



# **AISI STANDARD**

## **Commentary on Standard for Cold-Formed Steel Framing - Prescriptive Method for One and Two Family Dwellings**

**2007 Edition with Supplement 2**

**Revision of:  
AISI/COFS/PM-2006**

Endorsed by Steel Framing Alliance



### DISCLAIMER

The material contained herein has been developed by the American Iron and Steel Institute (AISI) Committee on Framing Standards. The Committee has made a diligent effort to present accurate, reliable, and useful information on cold-formed steel framing design and installation. The Committee acknowledges and is grateful for the contributions of the numerous researchers, engineers, and others who have contributed to the body of knowledge on the subject. Specific references are included in this *Commentary*.

With anticipated improvements in understanding of the behavior of cold-formed steel framing and the continuing development of new technology, this material will become dated. It is anticipated that AISI will publish updates of this material as new information becomes available, but this cannot be guaranteed.

The materials set forth herein are for general purposes only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a *design professional*. Indeed, in many jurisdictions, such review is required by law. Anyone making use of the information set forth herein does so at their own risk and assumes any and all liability arising therefrom.

The user is advised to check the availability of specific framing material in the region in which the dwelling is being constructed.

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## PREFACE

The American Iron and Steel Institute (AISI) Committee on Framing Standards (COFS) has developed this *Commentary on the Standard for Cold-Formed Steel Framing - Prescriptive Method for One and Two Family Dwellings [Commentary]* to provide the background, supplemental information, engineering assumptions and methods, and detailed calculations for the provisions of the *AISI 230 Prescriptive Method* (AISI S230-07 w/S2-08).

The loads, load combinations, and other design parameters used to develop the provisions in the *AISI S230* were based on the *International Residential Code* (ICC, 2006b), the *International Building Code* (ICC, 2006a) (where no provisions are included in the IRC) and ASCE 7 (ASCE, 2005).

*Commentary* is provided only for those sections of the *AISI S230* where background or supplemental information is of benefit to the user. Sections thought to need no explanation are left blank.

This document is divided into two sections. Section 1, *Commentary*, contains the background, supplemental information and engineering assumptions. Section 2, *Design Examples*, contains detailed calculations that demonstrate how the values in the *AISI S230* were derived.

Terms within the body of this *Commentary* that are shown in *italics* indicate that the italicized word is a defined term by the *AISI S230* or by the *General Provisions* (AISI S200-07).

The Committee acknowledges and is grateful for the contributions of the numerous engineers, researchers, producers and others who have contributed to the body of knowledge on the subjects. The Committee wishes to also express their appreciation for the support and encouragement of the Steel Framing Alliance.

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**PART 1 – COMMENTARY**  
**ON THE STANDARD FOR COLD-FORMED STEEL FRAMING –**  
**PRESCRIPTIVE METHOD FOR ONE- AND TWO-FAMILY DWELLINGS**

**A. GENERAL**

**A1 Scope**

*AISI S230* consists of prescriptive requirements for cold-formed steel floor, wall, and roof framing to be used in the construction of one- and two-family dwellings, townhouses, and other attached and detached single-family dwellings not more than three stories in height using *repetitive in-line framing* practices.

**A1.1 Limits of Applicability**

*AISI S230* is not applicable to all possible conditions of use and is subject to the applicability limits set forth in Section A1.1 and A1.2. The applicability limits are necessary to define reasonable boundaries to the conditions that must be considered in developing prescriptive construction requirements. The applicability limits should be carefully understood as they define important constraints on the use of *AISI S230*.

The applicability limits strike a reasonable balance between engineering theory, available test data, and proven field practices for typical residential construction applications. The applicability limits are intended to prevent misapplication while addressing a reasonably large percentage of new housing conditions. Special consideration is directed toward the following items related to the applicability limits.

**Building Geometry:** The provisions in *AISI S230* apply to detached one- and two-family dwellings, townhouses, and other attached single-family dwellings not more than three stories in height. Its application to homes with complex architectural configurations is subject to careful interpretation by the user and therefore, engineering design support may be required. The most common building widths (or depths) range from 24 feet to 40 feet (7.3 to 12.2 m), with axially load bearing wall heights up to 10 feet (3.1 m). The building width as used in *AISI S230* is the dimension measured along the length of the *joists* (floor or ceiling) between the outmost structural walls. The maximum length of building is limited to 60 feet (18.3 m) where the length is measured in the direction parallel to the roof ridge or perpendicular to the *floor joists* or *roof trusses*. In 2006, the maximum *mean roof height* was explicitly defined as 33 feet (9.14 m) above *average grade*, since this is what was actually used in the development of the standard.

**Site Conditions:** Conditions for each site must be established by the user. Local conditions include ground snow loads, basic wind speeds, and the *Seismic Design Category*.

**Snow Loads:** Snow load values are typically given in a ground snow load map such as provided in the building code, ASCE 7 (ASCE, 2005) or by local practice. The national model building codes in the U.S. either adopt the ASCE 7 snow map and load requirements or have a similar map published in the code. The 0 to 70 psf (0 to 3.35 kN/m<sup>2</sup>) ground snow load used in *AISI S230* covers approximately 90 percent of the United States, which was deemed to include the majority of the buildings that are expected to utilize this document. Buildings in areas with greater snow loads than 70 psf (3.35 kN/m<sup>2</sup>) should not use this document without consulting a design professional.

**Basic Wind Speed:** In 2006, in recognition of that all areas of the U.S. fall within the 90 to 150 mph (3-sec gust) (145 to 241 km/hr) range of design wind speeds, per ASCE 7 (ASCE, 2005) the maximum *basic wind speed* in the standard was increased from 130 mph (209 km/hr) to 150 mph (241 km/hr). Also per ASCE 7, the three-second-gust wind speeds were used in the development of *AISI S230*. The *wind exposure* category in *AISI S230* is limited to Exposures A, B, and C. Wind speed and exposure are defined in *AISI S230*. *Wind exposure* is a critical determinant of the wind loads to be expected at a given site, and it should be determined by good judgment on a case-by-case basis. Buildings built along the immediate coastline (i.e. beach front property) are classified as Exposure D and therefore, cannot use this document without consulting a design professional.

**Seismic Design Category:** *AISI S230* covers all residential constructions in *Seismic Design Categories* A, B, C, D<sub>0</sub>, D<sub>1</sub>, D<sub>2</sub> and E (within the limits of applicability of Tables A1-1 and A1-2).

**Loads:** Consistent values were established for design loads in accordance with a review of the major building codes and standards. The results of this load review are embodied in the applicability limits table in *AISI S230*. Loads and load combinations requiring calculations to analyze the structural components and assemblies of a home are presented in the design examples shown throughout this document. The load and resistance factor design (LRFD) load combinations as shown in ASCE 7 were used to develop the tables and other provisions in *AISI S230*.

*AISI S230*, however, does not limit the application of alternative methods or materials through engineering design.

## **A1.2 Limitations in High Seismic and High Wind Areas**

### **A1.2.1 Irregular Buildings in High Seismic and High Wind Areas**

In *high wind* and *high seismic areas* additional limitations were considered to be necessary. Plan and vertical offsets are not permitted in this edition of the *AISI S230* for simplicity. Where the user wishes to exceed the irregularity limits a design professional should be consulted.

## **A2 Definitions**

Many of the terms in *AISI S230* are self-explanatory. Only definitions of terms not self-explanatory or not defined in the referenced documents are provided in *AISI S230*.

## **A3 Referenced Documents**

The design tables contained in *AISI S230* were generated at different times and, consequently, using different editions of the *AISI Specification*. For example, the floor joist, ceiling joist and screw connection tables were developed using the 1996 edition with the 1999 Supplement whereas the wall stud, back-to-back header, box header, L-header, roof rafter and gable end wall tables were developed using the 2001 edition with the 2004 Supplement. Every effort was made in this *Commentary* to cite the applicable edition of the *Specification* and where appropriate *AISI S230* cites the latest edition of the *AISI Specification* as a referenced document because the documents would be compatible. The only major change in the 2007 *Specification* that might create a change of any significance to the tabulated values would be the new distortional buckling provisions for *studs* and *joists*. It is anticipated that *AISI S230* will

incorporate these provisions once the design standards; e.g., *AISI S210* and *S211*, have been updated to recognize the rotational restraint provided by sheathing.

#### A4 Limitations of Framing Members

##### A4.1 General

The *structural members* used in *AISI S230* are standard *C-shapes* produced by roll forming hot-dipped metallic coated sheet steel conforming to the *Standard for Cold-Formed Steel Framing – Product Data* (AISI, S201-07).

Because ASTM A1003 was a derivative of ASTM A653, in 2007 *AISI S230* recognized that sheet steel that is in compliance with the requirements of ASTM A653 Type SS or ASTM A792 Type SS complies with the material specification requirements *Product Data* (AISI S201, 2007).

##### A4.2 Physical Dimensions

Member section designations, in accordance with the *North American Standard for Cold-Formed Steel Framing – General Provisions* (AISI, S200-07) and the *North American Standard for Cold-Formed Steel Framing – Product Data* (AISI, S201-07), are used through out *AISI S230*. The designation system was developed in 1996 in order to standardize the identification of cold-formed steel framing based on specific shapes and material thickness. The designator consists of four parts, the first value represents the *web* depth, the second value represents the type of steel framing member, the third value represents the *flange* width, and the fourth value represents the minimum base steel thickness.

**Web Depth:** The actual *web* depths chosen for *AISI S230* are 3-1/2 inches, 5-1/2 inches, 8 inches, 10 inches, and 12 inches (89, 140, 203, 254 and 305 mm). The 3-1/2 and 5-1/2 inch (89 and 140 mm) *web* depths were chosen to accommodate current framing dimensions utilized in the residential building industry (i.e. to accommodate window and door jambs). These sizes can be used directly with conventional building materials and practices; however, the substitution of a slightly larger size member, such as using a 3-5/8 inch (92 mm) or 4 inch (102 mm) stud instead of a 3-1/2 inch (89 mm) stud are acceptable. The depth of the *web* for 8, 10, and 12-inch (203, 254, and 305 mm) members, versus traditional lumber sizes, are not of great significance because they are typically used for horizontal framing members (i.e. headers and joists).

**Flange Width:** *AISI S230* requires that the standard *C-shape* have a minimum of 1-5/8 inch (41 mm) *flange* with a maximum *flange* dimension of 2 inches (51 mm).

**Lip Size:** *AISI S230* also provides a minimum size for the stiffening *lip* of 1/2 inch (12.7 mm). This dimension is also common in the industry. Decreasing the *lip* size may have a detrimental effect on the structural capacity of structural members in many circumstances.

*AISI S230* requires steel *tracks* to have a minimum *flange* dimension of 1-1/4 inches (32 mm). This dimension ensures a sufficient *flange* width to allow fastening of the *track* to the framing members and finish materials. Steel *track webs* are measured from inside to inside of flanges and thus have wider overall *web* depths than the associated standard *C-shapes*. This difference in size allows the *C-shape* to be properly nested into the *track* sections. In *AISI S230*, *tracks* are always required to have a minimum steel thickness equal to or greater than the *structural members* to which they are attached.

The steel thickness indicated by *AISI S230* is the minimum uncoated steel thickness (excluding the thickness of the metallic coating) and is given in mils (1/1000 of an inch). This unit is a deviation from the historic practice, which uses a gauge designation for thickness. The “gauge” is an outdated reference that represents a range of thickness and is, therefore, a vague unit of measure when specifying minimums. The practice of using “gauge” as a basis for measurement has been discontinued in the industry. In order to achieve consistency, the *mil* designation was adopted. For example, the 33 mils (i.e., 0.033 inches or 0.84 mm), 43 mils (i.e., 0.043 inches or 1.09 mm), 54 mils (i.e., 0.054 inches or 1.37 mm), 68 mils (i.e., 0.068 inches or 1.73 mm), and 97 mils (i.e., 0.097 inches or 2.46 mm) are specified for the thickness.

The minimum thickness is the minimum delivered thickness that cannot be less than the *design thickness* multiplied by 0.95, which is permitted by the *AISI Specification* (AISI, 2004). The *design thickness* of the flat steel stock, exclusive of coatings, is used in the structural calculations.

The corner bend radius is measured on the inside of bends in cold-formed steel members. Strength increases are realized in the regions of bends due to a phenomenon known as cold working which locally increases the yield strength of the steel.

#### **A4.3 Material Properties**

*AISI S230* applies to steel with minimum *yield strength* of 33 ksi (230 MPa) or 50 ksi (345 MPa). The 33 ksi (230 MPa) steels are the minimum required for all steel floors, roofs, and header components. Steel multiple span *floor joist*, wall *stud*, header and roof *rafter* tables are provided for both 33 ksi (230 MPa) and 50 ksi (345 MPa) minimum *yield strength*. The 50 ksi (345 MPa) *yield strength* steel was included because of the structural benefits.

The user is advised to check the availability of specific framing material in the region in which the dwelling is being constructed. Not all material specified in *AISI S230* is expected to be available in all locations.

Strength increase from the cold work of forming (as permitted by the *AISI Specification*) is utilized for the design of *C-shaped* members in *AISI S230* used as flexural members, concentrically loaded compression members, and members with combined axial and bending loads. The reader is referred to Part 2 of this document for engineering calculations illustrating the application of the strength increase due to cold work of forming.

##### **A4.3.1 Material Properties in High Wind and High Seismic Areas**

Further limitations on material properties are imposed for the use of *AISI S230* in high wind and high seismic areas. These limitations were imposed to reflect the material properties used in the available *shear wall* test data.

#### **A4.4 Web Holes**

All *structural members* (i.e., *floor* and *ceiling joists*, wall *studs* and *headers*), except cantilevered portions of framing members, used in *AISI S230* are designed assuming maximum web hole dimensions as shown in Figure A4-1 and A4-2 of *AISI S230*. The maximum web hole dimensions are consistent with the *North American Standard for Cold-Formed Steel Framing – Product Data* (AISI, S201-07). The design procedure follows the *AISI Specification* (AISI, 2004).



**A4.5 Hole Reinforcing**

This section provides reinforcing options for web holes violating the requirements of S230 Section A4.4 and is based on engineering judgment and research at McMaster University (Siva, 2007).

**A4.6 Hole Patching**

In 2004, the limitations “that the depth of the hole does not exceed 70% of the flat width of the *web* and the length of the hole measured along the *web* does not exceed 10 inches (254 mm) or the depth of the *web*, whichever is greater” were added along with other editorial changes to better differentiate the permitted use of a patch versus when member replacement or engineering analysis would be required.

## B. CONNECTIONS

### B1 Fastening Requirements

Self-drilling screws conforming to the requirements of the *North American Standard for Cold-Formed Steel Framing – General Provisions* (AISI S200-07) are specified as the fastener for cold-formed steel framing members in AISI S230. Requirements for sharp point screws connecting gypsum board and sheathing to steel *studs* are found in ASTM C1002 (ASTM, 2007) and ASTM C954 (ASTM, 2007). The edge distance and center-to-center spacing of these screws follow industry recommendations and the AISI *Specification* (AISI, 2004). Although AISI S230 specifies the use of screws other fastening methods are permitted to be used, provided that the connection capacity can be shown to equal or exceed the connection capacity implied in AISI S230.

For practical purposes and added capacity in certain applications, No. 10 screws are specified in AISI S230. Because the point style of the screw may affect constructability, for example, a sharp point screw may be efficiently used to connect gypsum board and other panel products to steel framing members that are no thicker than 33 mils (0.84 mm), screw manufacturer recommendations should be consulted.

Screw capacities were calculated based on the design equations given in the AISI *Specification* (AISI, 2004). The *Specification* provides the equations necessary to calculate the shear, pullover, and pullout capacity of a connection based on the thicknesses of the steel, tensile strength of the steel and diameter of the screw.

AISI S230 also provides a screw substitution factor where larger screws can be used in lieu of the No. 8 screws or when one of the sheets of steel being connected is thicker than 33 mils (0.84 mm). This may result in a reduced number of screws.

### B2 Bearing Stiffeners

*Webs* of cold-formed steel members may cripple or buckle locally at locations of a concentrated load or a bearing support. The allowable reactions and concentrated loads for beams having single un-reinforced *webs* depend on *web* depth, bend radius, *web* thickness, *yield strength*, and actual bearing length.

The *floor joist* spans in AISI S230 were derived assuming *bearing stiffeners* (also called *web stiffeners*) are located at all support or bearing point locations. *Ceiling joist* span tables were developed for two cases, 1) assuming *bearing stiffeners* are located at all support or bearing point locations and 2) no *bearing stiffeners*. Where specified, *bearing stiffeners* are to be a minimum of 43 mil (1.09 mm) clip angle or *track* section or 33 mil (0.84 mm) *C-shaped* member.

Three types of *bearing stiffeners* are permitted in AISI S230, *C-shaped*, *track*, and clip angle. The requirements for the *C-shaped* and *track bearing stiffeners* are based on engineering judgment. The clip angle *bearing stiffener* requirements are stipulated in the *North American Standard for Cold-Formed Steel Framing – Floor and Roof System Design* (AISI, S210-07).

### B3 Clip Angles

All *clip angle* dimensions prescribed are shown as minimums. *Clip angles* that are of a greater base steel thickness or have greater overall dimensions, or both, are permitted to be

used up to a maximum thickness of 68 mils.

**B4 Anchor Bolts**

In the *high wind areas* and *high seismic areas*, the requirement for a minimum steel plate washer is based on engineering judgment.

## D. FLOOR FRAMING

### D1 Floor Construction

Floor *trusses* are not prescriptively addressed in *AISI S230*, but are permitted, in accordance with Section D8, and must be designed by a *design professional*. Also floor girders are also not addressed in *AISI S230*.

### D2 Floor to Foundation or Structural Wall Connection

*AISI S230* provides several details for connecting floor assemblies to foundations or structural walls. The details reflect common industry practice. In areas where wind speeds exceed 110 mph (177 km/hr) (exposure C) or in *Seismic Design Category* D<sub>1</sub>, D<sub>2</sub> or E, additional requirements for hold-downs and anchors are specified in Sections E11, E12 and E13.

### D3 Minimum Floor Joist Sizes

*AISI S230* provides *floor joist* tables with maximum allowable spans for two live load conditions: 30 psf and 40 psf (1.44 and 1.92 kN/m<sup>2</sup>). The two live load conditions are specified in the *International Building Code* (ICC, 2006a) and the IRC (ICC, 2006b). The 30 psf (1.44 kN/m<sup>2</sup>) is typically specified for sleeping areas, while the 40 psf (1.92 kN/m<sup>2</sup>) is specified for living areas. The spans shown in *AISI S230* assume *bearing stiffeners* are installed at each bearing point. *Bearing stiffener* requirements are provided in Section B2 of *AISI S230*.

For the design of *floor joists*, the following design considerations were evaluated:

- Flexural yielding
- Flexural buckling
- Web crippling
- Shear
- Vertical deflection
- Combined bending and shear (for multiple spans only)

All *joists* are considered to have *web* holes (a.k.a. “penetrations”, “utility holes”, “punchouts”), in accordance with Section A4.4. The compression *flange* (top *flange*) of a *floor joist* is assumed to be continuously braced by the sub-flooring, thus providing lateral bracing for the top *flanges*.

The *joist* span tables are calculated for a deflection limit of  $L/480$  for live load and  $L/240$  for total loads, where  $L$  is the clear horizontal distance between supports. The  $L/480$  limit may be more stringent than the minimum deflection limits established by building codes but was selected to achieve a satisfactory floor design for serviceability.

Multiple span *joists* are commonly used in the residential steel building market. With multiple spans, certain measures are necessary to address the responses of the loaded members. The magnitude of the reaction at the middle support will be greater than the end reactions, and may cause a web crippling failure at this location, which is controlled by requiring *bearing stiffeners* at all bearing points. The second issue with multiple span *joists* is the presence of negative moments (i.e. reversed bending) near the middle support resulting in the compression flange to be at the bottom rather than the top of the joists. If left unbraced, this could cause lateral instability and may cause premature failure of the joists under maximum loading

conditions. Furthermore, due to the presence of high shear and bending stresses at the middle reactions, shear and bending interaction was checked for multiple spans *joists*.

Bottom *flange bracing* at interior supports is provided by ceiling finishes (when present) and by positive connection to the interior bearing wall.

Since multiple spans are often limited by strength considerations instead of deflection, steels with higher *yield strengths* can result in longer spans. Therefore, an additional table for 50 ksi (345 MPa) steel is provided for multiple span *joists*. The 50 ksi (345 MPa) steel is not used for single spans because most of the entries in the single span tables are controlled by deflection rather than strength.

### D3.1 Floor Cantilevers

Cantilevers supporting structural walls may create special loading conditions that require an engineering analysis. In *AISI S230*, floor cantilevers are limited to a maximum of 24 inches (610 mm) for floors supporting one wall and roof only (one story). This limitation is imposed to minimize the impact of the added load on the *floor joists*. To fully utilize the strength of the *joist*, *web* holes are not permitted in cantilevered portions of a *joist*. *AISI S230* provides details for cantilevered floors. It is essential that *blocking* be installed between cantilevered *joists* at the bearing locations to adequately transfer floor *diaphragm* or *shear wall* loads (refer to Section D5.4).

### D4 Bearing Stiffeners

The *floor joist* spans in the *AISI S230* were calculated assuming *bearing stiffeners* (also called *web stiffeners*) are located at all support or bearing point locations. The *bearing stiffeners* are specified to be *C-shaped*, *track* or *clip angle bearing stiffeners* installed in accordance with Section B4. In 2006, language was added to clarify the requirements for *bearing stiffeners* when *floor joists* are lapped over interior bearing supports and to explicitly require that *floor joists* supporting *jamb studs* with multiple members have two *bearing stiffeners*.

### D5 Joist Bracing and Blocking

#### D5.1 Joist Top Flange Bracing

For typical residential floors, it has been assumed that the function of the floor sheathing is to transfer the loads to the *joists*, and to provide continuous lateral *bracing* to the compression *flanges*. Testing has indicated that using a single *joist* for strength calculation agrees with actual behavior when uniform loads are applied (WJE, 1977).

#### D5.2 Joist Bottom Flange Bracing/Blocking

*Bracing* the bottom *flanges* of *joists* as specified in *AISI S230* is based on industry practice and engineering judgment. Steel *strapping* and finished ceilings (e.g. application of gypsum board) are considered to be adequate *bracing* for the tension *flanges*. It is necessary, however, for steel *strapping* to have *blocking* installed at a maximum spacing of 12 feet (3.7 m) and at the termination ends of all *straps*. Alternatively, the ends of steel *straps* may be fastened to a stable component of the building in lieu of blocking (i.e. to a bearing wall or foundation).

### D5.3 Blocking at Interior Bearing Supports

Single span *floor joists* that are lapped over interior supports do not require *blocking* as the lapped sections provide adequate stiffness to prevent lateral movements. Continuous *joists* over interior supports, on the other hand, require *blocking* at every other joist to provide adequate stiffness to prevent lateral movement.

### D5.4 Blocking at Cantilevers

*Blocking* is required for cantilevered supports to transfer shear loads from the floor *diaphragm* or *shear wall*.

## D6 Splicing

Splicing of *structural members* is not permitted by *AISI S230*, however, there may be some situations where splicing would be useful. Applications may include repair of damaged *joists*, and simplified details for dropped floors. In these situations a design professional must be consulted.

The *floor joist* spans provided in *AISI S230* are based on the assumption that the *joists* are continuous, with no splices. Therefore, splicing of *joist* members in *AISI S230* requires an *approved* design except when lapped *joists* occur at interior bearing points.

## D7 Framing of Floor Openings

Openings in floors are needed for several reasons (such as at stairs, chases, chimneys). *AISI S230* limits the maximum width of the floor opening to 8 feet (2.4 m) and provides a provision for reinforcing the members around floor openings. All members around floor openings (i.e. *header* and *trimmer joists*) are required to be box-type members made by nesting *C-shaped joists* into a *track* and fastening them together along the top and bottom *flanges*. These built-up members are required to be equal to or a greater in size and steel thickness than the *floor joists*, which they are connecting to. Each *header joist* is required to be connected to the *trimmer joist* with a *clip angle* on each side of each connection. The *clip angle* is required to be of a thickness equivalent to the *floor joists*. The members around an opening are designed to support joists that have been displaced by the opening. The perimeter members given in *AISI S230* do not consider additional stair loads.

## D8 Floor Trusses

*AISI S230* does not contain provisions for floor *trusses*, which must have an *approved* design. This section is included so that pre-engineered floor *trusses* may be used in conjunction with this document. The *North American Standard for Cold-Formed Steel Framing – Truss Design* (AISI S214-07) should be consulted for the *truss* design.

## D9 Diaphragms

Floor *diaphragms* are required to adequately transfer shear loads to the foundation. In steel framed floors, the shear load transfer is typically accomplished by sheathing the top *flanges* of the *joists* with wood *structural sheathing* (such as OSB or plywood). Shear strength values used in verifying the adequacy of the floor *diaphragms* were taken from *North American Standard for Cold-Formed Steel Framing – Lateral Design* (AISI S213-07) for oriented-strand-board (OSB) panels

fastened to steel members with No. 8 screws at 6 inch (152 mm) on center spacing at panel edges and 12 inch (305 mm) on center spacing at intermediate supports. Additional requirements for steel floors constructed in *high wind* (110 mph (177 km/hr) or greater) or *high seismic areas* (*Seismic Design Category* D<sub>0</sub>, D<sub>1</sub>, D<sub>2</sub> and E) are specified in Section D9.1.

#### **D9.1 Floor Diaphragms in High Seismic and High Wind Areas**

Shear strength values used in verifying the adequacy of the floor *diaphragms* were taken from *North American Standard for Cold-Formed Steel Framing – Lateral Design* (AISI S213-07) for oriented-strand-board (OSB) panels fastened to steel members with No. 8 screws at 6 inch (152 mm) on center spacing at panel edges and 6 inch (152 mm) on center spacing at intermediate supports. The reduced fastener spacing from 12 inches (305 mm) to 6 inches (152 mm) is to ensure that the diaphragm adequately transfers shear loads to the foundation.

## E. WALL FRAMING

### E2 Wall to Foundation or Floor Connection

Historically the wall *track* was required to be connected through the floor sheathing to a steel member, i.e. the *floor joist* or *track* below. In 2004, Table E2-1 was revised to enable connection of the wall *track* to the floor sheathing alone (Figure E2-4). This revision was based on research by the NAHB Research Center (NAHBRC, 2003) in which five shear tests and six withdrawal tests were conducted where 33-mil (0.84 mm) *track* was connected to 23/32-inch-thick (18 mm) OSB sheathing using #8 screws. The average ultimate shear capacity was 412 lb (187 kg) and the average ultimate pullout capacity was 350 lb (159 kg). Considering that the minimum allowable fastener capacities for steel-to-steel connections for #8 screws and 33 mil (0.84 mm) material of 164 lb (74 kg) for shear and 72 lb (32.6 kg) for pullout were used to calculate the requirements for *AISI S230*, the Committee deemed that it would not be necessary to require that every fastener connect to a *floor joist* or *track* member. In 2007 *AISI S230* was expanded to include gable endwall to floor connection requirements for *studs* with heights greater than 10 feet, based on a study at the University of Missouri-Rolla (Downey et al., 2005).

### E3 Minimum Stud Sizes

This section dictates the minimum required thickness of steel *studs* for different wind speeds, *wind exposure* categories, wall heights, building widths, live loads, and ground snow loads. *Stud* selection tables are limited to buildings not greater than three stories with *structural wall* heights up to 10 feet (3.05 m). In 2007 *AISI S230* was expanded to include gable endwall *studs* with heights greater than 10 feet, based on a study at the University of Missouri-Rolla (Downey et al., 2005).

The 8-foot (2.44 m) wall height is widely used in residential construction; however, the higher strength of cold-formed steel wall studs enable light-steel framed construction to provide for higher ceilings such as 9- and 10-foot (2.74 and 3.05 m) walls. The 50 ksi (345 MPa) *yield strength stud* tables were developed to take advantage of the higher *yield strength*, which allows thinner *studs* in many cases. The user should verify the availability of steel member sizes and thickness in 33 or 50 ksi (230 and 345 MPa) *yield strengths* as many steel manufacturers do not produce certain studs in both 33 or 50 ksi (230 and 345 MPa) *yield strength*.

