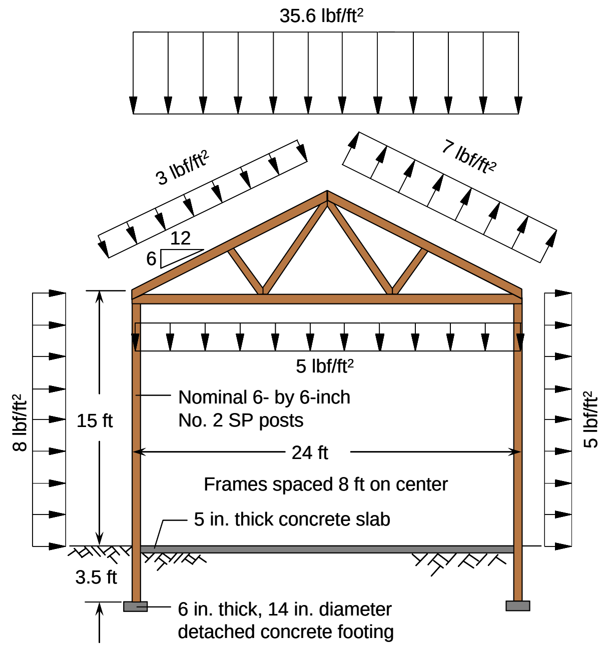


Figure 8-27. Interior post-frame of example 4 bay building.

1111 lbf. The difference of 1027 lbf between these two forces (1027 = 1111 – 84) is the total sidesway restraining force Q, which can be looked at as the load applied to the frame by the diaphragms, or as the eave load transferred away from the frame by the diaphragms.

After total sidesway restraining force Q is determined, it can be applied at the same location as the eave load (figure 8-28(a)), or it can be converted to a series of uniformly distributed forces ($q_{p,i}$ values) applied along diaphragm-to-frame attachment lines as shown in figure 8-28(b). The Q to $q_{p,i}$ conversions are accomplished using equation 8-1 as follows:

$$q_{p,i} = Q\,(c_{h,i} / C_h) / b_i$$

$$q_{p,roof} = 1027 \text{ lbf } (6000/16000) / 12 \text{ ft} = 32.1 \text{ lbf/ft}$$

$$q_{p,ceiling} = 1027 \text{ lbf } (4000/16000) / 24 \text{ ft} = 10.7 \text{ lbf/ft}$$

Post forces resulting from plane-frame structural analyses of the analogs in figures 8-28(a) and 8-28(b) are shown in figures 8-29(a) and 8-29(b), respectively. Although the two analogs provide virtually identical results for post shear and bending moments, there is almost a 5% difference in post axial forces. This is because the center of action for the forces uniformly distributed along the diaphragms is higher than it is for force Q when Q is applied at the eave. This higher center of action produces a greater "total building" overturning moment which increases axial force in the left post and decreases axial force in the right post.