The wall *studs* are grouped in three categories:

- *Studs* for one-story or second floor of two-story building or third floor of a three-story building (supporting roof only)
- *Studs* for first story of a two-story building or second story of a three-story building (supporting roof + one floor)
- *Studs* for first story of a three-story building (supporting roof + two floors)

For walls sheathed on both faces with wood structural panels (minimum 7/16 inch (11.1 mm) OSB or minimum 15/32 inch (11.9 mm) plywood), a reduction in thickness of the *stud* is allowed. All *studs* in exterior walls are treated as *structural* members in *AISI S230*. The following design assumptions were made in developing the wall *stud* selection tables.

- *Studs* are simply supported beam - columns
- The exterior *flanges* of the *studs* are braced by *structural sheathing* and the interior flanges are braced by mechanical *bracing* (mechanical bracing at mid-height for 8-



foot studs (2.4 m), 1/3 point for 9-foot (2.74 m) and 10-foot (3.05 m) and 11'-4" (3.45 m) studs)

- Maximum roof overhang of 24 inches (610 mm)
- Roof slopes limited to a range of 3:12 to 12:12
- Deflection limit of  $L/240$
- Ceilings, roofs, attics, and floors span the full width of the house (no interior *bearing* walls)
- Permitted attic live load is limited to 10 psf (0.48 kN/m<sup>2</sup>)
- Second floor of a two-story building and third floor of a three-story building live load is 30 psf (1.44 kN/m<sup>2</sup>). Second floor of a three-story building floor live load is 40 psf.
- Unbalanced snow loads in accordance with ASCE 7

### Stud Design

The design of the *studs* was based on the following design checks as stipulated by the *North American Standard for Cold-Formed Steel Framing – Wall Stud Design* (AISI S211-07):

- Combined bending and axial strength using Main Wind Force Resisting System (MWFRS) wind loads and the bracing as defined by Section E4.
- Bending strength based on Components and Cladding (C&C) loads and the bracing as defined by Section E4.
- Web crippling strength based on Components and Cladding (C&C) loads. Because bending alone was considered, Equation B2.2-1 was used for the development of the stud tables.
- Deflection limit based upon 70% of Components and Cladding (C&C) loads.

### Wind Design Loads

Both the Components and Cladding (C&C) and the Main Wind Force Resisting System (MWFRS) loads at the ends and corners of walls can be significantly higher than in the middle, or field, of the wall. However, historically for residential construction rather than design the entire wall for these increased corner loads, the loads in the middle of the wall were used to design the studs. Thus, the tables in *AISI S230* were developed for field of the wall wind loads.

### E4 Stud Bracing

*Studs* in *structural walls* are laterally braced on each *flange* by either a continuous 1-1/2 inch x 33 mil (38.1 x 0.84 mm) (minimum) *strap* at mid-height (or third points for 9-foot (2.74 m), 10-foot (3.05 m) and 11'-4" (3.45 m) studs) or by direct attachment of *structural sheathing* or rigid wall finishes (i.e. structural panels such as plywood, OSB or gypsum board) according to the requirements of the *AISI S230*. Therefore, for the evaluation of both the bending strength and axial strength all *studs* were considered to be braced at mid-height (or third points for 9-foot (2.74 m), 10-foot (3.05 m) and 11'-4" (3.45 m) studs) for the engineering analysis of the stud tables. As previously noted, the benefit achieved from *structurally sheathed* walls (both wall faces) on the required *stud* thickness and the composite wall strength are recognized in the allowance in dropping down a *stud* thickness.

Temporary *bracing* may be necessary to facilitate safe construction practices and to ensure that the structural integrity of the wall assembly is maintained. Prior to the installation of cladding or bridging, a wall *stud* is free to twist, thus making the *stud* potentially subject to

premature failure under heavy construction loads (i.e. stack of gypsum wallboard or roof shingles). In such cases, temporary *bracing* must be provided.

### E5 Splicing

The *stud* tables provided in AISI S230 are based on an assumption that the *studs* are continuous, with no splices. Therefore, *structural studs* shall not be spliced without an *approved* design. *Tracks* are permitted to be spliced according to the requirements and details in the AISI S230.

### E6 Corner Framing

AISI S230 utilizes a traditional three-stud practice for framing corners. The corner cavity should be insulated before the exterior sheathing is applied.

### E7 Headers

*Headers* are horizontal members used to transfer loads around openings in *structural walls*. *Headers* specified in AISI S230 are allowed only above the opening immediately below the wall top *track* (i.e. high *headers*) In 2007, an exception to this requirement was included in AISI S230 along with an alternative detail for box and back-to-back *headers* in gable endwalls, based on a study at the University of Missouri-Rolla (Downey et al., 2005). Historically, the two traditional ways of constructing *headers* was to put two *C-shaped* members back-to-back or in a box shape. However, recent testing of single and double L-shaped *headers* has proven that they as well as inverted L-headers may be an economical alternative to traditional *headers* in lightly loaded situations.

The following general design assumptions were made in determining *header* spans:

- *Headers* are simply supported beams
- Maximum roof overhang of 24 inches (610 mm)
- Roof slopes limited to a range of 3:12 to 12:12
- Ceilings, roofs, attics, and floors span the full width of the house, no interior load bearing walls, except as noted
- Deflection limit of  $L/240$

The design of *headers* is based on the *North American Standard for Cold-Formed Steel Framing – Header Design* (AISI S212-07).

#### E7.1 Box Headers

Box *headers* are formed from two equal sized *C-shaped* members placed toe-to-toe in a box type configuration and fastened to both the wall top *track* and a *track below*. *Tracks* used to frame around openings are required have a steel thickness equivalent to or greater than the wall *studs*. The orientation of the lower *track* is not critical to the structural performance of the box-header. Thus the lower track can be oriented to face either the top or the bottom of the wall. The following design assumptions were used when developing the header selection tables:

- Bending capacity is based on two C-sections alone, the track is not considered composite with the C-sections.
- Shear capacity is based on two C-sections alone.

- Interior-one-flange loading web crippling capacity is based on the *Header Design Standard* (AISI S212-07) with a bearing length,  $N = 1$ .
- End-one-flange loading web crippling capacity is not evaluated because the typical end detail precludes web crippling.
- Bending and web crippling capacity is based on the *North American Header Design Standard* (AISI S212-07).
- Deflection is based on two C-sections alone, the track is not considered composite with the C-sections.

## E7.2 Back-to-Back Headers

Back-to-back *headers* are formed from two equal sized *C-shaped* members in a back-to-back configuration creating an I-section. These *C-shaped* sections are fastened to the wall top *track* and a lower *track* spanning the width of the opening. *Tracks* used to frame around openings are required to have a steel thickness equal to or greater than the wall *studs*. The lower *track* can be oriented to face either towards the top or the bottom of the wall. It is more difficult to install *strapping* around back-to-back *headers* in *high wind areas*. The following design assumptions were made in developing the header selection tables:

- Bending capacity is based on two C-sections alone, the track is not considered composite with the C-sections.
- Shear capacity is based on two C-sections alone.
- Interior-one-flange loading web crippling capacity is based on the *Specification* (AISI, 1996) with a bearing length,  $N = 1$ .
- End-one-flange loading web crippling capacity is not evaluated because the typical end detail precludes web crippling.
- Bending and web crippling capacity is based on the *Specification* (AISI, 1996).
- Deflection is based on two C-sections alone, the track is not considered composite with the C-sections.

## E7.3 L-Headers

### E7.3.1 Double L-Headers

A double L-header is shown in Figure E7-5 of the *AISI S230*. Tables for gravity and uplift loads are provided for double L-headers. Double L-headers are easy to install. They can be installed during or after the wall has been framed. They do not require pre-insulation and provide a large surface to apply finishing materials. They also require less material (steel and screws) than back-to-back or box *headers*. Double L-headers do not need to be cut to exact lengths; however, they need to lap over a minimum of one *king stud* at each end. The design of the L-header is based on the *North American Standard for Cold-Formed Steel Framing – Header Design Standard* (AISI S212-07) which stipulates that the bending capacity be based on the angles alone. The *Header Design Standard* also stipulates that shear and web crippling alone, as well as combinations of shear, bending or web crippling, need not be checked.

### E7.3.2 Single L-Headers

A single L-header is shown in Figure E7-6 of *AISI S230*. Tables for gravity loads only are provided for single L-headers. They can be installed during or after the wall has been framed. They do not require pre-insulation and provide a large surface to apply finishing materials. They also require less material (steel and screws) than back-to-back or

box headers. Single L-headers do not need to be cut to exact lengths; however, they need to lap over the required *king studs*. The design of the L-header is based on the *North American Standard for Cold-Formed Steel Framing – Header Design Standard* (AISI S212-07) which stipulates that the bending capacity be based on the angle alone. The *Header Design* standard also stipulates that shear and web crippling alone, as well as combinations of shear, bending or web crippling, need not be checked.

### E7.3.3 Inverted L-Headers

An inverted L-header is shown in Figures E7-7 of *AISI S230*. Tables for gravity and uplift loads are provided for inverted L-headers. They can be installed during or after the wall has been framed. They do not require pre-insulation and provide a large surface to apply finishing materials. They also require less material (steel and screws) than back-to-back or box headers. Inverted L-headers need to be cut to exact lengths. The design of the L-header is based on the *Header Design Standard* (AISI S212-07). The *North American Standard for Cold-Formed Steel Framing – Header Standard* stipulates that for double inverted L-headers the bending capacity is determined by summing the gravity and uplift nominal moment capacities for the respective gravity and uplift capacities of the double L-header. For the single inverted L-header, the *Header Design* standard states that the gravity capacity for the single L-header is used when evaluating either gravity or uplift capacity for the inverted single L-header. The *Header Design* standard also stipulates that shear and web crippling alone, as well as combinations of shear, bending or web crippling, need not be checked.

### E7.4 Jack and King Studs

The required number of *jack* and *king studs* was calculated based on the size of the opening. The number was determined by taking the width of the opening, divided by the *stud* spacing, and rounding to the next higher whole number. The resulting number is further divided into *jack* and *king studs* based on the required axial capacity being provided by the *jack studs* only. *King* and *jack studs* are required to be the same size and thickness as the adjacent wall *studs*. *Jack* and *king studs* are interconnected by *structural sheathing* (plywood or OSB) to transfer lateral loads (when multiple *king* and *jack studs* are required).

### E7.5 Head and Sill Track

Head and sill *tracks* are those located at top (i.e., head) or bottom (i.e., sill) of window or door openings. Head and sill *tracks* span the full width of the opening and were designed to resist lateral wind loads only. The allowable head and sill *track* spans were calculated using C&C wind loads for a 48 inch (1.22 m) tributary span (i.e., assuming the opening covers the entire height of the 8-foot (2.44 m) wall.) As the tributary span decreases the head and sill *track* will have to resist less wind loads. Therefore, for a 4-foot (1.22 m) opening, the tributary opening width is 2 feet (0.61 m) and hence the allowable head and sill *track* span increases by a factor of 1.75. Similarly, for a 6-foot (1.83 m) opening, the tributary opening is 3 feet (0.92 m) and hence the allowable head and sill *track* span increases by a factor of 1.50.

## E8 Wall Bracing

The wall bracing provisions of this section are applicable to buildings classified as *Seismic Design Category A, B and C* and for buildings located where the *basic wind speed* is 90 mph (145 km/hr) or less.

Three different *bracing* methods are recognized in *AISI S230*:

- Steel *strap bracing* (diagonal X-bracing)
- *Structural sheathing* (plywood or OSB)
- Sheet steel (in *high wind* and *high seismic* regions)

### E8.1 Strap Bracing (X-brace)

The wall *bracing* in *AISI S230* was conservatively limited to the use of continuously sheathed walls with limitations on loading conditions and building geometry. The use of sheet steel diagonal strap bracing must be designed in accordance with *approved* engineering practices.

### E8.2 Structural Sheathing

The wall *bracing* requirements in *AISI S230* are based on an engineered approach that utilized available technical knowledge. The available shear strength for plywood and oriented-strand-board (OSB) sheathing are based on Table C2.1-1 of the *North American Standard for Cold-Formed Steel Framing – Lateral Design* (*AISI S213-07*). The shear strength for assemblies relevant to this document is summarized in Table C-E8.1.

**Table C-E8.1**  
**Cold-Formed Steel Framed Walls Shear Strength (*AISI S213-07*)**

| Assembly Description                                     | Nominal Shear Strength (plf) | Available Shear Strength (plf) |
|--|------------------------------|--------------------------------|
| 15/32" Plywood APA rated sheathing w/ panels on one side | 1065                         | 426                            |
| 7/16" OSB APA rated sheathing w/ panels on one side      | 910                          | 364                            |

The intent in *AISI S230* is to have the entire building fully sheathed (except for door and window openings, as limited by the minimum length of full height sheathing).

The lateral wind loads were calculated for a range of building surfaces using the orthogonal wind loading approach of ASCE 7 (ASCE, 2005). Tributary areas consisting of the leeward and windward wall surfaces were assigned to each exterior *shear wall* (i.e. sidewalls and endwalls) to determine the in-plane shear loads to be resisted by the walls,

No interior walls or alternative shear pathways were considered.

Using the more conservative available shear strength from C-E8.1 of 364 plf (5.39 kN/m) *AISI S230* wall *bracing* requirements were determined. The length of full-height sheathing required was then tabulated as a percentage of wall length for sidewalls and endwalls over the range of building geometries defined in *AISI S230* applicability limits. The length of wall with full-height sheathing is defined as the sum of wall segments that have sheathing extending from the bottom track to the top track, without interruption due to openings (i.e., the total of lengths of wall between window and door openings). Further, the individual wall segments must be 48 inches (1.22 m) in length or greater to contribute to the required length of full-height sheathing for a given wall line, unless permitted otherwise.

As a final step necessary for a basic prescriptive approach, the requirements were conservatively reduced to the minimum percent lengths of full-height sheathed wall shown in the wall-bracing table of the *AISI S230*. The only building geometry parameter retained was roof slope due to a significant impact on the wind loads transferred to the *shear walls*. Footnotes to the *shear wall* table provide additional information related to the proper applications of the requirements.

### E8.3 Structural Sheathing Fastening

Fastening of *structural sheathing* is typically done at 6 inch (152 mm) spacing at the perimeters and 12 inch spacing (305 mm) in the field. When this spacing is reduced to 4 inches (102 mm) (perimeter spacing only), the percentage of full-height sheathing is permitted to be multiplied by 0.72.

### E8.4 Hold-down Requirements

A *hold-down anchor* shall consist of an approved *strap* or bracket adequately attached to the *stud* and anchored to the foundation, floor, or wall below to form a continuous load path to the foundation. In wind conditions greater than 100 mph (161 km/hr) exposure C, *hold-down anchors* in accordance with Table E2-1 are required to stabilize the *shear walls*. *Hold-down anchors* may also be added to reduce the amount of full-height sheathing required, or to increase the shear (racking) strength of the wall.

## E9 Exterior Wall Covering

It is required that exterior coverings be installed in accordance with the recommendations of the manufacturer. *AISI S230* limits the total exterior envelope dead load (total load = stud framing plus wall coverings) to 10 psf (0.48 kN/m<sup>2</sup>). If the total exterior envelope dead load exceeds that value, then the walls must be engineered for that load (see Table A1-2 for maximum wall dead loads in high seismic areas).

## E11 Braced Walls in High Wind Areas and High Seismic Areas

### E11.1 General

This section provides additional *shear wall* requirements for buildings located in *high seismic areas* (i.e., *Seismic Design Categories* D<sub>0</sub>, D<sub>1</sub>, D<sub>2</sub> and E) or *high wind areas* (i.e., wind speed between 100 to 130 mph (161 to 209 km/hr)). In *high seismic areas*, buildings are required to comply with Sections E11 and E12; and in *high wind areas*, buildings are required to comply with the requirements in Sections E11 and E13.

The following general assumptions and building configurations were used in developing the high seismic tables and high wind provisions:

- Provisions and tables are limited to buildings no more than two-stories.
- Provisions and tables are limited to buildings on slab-on-grade or spread footing with stemwall foundation system with a single top of slab/top of stemwall elevation.
- Wall clear heights are limited to 8, 9, and 10 feet (2.44, 2.74 and 3.05 m).
- Maximum roof slope is limited to 6.9:12.
- All ceilings are considered leveled (i.e., no offsets or cathedral ceilings).

- Buildings are considered regular (rectangular shape).
- First and second story walls are assumed vertically stacked (no offset).

Weights used in calculating the entries of the tables in the *high seismic areas* are as follows:

- Roof/ceiling dead load = 25 psf (1.2 kN/m<sup>2</sup>) for heavy weight roofs  
= 15 psf (0.72 kN/m<sup>2</sup>) for normal weight roofs  
= 12 psf (0.57 kN/m<sup>2</sup>) for light weight roof systems
- Wall dead load = 14 psf (0.67 kN/m<sup>2</sup>) for heavy walls  
= 7 psf (0.34 kN/m<sup>2</sup>) for light walls
- Floor/ceiling dead load = 10 psf (0.48 kN/m<sup>2</sup>)
- Interior wall dead load = 5 psf (0.24 kN/m<sup>2</sup>) (based on 10 foot (3.04 m) wall)
- Ground snow load = 30 psf (1.44 kN/m<sup>2</sup>) for normal or light weight roofs  
= 70 psf (3.35 kN/m<sup>2</sup>) for heavy weight roof systems
- Roof weight includes a 2-foot (610 mm) overhang

The dead loads that were used in determining the seismic mass are given below:

| Wall Element         | Weight (psf)       |                    |
|----------------------|--------------------|--------------------|
|                      | Light Weight Walls | Heavy Weight Walls |
| Wallboard            | 1.8                | 1.8                |
| Steel Framing        | 0.6                | 0.6                |
| ½" Plywood Sheathing | 1.6                | 1.6                |
| Insulation           | 1.0                | 1.0                |
| 7/8" Stucco          | 0                  | 9.0                |
| Exterior Siding      | 1.5                | 0                  |
| Total                | 6.5                | 14.0               |

For SI: 1 psf = 0.0479 kN/m<sup>2</sup>.

| Roof Element                      | Weight (psf) |                     |
|-----------------------------------|--------------|---------------------|
|                                   | Flat Roofs   | Sloped Roofs (6:12) |
| Sheathing                         | 1.6          | 1.63                |
| Roof Framing or Trusses           | 2.5          | 2.5                 |
| Insulation                        | 1.0          | 1.0                 |
| Miscellaneous                     | 0.6          | 0.84                |
| Ceiling Covering                  | 1.8          | 1.8                 |
| Subtotal                          | 7.5          | 7.8                 |
| Total with Roof 3.7 psf Covering  | 11.5         | 12                  |
| Total with Roof 6.4 psf Covering  | 14.2         | 15                  |
| Total with Roof 15.3 psf Covering | 23.5         | 25                  |

For SI: 1 psf = 0.0479 kN/m<sup>2</sup>.



| Roof Category      | Roof/Ceiling Weight (psf) |
|--------------------|---------------------------|
| Light-Weight Roof  | 12                        |
| Normal-Weight Roof | 15                        |
| Heavy-Weight Roof  | 25                        |

For SI: 1 psf = 0.0479 kN/m<sup>2</sup>.

Design assumptions that are used in developing the shear wall and other tables in the high seismic areas are as follow:

- Nominal shear values are taken from Table 2211.1(3) of the *International Building Code* (IBC) (ICC, 2006a).
- *Seismic Design Category* (SDC) assignments in accordance with Table R301.2.2.1.1 of the *International Residential Code* (ICC, 2006b).
- Seismic base shears were calculated in accordance with the IBC (ICC, 2006a) using an  $R = 5.5$  and  $\Omega_0 = 2.5$ . Upper end  $S_{DS}$  values are used for each SDC.
- *Diaphragms* are considered to be flexible rather than rigid. No requirement for inclusion of accidental torsion and reduction of  $\Omega_0$  from 3 to 2.5 in accordance with the IBC (ICC, 2006a) was used.
- $F_u = 45$  ksi (310 MPa) and  $F_y = 33$  ksi (230 MPa) were used in lieu of the IRC (ICC, 2006b) allowed 1.08 tensile/yield ratio in determining screw capacities. *Shear wall* test values are based on the 33 ksi/45 ksi (230/310 MPa) steels.
- $\phi$  factor was used in combination with the  $\Omega_0$ , the over-strength factor, to determine screw requirements for chord splices. Chord splice screw requirements are based on the lesser of  $\Omega_0$  times  $T_{seismic}$  or  $T_n$ , divided by  $\phi V_n$ . Both the 3-1/2 inch (88.9 mm) and the 5-1/2 inch (140 mm) members were considered, as well as both the 33 mil (0.84 mm) and the 43 mil (1.09 mm) thickness.
- ASCE 7 requires the use of 20% of the uniform design snow load if the flat roof snow load,  $P_t$ , exceeds 30 psf (1.44 kN/m<sup>2</sup>). Where the ground snow load is 70 psf (3.35 kN/m<sup>2</sup>), the heavy roof system criteria applies.

Example: Load due to 70 psf ground snow load for normal weight roofs =  
 $15 \text{ psf} + 0.2 \times 0.7 \times 70 \text{ psf} = 24.8 \text{ psf}$  (1.19 kN/m<sup>2</sup>) (equals the heavy weight roof systems)

### E11.2 Braced Wall Lines

Two types of *braced wall lines* are presented in this section: *Type I* and *Type II Braced Walls*. *Type I braced walls* are traditional *shear walls* that have a *hold down anchor* at each end and have no openings between anchors. *Type II braced walls*, also known as *perforated shear walls*, are *shear walls* that have openings between anchors and there is no design for shear transfer around the openings.

### E11.3 Type I (Solid Sheathed) Braced Wall Panels

This section describes the traditional method of sheathing a steel-framed wall where continuous panels have hold down anchors at each end. The aspect ratio (height to width) used in the development of these provisions for this wall type is 2:1.



#### **E11.4 Type II (Perforated) Braced Wall Lines**

The *Type II Braced Wall*, or perforated *shear wall* method, requires *hold down anchors* at each end of each wall rather than at each end of continuous panels. The aspect ratio (height to width) is also 2:1 for this wall type. For a defined *Type II* (perforated *shear wall*), the adjustment factors given in Table E11-2 define the magnitude by which the strength of an otherwise solid wall must be divided to get the strength of the *Type II* (perforated) wall. The tabulated values, adopted from wood frame construction, were justified on the basis of a series of full-scale reversed cyclic tests by Vagh, Dolan and Easterling (2000) in which it was demonstrated that the tested wall capacities were greater than the reductions that are required by Table E11-2.

### **E12 Braced Wall Design in High Seismic Areas**

#### **E12.2 Braced Wall Anchorage and Chord Stud Requirements**

*AISI S230* permits the tabulated values to be divided by 1.4 when comparing requirements with manufacturer's published strengths expressed as allowable loads. The derivation of this adjustment factor is:  $\phi \times \Omega = 0.55 \times 2.5 = 1.4$ . In earlier editions of cold-formed steel lateral design provisions, the resistance factor was set at 0.55 to reflect the 1.4 value used in the UBC to compute allowable seismic loads (1.4 approx. =  $0.55 \times 2.5$ ).

### **E13 Braced Wall Design in High Wind Areas**

#### **E13.3 Connections of Walls in High Wind Areas**

##### **E13.3.2 Uplift Connection – Wall Assembly to Wall Assembly**

*AISI S230* permits the tabulated values to be divided by 1.3 when comparing requirements with manufacturers' published strengths expressed as allowable loads. The derivation of this adjustment factor is:  $\phi \times \Omega = 0.60 \times 2.0 = 1.3$ .

##### **E13.3.3 Header Uplift Connections**

For back-to-back *headers* supporting roof and ceiling only, these provisions require that uplift straps be installed on both sides of the *header* beam (inside and outside of the wall) in order to minimize any effect of torsion. This requirement is based on engineering judgment and recognizes that the back-to-back *header* lacks sufficient torsional strength and stiffness. For back-to-back *headers* supporting loads from one floor, roof and ceiling, and for any box and double L-headers, a single uplift strap is permitted and may be installed on either side of the *header* beam.

## F. ROOF FRAMING

### F1 Roof Construction

Roof *trusses* are not prescriptively addressed in *AISI S230*, but are permitted, in accordance with Section F6, and must be designed by a *design professional*. Also roof girders are also not addressed in *AISI S230*.

### F2 Ceiling Joists

#### F2.1 Minimum Ceiling Joist Size

*Ceiling joist* tables in *AISI S230* provide the maximum allowable *ceiling joist* spans for two loading conditions: 10 psf (0.48 kN/m<sup>2</sup>) and 20 psf (0.96 kN/m<sup>2</sup>) attic live loads.

For the design of *ceiling joists*, for the following design considerations were evaluated:

- Flexural yielding
- Flexural buckling
- Web crippling (not required if *bearing stiffeners* are specified)
- Shear
- Vertical deflection

The engineering approach used to develop *ceiling joist* span tables for *AISI S230* is similar to that used for *floor joists* with the exception of the magnitude of dead and live loads.

#### F2.2 Ceiling Joist Bearing Stiffeners

*AISI S230* enables the selection of *ceiling joists* based on the use of *bearing stiffeners*.

#### F2.3 Ceiling Joist Bottom Flange Bracing

Gypsum board (i.e. finished ceilings) is considered to be adequate *bracing* for the bottom (tension) *flanges* of the *ceiling joists*. Steel *strapping* can also be used as bottom *flange bracing* for *ceiling joists*.

#### F2.4 Ceiling Joist Top Flange Bracing

For braced top (compression) *flanges* it is necessary for steel *strapping* to have *blocking* (or bridging) installed at a maximum spacing of 12 feet (3.66 m) and at the termination of all *straps*. Moreover, the ends of steel *straps* are to be fastened to a stable component of the building if end *blocking* is not installed. *Ceiling joist* tables provide spans for braced, as well as unbraced, top *flanges*.

#### F2.5 Ceiling Joist Splicing

Splicing of *ceiling joists* in *AISI S230* requires an *approved* design except when lapped *ceiling joists* occur at an interior bearing wall.

### F3 Roof Rafters

#### F3.1 Minimum Roof Rafter Sizes

The *roof rafter* span table was designed based primarily on gravity loads, hence the *roof rafter* spans are based on the horizontal projection of the *roof rafter*, regardless of the slope. The gravity loads consist of a 7 psf (0.34 kN/m<sup>2</sup>) dead load and the greater of a 16 psf (0.77 kN/m<sup>2</sup>) live load or the applied roof snow load. Unbalanced snow loads in accordance with ASCE 7 were considered.

Wind load effects are developed by a procedure that equates the wind loads to equivalent snow loads as shown in Table F3.2 of *AISI S230*. Wind pressures were calculated using the ASCE 7 (ASCE, 2005) Components and Cladding coefficients. Wind loads acting perpendicular to the plane of the *roof rafter* were adjusted to represent loads acting orthogonal to the horizontal projection of the *roof rafter*. Wind loads were examined for both uplift and downward loads and the worst case was correlated to a corresponding snow load.

Permissible roof slopes range between 3:12 through 12:12 and more importantly, the roof system must consist of both *ceiling joists* (i.e. acting as rafter ties) and *roof rafters*. *AISI S230* does not currently address cathedral ceilings because a prescriptive ridge beam and post design is not provided.

Lapped *ceiling joists* must be connected with the same screw size and number (or more) as the heel joint connection to ensure adequate transfer of tension loads across the spliced joint. The splice must occur over an interior bearing wall.

##### F3.1.1 Eave Overhang

A 24 inch (610 mm) eave overhang was used when calculating the *roof rafter* spans in *AISI S230*.

##### F3.1.1 Rake Overhang

In 2007, limitations and details were added to *AISI S230* to clarify the installation requirements at gable endwalls, based on a study at University of Missouri-Rolla (Downey et al., 2005).