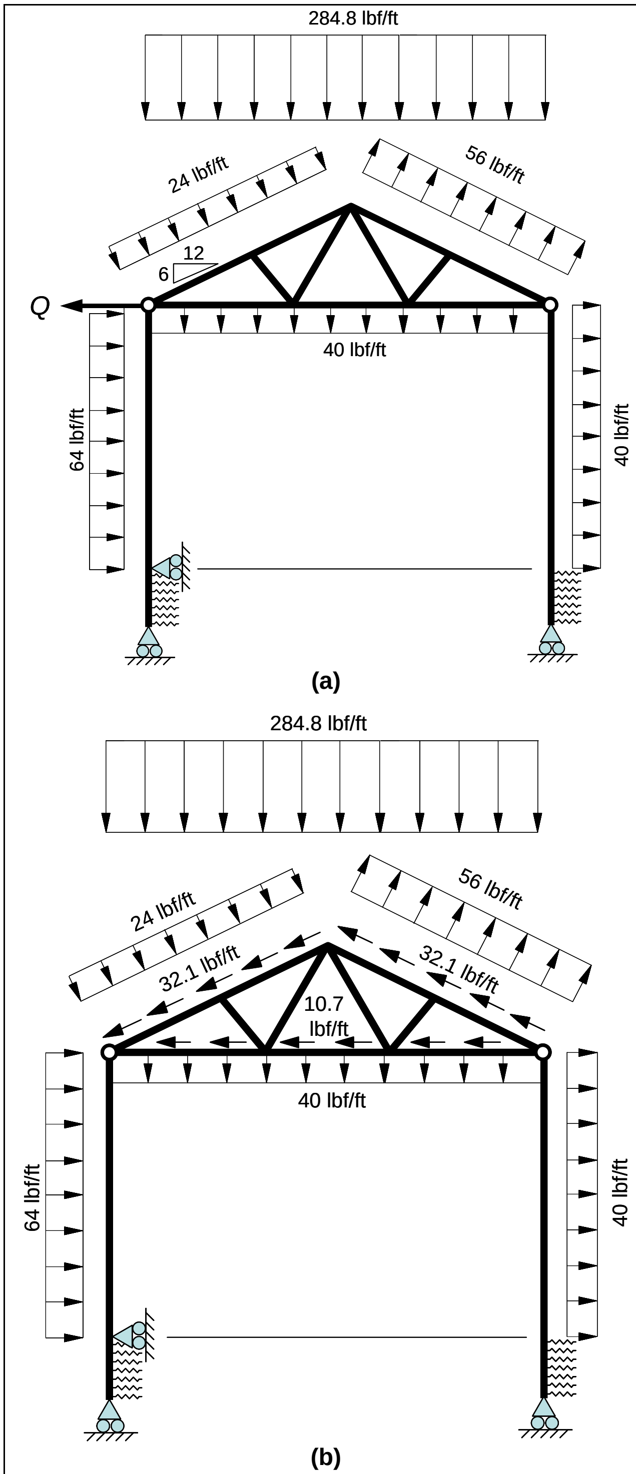


Figure 8-28. Structural analogs for determination of post forces featuring (a) direct application of the sidesway restraining force *Q*, and (b) replacement of *Q* with uniformly distributed loads acting in the plane of the diaphragms.

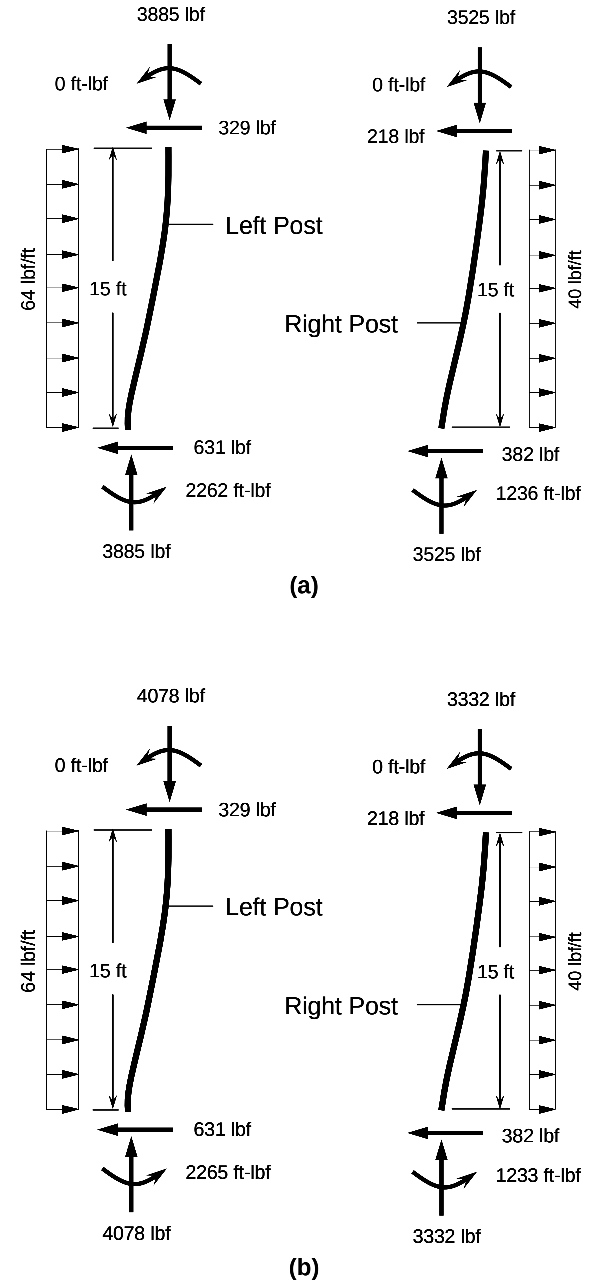


Figure 8-29. Results of the plane structural analyses with (a) the sidesway restraining force Q applied at the eave, and (b) uniformly distributed restraining forces applied along diaphragm-to-frame attachment lines.

Solution 2: Post Forces by Equations of Static Equilibrium

As an alternative to using a plane-frame structural analysis program, post fixity factors were used to determine frame stiffness and eave load values, DAFI was then used to determine eave displacements, and the equations in Table 8-1 were used to determine shear force and bending moments in the posts of the critical frame.