#### F3.2 Roof Rafter Support Brace

The support brace is used to increase the span of a particular member. When the brace is used, the *roof rafter* span is determined from the heel joint to the brace point or from the ridge member to the brace point (horizontal projection), whichever is greater.

#### F3.3 Roof Rafter Splice

The *roof rafter* spans provided in *AISI S230* are based on the assumption that the members are continuous, with no splices. Therefore, *roof rafters* are not to be spliced without an *approved* design.

#### F3.5 Roof Rafter Bottom Flange Bracing

The *bracing* requirements provided in *AISI S230* are commonly used in residential steel construction and are based on engineering judgment.

#### F4 Hip Framing

Prior to the 2007 edition of this standard, roof framing was limited to *roof rafters* and *ceiling joists*. Hip and valley framing options were added in 2007, based on research at the University of Missouri -Rolla (Waldo et al., 2006).

#### F5 Framing of Openings in Roofs and Ceilings

The requirements of this section are based on engineering judgment.

#### F6 Roof Trusses

AISI S230 does not contain provisions for *roof trusses*, which must have an *approved* design. This section is included so that pre-engineered *roof trusses* may be used in conjunction with this document. The *North American Standard for Cold-Formed Steel Framing – Truss Design* (AISI S214-07) should be consulted for the *truss* design.

#### F7 Ceiling and Roof Diaphragms

Roof *diaphragms* are required to adequately transfer shear loads to the *braced wall lines* in a structure. The load transfer typically accomplished by sheathing the roof-framing members with wood structural panels. Shear values used in the design of roof *diaphragms* were taken from the *North American Standard for Cold-Formed Steel Framing – Lateral Design* (AISI S213-07). Additional requirements for steel roof *diaphragms* in *high wind areas* (i.e., 110 mph (177 km/hr) or greater wind speed) or *high seismic areas* (i.e., *Seismic Design Category* D<sub>0</sub>, D<sub>1</sub>, D<sub>2</sub> and E) are specified in Section F6.1 and F6.2.

Ceiling *diaphragms* are also required to adequately transfer shear loads to the *braced wall lines* in a structure. The load transfer typically accomplished by sheathing the ceiling-framing members with gypsum board or wood structural panels. Shear values used in the design of ceiling *diaphragms* were taken from the *North American Standard for Cold-Formed Steel Framing – Lateral Design* (AISI S213-07).

## PART 2 – DESIGN EXAMPLES

### FOR THE STANDARD FOR COLD-FORMED STEEL FRAMING – PRESCRIPTIVE METHOD FOR ONE AND TWO FAMILY DWELLINGS

#### A. INTRODUCTION

Part 2 illustrates the basis for the development of the prescriptive requirements for cold-formed steel framing. Part 2 validates tabulated values through the use of design examples for *AISI S230* (AISI S230, 2007d). These design examples are based primarily on existing available reference standards such as the American Iron and Steel Institute *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2007), the American Society of Civil Engineers standard *ASCE-7 Minimum Design Loads for Buildings and Other Structures* (ASCE, 2005) and the American Iron and Steel Institute *North American Standard for Cold-Formed Steel Framing – Header Design* (AISI, 2007c).

These design examples are shown in U.S. customary units. For appropriate conversions to the International System of Units (SI), refer to Appendix A.

#### A1 Member Properties

All section properties and member capacities were calculated using either the 1996 *Specification* (AISI, 1996) or the 2001 *Specification* (AISI, 2001). Refer to Section A3 in Part 1 for a discussion of referenced documents. Floor and ceiling joist members were assumed to have holes with 2½" depth x 4 ½" length located along the centerline of the web. This is the maximum hole size permitted by *AISI S100*. Studs and other structural elements were assumed to have holes with 1½" wide x 4" long along the centerline of the web.

Minimum  $F_y = 33$  ksi      ➔      Minimum  $F_u = 45$  ksi  
Minimum  $F_y = 50$  ksi      ➔      Minimum  $F_u = 65$  ksi

The design thickness (minimum thickness divided by 0.95) used in calculating member capacity and section properties was based on the *Specification* Section A2.4.

#### A2 Design Loads

The following loads were used in designing the various steel-framing members:

|                        |   |
|------------------------|---|
| Ceiling dead load      | = 5 psf   |
| Roof dead load         | = 7 psf   |
| Floor dead load        | = 10 psf  |
| First floor live load  | = 40 psf  |
| Second floor live load | = 30 psf  |
| Wall dead load         | = 10 psf  |
| Attic live load        | = 10 psf for attics with no storage   |
| Attic live load        | = 20 psf for attics with storage  |
| Wind load              | = Varies by wind speed and exposure (3-sec. Gust)   |
| Seismic Load           | = Varies by Seismic Design Category: A, B, C, D <sub>0</sub> , D <sub>1</sub> , D <sub>2</sub> , or E |
| Roof snow load         | = 0.7 x Ground snow load (unbalanced snow loads were considered)                                      |

Roof live load = greater of 16 psf live load or the applied snow load

### A2.1 Roof Snow Loads

Applied roof snow loads were calculated by multiplying the ground snow load by a 0.7 conversion factor in accordance with ASCE 7 (ASCE, 2005). No further reductions were made for special cases.

The sloped roof snow load,  $P_s = (C_s)(P_f)$ , where  $P_f$  is the flat roof snow load.

$$P_f = 0.7 C_e C_t I P_g$$

$C_s$   $C_s$  is the roof slope factor ranging from approximately 0.1 to 1.0. A slope factor of 1.0 was judged to be conservative for houses with roof slopes from 3:12 to 12:12.

$C_e$   $C_e$  is the exposure factor depending on the location of the house.  $C_e$  varies from 0.8 for windy, unsheltered areas, to 1.2 for heavily sheltered areas. A factor of 1.0 was deemed reasonable for residential buildings that are partially exposed (ASCE 7, Table 7-2).

$C_t$   $C_t$  is a thermal factor that varies from 1.0 for heated structures to 1.2 for unheated structures. The thermal factor should be used based on the thermal condition that is likely to exist during the life of the structure. Houses are typically considered heated structures with  $C_t = 1.0$  (ASCE 7, Table 7-3).

$I$   $I$  is the importance factor based on building classification. Houses are typically Category II structures, with an importance factor of 1.0 (ASCE 7, Table 7-4).

$P_g$   $P_g$  is the ground snow load from the ASCE 7 estimated ground snow map (psf).

Unbalanced snow loads were considered but sliding snow loads, and snow drifts on lower roofs were not considered. Rain-on-snow surcharge load was also not considered in the calculations because the roof slopes in this document exceed the 1/2-inch per foot requirement by ASCE 7 for rain-on-snow surcharge to be considered. Therefore, the flat roof snow load was computed as:  $(1.0)(0.7)(1.0)(1.0)(P_g) = 0.7 P_g$ .

### A2.2 Wind Loads

The design tables contained in *AISI S230* were generated at different times and, consequently, using different editions of ASCE 7. For example, tables for the lateral provisions were developed using the 1998 edition whereas the wall stud tables were updated using the 2005 edition of ASCE 7. Every effort was made in this *Commentary* to cite the applicable edition of ASCE 7.

As an example, wind loads developed for floor and roof diaphragms and braced walls for wind were based on 3-second gust wind speeds ranging from 85 to 130 mph, Exposure A, B, or C in accordance with ASCE 7-98.

$$q = 0.00256 K_z (G C_p + G C_{pi}) (V^2 \times I)$$

where

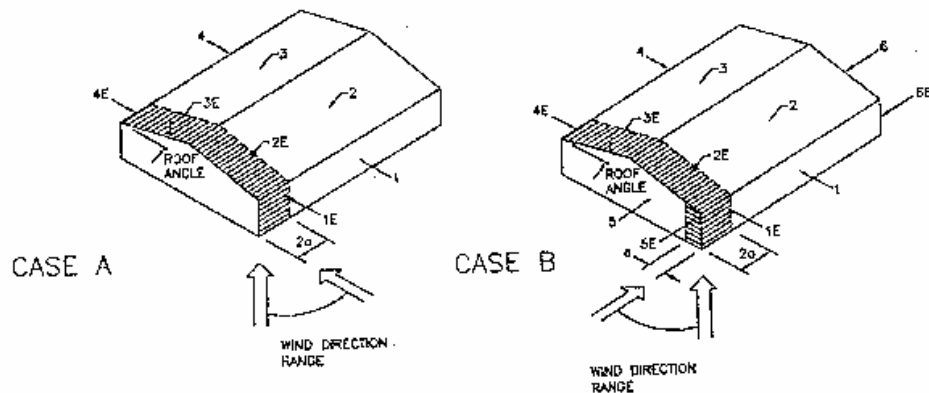
$K_z$  = 0.87 at 20 feet, for Exposure C

$G C_p$  = 0 for Zone 1, Tributary area = 75 ft<sup>2</sup>,

$G C_{pi}$  =  $\pm 0.25$  for Components and Cladding, interior pressure, enclosed buildings

$I$  = 1.0 for residential buildings in areas with wind speed < 100 mph

Tables A2.1 and A2.2 provide a summary of wind loads that were calculated in accordance with ASCE 7-98 (refer to Figure A2-1 for building surface).



**Figure A2-1 ASCE 7 Building Surfaces**

**Table A2.1**  
**ASCE 7-98 Main Wind Force Resisting System Design Pressures (psf)<sup>1,2,3,4,5,6,7</sup>**

| ROOF<br>PITCH | BUILDING<br>SURFACE<br>(Fig. 6-4) | LOADED<br>REGION   | 3-sec WIND SPEED (mph) and EXPOSURE |      |      |      |      |      |      |      |
|---------------|-----------------------------------|--------------------|-------------------------------------|------|------|------|------|------|------|------|
|               |                                   |                    | 85                                  |      | 90   |      | 100  |      | 110  |      |
|               |                                   |                    | A/B                                 | C    | A/B  | C    | A/B  | C    | A/B  | C    |
| 3:12          | 2r+3r                             | Roof               | 10                                  | 10   | 10   | 10   | 10   | 10   | 10   | 10   |
|               | 2Er+3Er                           | Roof Corner        | 10                                  | 10   | 10   | 10   | 10   | 10   | 10   | 10   |
|               | 1+4                               | Building           | 10                                  | 13.2 | 10.5 | 14.8 | 13   | 18.2 | 15.7 | 22.1 |
|               | 1E+4E                             | Building Corner    | 14.1                                | 19.8 | 15.8 | 22.2 | 19.5 | 27.4 | 23.6 | 33.1 |
|               | -                                 | Stud Design        | 10                                  | 10.2 | 10   | 11.4 | 10   | 14.1 | 12.6 | 17   |
|               | -                                 | Stud Design Corner | 10                                  | 14   | 11.2 | 15.7 | 13.8 | 19.3 | 16.7 | 23.4 |
| 6:12          | 2r+3r                             | Roof               | 1.7                                 | 2.4  | 1.9  | 2.7  | 2.4  | 3.3  | 2.9  | 4    |
|               | 2Er+3Er                           | Roof Corner        | 1.9                                 | 2.7  | 2.2  | 3.1  | 2.7  | 3.8  | 3.3  | 4.6  |
|               | 1+4                               | Building           | 10.4                                | 14.5 | 11.6 | 16.3 | 14.3 | 20.1 | 17.3 | 24.3 |
|               | 1E+4E                             | Building Corner    | 13.9                                | 19.5 | 15.6 | 21.9 | 19.3 | 27   | 23.3 | 32.7 |
|               | -                                 | Stud Design        | 10                                  | 11.3 | 10   | 12.6 | 11.1 | 15.6 | 13.5 | 18.9 |
|               | -                                 | Stud Design Corner | 10                                  | 14   | 11.2 | 15.7 | 13.8 | 19.4 | 16.7 | 23.5 |
| 9:12          | 2r+3r                             | Roof               | 4.2                                 | 5.9  | 4.7  | 6.6  | 5.9  | 8.2  | 7.1  | 9.9  |
|               | 2Er+3Er                           | Roof Corner        | 5.3                                 | 7.4  | 5.9  | 8.3  | 7.3  | 10.3 | 8.9  | 12.4 |
|               | 1+4                               | Building           | 10.2                                | 14.4 | 11.5 | 16.1 | 14.2 | 19.9 | 17.2 | 24.1 |
|               | 1E+4E                             | Building Corner    | 12.9                                | 18.1 | 14.4 | 20.3 | 17.8 | 25   | 21.6 | 30.3 |
|               | -                                 | Stud Design        | 10                                  | 11.4 | 10   | 12.8 | 11.3 | 15.8 | 13.7 | 19.1 |
|               | -                                 | Stud Design Corner | 10                                  | 13.4 | 10.7 | 15.1 | 13.3 | 18.6 | 16   | 22.5 |
| 12:12         | 2r+3r                             | Roof               | 5                                   | 7    | 5.6  | 7.8  | 6.9  | 9.7  | 8.3  | 11.7 |
|               | 2Er+3Er                           | Roof Corner        | 6.2                                 | 8.7  | 7    | 9.8  | 8.6  | 12.1 | 10.4 | 14.6 |
|               | 1+4                               | Building           | 10.2                                | 14.4 | 11.5 | 16.1 | 14.2 | 19.9 | 17.2 | 24.1 |
|               | 1E+4E                             | Building Corner    | 12.9                                | 18.1 | 14.4 | 20.3 | 17.8 | 25   | 21.6 | 30.3 |
|               | -                                 | Stud Design        | 10                                  | 11.4 | 10   | 12.8 | 11.3 | 15.8 | 13.7 | 19.1 |
|               | -                                 | Stud Design Corner | 10                                  | 13.4 | 10.7 | 15.1 | 13.3 | 18.6 | 16   | 22.5 |
| ALL           | 5+6                               | Bldg Gable End     | 10                                  | 10.7 | 10   | 11.9 | 10.5 | 14.7 | 12.7 | 17.8 |
|               | 5E+6E                             | Gable End Corner   | 11.5                                | 16.1 | 12.8 | 18   | 15.9 | 22.2 | 19.2 | 26.9 |

<sup>1</sup> Values based on Figure 6-4, Main Wind Force Resisting System, h<60 ft Walls and Gable Roof.

<sup>2</sup> Design pressures are based on a 30-ft mean roof height.

<sup>3</sup> (r = resultant horizontal component) The roof pressure is the total horizontal resultant pressure (windward & leeward) and should be applied to the vertical projected surface of the roof.

<sup>4</sup>  $K_D = 0.85$ , load combinations in Section 2 shall be used.

<sup>5</sup> Stud design pressures are for combined axial and bending load combinations. Load indicated is greater of cases a and b wind directions (side & end walls) with internal & external pressure applied.

<sup>6</sup> Gable end building pressure is for wind parallel to ridge of a gable roof (case b).

<sup>7</sup> Corner loads applied to a distance of a or 2a per Fig. 6-4, a = 10% of least width or 0.4h (whichever is smaller) but not less than either 4% of least width or 3 ft.



**Table A2.2**  
**ASCE 7-98 Components and Cladding Wall Design Pressures (psf)<sup>1,2,3,4</sup>**

| PRESSURE DIRECTION | LOADED REGION       | 3-sec WIND SPEED (mph) and EXPOSURE |       |       |       |       |       |       |       |       |       |       |       |
|--------------------|---------------------|-------------------------------------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
|                    |                     | 85                                  |       | 90    |       | 100   |       | 110   |       | 120   |       | 130   |       |
|                    |                     | A/B                                 | C     | A/B   | C     | A/B   | C     | A/B   | C     | A/B   | C     | A/B   | C     |
| Windward           | Stud Design Typical | 12.4                                | 17.4  | 13.9  | 19.5  | 17.2  | 24.1  | 20.8  | 29.1  |       |       |       |       |
|                    | Stud Design Corner  | 12.4                                | 17.4  | 13.9  | 19.5  | 17.2  | 24.1  | 20.8  | 29.1  |       |       |       |       |
| Leeward            | Stud Design Typical | -13.5                               | -18.9 | -15.1 | -21.1 | -18.7 | -26.2 | -22.6 | -31.6 | -26.9 | -37.7 | -31.6 | -44.2 |
|                    | Stud Design Corner  | -16.2                               | -22.7 | -18.2 | -25.5 | -22.5 | -31.5 | -24.6 | -34.4 |       |       |       |       |

<sup>1</sup> Values based on Figure 6-5A, Components and Cladding,  $h < 60$  ft Walls.

<sup>2</sup> Design pressures are based on a 30-ft mean roof height.

<sup>3</sup>  $K_d = 0.85$ , load combinations in Section 2 shall be used.

<sup>4</sup> Exposure C wind pressures are calculated by multiplying Exposure B pressures by 1.40.

The above two tables were further narrowed down to a smaller table, Table A2.3, to reduce the number of wall stud tables generated. The values for the stud design pressures rather than the stud design corner pressures were used because the corner pressures are only applicable to small areas around the building corners. If the corner pressures were to be used, the majority of the wall studs in the building will be over designed resulting in an uneconomic design. Furthermore, *AISI S230* requires a minimum of three studs at building corners thus compensating for the slightly increased pressures used in that region of the building.

**Table A2.3**  
**Design Pressures Used for Wall Stud Tables (psf)<sup>1,2</sup>**

| LOAD CASE | 3-sec WIND SPEED (mph) and EXPOSURE |      |      |      |      |      |      |      |      |      |      |      |
|-----------|-------------------------------------|------|------|------|------|------|------|------|------|------|------|------|
|           | 85                                  |      | 90   |      | 100  |      | 110  |      | 120  |      | 130  |      |
|           | A/B                                 | C    | A/B  | C    | A/B  | C    | A/B  | C    | A/B  | C    | A/B  | C    |
| MWFRS     | 10                                  | 11.4 | 10   | 12.8 | 11.3 | 16.8 | 13.7 | 19.1 |      |      |      |      |
| C&C       | 13.5                                | 18.9 | 15.1 | 21.1 | 18.7 | 26.2 | 22.6 | 31.6 | 26.9 | 37.7 | 31.6 | 44.2 |

<sup>1</sup> Values based on ASCE 7-98 Figure 6-5A, Components and Cladding (C&C),  $h < 60$  ft Walls.

<sup>2</sup> Design pressures are based on a 30-ft mean roof height.

Exposures A/B and C were also compared and tabulated in Table A2.4. The resulting comparison was used in developing the wall stud tables.

**Table A2.4**  
**Design Pressure Comparison Chart<sup>1</sup>**

| Wind Speed (mph) |     | MWFRS (psf)  |            | C&C (psf)    |            |
|------------------|-----|--------------|------------|--------------|------------|
|                  |     | Exposure A/B | Exposure C | Exposure A/B | Exposure C |
| 85               |     | 10           |            | 13.5         |            |
| 90               |     | 10           |            | 15.1         |            |
| 100              | 85  | 11.3         | 11.4       | 18.7         | 18.9       |
| 110              | 90  | 13.7         | 12.8       | 22.6         | 21.1       |
|                  | 100 |              | 16.8       |              | 26.2       |
|                  | 110 |              | 19.1       | 22.6         | 31.6       |
|                  | 120 |              |            | 26.9         | 37.7       |
|                  | 130 |              |            | 31.6         | 44.2       |

<sup>1</sup> Values based on ASCE 7-98

The following tables summarize the design wind pressures used for the tables that were updated in the 2007 edition of *AISI S230*.

**Table A2.5**  
**Main Wind Force Resisting System<sup>1,2</sup>**

| EXPOSURE | BASIC WIND SPEED (mph) |      |      |      |      |      |      |      |
|----------|------------------------|------|------|------|------|------|------|------|
|          | 85                     | 90   | 100  | 110  | 120  | 130  | 140  | 150  |
| <b>B</b> | 10.0                   | 10.0 | 11.3 | 13.6 | 16.2 | 19.0 | 22.1 | 25.4 |
| <b>C</b> | 11.4                   | 12.8 | 15.8 | 19.1 | 22.7 | 26.7 | 30.9 | 35.5 |

<sup>1</sup> Values based on ASCE 7-05.

<sup>2</sup> Design pressures are based on a 30-ft mean roof height.

**Table A2.5**  
**Components & Cladding<sup>1,2</sup>**

| EXPOSURE | BASIC WIND SPEED (mph) |      |      |      |      |      |      |      |
|----------|------------------------|------|------|------|------|------|------|------|
|          | 85                     | 90   | 100  | 110  | 120  | 130  | 140  | 150  |
| <b>B</b> | 14.1                   | 15.8 | 19.5 | 23.6 | 28.1 | 32.9 | 38.2 | 43.9 |
| <b>C</b> | 19.7                   | 22.1 | 27.3 | 33.0 | 39.3 | 46.1 | 53.5 | 61.4 |

<sup>1</sup> Values based on ASCE 7-05.

<sup>2</sup> Design pressures are based on a 30-ft mean roof height.

### **A3 Load Combinations**

The load and resistance factor design (LRFD) load combinations as shown in ASCE 7 (ASCE, 2005) were used. These load combinations are summarized in the Table A3.1.

**Table A3.1**  
**Summary of Load Combinations**

| Framing Component          | Load <sup>2</sup> Combinations  |
|----------------------------|---|
| Floor Joists               | 1.4D<br>1.2D + 1.6L   |
| Ceiling Joists             | 1.4D<br>1.2D + 1.6L   |
| Headers                    | 1.4D<br>1.2D + 1.6L + 0.5(L <sub>r</sub> or S)<br>1.2D + 0.5L + 1.6(L <sub>r</sub> or S)  |
| Header Uplift <sup>1</sup> | 0.9D - 1.6W<br>1.2D + 0.5(L <sub>r</sub> or S) + 0.5L - 1.6W<br>1.2D + 1.6(L <sub>r</sub> or S) - 0.8W  |
| Wall Studs                 | 1.4D<br>1.2D + 1.6L + 0.5(L <sub>r</sub> or S)<br>1.2D + 0.5L + 1.6(L <sub>r</sub> or S)<br>1.2D + 0.8W + 1.6(L <sub>r</sub> or S)<br>1.2D + 1.6W + 0.5L + 0.5(L <sub>r</sub> or S)<br>1.6W |
| Roof Rafters               | 1.4D<br>1.2D + 1.6L + 0.5(L <sub>r</sub> or S)<br>1.2D + 0.5L + 1.6(L <sub>r</sub> or S)<br>Wind loads will be converted to equivalent snow loads   |
| Sheathed Shear Walls       | 1.6W (MWFRS Walls and Roofs)  |

<sup>1</sup> Uplift loads were checked for L-headers only. The uplift loads do not control the design of back-to-back or box-beam headers.

<sup>2</sup> Load Definitions:

D = Dead Load  
L = Live Load

W = Wind Load  
S = Snow Load

L<sub>r</sub> = Roof Live Load

#### A4 Deflection Limits

**Table A4.1**  
**Deflection Limits**

| Framing Component       | Deflection Due to Live Load <sup>1</sup> | Deflection Due to Total Load <sup>1</sup> |
|-------------------------|--|---|
| Floor Joists            | L/480                                    | L/240                                     |
| Headers                 | L/360                                    | L/240                                     |
| Wall Studs <sup>2</sup> | L/240                                    | -   |
| Ceiling Joists          | L/360                                    | L/240                                     |
| Roof Rafters            | L/240                                    | L/180                                     |

<sup>1</sup> Unfactored loads were used to calculate deflections.

<sup>2</sup> A factor of 0.7 was applied to the deflection limit of load combinations including components and cladding wind loads in accordance with the AISI, S211-07.

## A5 Design Checks and Assumptions

Summarized in Table A5.1 are the design checks for each framing component.

**Table A5.1**  
**Design Checks**

| Framing Component           | Bending | Shear | Web Crippling | Bending & Shear | Bending & Web Crippling | Axial | Axial & Bending | Deflection |
|-----------------------------|---------|-------|---------------|-----------------|-------------------------|-------|-----------------|------------|
| Floor Joists <sup>1,2</sup> | √       | √     |               | √               |                         |       |                 | √          |
| Headers <sup>3</sup>        | √       | √     |               | √ <sup>4</sup>  | √                       |       |                 | √          |
| Wall Studs                  | √       | √     | √             |                 |                         | √     | √               | √          |
| Ceiling Joists              | √       | √     | √             | √               | √ <sup>5</sup>          |       |                 | √          |
| Roof Rafters                | √       | √     |               |                 |                         |       |                 | √          |

Comment [D01]: Footnote missing

<sup>1</sup> All joists must have *bearing stiffeners* at support locations.

<sup>2</sup> Combined bending and shear was checked for double spans.

<sup>3</sup> The clip angle at the header connection to the stud was considered a *bearing stiffener*.

<sup>4</sup> For two (continuous) ceiling joist spans without *bearing stiffener*.

### General Assumptions

- All members, except studs, are assumed to have 2-1/2" x 4" (1-1/2" x 4" for studs) web holes along the centerline of the web. Holes are not less than 24 inches on center, and are not within 10 inches from the end of the member or bearing condition.
- Steel may be provided in either a 33 ksi or 50 ksi yield stress.
- ASCE 7 (ASCE, 2005) wind load provisions are used.
- Design limited to Seismic Design Category A, B, C, D<sub>0</sub>, D<sub>1</sub>, D<sub>2</sub> and E, in accordance with the IBC (ICC, 2006a).
- LRFD load combinations are used.
- Increase in yield strength due to cold work of forming is used where allowed.
- Design thickness is minimum thickness divided by 0.95.
- House widths of 24', 28', 32', 36' and 40'.

### Floor Joists

- Joist Spacing: 12", 16", 19.2" and 24" oc
- Live Loads: 30 psf and 40 psf
- Span: Single (simply-supported span) and Two-equal spans with stiffeners at bearing locations (uniform load over the entire two-spans; i.e., no alternate loading is considered)
- Joist Size Range: 550S162-33 to 1200S162-97
- Yield Stress: 33 ksi steel for single spans and 33 and 50 ksi steel for multiple spans
- Floor Dead Load: 10 psf
- Design Checks: Bending, and shear for single and two spans and combined bending and shear for two-spans only
- Deflection Criteria: L/480 for live loads and L/240 for total loads
- Bracing: Joists are considered to be continuously braced at the top flange by floor sheathing

- Bearing Stiffeners: All floor joists are assumed to have stiffeners at bearing support locations
- Bearing Length: Minimum bearing length of 1½" at end supports and 3½" at interior supports

#### Structural Wall Studs

- Stud Spacing: 16" and 24" on center
- Wind Speed: 85 through 130 mph Exposures A/B (3-sec. gust wind speeds) with adjustments for Exposure C (enclosed buildings)
- Seismic Design Category: A, B, C, D<sub>0</sub>, D<sub>1</sub>, D<sub>2</sub> and E
- Stud Heights: 8', 9', and 10'
- Stud Size Range: 350S162 and 550S162 for 33 to 97 mil thickness
- Ground Snow Loads: 20 psf, 30 psf, 50 psf, and 70 psf
- Attic Live Load: No attic live load acting on the studs. Tables are footnoted to allow attics with storage by using to the next snow load value
- Snow Load: 0.7xGround Snow Load (16 psf minimum). Unbalanced snow loads were computed in accordance with ASCE 7-05.
- Stud Bracing: Bracing of the interior and exterior flanges of the studs by sheathing or mechanical bracing (mechanical bracing at mid-height for 8' studs, 1/3 point for 9' and 10' studs)
- Yield Stress: 33 and 50 ksi
- 2<sup>nd</sup> Floor Dead Load: 10 psf
- 2<sup>nd</sup> Floor Live Load: 30 psf
- 2<sup>nd</sup> Floor Wall Dead Load: 10 psf
- Roof Dead Load: 7 psf
- Ceiling Dead Load: 5 psf
- Design Checks: Bending only: C&C loads per ASCE 7-05  
Combined axial and bending: MWFRS loads
- Deflection Criteria: L/240 for C&C ASCE 7 wind loads with 0.7 reduction factor
- Other Assumptions:
  - Studs are simply supported beams.
  - Maximum roof overhang is 24" on either side of roof.
  - Roof slopes limited to a range of 3:12 to 12:12.
  - Ceilings, roofs, and floors span full width of the house; no interior load bearing walls.