Two separate analyses were conducted. For the first analysis, posts were assumed to be fixed at grade and pin-connected to trusses (i.e., post fixity case 3 in Tables 6-1, 6-5 and 8-1). For the second analysis, the left post was treated as a surface-constrained embedded post pinned to the truss (post fixity case 7) and the right post was modeled as a non-constrained embedded post pinned to the truss (fixity case 5). Results of these two analyses are summarized in Table 8-26. Color coding in this table represents different steps in the analysis. Pink, green and blue represent application of the equations in Tables 6-1, 6-5 and 8-1, respectively. Yellow identifies values from DAFI.

Table 8-26. Results for Analyses Utilizing Post Fixity Factors(a)

Property	Analysis 1 (fixed-pinned)	Analysis 2 (embedded-pinned)
Stiffness of left post, lbf/in	47.0	46.4
Stiffness of right post, lbf/in	47.0	40.5
Interior frame stiffness, *k*, lbf/in	94.1	86.9
Eave load, *R*, lbf	1065	1083
Eave displacement of middle (i.e., critical) frame, Δ, inches	1.11	1.13
Load resisted by critical frame, lbf	104.3	98.5
Total sidesway restraining force for critical frame, *Q*, lbf	960.7	984.5
Groundline shear, left post, lbf	652.2	651.1
Goundline shear, right post, lbf	427.2	406.2
Top shear, left post, lbf	-307.8	-308.9
Top shear right post, lbf	-172.8	-193.8
Moment at grade, left post, ft-lbf	-2582	-2566
Moment at grade, right post, ft-lbf	-1907	-1594

(a) For s = 8 ft; H = 15 ft; d = 3.5 ft; E = 1,200,000 lbf/in^2; I = 76.25 in^4; A_E = 73.33 lbf/in^2; C = 74.81; q_{ww} = 8 lbf/ft^2, q_{lw} = -5 lbf/ft^2, q_{wr} = 3 lbf/ft^2; q_{lr} = -7 lbf/ft^2.

To obtain post axial forces, a free body diagram of the truss was drawn (figure 8-30). Moments were summed around the top of the left post to obtain the force in the right post. Forces were then summed in the vertical direction to obtain the force in the left post.

The loads on the FBD in figure 8-30 include those for a total sidesway restraining force of 1027 lbf, which induces the in-plane diaphragm forces shown in Figure 8-28(b). With these loads, the axial forces match within round-off error (as expected) those shown in figure 8-29(b). Using the 961 lbf sideway restraining force from Analysis 1 in Table 8-26 results in axial forces of 4062 lbf and 3349 lbf for the left and right posts, respectively. These are only 0.3% different from those in figure 8-30.

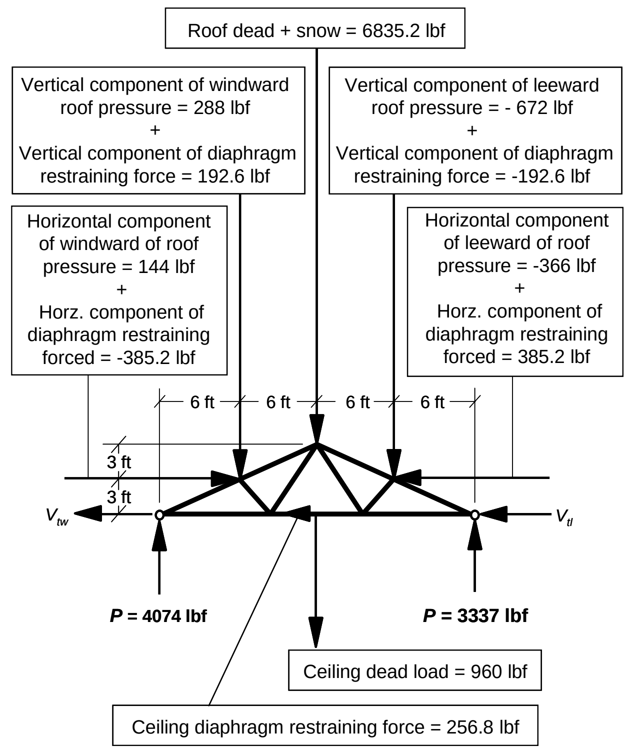


Figure 8-30. Free body diagram of the truss used to determine post axial forces.

8.13.2 Post Design Check

Problem Statement

Using allowable stress design, determine if the windward post in the previous problem (Section 8.13.1) is adequate.