#### Headers

- Headers Supporting: Roof and ceiling and one floor, roof and ceiling, roof and ceiling with center beam supporting first floor
- Header Types: Back-to-Back, Box Beam and L-Header
- Header Size Range: 350S162-33 through 1200S162-97
- L-Header Size Range: 600L150-43 through 1000L150-68
- Ground Snow Loads: 20 psf, 30 psf, 50 psf, and 70 psf

- Attic Live Load: No attic live load is considered in the header design. Tables are footnoted to allow for attics with storage by using the next snow load value.
- Snow Load:  $0.7 \times \text{Ground Snow Load}$  (16 psf minimum). Unbalanced snow loads were computed in accordance with ASCE 7-05.
- 2<sup>nd</sup> Floor Dead Load: 10 psf
- 2<sup>nd</sup> Floor Live Load: 30 psf
- 2<sup>nd</sup> Floor Wall Dead Load: 10 psf (for 8 feet)
- Roof Dead Load: 7 psf
- Ceiling dead Load: 5 psf
- Yield Strength: 33 ksi steel
- Design Checks: All headers designed in accordance with *Standard for Cold-Formed Steel Framing – Header Design* (AISI S212-07, 2007c).
- Deflection Criteria:  $L/240$  for total load
- Other Assumptions:
  - Headers are simply supported beams.
  - Maximum roof overhang is 24".
  - Roof slopes limited to a range of 3:12 to 12:12.
  - Ceilings, roofs, and floors span the full width of the house with no interior load bearing walls, except as noted.
  - The allowable capacity of each header is calculated in accordance with AISI S212.
  - The number of king and jack studs was determined by as the width of the opening and divided by the stud spacing. The results were rounded to the next stud.

#### Ceiling Joists

- Joist Spacing: 16" and 24"
- Live Loads: 10 psf and 20 psf
- Spans: Single and two-equal spans, with and without web stiffeners (uniform load over the entire two-spans; no alternate span loading was considered)
- Joist Size Range: 350S162 through 1200S162 with 33 to 97 mil thickness
- Ceiling Dead Load: 5 psf
- Yield Stress: 33 ksi
- Bracing: Unbraced, mid-span, or third point bracing
- Deflection Criteria:  $L/240$  for total load
- Basic Load Combinations: LRFD loads and load combinations
- Design Checks: Bending, shear, combined bending and shear, and combined bending and web crippling
- Bearing Width: 3-1/2" at ends and at interior supports

#### Rafters

- Rafter Spacing: 16" and 24"
- Ground Snow Loads: 20 psf, 30 psf, 50 psf, and 70 psf
- Wind Speeds: Same as wall studs wind speeds
- Seismic Design Categories: A, B, and C
- Spans: Single and two-equal spans

- Joist Size Range: 550S162-33 through 1200S162-97
- Roof Dead Load: 12 psf
- Yield Stress: 33 ksi
- Roof Pitch: 3:12 to 12:12
- Deflection Criteria:  $L/240$  for live load and  $L/180$  for total load
- Snow Load =  $0.7 \times$  ground snow load (16 psf minimum). Unbalanced snow loads were computed in accordance with ASCE 7-05.
- Basic Load Combinations: LRFD loads and load combinations
- Bracing: Bottom flange bracing located at mid-span
- Other Assumptions:
  - Rafter spans are designed based on gravity loads; hence the rafter spans are reported on the horizontal projection of the rafter, regardless of the slope. The gravity loads consist of a roof dead load and the greater of a minimum 16 psf live load or the applied roof snow load.
  - Wind load effects are correlated to equivalent snow loads. Wind pressures were calculated using the ASCE 7-05 Components and Cladding pressure coefficients. Wind loads acting perpendicular to the plane of the rafter were adjusted to represent loads acting orthogonal to the horizontal projection of the rafter. Wind loads were examined for both uplift and downward loads and the worst case was correlated to a corresponding snow load.
  - The roof system must consist of both ceiling joists (i.e. acting as rafter ties) and rafters.
  - Rafters are assumed to be simply supported beams.

Summarized in Table A5.2 are the loads and other design assumptions used in developing *AISI S230*.

**Table A5.2**  
**Summary of Loads and Design Assumptions**

|                                | Floor Joists  | Wall Studs         | Headers   |     |                                | Ceiling Joists                             | Rafters   | Shear Walls                   |
|--------------------------------|---|--------------------|---|-----|--------------------------------|--|---|-------------------------------|
|                                |   |                    | B-to-B  | Box | L                              |  |   |                               |
| Member Size                    | 550S162<br>800S162<br>1000S162<br>1200S162                    | 350S162<br>550S162 | 350S162<br>550S162<br>800S162<br>1000S162<br>1200S162 |     | 600L150<br>800L150<br>1000L150 | 550S162<br>800S162<br>1000S162<br>1200S162 | 350S162<br>550S162<br>800S162<br>1000S162<br>1200S162 | OSB<br>Plywood<br>Sheet Steel |
| Member Thickness               | 33 mil to 97 mil  |                    |   |     | 43 mil to 68 mil               | 33 mil to 97 mil                           |   | 27 mil<br>(see note 1)        |
| Member Height                  | N/A   | 8', 9' and 10'     | N/A   |     |                                |  |   |                               |
| Yield Stress (ksi)             | 33 & 50   |                    | 33  |     |                                |  |   |                               |
| Snow Load (psf)                | N/A   | 20 to 70           |   |     |                                |  |   |                               |
| Wall DL (psf)                  | N/A   | 10                 |   |     |                                | N/A  |   | 10                            |
| Floor DL (psf)                 | 10  |                    |   |     | N/A                            |  | 10  |                               |
| 1 <sup>st</sup> Floor LL (psf) | 40  | N/A                |   |     |                                |  |   | 40                            |
| 2 <sup>nd</sup> Floor LL (psf) | 30  |                    |   |     | N/A                            |  | 30  |                               |
| Roof LL (psf)                  | N/A   | 16                 |   |     |                                | N/A  | 16  |                               |
| Roof DL (psf)                  | N/A   | 7                  |   |     |                                | N/A  | 7   |                               |
| Ceiling DL (psf)               | N/A   | 5                  |   |     |                                |  | N/A   | 5                             |
| Attic LL (psf)                 | N/A   | N/A                |   |     |                                | 10 and 20                                  | N/A   |                               |
| SDC                            | A, B, C, D <sub>0</sub> , D <sub>1</sub> , D <sub>2</sub> , E |                    |   |     |                                |  |   |                               |
| Wind Speed (mph)               | 85, 90, 100, 110, 120, 130 (3-sec. gust, Exposure A, B and C) |                    |   |     |                                |  |   |                               |
| Deflection Limit               | L/480 and L/240   | L/240              |   |     |                                |  | L/180   | N/A                           |

<sup>1</sup> Minimum 7/16" thickness for OSB and plywood.



## D. FLOOR FRAMING DESIGN EXAMPLES

### D1 Floor Joist Design

Calculate the allowable span for a 1000S162-54,  $F_y = 33$  ksi, single span joist spaced at 24" (2') on center, supporting a live load of 40 psf and a dead load of 10 psf. using the provisions of the *AISI Specification* (AISI,1996). The compression flange of the joist is laterally restrained by floor sheathing. For deflection checks,  $\Delta = L/480$  for live load only and  $\Delta = L/240$  for total loads (dead load + live load).

The maximum allowable joist span is the minimum span calculated based on shear, moment, and deflection.

#### D1.1 Shear Capacity

In accordance with Section C3.2 of *AISI Specification*, the nominal shear strength,  $V_n$ , is calculated to be 2,673 lb for a joist with 2 1/2" deep web holes.

Use the design shear strength,  $\phi V_n$ , to calculate the maximum unsupported span length.

$$\phi V_n = \frac{wL}{2} \rightarrow L = \frac{2\phi V_n}{w}$$

where  $w$  = factored uniform load (plf) =  $(1.2 \times 10 + 1.6 \times 40)(2) = 152$  plf

$$L = \frac{2 \times 0.9 \times 2673}{152} = L = 31.66' = 31'-8"$$

#### D1.2 Moment Capacity

The nominal flexural strength,  $M_n$ , is 5,461 ft-lb, as calculated in accordance with Section C3.1.1 of the *AISI Specification* for a joist with 2-1/2" deep web hole.

For a simply supported span with the top flanges laterally supported:

$$\phi M_n = \frac{wL^2}{8} \rightarrow L = \sqrt{\frac{8\phi M_n}{w}}$$

$$L = \sqrt{\frac{8 \times 0.95 \times 5461}{152}}$$

$$L = 16.52 \text{ feet} = 16'-6"$$

#### D1.3 Deflection Limit

The deflection equation for a simply supported span with distributed load is:

$$\Delta = \frac{5wL^4}{384EI} \rightarrow L = \sqrt[4]{\frac{385EI_x}{5w}} \text{ (Deflection Limit)}$$

where

$L$  = Single span length (inches)  
 $I_x$  = Effective moment of inertia for deflection = 9.8815 in<sup>4</sup>  
 $w$  = 50 psf (24/12) for total load deflection check and 40 psf (24/12) for live load deflection check

$E =$  Modulus of elasticity = 29,500,000 psi  
 Deflection Limit =  $L/240$  for total loads and  $L/480$  for live loads,  
 inches

The maximum span for the deflection limits for total and live loads:

$L_{TL} = 18'-8"$  (for total loads)

$L_{LL} = 15'-11"$  (for live load only)

#### D1.4 Web Crippling Capacity

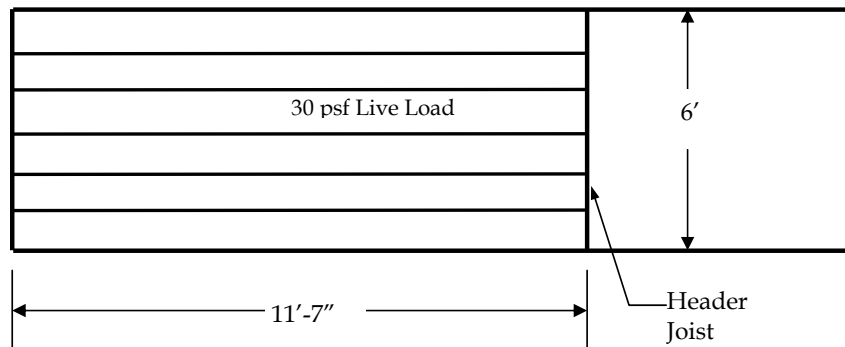
Floor joists are required to have web stiffeners at both ends and at concentrated load locations hence, there is no need to check the web crippling strength.

#### D1.5 Maximum Allowable Joist Span

The maximum allowable joist span is the minimum span calculated based on shear, moment, and deflection for a single span. The resulting span is 15'-11" (controlled by the live load deflection). This result confirms the value published in *AISI S230*, Table D3-1.

### D2 Header Joist Design

Check the adequacy of a 550S162-33,  $F_y = 33$  ksi, header joist for a 6-foot floor opening. The header joist is fabricated using a C-section nested in a track (550T125-33,  $F_y = 33$  ksi) in accordance with Detail D7-1 of *AISI S230*. The joists are spaced at 12" on center. The floor live load is 30 psf and the dead load is 10 psf.



The maximum joist span for a 550S162-33,  $F_y = 33$  ksi joists from Table D3.1 of *AISI S230* is 11'-7" (11.6').

The load on the header joist is as follows:

$$W_T = \frac{[1.2(10) + 1.6(30)](11.6')}{2} = 348 \text{ plf}$$

$$M = \frac{348(6)^2}{8} = 1,566 \text{ ft-lb}$$

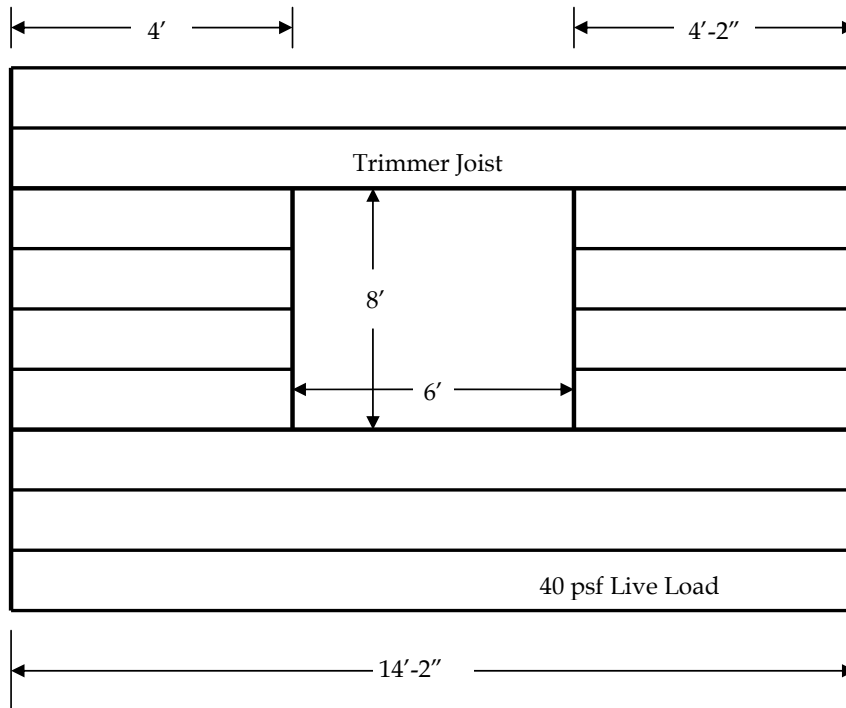
The design flexural strength of the header joist is calculated in accordance with Section C3.1.1 of the *AISI Specification* (AISI, 1996).

$$\begin{aligned}\phi M_1 &= 783.65 \text{ ft-lb} && (\text{for } 550\text{T}125-33) \\ \phi M_2 &= 1,099.57 \text{ ft-lb} && (\text{for } 550\text{S}162-33) \\ \phi M_T &= 783.65 \text{ ft-lb} + 1,099.57 \text{ ft-lb} = 1,883.22 \text{ ft-lb} \\ M &< \phi M_T = 1,566 \text{ ft-lb} < 1,883.22 \text{ ft-lb} && \text{ok}\end{aligned}$$

This verifies the header joist design as defined by Section D7 of *AISI S230* is adequate.

### D3 Trimmer Joist Design

Check the adequacy of an 800S162-68 trimmer joist for an 8-foot wide by 6-foot long opening. The joists are spaced at 24" on center. The floor live load is 40 psf and the dead load is 10 psf.



The maximum joist span from Table D3-1 of *AISI S230* is 14'-2".

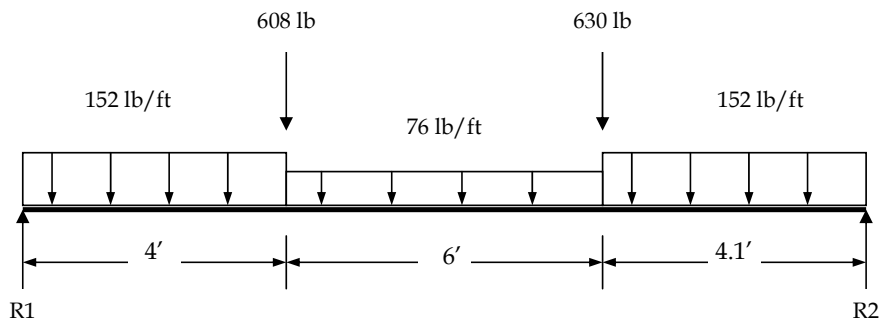
The design flexural strength for the trimmer joist is calculated in accordance with Section C3.1.1 of the *AISI Specification* (AISI, 1996):

$$\begin{aligned}\phi M_1 &= 3,501 \text{ ft-lb} && (\text{for } 800\text{T}125-68) \\ \phi M_2 &= 5,332 \text{ ft-lb} && (\text{for } 800\text{S}162-68)\end{aligned}$$

Because the opening width is 8 feet, Section D-7 and Figure D7-2 of *AISI S230* require the trimmer joist to be fabricated from two C-section joists and a track. Thus, the design strength for the trimmer joist is the sum of the strengths of the two C-sections and the track.

$$\phi M_T = 2(5,332) + 3,501 = 14,165 \text{ ft-lb}$$

The loading on the trimmer joist is as shown below. All loads are factored loads.



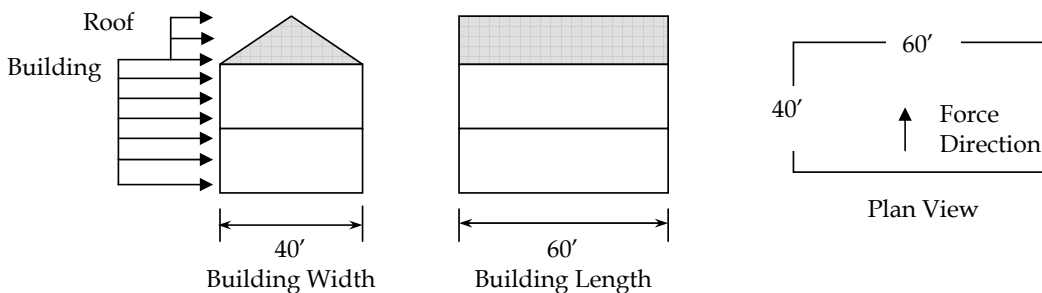
Based on the above loading, the maximum moment is 5,080 ft-lb

$$M < \phi M_T \Rightarrow 5,080 \text{ ft-lb} < 14,165 \text{ ft-lb} \quad \underline{ok}$$

This verifies the trimmer joist design as defined by Section D7 of *AISI S230* is adequate.

#### D4 Floor Diaphragm Design

Check the adequacy of a 19/32" OSB unblocked floor diaphragm for a 40x60 ft, two-story building subjected to 110 mph wind speed. The building has a roof slope of 12:12, mean roof height of 30 ft, and 8' wall studs at each floor. The building is located in Exposure Category C. The OSB floor sheathing is fastened to the floor joists with No. 8 screws spaced at 6" o.c. at panel edges and at 12" o.c. at intermediate supports.



From Table C2.1 of the *AISI S230*, the following wind pressures were obtained for the given wind speed, exposure and roof slope:

|                               |            |
|-------------------------------|------------|
| Roof pressure                 | = 11.7 psf |
| Roof corner pressure          | = 14.6 psf |
| Main building pressure        | = 24.1 psf |
| Main building corner pressure | = 30.3 psf |

The corner area width is  $2a$ , where " $a$ " is 10% of the least horizontal dimension (i.e.,

building width or building length) or 0.4 times the mean roof height,  $h$  which ever is smaller, but not less than either 4% of least horizontal dimension or 3 feet.

$$a = 0.10(40) = 4'$$

$$a = 0.4(30) = 12'$$

$$\text{Therefore } a = 4' > 0.04(40) = 1.6' \text{ or } 3'$$

$$\text{Use } a = 4'$$

$$\text{Corner area} = 2a = 8'$$

$$\text{Shear} = (60' - 16')(8')(24.1 \text{ psf}) + 16'(8')(30.3 \text{ psf}) = 12,361 \text{ lbs}$$

$$\text{Floor level diaphragm load} = (12,361/2)/40 = 155 \text{ plf}$$

$$\text{Factored diaphragm shear load} = 155 \times 1.6 = 248 \text{ plf}$$

The *North American Standard for Cold-Formed Steel Framing – Lateral Standard* (AISI S213-07) does not stipulate the design shear strength for an unblocked 19/32" diaphragm. However S213-07 does stipulate a value of 442 plf for 7/16" OSB with No. 8 screws spaced at 6".

The diaphragm in this example is therefore adequate ( $442 \text{ plf} > 248 \text{ plf}$ ).

## E. WALL FRAMING DESIGN EXAMPLES

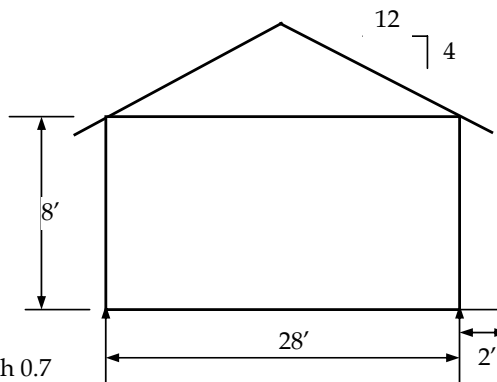
### E1 Wall Stud Design

Calculate the minimum size and thickness for an 8-foot wall stud located at the upper story of a 28-foot wide two-story house that is subjected to a 90 mph, Exposure Category B wind. The studs are spaced at 24" on center. The maximum ground snow load is 30 psf. The wall studs are laterally restrained at mid-height.

All referenced equations and sections are to the AISI *Specification* (AISI, 2001) unless noted.

#### E1.1 Design Assumptions

|        |                                       |
|--------|---------------------------------------|
| 28'    | Building Width                        |
| 2'     | Roof Overhang                         |
| 2'     | Stud Spacing                          |
| 33 ksi | Yield Stress                          |
| 4:12   | Roof Pitch                            |
| 30 psf | Ground Snow Load                      |
| 16 psf | Minimum Roof Live Load                |
| 7 psf  | Roof Dead Load                        |
| 5 psf  | Ceiling Dead Load                     |
| 45'    | Building Length                       |
| 8'     | Wall Height                           |
| L/240  | C&C ASCE 7 wind loads with 0.7 factor |



#### E1.2 Design Loads

##### Dead Load:

$$\begin{aligned}\text{Ceiling Dead Load} &= 5(28)/2 &= 70 \text{ plf} \\ \text{Roof Dead Load} &= 7(32)/2 &= 112 \text{ plf} \\ \text{Total Dead Load (D)} & &= 182 \text{ plf}\end{aligned}$$

##### Live Loads:

$$\begin{aligned}\text{Roof Live Load (L}_r\text{)} &= 16(2 + 28/2) &= 256 \text{ plf} \\ \text{Balanced Roof Snow Load (S)} &= (0.7 \times 30)(2 + 28/2) &= 336 \text{ plf} \\ \text{Unbalanced Roof Snow Load (S)} &= (30)(2 + 28/2)(0.75 \times 28 + 2/2)/28 = 377 \text{ plf} \quad \leftarrow \text{controls}\end{aligned}$$

##### Wind Loads:

Wind loads were calculated in accordance with ASCE 7 (ASCE, 2005) equations using MWFRS and components and cladding.

##### MWFRS:

MWFRS wind pressure for this design example is 10 psf.

##### Components and Cladding:

The C&C wind pressure for this design example is 15.1 psf.

### E1.3 Load Combinations

1. 1.4D
2. 1.2D + 1.6L + 0.5(L<sub>r</sub> or S)
3. 1.2D + 0.5L + 1.6(L<sub>r</sub> or S)
4. 1.2D + 0.8W + 1.6(L<sub>r</sub> or S) (using MWFRS for W)
5. 1.2D + 1.6W + 0.5L + 0.5(L<sub>r</sub> or S) (using MWFRS for W)
6. 1.6W (C&C wind coefficients used to check bending)
7. 0.7W (C&C wind coefficients used to check deflections)

The load combinations listed below will be checked:

- |    |   |                                 |
|----|---|---------------------------------|
| 1. | 1.4D = 255 plf,                                   | No lateral load                 |
| 2. | 1.2D + 1.6L + 0.5(L <sub>r</sub> or S) = 407 plf, | No lateral load                 |
| 3. | 1.2D + 0.5L + 1.6(L <sub>r</sub> or S) = 822 plf, | No lateral load                 |
| 4. | 1.2D + 1.6(L <sub>r</sub> or S) = 822 plf,        | 0.8W = 0.8(10 x 2) = 16 plf     |
| 5. | 1.2D + 0.5(L <sub>r</sub> or S) = 407 plf,        | 1.6W = 1.6(10 x 2) = 32 plf     |
| 6. |   | 1.6W = 1.6(15.1 x 2) = 48.3 plf |

Therefore, the controlling load combinations to be checked are:

- |    |                               |     |                       |
|----|-------------------------------|-----|-----------------------|
| 3. | 822 x 2 = 1,643 lb axial load | and | 0 lateral load        |
| 4. | 822 x 2 = 1,643 lb axial load | and | 16 plf lateral load   |
| 5. | 407 x 2 = 814 lb axial load   | and | 32 plf lateral load   |
| 6. | 0 axial load                  | and | 48.3 plf lateral load |

### E1.4 Member Properties

Try a 350S162-33 member and check its adequacy. The calculated design flexure strength and section properties for this member in accordance with the *AISI Specification* are:

|                                       |                          |
|---------------------------------------|--------------------------|
| $\phi M_n$                            | = 664.5 ft-lb            |
| Effective section modulus, $S_{xx}$   | = 0.2543 in <sup>3</sup> |
| Effective moment of inertia, $I_{xx}$ | = 0.5051 in <sup>4</sup> |
| Radius of gyration, $r_x$             | = 1.403"                 |
| Radius of gyration, $r_y$             | = 0.616"                 |

### E1.5 Combined Axial and Bending Capacity

Check combined axial and bending in accordance with the *AISI Specification*, Section C5.2, as applicable for wall stud design.

$$\frac{P_u}{\phi_c P_n} + \frac{C_{mx} M_{ux}}{\phi_b M_{nx} \alpha_x} \leq 1.0 \quad (\text{Eq. C5.2.2-1})$$

$$\frac{P_u}{\phi_c P_{no}} + \frac{M_{ux}}{\phi_b M_{nx}} \leq 1.0 \quad (\text{Eq. C5.2.2-2})$$

$$\text{When } \frac{P_u}{\phi_c P_n} \leq 0.15 \rightarrow \frac{P_u}{\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{nx}} \leq 1.0 \quad (\text{Eq. C5.2.2-3})$$

$C_{mx} = 1$  conservatively taken as unity in accordance with Section C5.2 of the *AISI Specification*

$$\alpha_x = [1 - P_u/P_{Ex}] \quad (\text{Eq. C5.2.2-4})$$

$P_u$  = Required axial strength (LRFD)

$M_{ux}$  = Required bending strength (LRFD)

$M_{nx} = (664.5/0.95) = 699$  ft-lb (Previously calculated)

$P_n$  = Nominal axial strength determined in accordance with Section C4.

$P_{no}$  = Nominal axial strength determined in accordance with Section C4, with  $F_n = F_y$

$\phi_c$  = Factor of safety = 0.85

$\phi_b$  = Factor of safety = 0.95

$$P_{Ex} = \pi^2 EI_x / (K_x L_x)^2 \quad (\text{Eq. C5.2.2-6})$$

$I_x$  = Effective moment of inertia about the x-axis at unfactored total load = 0.5051 in<sup>4</sup>

$L_x$  = Actual unbraced length for bending about the x-axis = 96"

$L_y$  = Actual unbraced length for bending about the y-axis = 48"

$K_x$  = Effective length factor for buckling about the x-axis = 1.0

$K_y$  = Effective length factor for buckling about the y-axis = 1.0

The nominal axial strength,  $P_n$  and  $P_{no}$  for the stud as calculated in accordance with Section C4 of the AISI *Specification* (AISI, 2001):

$P_n = 3,267$  lbs

$P_{no} = 5,631$  lbs

Load Combination 3: 1,643 lb axial load and 0 plf lateral load

$$\frac{1,643}{0.85(3,267)} = 0.59 < 1.0 \quad \underline{ok} \quad (\text{Eq. C5.2.2-1})$$

$$\frac{1,643}{0.85(5,631)} = 0.34 < 1.0 \quad \underline{ok} \quad (\text{Eq. C5.2.2-2})$$

Load Combination 4: 1,643 lb axial load and 16 plf lateral load

$$M_x = wL^2 / 8 = (16)(8)^2 / 8 = 128 \text{ ft-lb}$$

$$\frac{1,643}{0.85(3,267)} + \frac{1(128)}{0.95(699)(0.9052)} = 0.80 < 1.0 \quad \underline{ok} \quad (\text{Eq. C5.2.2-1})$$

$$\frac{1,643}{0.85(5,631)} + \frac{1(128)}{0.95(699)} = 0.54 < 1.0 \quad \underline{ok} \quad (\text{Eq. C5.2.2-2})$$

Load Combination 5: 822 lb axial load and 32 plf lateral load

$$M_x = wL^2 / 8 = (32)(8)^2 / 8 = 256 \text{ ft-lb}$$

$$\frac{822}{0.85(3,267)} + \frac{1(256)}{0.95(699)(0.9516)} = 0.70 < 1.0 \quad \underline{ok} \quad (\text{Eq. C5.2.2-1})$$



$$\frac{822}{0.85(5,631)} + \frac{1(256)}{0.95(699)} = 0.56 < 1.0 \quad \text{ok} \quad (\text{Eq. C5.2.2-2})$$

**Load Combination 6:** 0 axial load and 48.3 plf lateral load

$$M_x = wL^2 / 8 = (48.3)(8)^2 / 8 = 386.4 \text{ ft-lb}$$

$$\frac{386.4}{0.95(699)} = 0.58 < 1.0 \quad (\text{since } P_u / \phi P_n < 0.15) \quad \text{ok} \quad (\text{Eq. C5.2.2-3})$$

### E1.6 Deflection Limit

Check the stud deflection using the components and cladding wind pressure with a 0.7 factor in accordance with the *North American Standard for Cold-Formed Steel Framing - Wall Stud Design* (AISI S211-07):

$$\delta = \frac{5wL^4}{384EI} \quad \text{where } w = 15.1 \text{ psf} \times 2' \times 0.7 = 21.14 \text{ plf}$$

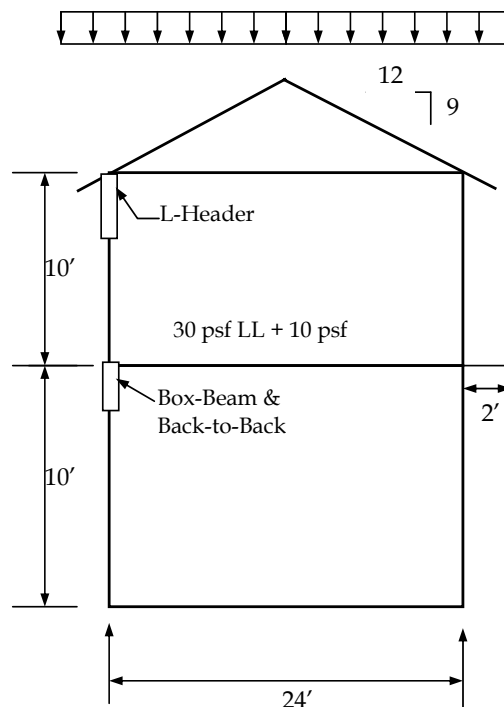
$$\delta = \frac{5(21.14/12)(96)^4}{(384)(29,500,000)(0.5051)} = 0.131" < L/240 = 0.40" \quad \text{ok}$$

Therefore, 350S162-33 studs spaced at 24" on center are adequate. This verifies the wall stud selection from Table E3.2a of AISI S230.

### E2 Box Header Design

Calculate the maximum allowable span for a 2-800S162-43 box header supporting an opening located on the first floor of the two-story, 24-foot wide building described below. Headers are designed in accordance with the *North American Standard for Cold-Formed Steel Framing Header Design* (AISI S212-07) and the *AISI Specification* (AISI, 2001). All referenced equations and sections are to the *AISI Specification* unless noted.

|        |                                   |
|--------|-----------------------------------|
| 24'    | Building Width                    |
| 2'     | Eave Overhang                     |
| 24"    | Joist/Truss/Rafter Spacing        |
| 10'    | Wall Height                       |
| 1.625" | Joist/Truss/Rafter Bearing Length |
| 33 ksi | Yield Stress                      |
| 50 psf | Ground Snow Load                  |
| 16 psf | Minimum Roof Live Load            |
| 7 psf  | Roof Dead Load                    |
| 5 psf  | Ceiling Dead Load                 |
| 30 psf | Top Floor Live Load               |



|        |                             |
|--------|-----------------------------|
| 10 psf | Top Floor Dead Load         |
| L/240  | Total Load Deflection Limit |
| L/360  | Live Load Deflection Limit  |
| 10 psf | Wall Dead Load              |

### E2.1 Design Loads

#### Dead Loads:

|                                  |           |
|----------------------------------|-----------|
| Ceiling Dead Load = $5(24/2)$    | = 60 plf  |
| Roof Dead Load = $7(28/2)$       | = 98 plf  |
| Wall Dead Load = $10(10)$        | = 100 plf |
| Top Floor Dead Load = $10(24/2)$ | = 120 plf |
| Total Dead Load                  | = 378 plf |

#### Live Loads:

|  |                      |
|--|----------------------|
| Roof Live Load = $16(28/2)$                                      | = 224 plf            |
| Balanced Snow Load = $0.7(50)(28/2)$                             | = 490 plf            |
| Unbalanced Snow Load = $(50)(2 + 24/2)(0.75 \times 24 + 2/2)/24$ | = 554 plf ← controls |
| Top Floor Live Load = $30(24/2)$                                 | = 360 plf            |

### E2.2 Load Combinations

1.  $1.4D = 1.4(378) = 529$  plf
2.  $1.2D + 1.6L + 0.5(L_r \text{ or } S) = 1.2(378) + 1.6(360) + 0.5(554) = 1,306$  plf
3.  $1.2D + 0.5L + 1.6(L_r \text{ or } S) = 1.2(378) + 0.5(360) + 1.6(554) = 1,520$  plf ← controls

### E2.3 Member Properties

|                              |                                  |
|------------------------------|----------------------------------|
| $d = 8.0''$                  | Depth of section                 |
| $t = 0.0451''$               | Design thickness                 |
| $R = 0.09375''$              | Inside bend radius               |
| $I_{xx} = 9.00 \text{ in}^4$ | Moment of inertia for deflection |

### E2.4 Bending Capacity

|                                    |   |
|------------------------------------|---|
| $M_n = 5,606 \text{ ft-lb}$        | Nominal flexural strength (calculated per <i>Specification</i> )    |
| $\phi_b = 0.95$                    | Resistance factor for flexural strength (per <i>Specification</i> ) |
| $\phi_b M_n = 5,326 \text{ ft-lb}$ | (Design strength for two sections with punched webs)                |

$$M = \frac{wL^2}{8} \rightarrow L = \sqrt{\frac{8M}{w}} = \sqrt{\frac{8(5,326)}{1,520}}$$

$$L = 5.29 \text{ ft} = 5'-3''$$

### E2.5 Deflection Limit

$$\Delta = L/240 \text{ (total loads)}$$

The deflection equation for a simply supported span with distributed load is:

$$\Delta = \frac{5wL^4}{384EI} = L/240$$

$$L = \sqrt[4]{\frac{384EI_x(L/240)}{5w}} = \sqrt[3]{\frac{384(29,500,000)(9.00)}{5(1,292/12)(240)}} = 7'-8''$$

where:

- L = Single span length (inches)
- $I_x$  = Effective moment of inertia for deflection = 9.00 in<sup>4</sup>
- w = 1292 plf
- E = Modulus of elasticity = 29,500,000 psi
- Deflection Limit = L/240 for total loads and L/360 for live loads, inches

$$\Delta = L/360 \text{ (live loads)}$$

$$\Delta = \frac{5wL^4}{384EI} = L/360 \rightarrow L = \sqrt[3]{\frac{384(29,500,000)(9.00)}{5(360/12)360}} = 10'-3"$$

where: w = 360 plf

## E2.6 Shear Capacity

In accordance with Section C3.2 of the AISI S100 the nominal shear strength,  $V_n$ , is calculated to be 5144 lb for the box header.

Use the design shear strength,  $\phi V_n$ , to calculate the maximum unsupported span length.

$$\phi V_n = \frac{wL}{2} \rightarrow L = \frac{2\phi V_n}{w}$$

where: w = required strength (factored uniform load) = 1,520 plf

$$L = \frac{2(0.95)(5,144)}{1,520} \rightarrow L = 6.43' = 6'-5"$$

## E2.7 Combined Bending and Web Crippling Capacity

In accordance with Section C3.4.1 of the AISI *Specification*, the nominal web crippling strength,  $P_n$ , is calculated to be 1983 lb for the box header assuming flanges are unfastened to the support and no reduction is required for the presence of web holes.

$P_n$  should be adjusted for presence of web holes in each of the C-section members in accordance with Section C3.4.2 of the AISI *Specification* (AISI, 2001).

Check applicability of AISI *Specification* Section C3.4.2:

$$d_0/h = 1.5/7.722 = 0.19 < 0.7$$

$$h/t = 7.722/0.0451 = 171.2 < 200$$

Holes centered at mid-depth of the web

Clear distance between holes  $\geq 18"$

Circular hole diameter  $\leq 6"$

Non-circular holes, hole depth ( $d_0$ ) =  $1.5" \leq 2.5"$  and  $b = 4" \leq 4.5"$

$d_0 > 9/16"$

$$R_c = 0.90 - 0.047d_0/h + 0.053x/h \leq 1.0 \quad (\text{Eq. C3.4.2-2})$$

$$R_c = 0.90 - 0.047(1.5)/7.722 + 0.053(10)/7.722 = 0.96$$

$P_n$  may also be adjusted using the parameter  $\alpha$  to account for the increased strength due to the track in accordance with Section B2.3 in the *North American Standard for Cold-Formed Steel Framing – Header Design Standard* (AISI S212-07).

$$\alpha = 2.3(t_t/t_c)$$

where:  $t_t = 0.0346$  in  
 $t_c =$  Design thickness of the C-shape section

$$\alpha = 2.3\left(\frac{0.0346}{0.0451}\right) = 1.765$$

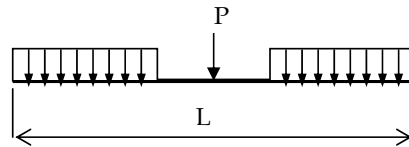
The nominal web crippling strength of the box-beam header is therefore:

$$P_n = 1,983(0.96)(1.765) = 3,360 \text{ lb} \quad (\text{for 2 webs with holes})$$

For combined bending and web crippling using the *North American Standard for Cold-Formed Steel Framing – Header Design Standard* (AISI S212-07):

$$\left(\frac{P_u}{P_n}\right) + \left(\frac{M_u}{M_{nxo}}\right) \leq 1.5\phi \quad (\text{Eq. B2.5-2})$$

The following load model is used in checking the combined bending and web crippling equation. The concentrated load is located at mid-span of the beam. The distributed load is located one foot away from the concentrated load on each side for a 2-foot joist and rafter spacing.



$$\text{Maximum moment, } M = \frac{PL}{4} + w\left(\frac{L-2}{2}\right)\left(\frac{L}{2}\right) - w\left(\frac{L-2}{2}\right)\left[\left(\frac{L}{2}\right) - \left(\frac{L-2}{4}\right)\right]$$

Substitute for all the variables in Eq. B2.5-2 and solve for L.

where:  $M_{nxo} = 5,505 \text{ ft-lb}$   
 $\phi = 0.85$   
 $w = 1,520 \text{ plf (Section E2.2)}$   
 $P = 2(w) = 3,040 \text{ lbs}$

$$\left(\frac{3,040}{3,360}\right) + \left(\frac{M_u}{5,505}\right) \leq 1.5(0.85) \rightarrow M_u = 2,038 \text{ ft-lb}$$

$$\text{Set } M_u = M = \frac{PL}{4} + w\left(\frac{L-2}{2}\right)\left(\frac{L}{2}\right) - w\left(\frac{L-2}{2}\right)\left[\left(\frac{L}{2}\right) - \left(\frac{L-2}{4}\right)\right]$$

Solving the above equation for L gives:

$$L = 2.594 \text{ ft} = 2'-7''$$

Therefore, the maximum span for a 2-800S162-43 header for the conditions in this example is 2'-7". This confirms the maximum span given in Table E7.4a of *AISI S230*.

### E3 Back-to-Back Header Design

Calculate the maximum allowable spans for a 2-800S162-43 back-to-back header supporting an opening located on the first floor of the two-story building described for the example in Section E2. All referenced equations and sections are to the *Specification* unless noted.

#### E3.1 Design Loads

Dead Loads:

|                                |           |
|--------------------------------|-----------|
| Ceiling Dead Load = 5(24/2)    | = 60 plf  |
| Roof Dead Load = 7(28/2)       | = 98 plf  |
| Wall Dead Load = 10(10)        | = 100 plf |
| Top Floor Dead Load = 10(24/2) | = 120 plf |
| Total Dead Load                | = 378 plf |

Live Loads:

|   |                      |
|---|----------------------|
| Roof Live Load = 16(28/2)                             | = 224 plf            |
| Balanced Snow Load = 0.7(50)(28/2)                    | = 490 plf            |
| Unbalanced Snow Load = (50)(2 + 24/2)(0.75x24+2/2)/24 | = 554 plf ← controls |
| Top Floor Live Load = 30(24/2)                        | = 360 plf            |

#### E3.2 Load Combinations

1.  $1.4D = 1.4(358) = 501 \text{ plf}$
2.  $1.2D + 1.6L + 0.5(L_r \text{ or } S) = 1.2(358) + 1.6(360) + 0.5(490) = 1,251 \text{ plf}$
3.  $1.2D + 0.5L + 1.6(L_r \text{ or } S) = 1.2(358) + 0.5(360) + 1.6(490) = 1,394 \text{ plf}$  ← controls

#### E3.3 Member Properties

Same as previous example in E2.3

#### E3.4 Bending Capacity

Same as previous example in E2.4

#### E3.5 Deflection Limit

Same as previous example in E2.5

#### E3.6 Shear Capacity

Same as previous example in E2.6

### E3.7 Combined Bending and Web Crippling Capacity

In accordance with Section C3.4.1 and Table C3.4.1-1 of the *AISI Specification*, the nominal web crippling strength,  $P_n$ , is calculated to be 3,546 lb for the back-to-back header assuming flanges are unfastened to the support and no reduction is required for the presence of web holes. According to the *North American Standard for Cold-Formed Steel Framing – Header Design Standard* (AISI S212-07), back-to-back C-section headers can be treated as built-up sections for web crippling.

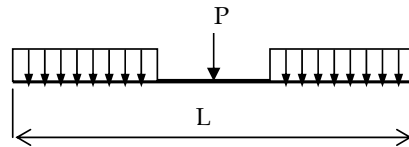
$P_n$  should be adjusted for presence of web holes in each of the C-section members in accordance with Section C3.4.2 of the *Specification* (AISI, 2001).

$R_c = 0.96$  (Same as previous example in E2).

$P_n = 3,546 \times 0.96 = 3,404 \text{ lb}$  (for two webs with holes)

$$0.82 \left( \frac{P_u}{\phi_w P_n} \right) + \left( \frac{M_u}{\phi_b M_{nxo}} \right) \leq 1.32 \quad (\text{Eq. C3.5.2-2})$$

The following load model is used in checking the combined bending and web crippling equation. The concentrated load is located at mid-span of the beam. The distributed load is located one foot away from the concentrated load on each side for a 2-foot joist and rafter spacing.



$$\text{Maximum moment, } M = \frac{PL}{4} + w \left( \frac{L-2}{2} \right) \left( \frac{L}{2} \right) - w \left( \frac{L-2}{2} \right) \left[ \left( \frac{L}{2} \right) - \left( \frac{L-2}{4} \right) \right]$$

Substitute for all the variables in Eq. C3.5.2-2 and solve for L.

where  $M_{nxo} = 5,505 \text{ ft-lb}$  (Nominal flexural strength)  
 $\phi_w = 0.85$  for back-to-back headers  
 $\phi_b = 0.95$   
 $w = 1,520 \text{ plf}$  (Section E2.2)  
 $P = 2(w) = 3,040 \text{ lbs}$

$$0.82 \left( \frac{3,040}{0.85(3,404)} \right) + \left( \frac{M_u}{0.95(5,505)} \right) \leq 1.32 \rightarrow M_u = 2,398 \text{ ft-lb}$$

$$\text{Set } M_u = M = \frac{PL}{4} + w \left( \frac{L-2}{2} \right) \left( \frac{L}{2} \right) - w \left( \frac{L-2}{2} \right) \left[ \left( \frac{L}{2} \right) - \left( \frac{L-2}{4} \right) \right]$$

Solve the above equation for L gives:

$$L = 2.936 \text{ ft} = 2'-11''$$

Therefore, the maximum span for a 2-800S162-43 back-to-back header for the conditions in this example is 2'-11". This confirms the maximum span given in Table E7-10a of *AISI S230*.

#### E4 Double L-Header Design (Gravity Loading)

Calculate the maximum allowable span for a 2-800L150-54 double L-header supporting an opening located at the roof level of the two-story building described in Section E2.

##### E4.1 Design Loads

###### Dead Loads:

$$\begin{aligned} \text{Ceiling Dead Load} &= 5(24/2) &&= 60 \text{ plf} \\ \text{Roof Dead Load} &= 7(28/2) &&= 98 \text{ plf} \\ \text{Total Dead Load} &&&= 158 \text{ plf} \end{aligned}$$

###### Live Loads:

$$\begin{aligned} \text{Roof Live Load} &= 16(28/2) &&= 224 \text{ plf} \\ \text{Balanced Snow Load} &= 0.7(50)(28/2) &&= 490 \text{ plf} \\ \text{Unbalanced Snow Load} &= (50)(2 + 24/2)(0.75 \times 24 + 2/2)/24 = 554 \text{ plf} \quad \leftarrow \text{controls} \end{aligned}$$

##### E4.2 Load Combinations

$$\begin{aligned} 1. \quad 1.4D &= 1.4(158) = 221 \text{ plf} \\ 2. \quad 1.2D + 0.5L + 1.6(L_r \text{ or } S) &= 1.2(158) + 1.6(554) = 1,076 \text{ plf} \quad \leftarrow \text{controls} \end{aligned}$$

##### E4.3 Member Properties

$$\begin{aligned} L_1 &= 8.0'' && \text{Long leg of angle} \\ L_2 &= 1.5'' && \text{Short leg of angle} \\ t &= 0.0566'' && \text{Design thickness} \end{aligned}$$

##### E4.4 Bending Capacity

$$M_{ng} = 4,950 \text{ ft-lb} \quad \text{Nominal flexural strength (calculated in accordance with AISI S212 Section B3.1.1) calculated for two angles.}$$

The design flexural strength is determined from Section B3.1.3 of AISI S212 as follows:

$$\begin{aligned} \phi M_{ng} &= (0.90) 4,950 = 4,455 \text{ ft-lb} \\ \phi M_{ng} &= \frac{wL^2}{8} \quad \rightarrow \quad L = \sqrt{\frac{8\phi M_{ng}}{w}} \\ L &= \sqrt{\frac{8(4,455)}{1,076}} = 5.755 \text{ ft} = 5'-9'' \end{aligned}$$

Therefore, the maximum span for a 2-800L150-54 double L-header for the conditions in this example is 5'-9". This confirms the maximum span given in Table E7-15a of *AISI S230*.

#### E5 Double L-Header Design (Uplift Loading Case 1)

Calculate the maximum span for uplift for an L800S150-54 ( $F_y = 33 \text{ ksi}$ ) double L-header supporting an opening located at the roof level of the two-story building described in Section

E2. The fastest-mile wind speed is 100 mph, Exposure Category C. Use Section B3 of AISI S212 *North American Standard for Cold-Formed Steel Framing – Header Design*.

### E5.1 Design Loads

#### Dead Loads:

$$\begin{aligned}\text{Ceiling Dead Load} &= 5(24/2) &= 60 \text{ plf} \\ \text{Roof Dead Load} &= 7(28/2) &= \underline{98 \text{ plf}} \\ \text{Total Dead Load} &= &= 158 \text{ plf}\end{aligned}$$

#### Wind Uplift Load:

Wind pressures are taken from Figure 6-2 using Method 1 from ASCE 7 (ASCE, 2005) for roof corner MWFRS wind pressures for a 20 degree roof slope, 100 mph Exposure Category C (same as 120 mph, Exposure B). The calculated pressures are perpendicular to the vertical projection of the roof and a 20 degree slope was used as it produces the greatest uplift force for the range of slopes allowed in AISI S230. The uplift pressure on the header is calculated as follows:

$$\begin{aligned}\text{Wind pressure on overhang} &= -30.1 \text{ psf} \\ \text{Wind pressure on windward slope} &= -19.1 \text{ psf} \\ \text{Wind pressure on leeward slope} &= -14.5 \text{ psf}\end{aligned}$$

Summing moments about the leeward bearing wall gives the following calculation for the reaction at the windward bearing wall (uniform uplift load on the header):

$$W = \frac{(-30.1)(2)(24+1) + (-19.1)(24/2)(3 \times 24/4) + (-14.5)(24/2)(24/4)}{24} = -278 \text{ plf}$$

### E5.2 Load Combinations:

$$1. \quad 0.9D + 1.6W = 0.9(158) + 1.6(-278) = -303 \text{ plf}$$

### E5.3 Member Properties

$$\begin{aligned}L_1 &= 8.0'' && \text{Long leg of angle} \\ L_2 &= 1.5'' && \text{Short leg of angle} \\ t &= 0.0566'' && \text{Design thickness}\end{aligned}$$

### E5.4 Bending Capacity

$$M_{nu} = R M_{ng} \quad (\text{Eq. B3.1.2-1})$$

where

$$\begin{aligned}M_{ng} &= 4950 \text{ ft-lbs} && (\text{from example E4}). \\ R &= \text{uplift reduction factor} \\ &= 0.25 \text{ for } L_h/t \leq 150 \\ &= 0.20 \text{ for } L_h/t \geq 170 \\ &= \text{use linear interpolation for } 150 < L_h/t < 170 \\ L_h &= \text{vertical leg dimension of the angle} \\ t &= \text{design thickness}\end{aligned}$$



For LRFD, calculate the design flexural strength for uplift:

$$L_h/t = 8.0/0.0566 = 141.3 \leq 150, \text{ Therefore, } R = 0.25$$

$$M_{nu} = 0.25(4,950) = 1,238 \text{ ft-lb}$$

$$M_u \leq \phi M_{nu} \quad (\text{Eq. B3.1.3-4})$$

Calculate the maximum span for uplift:

$$\phi = 0.80$$

$$\phi M_{nu} = \frac{wL^2}{8} \rightarrow L = \sqrt{\frac{8\phi M_{nu}}{w}}$$

$$L = \sqrt{\frac{8(0.8)(1,238)}{303}} = 5.114' = 5'-2''$$

Therefore, the maximum span for a 2-800L150-54 double L-header for the conditions in this example is 5'-2". This confirms the maximum span given in Table E7-20a of *AISI S230*.

## E6 Double L-Header Design (Uplift Loading Case 2)

Calculate the maximum span for a 2-L800S150-54 L-header located in the first story of a the two-story building described in Section F1, subjected to 110 mph Exposure Category C wind speed.

### E6.1 Design Loads

#### Dead Loads:

|                               |                   |
|-------------------------------|-------------------|
| Ceiling Dead Load = $5(24/2)$ | = 60 plf          |
| Roof Dead Load = $7(28)/2$    | = 98 plf          |
| Floor Dead Load = $10(24/2)$  | = 120 plf         |
| Wall Dead Load = $10(10)$     | = <u>100* plf</u> |
| Total Dead Load               | = 378 plf         |

\* Note: Use 10' high wall which is the maximum allowed by *AISI S230*

#### Wind Uplift Load:

Wind pressures are taken from Figure 6-2 using Method 1 from ASCE 7 (ASCE, 2005) for roof corner MWFRS wind pressures for a 20 degree roof slope, 110 mph Exposure Category C (same as 130 mph, Exposure B). The calculated pressures are perpendicular to the vertical projection of the roof and a 20 degree slope was used as it produces the greatest uplift force for the range of slopes allowed in *AISI S230*. The uplift pressure on the header is calculated as follows:

|                                 |             |
|---------------------------------|-------------|
| Wind pressure on overhang:      | = -35.4 psf |
| Wind pressure on windward slope | = -22.4 psf |
| Wind pressure on leeward slope  | = -17.1 psf |

Summing moments about the leeward bearing wall gives the following calculation for the reaction at the windward bearing wall (uniform uplift load on the header):

$$W = \frac{(-34.4)(2)(24+1) + (-22.4)(24/2)(3 \times 24/4) + (-17.1)(24/2)(24/4)}{24} = -325 \text{ plf}$$

### E6.2 Load Combinations

$$1. \quad 0.9D + 1.6W = 0.9(378) + 1.6(-325) = -180 \text{ plf}$$

### E6.3 Member Properties

|                |                    |
|----------------|--------------------|
| $L_1 = 8.0''$  | Long leg of angle  |
| $L_2 = 1.5''$  | Short leg of angle |
| $t = 0.0566''$ | Design thickness   |

### E6.4 Bending Capacity

$$M_{nu} = R M_{ng} \quad (\text{Eq. B3.1.2-1})$$

where:

- $M_{ng} = 4950 \text{ ft-lbs}$  (from example E4)
- $R$  = uplift reduction factor
  - = 0.25 for  $L_h/t \leq 150$
  - = 0.20 for  $L_h/t \geq 170$
  - = use linear interpolation for  $150 < L_h/t < 170$
- $L_h$  = vertical leg dimension of the angle
- $t$  = design thickness

For LRFD, calculate the design moment capacity for uplift:

$$L_h/t = 8.0/0.0566 = 141.34 \leq 150, \text{ Therefore, } R = 0.25$$

$$M_{nu} = 0.25(4,950) = 1,238 \text{ ft-lb}$$

Calculate the maximum span for uplift:

$$\phi = 0.80$$

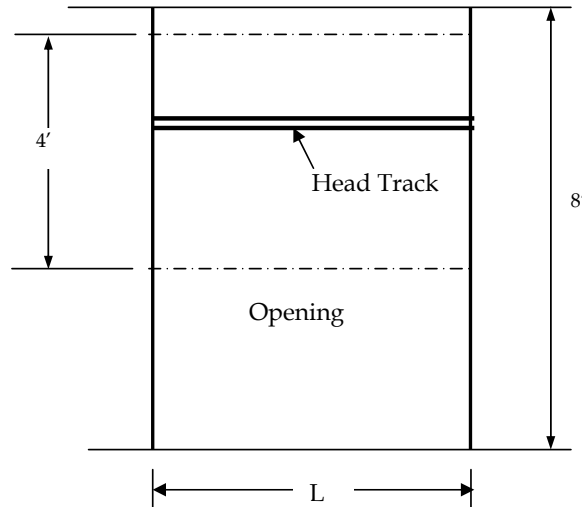
$$\phi M_{nu} = \frac{wL^2}{8} \rightarrow L = \sqrt{\frac{8\phi M_{nu}}{w}}$$

$$L = \sqrt{\frac{8(0.8)(1,238)}{180}} = 6.63' = 6'-8''$$

Therefore, the maximum span for a 2-800L150-54 double L-header for the conditions in this example is 6'-8". This confirms the maximum span given in Table E7-25a of AISI S230.

## E7 Head Track Design

Calculate the maximum allowable span for a 350T125-33 head track for an opening in a building subjected to 120 mph Exposure Category C wind speed.



### E7.1 Design Loads and Assumptions

Wind Load = 37.7 psf (Table A2.2, 120 mph Exposure Category C wind speed)

Deflection Limit =  $L/240$

End Bearing Length = 6" (so bearing does not control)

8' high walls

### E9 Shear Wall Design (One Story Building)

The segmented shear wall line has the following dimensions:

Wall construction:

Building Width = 30'

Building Length = 45'

Wall Height = 8'

Roof Slope = 6:12

Exterior sheathing is 7/16-inch-thick OSB with No. 8 screws spaced 6 inches on center on panel edges and 12 inches on center in panel field

Interior sheathing is 1/2-inch-thick gypsum wall board with No. 6 screws at 12 inches on center

Framing studs are 350S162-33 spaced at 24 inches on center

Calculate the sheathing requirements for the side and end walls. The building is subjected to a wind speed of 100 mph, Exposure Category C.