Solution

For a nominal 6- by 6-inch timber:

b = 5.5 inches
d = 5.5 inches
A = 30.25 in^2
S = 27.73 in^3
I = 76.25 in^4

For No. 2 Southern Pine timber, the tabulated reference design values from Table 8-2 are:

F_b = 850 lbf/in.^{2}
F_v = 165 lbf/in.^{2}
F_c = 525 lbf/in.^{2}
E = 1,200,000 lbf/in.^{2}
E_{min} = 440,000 lbf/in.^{2}

Applicable adjustment factors for these design values from Table 8-10 are:

$F_b' = F_b\, C_D\, C_t\, C_F\, C_L$
$F_v' = F_v\, C_D\, C_t$
$F_c' = F_c\, C_D\, C_M\, C_t\, C_P$
$E' = E\, C_t$
$E_{min}' = E_{min}\, C_t$

where:

C_D = 1.60 since the shortest duration load in the combination of loads is wind
C_t = 1.00 for use under normal temperatures
C_M = 0.91 for wood located near the soil surface
C_F = 1.00 for nominal 6- by 6-inch No.2 Southern Pine
C_L = 1.00 since post is square
C_P = 1.00 at the base of the post where support is provided in both directions
C_P = less than 1.00 between points of lateral support. Calculated using equation 8-8

Substituting in known values yields:

F_b' = 850 lbf/in^2(1.60)(1.0)(1.0)(1.0) = 1360 psi
F_v' = 165 lbf/in^2(1.60)(1.0) = 264 lbf/in^2
F_c' = 525 lbf/in^2(1.60)(0.91)(1.0)C_P = 765 C_P psi
E' = 1,200,000 lbf/in.^{2} (1.0) = 1,200,000 lbf/in.^{2}
E_{min}' = 440,000 lbf/in.^{2}(1.0) = 440,000 lbf/in.^{2}

For calculation of the column stability factor, L_u is taken as the post height of 180 inches, and K_e is set equal to 0.80 resulting in an L_e value of 144 inches and an L_e/d ratio 26.2. This value is assumed to far exceed the value for buckling within the plane of the wall because of lateral support provided in the plane of the wall by girts.

Once the slenderness ratio is known, the critical buckling design value for compression F_{cE} can be calculated as:

$$F_{cE} = 0.822 \cdot E_{min}'/(L_e/d)^2$$
$$= 0.822(440{,}000)/(26.2)^2 = 527 \text{ lbf/in}^2$$

α_c is then calculated as F_{cE}/F_c^* where F_c^* is the reference design value for compression multiplied by all factors except C_P, or:

$$\alpha_c = F_{cE}/F_c^* = 527 \text{ lbf/in}^2 / 765 \text{ lbf/in}^2 = 0.689$$

For an α_c of 0.689, C_P from equation 8-8 is equal to 0.552, and $F_c' = F_c^*\, C_P$ = 423 lbf/in^2

Actual stresses are based on maximum forces, which for the loading in Section 8.13.1 (figure 8-27b) are:

M_{neg} (at base) = 2265 ft-lbf = 27180 in-lbf
M_{pos} (119 inches from base) = 10148 in-lbf
V (at base) = 631 lbf
P = 4078 lbf

Actual maximum stresses:

f_b (at base) = M/S =27180 in-lbf/27.73 in^3 = 980 lbf/in^2
f_b (at 119 in.) = 10148 in-lbf/27.73 in^3 = 366 lbf/in^2
f_v = 1.5 V/A = 1.5(631 lbf)/ 30.25 in^2 = 31.3 lbf/in^2
f_c = P/A = 4078 lbf/30.25 in^2 = 135 lbf/in^2

Check of controlling equation 7-11:

f_v = 31.3 lbf/in^2 $\leq$ F_v' = 264 lbf/in^2 (O.K.)

Check of controlling equation 7-13 for combined loading at the base of post (C_P = 1.0 and F_c' = 765 lbf/in^2). In this case, L_e is taken to be very small because of surrounding support (and thus the "f_c/F_{cE}" term in the equation approaches zero) and equation 8-13 becomes:

$$(f_c/F_c')^2 + f_b/F_b' \leq 1.0$$

$(135/765)^2 + 980/1360 = 0.031+0.720 \leq 1.0$ (O.K.)

Check of controlling equation 8-13 for combined loading at the point of maximum positive moment:

$$(f_c/F_c')^2 + f_b/\{F_b'\,[1 - (f_c/F_{cE})]\} \leq 1.0$$

$(135/423)^2 + 366/\{1360[1-(135/527)]\}$ =
$0.102 + 0.362 = 0.463 \leq 1.0$ (O.K.)

8.14 References

8.14.1 Non-Normative References

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8.14.2 Normative References

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IIC-ES Evaluation Report ESR-2403. Evaluation Subject: *LP® Solidstart® Laminated Strand Lumber (LSL) and Laminated Veneer Lumber (LVL).* Report Holder: Louisiana Pacific Corporation. International Code Council Evaluation Service. www.icc-es.org

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