### E9.1 Design Loads

Calculated in accordance with ASCE 7:

MWRFS building corner wind pressure = 27 psf

|                              |            |
|------------------------------|------------|
| MWRFS building wind pressure | = 20.1 psf |
| Roof corner wind pressure    | = 3.8 psf  |
| Roof wind pressure           | = 3.3 psf  |
| Building gable end           | = 14.7 psf |
| Building gable end corner    | = 22.2 psf |

Nominal shear value per foot of shear wall for 7/16 inch thick OSB at the stated fastening and a 2:1 aspect ratio is 910 plf (Table 2211.1(3) of IBC 2000 (ICC, 2000a)). The IBC provides shear values for GWB with 7 inch o.c. fastener spacing. It provides no shear values for 12 inch o.c. fastener spacing.

Where LRFD is used, the IBC 2000 requires the factored design shear value to be determined by multiplying the ultimate shear value by a resistance factor ( $\phi$ ) of 0.55.

$$\text{Area End} = 6/12(15')(15') + \frac{1}{2}(8')(30') = 233 \text{ ft}^2$$

$$\text{Area Side} = (8'/2)(45') = 180 \text{ ft}^2$$

$$\text{Building aspect ratio} = 45/30 = 1.50$$

$$\text{Actual shear at each side wall} = (20.1 \text{ psf})(1.6)(180 \text{ ft}^2)/2 = 2,894 \text{ lb}$$

$$\text{Actual shear at each end wall} = (20.1 \text{ psf})(1.6)(233 \text{ ft}^2)/2 = 3,747 \text{ lb}$$

$$a = 10\%(30) = 3'$$

$$a = 0.4(15.5) = 6.2' \quad (\text{building height} = 8' + 7.5' = 15.5')$$

$$\text{Use } a = 3'$$

$$2a = 6'$$

$$\text{End wall corner area} = \frac{1}{2}(8')(2 \times 3) = 24 \text{ ft}^2$$

$$\text{End wall area} = \frac{1}{2}(8')(30') - 24 = 968 \text{ ft}^2$$

$$\text{End wall roof corner area} = 1/2(7.5)(3) = 11.25 \text{ ft}^2$$

$$\text{End wall roof area} = (6/12)(15')(15') - 11.25 = 101.25 \text{ ft}^2$$

$$\text{Side wall corner area} = \frac{1}{2}(8')(6) = 24 \text{ ft}^2$$

$$\text{Side wall area} = \frac{1}{2}(8')(45') - 24 = 156 \text{ ft}^2$$

$$\text{Side wall roof corner area} = (7.5)(6) = 45 \text{ ft}^2$$

$$\text{Side wall roof area} = (7.5')(45') - 45 = 292.5 \text{ ft}^2$$

$$\text{Building aspect ratio} = 45/30 = 1.50$$

Design shear at each side wall

$$= [24 \text{ ft}^2 (22.2 \text{ psf}) + 96 \text{ ft}^2 (14.7 \text{ psf}) + 11.25 \text{ ft}^2 (22.2) + 101 \text{ ft}^2 (14.7)](1.6)/2 = 2,943 \text{ lb}$$

Design shear at each end wall

$$= [24 \text{ ft}^2 (27 \text{ psf}) + (156 \text{ ft}^2 (20.1 \text{ psf}) + 45 \text{ ft}^2 (3.8 \text{ psf}) + 292.5 \text{ psf ft}^2 (3.3)](1.6)/2 = 3,936 \text{ lb}$$

## E9.2 Required Sheathing

$$\text{Each end wall: } \frac{3,936}{910(0.55)} = 7.86 \text{ ft} \quad \text{This equates to 26\% length of sheathed wall}$$

required for an engineered design. Using Tables E8-1 and E8-4, 39% of sheathed wall length must be provided.

$$\text{Each side wall: } \frac{2,943}{910(0.55)} = 5.88 \text{ ft} \quad \text{This equates to 13\% length of sheathed wall}$$

required for an engineered design. Using Tables E8-1 and E8-4, 20% of sheathed wall length must be provided.

**E10 Shear Wall Design (Two Story Building)**

The segmented shear wall line has the following dimensions:

Wall construction:

Building width = 30'

Building length = 50'

Wall Height = 8'

Roof slope = 6:12

Exterior sheathing is 7/16-inch-thick OSB with No. 8 screws spaced 6 inches on center on panel edges and 12 inches on center in panel field.

Interior sheathing is 1/2-inch-thick gypsum wall board with No. 6 screws at 12 inches on center.

Framing studs are 350S162-33 spaced at 24 inches on center.

Calculate the sheathing requirements for the side and end walls. The building is subjected to a wind speed of 100 mph, Exposure Category C.

**E10.1 Design Loads**

Calculated in accordance with ASCE 7 (ASCE, 1998):

|                              |            |
|------------------------------|------------|
| MWRFS building corner        |            |
| wind pressure                | = 27 psf   |
| MWRFS building wind pressure | = 20.1 psf |
| Roof corner wind pressure    | = 3.8 psf  |
| Roof wind pressure           | = 3.3 psf  |
| Building gable end           | = 14.7 psf |
| Building gable end corner    | = 22.2 psf |

Nominal shear value per foot of shear wall for 7/16 inch thick OSB at the stated fastening and a 2:1 aspect ratio is 910 plf (Table 2211.1(3) of IBC 2000 (ICC, 2000a)). The IBC provides shear values for GWB with 7 inch o.c. fastener spacing. It provides no shear values for 12 inch o.c. fastener spacing.

Where LRFD is used, the IBC 2000 (ICC, 2000a) requires the factored design shear value to be determined by multiplying the ultimate shear value by a resistance factor ( $\Phi$ ) of 0.55.

Calculate corner area width (2a), where "a" equals 10% of least width or 0.4h (whichever is smaller) but not less than either 4% of least width or 3 feet.

$$\begin{aligned} a &= 10\%(30) = 3' \\ a &= 0.4(30) = 12' \\ \text{Use } a &= 3' \\ 2a &= 6' \end{aligned}$$

$$\text{End wall corner area} = (4' + 8')(6') = 72 \text{ ft}^2$$

$$\text{End wall area} = (4' + 8')(30') - 72 = 288 \text{ ft}^2$$

$$\text{End wall roof corner area} = (1/2)(7.5)(3') = 11.25 \text{ ft}^2$$

$$\text{End wall roof area} = (6/12)(15')(15') - 11.25 = 101 \text{ ft}^2$$

$$\text{Side wall corner area} = (4' + 8')(6') = 72 \text{ ft}^2$$

$$\text{Side wall area} = (4' + 8')(50') - 72 = 528 \text{ ft}^2$$

$$\text{Side wall roof corner area} = (1/2)(7.5')(6')(2) = 45 \text{ ft}^2$$

$$\text{Side wall roof area} = (7.5')(50') - 45 = 330 \text{ ft}^2$$

$$\text{Building aspect ratio} = 50/30 = 1.67$$

Design shear at each side wall

$$= [72 \text{ ft}^2 (22.2 \text{ psf}) + 288 \text{ ft}^2 (14.7 \text{ psf}) + 11.25 \text{ ft}^2 (22.2 \text{ psf}) + 101 \text{ ft}^2 (14.7 \text{ psf})](1.6)/2 = 5,954 \text{ lb}$$

Design shear at each end wall

$$= [72 \text{ ft}^2 (27 \text{ psf}) + (528 \text{ ft}^2 (20.1 \text{ psf}) + 22.5 \text{ ft}^2 (3.8 \text{ psf}) + 330 \text{ ft}^2 (3.3 \text{ psf}))(1.6)/2 = 10,985 \text{ lb}$$

### E10.2 Required Sheathing (First Floor Walls)

$$\text{Each end wall: } \frac{10,985}{910(0.55)} = 21.95 \text{ ft} \quad \text{This equates to 73\% of length of sheathed wall}$$

required for an engineered design. Using Tables E8-1 and E8-4, 97% of sheathed wall length must be provided.

$$\text{Each side wall: } \frac{5,954}{910(0.55)} = 11.90 \text{ ft} \quad \text{This equates to 24\% of side wall length of sheathed}$$

wall required for an engineered design. Using Tables E8-1 and E8-4, 35% of sheathed wall length must be provided.

## E11 Shear Wall Design (High Seismic Area)

Design the lateral force resisting system for the example building shown below. Use Type I braced wall for the side walls and Type II braced walls for the end walls.

### E11.1 Design Assumptions

Building Width = 30'

Building Length = 50'

Roof Slope = 6:12

Seismic Design Category = D2

Roof Dead Load = 15 psf

Exterior Wall Dead Load = 7 psf

Ground Snow Load = 25 psf

Slab-on-grade foundation

Interior Wall Dead Load = 5 psf

Overhang = 2 ft

Studs: 350S162-33

Stud Spacing = 16" on center

### E11.2 Design Loads

Roof Dead Load = 15 psf

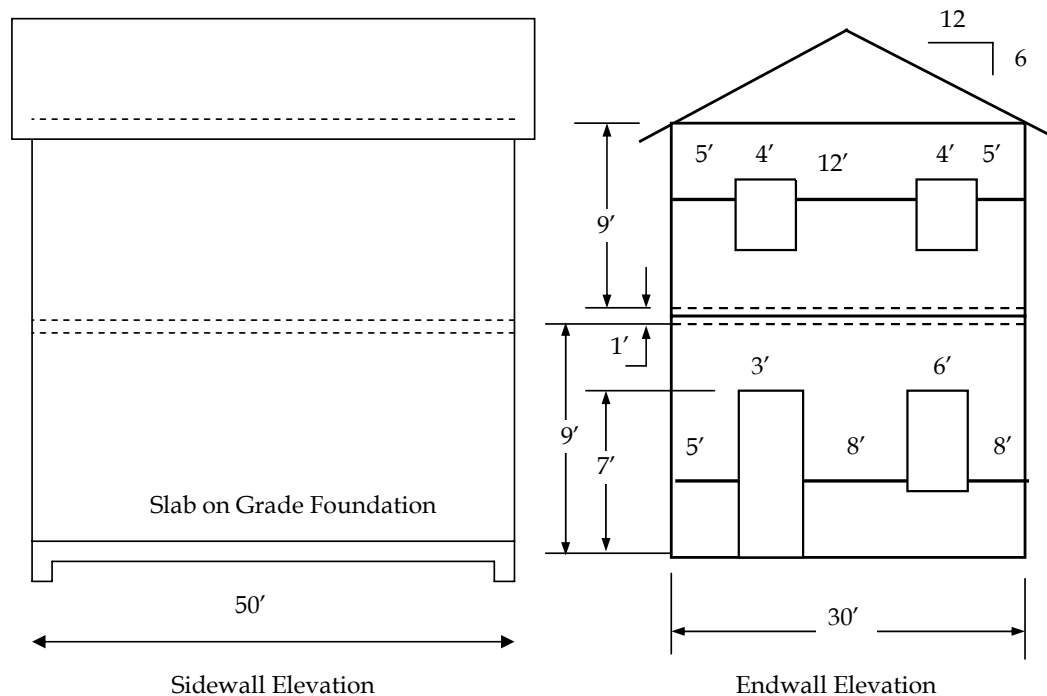
Floor Dead Load = 10 psf

Exterior Wall Dead Loads = 7 psf

Interior Walls Dead Load = 5 psf

Floor area =  $(50')(30') = 1500 \text{ ft}^2$

Roof area =  $[50' + 2(2)][30' + 4'] = 1836 \text{ ft}^2$ . (2' overhangs)



Interior wall weight per square foot of floor area =

@ Roof =  $\frac{1}{2}(5 \text{ psf}) = 2.5 \text{ psf}$

@ Floor =  $\frac{1}{2}(2)(5 \text{ psf}) = 5 \text{ psf}$

|                       |   |                                 |   |           |                |
|-----------------------|---|---------------------------------|---|-----------|----------------|
| Roof weight           | = | 15 psf(1,836 ft <sup>2</sup> )  | = | 2,7540 lb |                |
| Floor weight          | = | 10 psf(1,500 ft <sup>2</sup> )  | = | 1,5000 lb |                |
| Interior walls weight | = | 5 psf(1,500 ft <sup>2</sup> )   | = | 7,500 lb  | @ Second floor |
|                       | = | 2.5 psf(1,500 ft <sup>2</sup> ) | = | 3,750 lb  | @ Roof         |

Exterior Walls (Long Walls)

|                          |   |                      |   |          |
|--------------------------|---|----------------------|---|----------|
| To Roof                  | = | $(2)(50)(9)(1/2)(7)$ | = | 3,150 lb |
| To 2 <sup>nd</sup> Floor | = | $(2)(3,150)$         | = | 6,300 lb |

Exterior Walls (Short Walls)

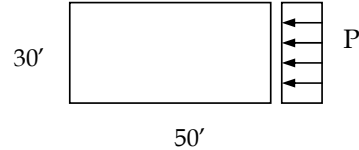
|                          |   |                      |   |          |
|--------------------------|---|----------------------|---|----------|
| To Roof                  | = | $(2)(30)(9)(1/2)(7)$ | = | 1,890 lb |
| To 2 <sup>nd</sup> Floor | = | $(2)(1890)$          | = | 3,780 lb |

$$\begin{aligned}\text{Total Load at Roof} &= 3,150 + 1,890 = 5,040 \text{ lb} \\ \text{Total Load at 2nd Floor} &= 6,300 + 3,780 = 10,080 \text{ lb}\end{aligned}$$

### Sum of Weights

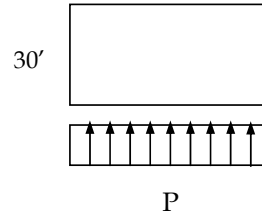
For Base Shear, Vertical Distribution and Shear Wall Design:

$$\begin{aligned}\text{At Roof:} & 27,540 + 3,750 + 5,040 = 36,330 \text{ lb} \\ \text{At 2nd Floor:} & 15,000 + 7,500 + 10,080 = \underline{32,580 \text{ lb}} \\ \text{Sum:} & 36,330 + 32,580 = 68,910 \text{ lb}\end{aligned}$$



For Diaphragm Design

$$\begin{aligned}P_{\text{Roof}} &= 36,330 - 3,150 = 33,180 \text{ lb} \\ P_{\text{2nd Floor}} &= 32,580 - 6,300 = 26,280 \text{ lb}\end{aligned}$$



$$\begin{aligned}P_{\text{Roof}} &= 36,330 - 1,890 = 34,440 \text{ lb} \\ P_{\text{2nd Floor}} &= 32,580 - 3,780 = 28,800 \text{ lb}\end{aligned}$$

### Determine Base Shear - SDC D<sub>2</sub>

Base Shear,  $V = C_s W$

(IBC Eq. 16-34)

$$C_s = S_{DS}/R$$

$$S_{DS} = 1.17$$

(IRC Table R301.2.2.1.1)

$$R = 6$$

(IBC Table 1617.6, System 1-K)

$$C_s = 1.17/6 = 0.195$$

$$V = (0.195)(68,910) = 13,437 \text{ lb}$$

### Vertical Distribution

$$F_x = C_{vx} V$$

(IBC Eq. 16-41)

$$C_{vx} = \frac{W_x H_x}{\sum_{i=1}^n W_i H_i}$$

(IBC Eq. 16-42 with  $K = 1$ )

$$V_x = \sum_{i=1}^n F_i$$

(IBC Eq. 16-43)

$$\text{Height}_{\text{2nd}} = 9' + 1' = 10 \text{ ft}$$

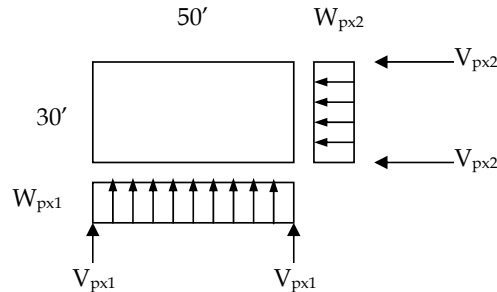
$$\text{Height}_{\text{Roof}} = (2)(9) + 1 + \frac{1}{2}(7.5') = 22.75 \text{ ft}$$



| Diaphragm             | Weight<br>(lb) | Height<br>(ft) | Weight x Height | C <sub>vx</sub> | F <sub>x</sub><br>(lb) | V <sub>x</sub><br>(lb) |
|-----------------------|----------------|----------------|-----------------|-----------------|------------------------|------------------------|
| Roof                  | 36,330         | 22.75'         | 826,508         | 0.7173          | 9,638                  | 9,638                  |
| 2 <sup>nd</sup> Floor | 32,580         | 10'            | 325,800         | 0.2827          | 3,799                  | 13,437                 |
| Total                 | 68,910         |                | 1,152,308       |                 |                        |                        |

### Diaphragm Loads

$$F_{px} = \frac{\sum_{i=x}^n F_x}{\sum_{i=x}^n W_i} W_{px}$$



| Diaphragm             | F <sub>x</sub><br>(lb) | W <sub>x</sub><br>(lb) | W <sub>px</sub><br>(lb)  | ΣF <sub>i</sub><br>(lb) | ΣW <sub>i</sub><br>(lb) | F <sub>px</sub><br>(lb) | V <sub>px</sub><br>(lb/ft) |
|-----------------------|------------------------|------------------------|--------------------------|-------------------------|-------------------------|-------------------------|----------------------------|
| Roof                  | 9,638                  | 36,330                 | W <sub>px1</sub> =34,440 | 9,638                   | 36,330                  | 9,137                   | 152                        |
|                       |                        |                        | W <sub>px2</sub> =33,180 |                         |                         | 8,802                   | 88                         |
| 2 <sup>nd</sup> Floor | 3,799                  | 32,580                 | W <sub>px1</sub> =28,800 | 13,437                  | 68,910                  | 5,616                   | 94                         |
|                       |                        |                        | W <sub>px2</sub> =26,280 |                         |                         | 5,124                   | 51                         |

Compare with 0.15(S<sub>DS</sub> I<sub>E</sub> W<sub>px</sub>) and 0.3 (S<sub>DS</sub> I<sub>E</sub> W<sub>px</sub>)

(IBC Eq. 1620.3.3)

S<sub>DS</sub> = 1.17; I<sub>E</sub> = 1.0

Roof 6,044 lb < 9,137 lb < 12,088 lb ok

2<sup>nd</sup> 5,054 lb < 5,616 lb < 10,109 lb ok

### Diaphragm Chord Splice Requirements

Determine Chord Force and Splice Requirements with Amplified Chord Force

| Diaphragm             | W <sub>p</sub><br>(lb/ft) | L<br>(ft) | C=T (lb) | Ω <sub>0</sub> | Ω <sub>0</sub> T | Required No. 8<br>Screws @ Splice |
|-----------------------|---------------------------|-----------|----------|----------------|------------------|-----------------------------------|
| Roof                  | 183                       | 50        | 1,906    | 2.5            | 4,765            | 20                                |
|                       | 293                       | 30        | 659      |                | 1,648            | 7                                 |
| 2 <sup>nd</sup> Floor | 113                       | 50        | 1,177    | 2.5            | 2,943            | 12                                |
|                       | 171                       | 30        | 385      |                | 963              | 4                                 |

where W<sub>p</sub> = F<sub>px</sub>/L; C=T= W<sub>p</sub>L<sup>2</sup>/8L<sup>2</sup>

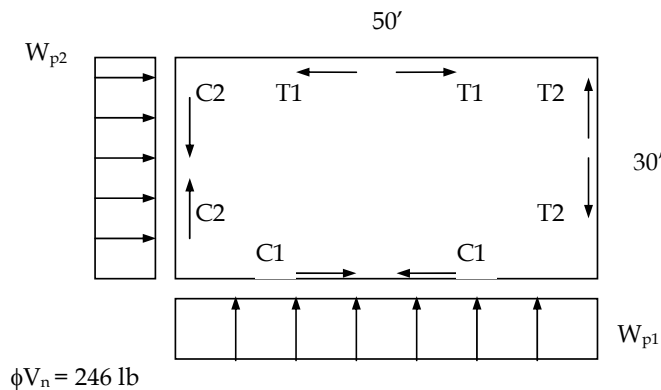
These are the required number of screws for an engineered design. From Table E12-20, in AISI S230, 23, 9, 15 and 5 screws respectively must be provided.

Note: Ω<sub>0</sub> = 2.5 based on system 1.K from Table 1617.6 of the IBC 2000 with 0.5 reduction for flexible diaphragm (3.0 - 0.5 = 2.5)

Note: For No. 8 screws, assuming 33 mil top track:

$\phi = 0.5$  for screws in shear

$V_n = 492 \text{ lb}$



### E11.3 Required Sheathing (Side Walls)

Per Table 2211.1(3) and Section 2211.6 of the 2000 IBC:

$V_n = 700 \text{ plf}$  (7/16" OSB w/6" screw spacing at panel edges and 12" at interior supports)

$\phi = 0.55$

$\phi V_n = 385 \text{ lb/ft}$

| Wall Level      | $V_x$ (lb) | $V_{\text{wall}} (V_x/2)$ (lb) | $L_{\text{Required}} (V_{\text{wall}} / \phi V_n)$ (ft) |
|-----------------|------------|--------------------------------|---|
| 2 <sup>nd</sup> | 9,638      | 4,819                          | 12.60   |
| 1 <sup>st</sup> | 13,437     | 6,719                          | 17.45   |

Provide (1) shear wall at each corner of each side and end walls (2 shear walls per wall)

Second Floor:  $L = 12.52 \text{ ft}$  Use 7 feet each end

First Floor:  $L = 17.45 \text{ ft}$  Use 9 feet each end

These are the required lengths for an engineered design. Using Tables E12-3 and E12-4 in *AISI S230*, 9 ft for the second floor and 13 ft for the first floor must be provided at each end.

### E11.4 Hold Downs and Multiple Stud Posts (Side Walls)

Second Floor:  $V = 346 \text{ plf}$   $T/C = 3,120 \text{ lb}$

First Floor:  $V = 373 \text{ plf}$   $T/C = 3,730 \text{ lb}$

Multiple Studs Required at Shear Wall Ends

|               |         |          |                            |
|---------------|---------|----------|----------------------------|
| Second Floor: | $P_u =$ | 3,120 lb |                            |
| First Floor:  | $P_u =$ | 6,850 lb | (when chords align)        |
| First Floor:  | $P_u =$ | 3,730 lb | (when chords do not align) |

Allowable loads:

$$(2) 350S162-33: \quad \phi P_n = 4,945 \text{ lb} \quad \phi P_n = 4,945 \text{ lb} > 3,730 \text{ lb}$$

$$(2) 350S162-54: \quad \phi P_n = 8,102 \text{ lb} \quad \phi P_n = 8,102 \text{ lb} > 6,850 \text{ lb}$$

These are the multiple studs for an engineered design. Using Table E12-19, 54 mil studs are not an option, therefore the *AISI 230* would not apply for this design.

Hold Downs Required at Shear Wall Ends

|                       |         |           |             |          |
|-----------------------|---------|-----------|-------------|----------|
| Single Shear Wall:    | $T_u =$ | 3,730 lbs | $T_{ASD} =$ | 2,710 lb |
| Aligning Shear Walls: | $T_u =$ | 6,850 lbs | $T_{ASD} =$ | 4,893 lb |

Using a Hold Down with 5/8" diameter bolt for single shear wall:

$$T_{\text{allowable}} = 5,260 \text{ lbs}$$

$$T_{\text{allowable}} = 3,950 \text{ lbs} \quad (\text{without } 1/3 \text{ increase for wind/seismic})$$

$$T_{\text{allowable}} > T_{ASD} = 2,710 \text{ lb}$$

Using Hold Down with 5/8" diameter bolt for aligning shear wall:

$$T_{\text{allowable}} = 7,920 \text{ lbs}$$

$$T_{\text{allowable}} = 5,940 \text{ lbs} \quad (\text{without } 1/3 \text{ increase for wind/seismic})$$

$$T_{\text{allowable}} > T_{ASD} = 4,940 \text{ lb}$$

**E11.5 Required Sheathing (End Walls)**

End wall shear walls use 7/16" OSB fastened with No. 8 screws at 6" at panel edges and 12" at intermediate supports. Per previous calculations,  $L_{\text{Required}}$  at 2<sup>nd</sup> floor = 12.6'; and  $L_{\text{Required}}$  at 1<sup>st</sup> floor = 17.45'.

Try Type II walls (perforated shear wall):

$$\text{Determine percentage of fully sheathed wall} \quad @ 2^{\text{nd}} \text{ floor} = (5+12+5)/30 = 0.73 = 73.3\%$$

$$@ 1^{\text{st}} \text{ floor} = (5+8+8)/30 = 0.70 = 70\%$$

Maximum unrestrained opening at 1<sup>st</sup> and 2<sup>nd</sup> floor = 7', with wall fully sheathed above and below.

Interpolate for length adjustment factor from Table E11-2 for both "Percent Fully Sheathed Wall" and "Maximum Unrestrained Opening Height":

At 2<sup>nd</sup> Floor:

| % Fully Sheathed | 6.75' | 7'    | 7.5'  |
|------------------|-------|-------|-------|
| 60               | 1.28  |       | 1.32  |
| 73.3             | 1.176 | 1.182 | 1.203 |
| 80               | 1.12  |       | 1.14  |

Therefore, the required length of fully sheathed (Type II) wall =  $(1.182)(12.60') = 14.89'$   
 $14.89' < 22'$  provided by the full height sheathing for the upper wall. ok

At 1<sup>st</sup> Floor:

| % Fully Sheathed | 6.75' | 7'    | 7.5' |
|------------------|-------|-------|------|
| 60               | 1.28  | 1.293 | 1.32 |
| 70               | 1.2   | 1.21  | 1.23 |
| 80               | 1.12  | 1.127 | 1.14 |

Therefore, the required length of fully sheathed (Type II) wall =  $(1.21)(17.45') = 21.11'$   
 $21.11'$  is close to  $21'$  provided by the full height sheathing for the lower wall. ok

#### E11.6 Hold Downs and Multiple Stud Posts (End Walls)

Second Floor:  $V = 162$  plf       $T/C = 1,460$  lb (when chords do not align)  
 First Floor:  $V = 228$  plf       $T/C = 3,730$  lb (when chords align)

Allowable loads:

(2) 350S162-33:  $\phi P_n = 4,945$  lb       $\phi P_n = 4,945 > 3,730$  lb ok

#### Hold Downs Required at Shear Wall Ends

Single Shear Wall:  $T_u = 1,460$  lbs       $T_{ASD} = 1,040$  lb  
 Aligning Shear Walls:  $T_u = 3,730$  lbs       $T_{ASD} = 2,670$  lb

Using a Hold Down with 5/8" diameter bolt. For single shear walls:

$T_{allowable} = 5,260$  lbs  
 $T_{allowable} = 3,950$  lbs (without 1/3 increase for wind/seismic)  
 $T_{allowable} > T_{ASD} = 1,040$  lb

These are the multiple studs and hold down requirements for an engineered design.  
 Use Table E12-17 to determine the chord strength and hold down strength required for this design.

#### Determination of Top Track Requirements to Accommodate Diaphragm Chord Forces

From the above,      @Roof       $C_{max} = 1,906$  lb  
                                  @2<sup>nd</sup> Floor       $C_{max} = 1,177$  lb

Using computer software, the maximum axial capacity of a 350T125-33 (top track) is 2,882 lbs (concentric load with no weak axis bracing and a maximum track length of 1.33 ft).

For  $L = 16''$  with track considered fully braced,  $\phi P_n = 2,882 \text{ lb} > 1,906 \text{ lb}$

Note: The diaphragm chord forces were tabulated for the various roof and wall height combinations at various diaphragm spans with some breakdown of diaphragm aspect ratios where warranted.

#### Establish Shear Anchor Requirements Based on Shear Wall Edge Screw Spacing

Use shear wall capacity, based on shear wall edge screw spacing (6" o.c. at panel edges) to determine required shear anchor capacity:

$$\phi V_n = 385 \text{ plf @ } 6'' \text{ panel edge screw spacing}$$

Check 350T125-33 bottom track with nested 350S162-33 stud:

$$\phi P_{n(\text{Track})} = 2,420 \text{ lb bearing capacity of track plus stud} \quad (\text{Table E3.3-2})$$

$$\begin{aligned} \text{where: } F_u &= 45 \text{ ksi} \\ t &= 0.0346'' \\ d &= 0.5'' \\ \phi &= 0.7 \end{aligned} \quad (\text{Table E3.3-2})$$

$$\phi P_n = 3,444 \text{ lb shear capacity of } \frac{1}{2}'' \text{ diameter bolt} \quad (\text{Eq. E3.4-1})$$

$$\begin{aligned} \text{where: } A_b &= 0.196 \text{ in}^2 \\ F_{nv} &= 27 \text{ ksi} \\ \phi &= 0.65 \end{aligned} \quad \begin{aligned} &(\text{Table E3.4-1}) \\ &(\text{Table E3.4-1}) \end{aligned}$$

$$\text{Required Fastener Spacing} = 2,420/385 = 6.28 \text{ feet}$$

This is the fastener spacing for an engineered design. Using Table E12-18, the required spacing is 5.0 feet.

#### **E11.7 Continuous Strap for Drag Force**

##### **A. Tensile Capacity of 2.5" x 43-mil Strap**

$$\begin{aligned} \phi T_n &= (0.95)(33 \text{ ksi})(2.5'')(0.0451'') = 3.535 \text{ kips} \\ \text{Where: } F_y &= 33 \text{ ksi} \\ A_n &= 0.113 \text{ in}^2 \\ \phi &= 0.95 \end{aligned}$$

##### **B. Screw Shear Capacity for Strap End Connections:**

| Screw Size   | $V_a^*(\text{lb})$ | $\Omega$ | $\phi$ | $\phi V_n$ | No. Required** |
|--------------|--------------------|----------|--------|------------|----------------|
| No. 8 Screw  | 244                | 3        | 0.5    | 366        | 11             |
| No. 10 Screw | 263                | 3        | 0.5    | 395        | 10             |

\*Values for screw shear capacity was obtained from Section 1, Table C-B1.

\*\*No. 8 screw:  $\phi V_n = (244)(\Omega\phi) = 244(3)(0.5) = 366 \text{ lb/screw}$

No. 10 screw:  $\phi V_n = (263)(\Omega\phi) = 263(3)(0.5) = 395 \text{ lb/screw}$

$3,535/366 = 9.66$  Use (10) No. 8 screws, or,

$3,535/395 = 8.95$  Use (9) No. 10 screws

Maximum diaphragm shear = 346 plf (at roof level)

#### C. Maximum Blocking Spacing Based on Tensile Capacity of Strap

$$\text{Spacing} = 3,535/346 = 10.22 \text{ feet}$$

#### D. Maximum Blocking Spacing Based on Capacity of Blocking

Block properties:  $h = 7.5 \text{ in}$   
 $t = 0.0566 \text{ in}$   
 $h/t = 132.51$   
 $k_v = 5.34$  (assume unreinforced web)  
 $F_y = 33 \text{ ksi}$   
 $[Ek_v/F_y]^{1/2} = 68.5$

$$0.96(68.5) = 65.76 < 132.51$$

$$1.415(68.5) = 96.93 < 132.51$$

$$V_n = 0.905Ek_v t^3/h$$

$$V_n = 3,388 \text{ lb}$$

$$\phi V_n = 0.90(3,388) = 3,049 \text{ lb}$$

(Eq. C3.2-3)

$$\text{Clip Spacing} = 3,049/346 = 8.81 \text{ feet}$$

Space blocking at 8-foot maximum and at each end.

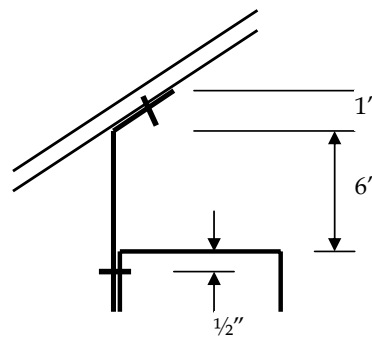
This spacing is for an engineered design. Using Figure E11-6 of *AISI S230*, blocking is required at 4-foot maximum spacing and 13 No. 8 screws from strap to blocking panel.

#### E11.8 Stabilizing Clip at Eave Block

Use calculated value for  $\phi V_n = 3,049 \text{ lb}$

$V_{\max} = \phi V_n$  for the block

Assume 6" blocking depth + 2" to connection:

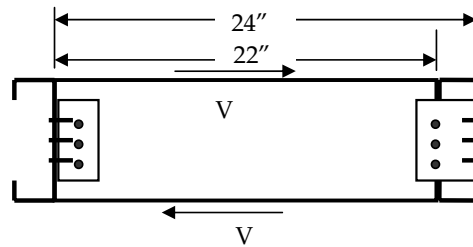


$$M_{\text{Block}} = (8'')(3.049) = 24.4 \text{ in-k}$$

$$V_{\text{clip}} = 24.4/22 = 1.11 \text{ k}$$

$$\text{No. of Screws Required} = 1,110 / [(0.5)(3)(244)] = 3.03 \quad (\text{assume 43 mil rafters})$$

Use (4) No. 8 screws. This is for an engineered design. Using Figure E11-6 of *AISI S230*, blocking is required the clip requires 4 No. 8 screws for blocking spaced at 4'-0" o.c.



### E11.9 Connection of Shear Wall to Floor Diaphragm to Shear Wall Below

Sheathing Screw Spacing 6"

Sheathing Shear capacity,  $\phi V_n$  (plf) 385 lbs

No. 8 Screw Capacity 246 lbs

Number of No. 8 Screws Required Per Foot 2

This validates the note "Use No. 8 screws to match edge screw spacing for braced wall above" as stipulated by Figure E11-8.

## E12 Shear Wall Design (High Wind Area)

Design the lateral force resisting system for the example building shown below. Use Type I braced wall for the side walls and Type II braced walls for the end walls.

### E12.1 Design Assumptions

Building Width = 32'

Building Length = 50'

Roof Slope = 3:12

Design Wind Speed = 120 mph Exposure Category B

Roof Dead Load = 15 psf

Wall Dead Load = 7 psf

Ground Snow Load = 25 psf

Slab-on-Grade Foundation

Framing Studs: 350S162-33

Stud Spacing = 16" on center

### Calculate Wind Load

In accordance with ASCE 7 (ASCE, 1998):

Directionality Factor ( $K_d$ ) = 0.85

(Table 6-6)

Importance Factor = 1.0

(Section 6.5.5)

Topographic Factor ( $K_{zt}$ ) = 1.0 (Figure 6-2)

Velocity Exposure ( $K_z$ ) = 0.70

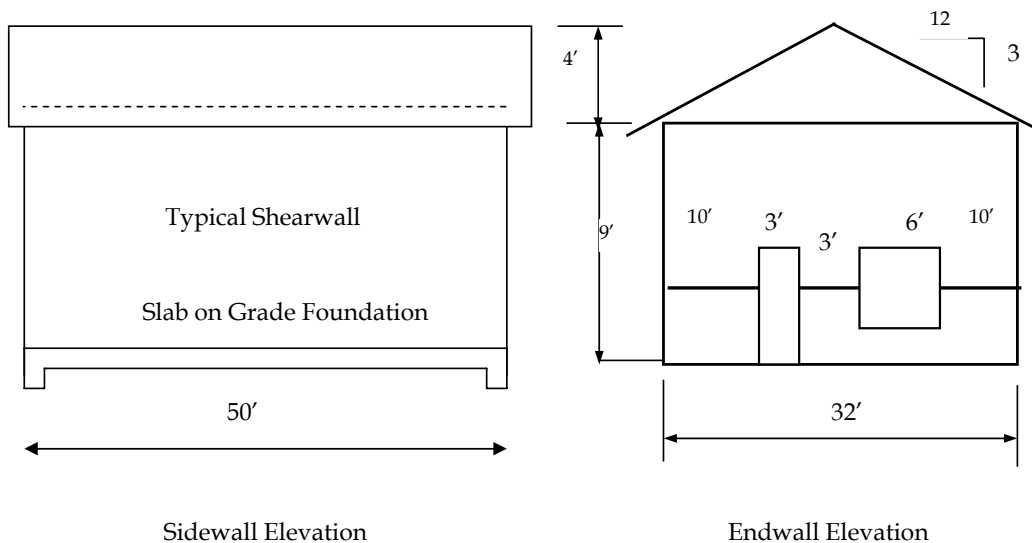
(Table 6-5)

Velocity Pressure =  $q_z = 0.00256 \times K_z \times K_{zt} \times K_d (V^2 \times I) = 21.934$

(Eq. 6-13)

Pressure =  $p = q_z[(GC_{pf}) - (GC_{pi})]$

(Eq. 6-16)



#### $GC_{pf}$ (ASCE Figure 6-4) CASE A

Roof Slope = 14.03 degrees

| Area 1 | Area 2 | Area 3 | Area 4 | Area 1E | Area 2E | Area 3E | Area 4E |
|--------|--------|--------|--------|---------|---------|---------|---------|
| 0.478  | -0.690 | -0.436 | -0.374 | 0.724   | -1.070  | -0.626  | -0.556  |

#### $GC_{pi}$ (ASCE Figure 6-4) CASE A

Enclosed Buildings

|       |       |
|-------|-------|
| +0.18 | -0.18 |
|-------|-------|

#### $GC_{pf}$ (ASCE Figure 6-4) CASE B

Roof Slope = 14.03 degrees

| Area 1 | Area 2 | Area 3 | Area 4 | Area 5 | Area 6 | Area 1E | Area 2E | Area 3E | Area 4E | Area 5E | Area 6E |
|--------|--------|--------|--------|--------|--------|---------|---------|---------|---------|---------|---------|
| -0.45  | -0.69  | -0.37  | -0.45  | 0.4    | -0.29  | -0.48   | -1.07   | -0.53   | -0.48   | 0.61    | -0.43   |



**p (psf) - CASE A**

Roof Slope = 14.03 degrees

| Area 1 | Area 2 | Area 3 | Area 4 | Area 1E | Area 2E | Area 3E | Area 4E |
|--------|--------|--------|--------|---------|---------|---------|---------|
| 14.43  | -19.08 | -13.51 | -12.15 | 19.83   | -27.42  | -17.68  | -16.14  |

**p (psf) - CASE B**

Roof Slope = 14.03 degrees

| Area 1 | Area 2 | Area 3 | Area 4 | Area 5 | Area 6 | Area 1E | Area 2E | Area 3E | Area 4E | Area 5E | Area 6E |
|--------|--------|--------|--------|--------|--------|---------|---------|---------|---------|---------|---------|
| -      | -      | -      | -13.82 | 12.72  | -      | -14.48  | -27.42  | -15.57  | -14.48  | 17.33   | -13.38  |
| 13.82  | 19.08  | 12.06  |        |        | 10.31  |         |         |         |         |         |         |

The above are based on corner length as follows:

|                       |           |                     |
|-----------------------|-----------|---------------------|
| 0.4 x Building Height | = 5.20 ft | (height = 13.00 ft) |
| 0.1 x min. Width      | = 3.20 ft |                     |
| Not less than 3'      | = 3.00 ft |                     |
| a                     | = 3.20 ft |                     |
| Corner Length = 2a    | = 6.40 ft |                     |

Roof Area 2 Factored Wind Load = 19.08 (1.6) = 30.52 psf

Roof Area 3 Factored Wind Load = 13.51 (1.6) = 21.62 psf

Factored Roof Dead Load = 0.9(15 psf) = 13.5 psf

Determine Forces in Walls

## Side Walls

P = 12.72 + 10.31 = 23.03 psf (interior)

P = 17.33 + 13.38 = 30.71 psf (exterior, corner)

## Force/Wall

$$= \left[ 23.03 \text{ psf} \left( \frac{1}{2} \right) (9') \frac{32' - 6.4'}{2 \text{ walls}} \right] + \left[ 23.03 \text{ psf} \frac{(1/2)(32')(4')}{2 \text{ walls}} \right] - \left[ \frac{23.03 \text{ psf} (3.2')(0.8)}{2} \right]$$

$$+ \left[ \frac{30.71 \text{ psf} (3.2')(9')}{2} \right] + \left[ \frac{(30.71 \text{ psf})(3.2')(0.8)}{2} \right]$$

Force/Wall = 2515 lb/side wall

## End Walls

P = 14.43 + 12.15 = 26.58 psf (interior)

P = 19.82 + 16.14 = 35.96 psf (corner)

Roof: P<sub>1</sub> = -19.08 psf (Area 2),

[Sin(14.03)][-19.08] = -4.62 psf

P<sub>1</sub> = -27.41 psf (Area 2E),P<sub>2</sub> = -13.51 psf (Area 1)

[Sin(14.03)][13.51] = 3.27 psf

P<sub>2</sub> = -17.67 psf (Area 1E)

Total load from roof =  $4.62 - 3.27 = 1.35$  psf. (typ.)

Total load from roof = 2.36 psf (corner)

$$\begin{aligned} \text{Force/Wall} &= (26.58 \text{ psf})(9' \times 1/2)(50' - 12.8')/2 \text{ walls} + (35.96 \text{ psf})(1/2)(9')(6.4') \\ &\quad + (-4.35 \text{ psf})(4')[(50' - 12.8')/2] + (-2.36 \text{ psf})(4')(6.4') = 3,099 \text{ lb/end wall} \end{aligned}$$

However, in accordance with ASCE 7 the total horizontal shear shall not be less than that determined by neglecting wind roof surfaces. Therefore,

$$\begin{aligned} \text{Force/wall} &= (26.58 \text{ psf})(9'/2)(37.2') + (35.96 \text{ psf})(9'/2)(12.8') \\ &= 6,520 \text{ lb}/2 \text{ sides} \\ &= 3,260 \text{ lb} \end{aligned}$$

#### Chords

$v = 1,258 \text{ lb Side wall} / 4' \text{ long} = 315 \text{ plf}$

$v = 1,630 \text{ lb End wall} / 5' \text{ long} = 326 \text{ plf}$

Wall height = 9 feet

Maximum chord force =  $326(9) = 2,934 \text{ lb}$

This force is for an engineered design. Using Table E13-10 a maximum chord strength of 4419 lb is required.

Post: Use (2) 350S162-33 studs braced at 4' ( $P_{all} = 4,945 \text{ lb}$ )

#### Hold Downs

$P = 2,934 \text{ lb}$

Attach hold down to each end of each shear wall panel. Use hold-downs with 5/8" diameter bolt ( $P_{all} = 3,295 \text{ lb}$ ).

The 3,295 lb hold down capacity is calculated as follows:

$P_{all} = 4,385 \text{ lb}$  for 54 mil member with 14 No. 10 screws.

$P_{all} = (177 \text{ lb/screw})(14)(1.33) = 3,295 \text{ lb}$  for 33 mil member.

#### **E12.2 Length of Shear Panel (Side Walls)**

$P = 2,515/2 \text{ panels} = 1,258 \text{ lb per shear wall}$

In accordance with Table 2211.1(1) of IBC 2000:  $V_n = 910/2.5 = 364 \text{ lb}$

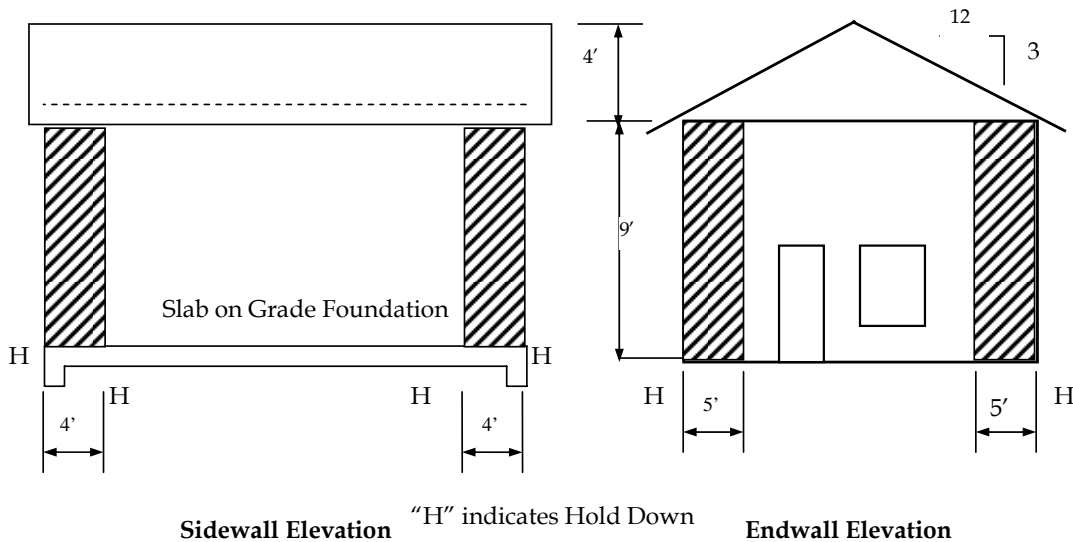
Length of shear panel required =  $1,258/364 = 3.46 \text{ ft/shear wall}$

#### **E12.3 Length of Shear Panel (End Walls)**

$P = 3,260/2 \text{ panels} = 1,630 \text{ lb per shear wall}$

In accordance with Table 2211.1(1) of IBC 2000:  $V_n = 910/2.5 = 364$  lb with 6" fastener spacing.

Length of shear panel required =  $1,630/364 = 4.48$  ft/shear wall



Verify the above design with the requirements of AISI S230:

Range of allowable side wall lengths of one-story slab-on-grade (Table E13-1)

Minimum =  $14 \text{ feet}/1.21 = 11.57'$  (adjusted for Exposure Category B)

Maximum =  $60 \text{ feet}/1.21 = 66.11$  feet (adjusted for Exposure Category B)

Required minimum length of full height sheathing on side walls (Type I):

Minimum =  $7'(1.13) = 7.91$  feet (6" fastener spacing) (Table E13-3)

Required minimum length of full height sheathing on side walls (Type I):

Minimum sheathing length =  $8'(1.13)(1/1.21)(0.8)$   
= 5.23 feet (6" fastener spacing) (Table E13-4)

(1.21 adjustment factor for Exp. B; 1.13 adjustment factor for 9' walls; 0.8 adjustment factor for mean roof height < 15')

Use (2) 4' long shear walls with 7/16" OSB fastened with No. 8 screws at 6" o.c. at edges.

Required minimum length of full height sheathing on end walls (Type I):

Minimum sheathing length =  $8'(1.13)(0.8)(1/1.21) = 5.97$  feet

(1.21 adjustment factor for Exp. B; 1.13 adjustment factor for 9' walls; 0.8 adjustment factor for mean roof height < 15')

Since this is a Type II shear wall, check the applicability of any factors:

The length of full height sheathing cannot be multiplied by any factor from Table E12.1 because the roof dead load is greater than 11 psf and the wall dead load is not less than 7 psf.

$L = 5.97$  feet. (6" o.c. edge fastener spacing)

Use (2) 5' long shear walls with 7/16" OSB fastened with No. 8 screws at 6" o.c. at edges.

#### **E12.4 Braced Wall Hold Down Anchorage**

Required hold down anchor force from Table E12.2 = 3,535 lbs. (9-foot walls and 6" screw spacing).

$F = 3,535 / 1.4 = 2,525$  lb (The 1.4 factor is from Table E12-2 of *AISI S230*)

## F. ROOF FRAMING DESIGN EXAMPLES

### F1 Ceiling Joist Design

Calculate the maximum allowable single span for a 550S162-33 ceiling joist (attics with limited storage) with bearing stiffeners at each support. The deflection limit is  $L/240$  (total loads). The joists are spaced at 24 inches on center and laterally braced (on compression flange) at mid-span.

#### F1.1 Design Assumptions

Joist Spacing = 24"  
 Steel Yield Strength = 33 ksi  
 Ceiling Dead Load = 5 psf  
 Attic Live Load for Attics with Limited Storage = 20 psf  
 Live Load + Dead Load Deflection Limit =  $L/240$   
 Punchouts: 2-1/2" wide x 4" long along centerline of joist

#### F1.2 Design Loads

Dead Load:  
 Ceiling Dead Load (D) = 5 psf  
Live Loads:  
 Attic Live Load (L) = 20 psf

#### F1.3 Load Combinations

1.  $1.4D = 14 \text{ plf}$
2.  $1.2D + 1.6L = 76 \text{ plf}$

Therefore, the controlling factored uniform design load,  $w_u$ , is 76 plf

#### F1.4 Member Properties

Select a 550S162-33 member and check its adequacy. The calculated design flexure strength and section properties for this member in accordance with the *Specification* are:

|                                       |                          |
|---------------------------------------|--------------------------|
| $\phi M_n$                            | = 969 ft-lb              |
| $\phi V_n$                            | = 744.1 lb               |
| Effective Moment of Inertia, $I_{xx}$ | = 1.4506 in <sup>4</sup> |
| Effective Section Modulus, $S_{xx}$   | = 0.4209 in <sup>3</sup> |

#### F1.5 Bending Capacity

$$\phi_b M_n = 969 \text{ ft-lb}$$

$$M = \frac{wL^2}{8} \rightarrow L = \sqrt{\frac{8M}{w}} = \sqrt{\frac{8(969)}{76}}$$

$$L = 10.10 \text{ ft} = 10'-2"$$

#### F1.6 Shear Capacity

$\phi V_n = 744 \text{ lbs}$       Nominal shear strength (calculated per AISI *Specification*)

Use the design shear strength to calculate the maximum span.

$$V = \frac{wL}{2} \rightarrow L = \frac{2V}{w} = \frac{2(744.13)}{76} = 19'-7''$$

### F1.7 Deflection Limit

$$\Delta = L/240 \text{ (total loads)}$$

The deflection equation for a simply supported span with distributed load is:

$$\Delta = \frac{5wL^4}{384EI} = L/240$$

$$L = \sqrt[4]{\frac{384EI_x(L/240)}{5w}} = \sqrt[3]{\frac{384(29,500,000)(1.4506)}{5(50/12)(240)}} = 12'-5''$$

where:

|                  |   |
|------------------|---|
| L =              | Single span length (inches)   |
| I <sub>x</sub> = | Effective moment of inertia for deflection = 1.4506 in <sup>4</sup> |
| w =              | 50 plf  |
| E =              | Modulus of elasticity = 29,500,000 psi                              |

Therefore, the maximum span for a 550S162-33 ceiling joist for the conditions in this example is 10'-2'' (controlled by bending). This confirms the maximum span given in Table F2-3 of the AISI S230.

## F2 Rafter Design

Determine if an 1000S162-54 steel rafter (6:12 slope) without a supplemental brace (Figure F2-1) is adequate for a 32-foot building subjected to 90 mph (Exposure Category B) wind speed and 30 psf ground snow load. Assume ceiling joist is 43 mil thick.

### F2.1 Design Assumptions

|        |                              |
|--------|------------------------------|
| 24"    | Rafter Spacing               |
| 3.5"   | Ceiling Joist Bearing Length |
| 33 ksi | Steel Yield Strength         |
| 7 psf  | Roof Dead Load               |
| 5 psf  | Ceiling Dead Load            |
| L/180  | Total Load Deflection Limit  |
| L/240  | Live Load Deflection Limit   |

### F2.2 Design Methodology

The rafter span table was designed based primarily on gravity loads, hence the rafter spans are reported on the horizontal projection of the rafter, regardless of the slope. The gravity loads consist of a 7 psf dead load and the greater of a minimum 16 psf live load or the applied unbalanced roof snow load (unbalanced snow load is calculated by multiplying the ground snow load by 1.0 and no further reductions or increases are made for special cases).

Wind load effects are correlated to equivalent snow loads wind pressures were calculated using the ASCE 7 (ASCE 2005) components and cladding coefficients. Wind loads acting perpendicular to the plane of the rafter were adjusted to represent loads acting orthogonal to the horizontal projection of the rafter (as shown in Figure F2-1. Wind loads were examined for both uplift and downward loads and the worst case was correlated to a corresponding snow load.

### F2.3 Design Loads

#### Dead Load:

$$\text{Roof dead load} = (7 \text{ psf} / \cos 26.56^\circ)(2') = 15.65 \text{ psf}$$

#### Live Load:

$$\text{Unbalanced Roof Snow Load (S)} = (30 \text{ psf})(2') = 60 \text{ plf} \quad \leftarrow \text{controls}$$

$$\text{Roof Live Load (L}_r\text{)} = 16 \text{ psf (2')} = 32 \text{ plf}$$

Both uplift and inward acting wind loads must be examined and the worst case converted to an equivalent snow load effect. ASCE 7 (ASCE, 1998) components and cladding pressure coefficients are used to calculate the wind load.

$$\rho = q_h[(GC_p) - (GC_{pi})] \text{ lb/ft}^2 \quad (\text{Eq. 6-18})$$

$$q_h = 0.00256K_zK_{zt}K_dV^2I \quad (\text{Eq. 6-13})$$

$$K_z = 0.70 \text{ for exposure B} \quad (\text{Table 6-5})$$

$$K_{zt} = 0.72 \text{ (maximum value used)} \quad (\text{Figure 6-2})$$

$$K_d = 0.85 \quad (\text{Table 6-6})$$

$$GC_p = -1.4 \text{ (Tributary area} = 100 \text{ ft}^2\text{)} \quad (\text{Figure 6-5B})$$

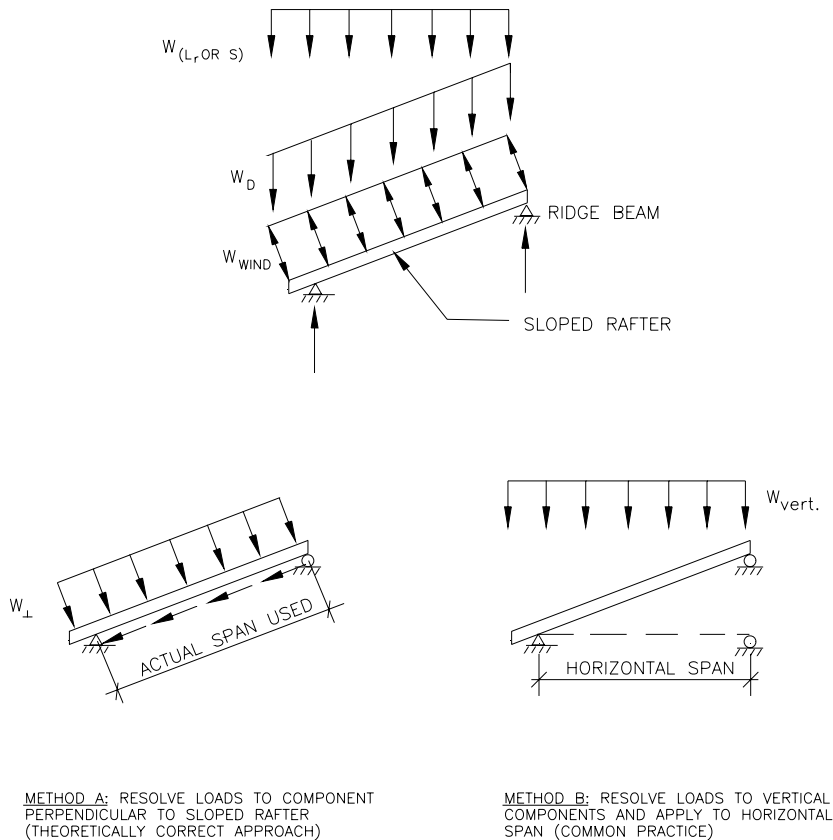
$$GC_p = +0.3 \text{ (Tributary area} = 100 \text{ ft}^2\text{)} \quad (\text{Figure 6-5B})$$

$$GC_{pi} = \pm 0.18 \quad (\text{Table 6-7})$$

$$\text{Downward load (inward): } q = 0.00256(0.70)(0.72)(0.85)(90)^2[0.3 - (-0.18)] = 4.26 \text{ psf}$$

$$\text{Upward load (uplift or outward): } q = 0.00256(0.70)(0.72)(0.85)(90)^2[-1.4 - (0.18)] = -14 \text{ psf}$$

Determine rafter transverse bending and shear loads (using Method B of Figure F2.1).



NOTE: SNOW AND LIVE LOADS IN CODES ARE GENERALLY CONSIDERED RELATIVE TO THE HORIZONTAL AREA. WIND LOADS ACT PERPENDICULAR TO THE BUILDING SURFACE. DEAD LOAD IS A VERTICAL LOAD AND APPLIES TO THE ACTUAL AREA/LENGTH OF THE MEMBER AND SUPPORTED COMPONENTS.

**Figure F2.1 Design Method for a Sloped Rafter**

The wind load acts transverse (i.e., perpendicular) to the rafter; and must be resolved to its components. Generally, the axial component of the gravity load along the rafter (which varies unknowingly depending on end connectivity) is ignored and has negligible impact considering the roof system effects that are also ignored. Also, given the limited overhang length, this too will have a negligible impact on the design of the rafter itself. Thus, the rafter can be reasonably analyzed as a sloped, simply supported bending member. In analyzing wind uplift connection forces at the outside bearing of the rafter, the designer should consider the additional uplift created by the small overhang, though for the stated condition it would amount only to small percentage of the uplift load.

$$\begin{aligned} W_{D, \text{vertical}} &= 15.65 \text{ plf} \\ W_{\text{snow}} &= 60 \text{ plf} \end{aligned}$$



$$\begin{aligned}
 w_{y(\text{upward})} &= -14(\cos 26.56^\circ)^2(2') = -25 \text{ psf} \\
 w_{y(\text{downward})} &= 4.26(\cos 26.56^\circ)^2(2') = 7.62 \text{ psf}
 \end{aligned}$$

#### F2.4 Load Combinations

1.  $1.4D = 1.4(15.65) = 21.91 \text{ plf}$  (downward)
2.  $1.2D + 1.6S = 1.2(15.65) + 1.6(60) = 114.8 \text{ plf}$  (downward)
- 3a.  $0.9D + 1.6W = 0.9(15.65) + 1.6(-25) = -25.9 \text{ plf}$  (upward)
- 3b.  $0.9D + 1.6W = 0.9(15.65) + 1.6(7.62) = 26.3 \text{ plf}$  (downward)

Load combination 2 controls rafter design in inward-bending direction (compression side of rafter laterally supported). Load combination 3a would also need to be checked, as it may control rafter design in outward-bending direction since the compression side now has no lateral bracing unless specified; also important to rafter connections at the bearing wall and ridge member. In this case, load combination 3a was not found to control.

$$M_u = \frac{wL^2}{8} = \frac{114.8(16)^2}{8} = 3,674 \text{ ft-lb}$$

#### F2.5 Member Properties

The calculated design flexure strength, design shear strength and section properties for an 1000S162-54 in accordance with the *Specification* are:

$$\begin{aligned}
 \phi M_n &= 4,499 \text{ ft-lb} \\
 \phi V_n &= 2,524 \text{ lb} \\
 \text{Effective section modulus, } S_{xx} &= 1.6936 \text{ in}^3 \\
 \text{Effective moment of inertia, } I_{xx} &= 9.2502 \text{ in}^4
 \end{aligned}$$

#### F2.6 Bending Capacity

*Specification* Section C3.1.1 nominal section strength applies

$$\begin{aligned}
 \phi_b M_n &= 4,499 \text{ ft-lb} \\
 M &= \frac{wL^2}{8} \rightarrow L = \sqrt{\frac{8M}{w}} = \sqrt{\frac{8(4,423)}{114.8}} \\
 L &= 17.71 \text{ ft} = 17'-9''
 \end{aligned}$$

#### F2.7 Shear Capacity

$$\phi V_n = 2,524 \text{ lb (punched section)}$$

Use the design shear strength to calculate the maximum span.

$$V = \frac{wL}{2} \rightarrow L = \frac{2V}{w} = \frac{2(2,524)}{114.8} = 44'-0''$$

#### F2.8 Deflection Limit

$$\Delta = L/180 \text{ (total loads)}$$

The deflection equation for a simply supported span with distributed load is:

$$\Delta = \frac{5wL^4}{384EI} = L/180$$

$$L = \sqrt[4]{\frac{384EI_x(L/180)}{5w}} = L = \sqrt[3]{\frac{384(29,500,000)(9.2502)}{5(75.65/12)(180)}} = 22'-0"$$

where:

|                  |  |
|------------------|--|
| L =              | Single span length (inches)  |
| I <sub>x</sub> = | Effective moment of inertia for deflection= 9.2502 in <sup>4</sup> |
| w =              | 15.65 + 60 = 75.65 plf   |
| E =              | Modulus of elasticity = 29,500,000 psi                             |

$$\Delta = L/240 \text{ (live loads)}$$

$$\Delta = \frac{5wL^4}{384EI} = L/240$$

$$L = \sqrt[4]{\frac{384EI_x(L/240)}{5w}} = L = \sqrt[3]{\frac{384(29,500,000)(9.2502)}{5(60/12)(240)}} = 21'-7"$$

where:

|                  |  |
|------------------|--|
| L =              | Single span length (inches)  |
| I <sub>x</sub> = | Effective moment of inertia for deflection= 9.2502 in <sup>4</sup> |
| w =              | 60 plf   |
| E =              | Modulus of elasticity = 29,500,000 psi                             |

Therefore, the maximum span for a 1000S162-54 roof rafter for the conditions in this example is 17'-9" (controlled by bending). This confirms the maximum span given in Table F3-1a of the AISI S230.

### F3 Ridge Member Shear Connection

Consider the horizontal projection of a simply supported rafter.

$$V_{\max} = wL/2 \quad (L=16 \text{ ft}, w = 115 \text{ plf})$$

$$V_{\max} = 115 \times 16/2 = 920 \text{ lb}$$

*Screw Shear Capacity* (calculated per Section E4.3 of the *Specification* (AISI, 2001)).

Screw diameter, d = 0.19 inches for a # 10 screw as given in the AISI *Commentary to Specification* Table C-E4-1).

Ultimate capacity of steel = 45 ksi (tensile)

Steel design thickness = 0.0451" (43 mils minimum thickness)

$$\phi = 0.50$$

$t_2/t_1 \leq 1.0$ , the nominal shear strength per screw,  $P_{ns}$ , is the smallest of:

$$P_{ns} = 4.2 \left( \sqrt{t_2^3 d} \right) F_{u2} \quad (\text{Eq. E4.3.1-1})$$

$$P_{ns} = 2.7 t_1 d F_{u1} \quad (\text{Eq. E4.3.1-2})$$

$$P_{ns} = 2.7 t_2 d F_{u2} \quad (\text{Eq. E4.3.1-3})$$

where:  $t_1 = t_2 = 0.0451''$  and  $F_{u1} = F_{u2} = 45 \text{ ksi}$

$$P_{ns} = 789 \text{ lbs.}$$

*Screw Pull-Out Capacity* (calculated in accordance with Section E4.4 of the AISI Specification (AISI, 2001).

$$P_{not} = 0.85(t_c)(d)F_{u2} \quad (\text{Eq. E4.4.1.1})$$

$$P_{not} = 327 \text{ lbs}$$

Therefore the design strength for screw shear,  $\phi P_{ns} = 789(0.5) = 394.5 \text{ lbs}$

Number of screws required =  $920/394.5 = 2.33$  screws, Use 3 screws. This confirms the Table F3-3 requirement in AISI S230 of 3 screws.

#### F4 Ceiling Joist to Rafter Connection

Snow load = 21 psf ( $30 \times 0.7$ )

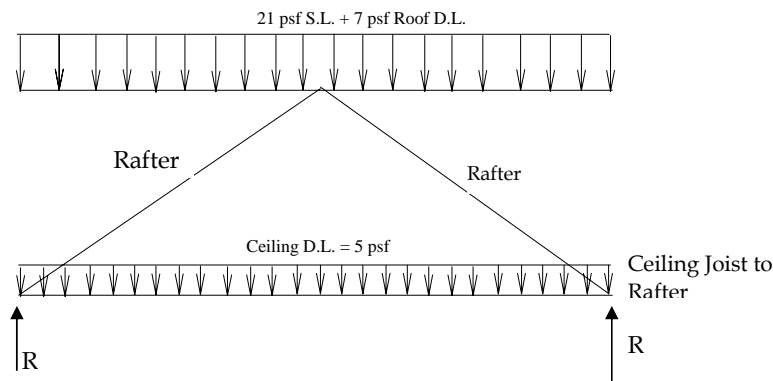
House width = 32 ft

Roof dead load = 7.83 psf

Ceiling dead load = 5 psf

Spacing = 24 in o.c.

No. 10 screw nominal shear capacity = 395 lb (from F3 above)



**Figure F4.1 - Roof Loading Diagram**

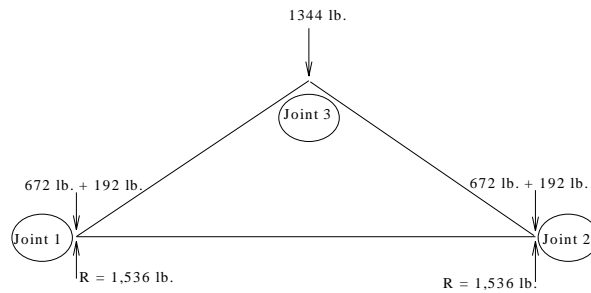
Find the reactions (from Figure F4.1).

$$R = wL/2$$

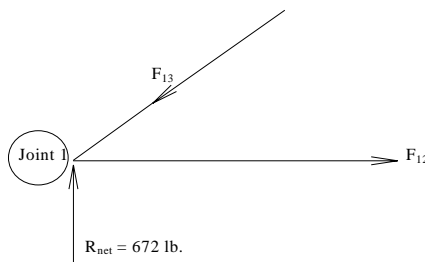
$$R = [(21 \times 1.6 + 7.83 \times 1.2 + 5 \times 1.2) \times 2] \times 32' / 2 = 1,536 \text{ lb}$$

A distributed load of  $[(21 \times 1.6 + 7.83 \times 1.2) \times 2]$  psf or 86 plf is spread across the length of the building. One fourth of this load will be concentrated at the end walls, joints 1 and 2 (86

plf x 8 or 688 lb), and  $\frac{1}{2}$  the load will be concentrated at the ridge member connection, joint 3 (refer to Figure F4.2). Similarly, a distributed load of 12 plf (5 psf x 1.2x2) is spread along the length of the ceiling joist. This load will be divided equally at each end of the wall (i.e. at joints 1 and 2).



**Figure F4.2 - Free Body Diagram**



**Figure F4.3 - Joint 1**

$$\theta = \tan^{-1} (6/12) = 26.565 \text{ degrees}$$

$$F_{13} = 688 / \sin (26.565) = 1538 \text{ lb}$$

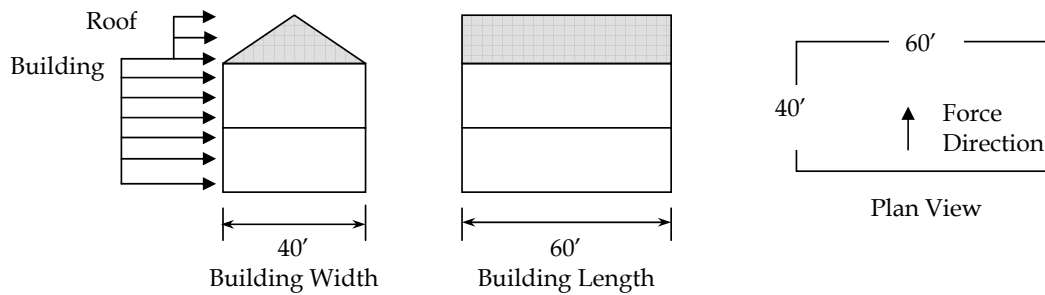
$$F_{12} = 688 \times \cot (26.565) = 1376 \text{ lb}$$

Heel joint connection shall be designed from the compression in  $F_{13}$  since it represents the worst case.

$$\text{Number of screws} = 1538 / 395 = 3.89 \text{ screws} \quad \text{Use 4 screws}$$

This confirms the Table F2-9 requirement in AISI S230 of 4 screws.

### F5 Roof Diaphragm Design (First Example)



Check the adequacy of the roof diaphragm for a 40x60 ft, two-story building with 12:12 roof slope and 8' wall studs, subjected to 110 mph wind speed, Exposure Category C.

Based on ASCE 7 the following wind pressures were obtained for the given wind speed, exposure and roof slope:

|                               |            |
|-------------------------------|------------|
| Roof pressure                 | = 11.7 psf |
| Roof corner pressure          | = 14.6 psf |
| Main building pressure        | = 24.1 psf |
| Main building corner pressure | = 30.3 psf |

Calculate corner area width (2a), where "a" equals 10% of least width or 0.4h (whichever is smaller) but not less than either 4% of least width or 3 feet.

$$\begin{aligned}
 a &= 10\%(40) = 4' \\
 a &= 0.4(30) = 12' \\
 a &= 3'
 \end{aligned}
 \quad \rightarrow \quad
 \begin{aligned}
 &\text{Use } a = 4' \\
 &2a = 8'
 \end{aligned}$$

$$\begin{aligned}
 \text{Shear} &= (60' - 8')(8' / 2)(24.1 \text{ psf}) + 8'(8' / 2)(30.3 \text{ psf}) + (60' - 8')(20')(11.7 \text{ psf}) + 20'(8')(14.6 \text{ psf}) \\
 &= 20,486 \text{ lbs.}
 \end{aligned}$$

$$\text{Roof diaphragm load} = (20,486 / 2) / 40 = 256 \text{ plf}$$

IBC Table 2306.3.1 (ICC, 2000a) provides recommended shear values for wood structural shear panel diaphragms. For 7/16" OSB with 8d-nail spacing, the unblocked shear value is 230 plf (multiplied by wood species reduction factor). The allowable shear values for unblocked diaphragms for 7/16" OSB with No. 8 screws spaced at 6 inches the unblocked shear value is 252 plf (AISI S213-07).

Using the AISI S213 diaphragm value, the diaphragm in this example would be adequate.

### F6 Roof Diaphragm Design (Second Example)

Check the adequacy of the roof diaphragm in Section F5 for a 30x60 ft two-story building.

Calculate corner area width ( $2a$ ), where " $a$ " equals 10% of least width or  $0.4h$  (whichever is smaller) but not less than either 4% of least width or 3 feet.

$$\begin{array}{ll} a = 10\%(30) & = 3' \\ a = 0.4(30) & = 12' \\ a = 3' & \end{array} \quad \rightarrow \quad \begin{array}{l} \text{Use } a = 3' \\ 2a = 6' \end{array}$$

$$\text{Shear} = (60' - 6')(8' / 2)(24.1 \text{ psf}) + 6'(8' / 2)(30.3 \text{ psf}) + (60' - 6')(20')(11.7 \text{ psf}) + 20'(6')(14.6 \text{ psf}) = 20,321 \text{ lb}$$

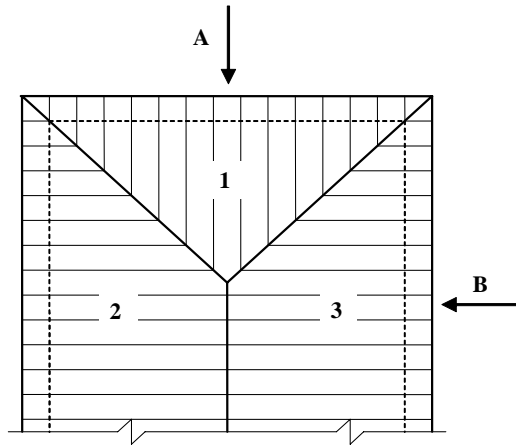
Roof diaphragm load =  $(20,321 / 2) / 30 = 339 \text{ plf} > 252 \text{ plf}$  This is not acceptable and therefore the roof diaphragm must be blocked (380 plf) and therefore an engineered design is required.

## F7 Hip Member Design

Determine if a hip member built-up from a 1000S162-97 and a 1000T150-97 is adequate for the design assumptions listed below. The member is to be checked for all appropriate LRFD load combinations. Bending moment will be calculated based on the horizontal projected span. All references are to the *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2001), unless otherwise noted.

### F7.1 Design Assumptions

Rafter Spacing = 24 in  
 Roof Pitch = 3:12  
 Roof Slope =  $\tan^{-1}(3:12) = 14.04^\circ$   
 Building Width = 32 ft  
 Building Length = 60 ft  
 Building Eave Height = 21 ft (2-story)  
 Wall height = 10 ft  
 Roof Dead Load = 7 psf  
 Wind Speed = 110 mph  
 Wind Exposure Category = C  
 Ground Snow Load = 30 psf



### F7.2 Design Loads:

#### Dead Load:

Uniform Roof Dead Load = 7 psf  
 Total roof dead load on the horizontal projected area of each hip:

$$D = \frac{(16 \text{ ft})^2}{2} \cdot \frac{7 \text{ psf}}{\cos \theta} = 924 \text{ lbs}$$

#### Roof Live Load:

Minimum roof live load = 16 psf  
 Total roof live load on the horizontal projected area:

$$L_r = \frac{(16\text{ ft})^2}{2} (16\text{ psf}) = 2048\text{ lbs}$$

#### Roof Snow Load:

Roof snow load =  $0.7(30\text{ psf}) = 21\text{ psf}$  (Governs)

Total snow load on the horizontal projected area:

$$S = \frac{(16\text{ ft})^2}{2} (21\text{ psf}) = 2688\text{ lbs}$$

#### Wind Loads:

Wind loads are calculated in accordance with ASCE 7 Method 2 – Analytical Procedure for MWFRS (ASCE, 2005).

| Equation:                              | Comments:                  | ASCE 7 Reference:  |
|--|----------------------------|--------------------|
| $p = qGC_p - q_iGC_{pi}$               | Rigid Building of All Hts. | 6.5.12.2, Eq. 6-17 |
| $q = q_i = q_h = 0.00256K_zK_dK_aV^2I$ |                            | 6.5.10, Eq. 6-15   |

Where:

|                      |                                    |                      |
|----------------------|------------------------------------|----------------------|
| $K_z = 0.924$        | Exp. C, Case 1, $z = 23\text{ ft}$ | Table 6-3            |
| $K_{zt} = 1.0$       | No Topographic Factor              |                      |
| $K_d = 0.85$         |                                    | Table 6-4            |
| $V = 110\text{ mph}$ | 3-Second Gust Wind Speed           |                      |
| $I = 1.0$            | Residential Building (Cat I)       | Table 1-1, Table 6-1 |

$$q_h = 0.00256(0.924)(1.0)(0.85)(110)^2(1.0) = 24.3\text{ lb/ft}^2$$

|                                   |                    |                 |
|-----------------------------------|--------------------|-----------------|
| $G = 0.85$                        | Rigid Structure    | Section 6.5.8.1 |
| $GC_{pi} = -0.18\text{ or }+0.18$ | Enclosed Buildings | Figure 6-5      |

Wind pressures for wind direction A ( $h/L = 23/60 = 0.38$ ):

|                                |            |
|--------------------------------|------------|
| Surface 1: $C_p = -0.6, -0.09$ | Figure 6-6 |
| Surface 2: $C_p = -0.9, -0.18$ | Figure 6-6 |
| Surface 3: $C_p = -0.9, -0.18$ | Figure 6-6 |

$$p_1 = 24.3(0.85)(-0.6) - 24.3(\pm 0.18) = \underline{-16.8\text{ psf}}, -8.0\text{ psf} \quad \text{Uplift}$$

or

$$p_1 = 24.3(0.85)(-0.09) - 24.3(\pm 0.18) = \underline{-6.2\text{ psf}}, +2.5\text{ psf} \quad \text{Downward}$$

$$p_2 = p_3 = 24.3(0.85)(-0.9) - 24.3(\pm 0.18) = \underline{-23.0\text{ psf}}, -14.2\text{ psf} \quad \text{Uplift}$$

or

$$p_2 = p_3 = 24.3(0.85)(-0.18) - 24.3(\pm 0.18) = \underline{-8.1\text{ psf}}, +0.7\text{ psf} \quad \text{Downward}$$

Wind pressures for wind direction B ( $h/L = 23/32 = 0.72$ ):

|                                 |            |
|---------------------------------|------------|
| Surface 1: $C_p = -1.08, -0.18$ | Figure 6-6 |
| Surface 2: $C_p = -0.54$        | Figure 6-6 |
| Surface 3: $C_p = -0.83, -0.18$ | Figure 6-6 |

$$p_1 = 24.3(0.85)(-1.08) - 24.3(\pm 0.18) = \underline{-26.7 \text{ psf}}, -17.9 \text{ psf} \quad \text{Uplift}$$

or

$$p_1 = 24.3(0.85)(-0.18) - 24.3(\pm 0.18) = -8.1 \text{ psf}, \underline{+0.7 \text{ psf}} \quad \text{Downward}$$

$$p_2 = 24.3(0.85)(-0.54) - 24.3(\pm 0.18) = \underline{-15.5 \text{ psf}}, -6.8 \text{ psf} \quad \text{Uplift}$$

$$p_3 = 24.3(0.85)(-0.83) - 24.3(\pm 0.18) = \underline{-21.5 \text{ psf}}, -12.8 \text{ psf} \quad \text{Uplift}$$

or

$$p_3 = 24.3(0.85)(-0.18) - 24.3(\pm 0.18) = -8.1 \text{ psf}, \underline{+0.7 \text{ psf}} \quad \text{Downward}$$

Wind direction A governs for downward pressure with  $P_1 = +2.5 \text{ psf}$  and  $P_2 = P_3 = +0.7 \text{ psf}$   
Vertical component of downward wind load:

$$P_1 = 2.5 \text{ psf} \times \cos \theta = 2.4 \text{ psf}$$

$$P_2 = P_3 = 0.7 \text{ psf} \times \cos \theta = 0.7 \text{ psf}$$

Total downward wind load on the horizontal projected area:

$$W_{\text{down}} = \frac{(16 \text{ ft})^2}{4 \times \cos \theta} [2.4 \text{ psf} + (0.7 \text{ psf})] = 205 \text{ lbs}$$

Wind direction B governs for uplift with  $P_1 = -26.7 \text{ psf}$  and  $P_3 = -21.5 \text{ psf}$   
Vertical component of uplift wind load:

$$P_1 = -26.7 \text{ psf} \times \cos \theta = -25.9 \text{ psf}$$

$$P_3 = -21.5 \text{ psf} \times \cos \theta = -20.9 \text{ psf}$$

Total uplift wind load on the horizontal projected area:

$$W_{\text{uplift}} = \frac{16^2}{4 \times \cos \theta} [-25.9 \text{ psf} + (-20.9 \text{ psf})] = -3087 \text{ lbs}$$

### F7.3 Load Combinations

The load combinations listed below will be checked:

1.  $1.2D + 1.6(L_r \text{ or } S) + 0.8W_{\text{down}}$
2.  $1.2D + 1.6W_{\text{down}} + 0.5(L_r \text{ or } S)$
3.  $0.9D + 1.6W_{\text{uplift}}$

Uplift check

Controlling design loads:

Max. Downward (load case 1),  $W_u = 1.2(924 \text{ lbs}) + 1.6(2688 \text{ lbs}) + 0.8(205 \text{ lbs}) = 5573 \text{ lbs}$

Max. Uplift (load case 3),  $W_u = 0.9(924 \text{ lbs}) + 1.6(-3087 \text{ lbs}) = -4107 \text{ lbs}$

### F7.4 Material and Section Properties

The calculated design flexure strength, design shear strength and section properties for a 1000S162-97 and a 1000T150-97 in accordance with the *Specification* are:

$F_y$  (track and C-section) = 50 ksi

$S_x = 6.4479 \text{ in}^3$

$I_x = 33.387 \text{ in}^4$



$$\phi M_n = 271.39 \text{ k-in}$$

$$\phi V_n = 9.542 \text{ k}$$

### F7.5 Bending Check

Maximum downward load governs for bending. Unsupported length of the compression flange for both positive and negative moment is the same. The jack rafters connected to the hip member resists lateral torsional buckling for both gravity loads and uplift.

Total triangular load supported by the hip member:  $W_u = 5573 \text{ lbs}$

Applied bending moment:

$$M_u = 0.128W_u L = 0.128(5573 \text{ lbs})(16 \text{ ft})\sqrt{2}\left(\frac{12}{1000}\right) = 194 \text{ k-in}$$

$$M_u = 194 \text{ k-in} \leq \phi M_n = 271 \text{ k-in} \quad \text{ok}$$

Therefore, the hip member composed of 1000S167-97 and 1000T1500-97 is adequate for bending.

### F7.6 Shear Check

Maximum downward load governs for bending. Unsupported length of the compression flange for both positive and negative moment is the same. The jack rafters connected to the hip member resists lateral torsional buckling for both gravity loads and uplift.

Total triangular load supported by the hip member:  $W_u = 5573 \text{ lbs}$

Applied shear:

$$V_u = \frac{2}{3}W_u = \frac{2}{3}(5573 \text{ lbs}) = 3715 \text{ lbs}$$

$$V_u = \frac{3715 \text{ lbs}}{1000} = 3.715 \text{ k} \leq \phi V_n = 9.54 \text{ k} \quad \text{ok}$$

Therefore, the hip member composed of 1000S167-97 and 1000T1500-97 is adequate for shear.

### F7.7 Deflection Check

Total load deflection limit is  $L/180$  and is based on service dead load plus roof snow.

Total service load supported by the hip member:  $W = 924 \text{ lbs} + 2688 \text{ lbs} = 3612 \text{ lbs}$

$$\Delta_{allow} = \frac{L}{180} = \frac{22.63 \text{ ft} \times 12}{180} = 1.51 \text{ in}$$

$$\Delta_{tot} = \frac{0.01304WL^3}{EI_x} = \frac{0.01304(3612 \text{ lbs})(22.63 \text{ ft})^3(1728)}{29,500,000 \text{ psi}(33.387 \text{ in}^4)} = 0.96 \text{ in}$$

$$\Delta_{tot} = 0.96 \text{ inches} < \Delta_{allow} = 1.51 \text{ inches} \quad \text{ok}$$

Therefore, hip member composed of 1000S162-97 and 1000T150-97 is adequate for deflection.

Therefore, a hip member composed of 1000S162-97 and 1000T150-97 is adequate for the conditions of this hip roof. This confirms the maximum span given in Table F4-1 of *AISI S230*.

## F8 Hip Member Support Column above Ceiling

Determine if a hip member support column built-up from 2-550S162-68 is adequate for the design assumptions from example F7. The member is to be checked for all appropriate LRFD load combinations. All references are to the *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2001), unless otherwise noted.

### F8.1 Design Assumptions

All design assumptions will be the same as used in example F7.

### F8.2 Design Loads

#### Dead Load:

Uniform Roof Dead Load = 7 psf

Total roof dead load on the horizontal projected area:

$$D = \frac{16^2}{\cos \theta} \cdot \frac{2}{3} (7 \text{ psf}) = 1231 \text{ lbs}$$

#### Roof Live Load:

Minimum roof live load = 16 psf

Total roof live load on the horizontal projected area:

$$L_r = 16^2 \cdot \frac{2}{3} (16 \text{ psf}) = 2730 \text{ lbs}$$

#### Roof Snow Load:

Roof snow load =  $0.7(30 \text{ psf}) = 21 \text{ psf}$  (Governs)

Total snow load on the horizontal projected area:

$$S = 16^2 \cdot \frac{2}{3} (21 \text{ psf}) = 3584 \text{ lbs}$$

#### Wind Loads:

Wind loads are calculated in accordance with ASCE 7-2005 Method 2 – Analytical Procedure for MWFRS (ASCE 7, 2005).

Wind pressures for wind direction A ( $h/L = 23/60 = 0.38$ ):

Surface 1:  $C_p = -0.6, -0.09$

Figure 6-6

Surface 2:  $C_p = -0.9, -0.18$

Figure 6-6

Surface 3:  $C_p = -0.9, -0.18$

Figure 6-6

$$p_1 = 24.3(0.85)(-0.6) - 24.3(\pm 0.18) = \underline{-16.8 \text{ psf}}, -8.0 \text{ psf} \quad \text{Uplift}$$

or

$$p_1 = 24.3(0.85)(-0.09) - 24.3(\pm 0.18) = -6.2 \text{ psf}, \underline{+2.5 \text{ psf}} \quad \text{Downward}$$

$$p_2 = p_3 = 24.3(0.85)(-0.9) - 24.3(\pm 0.18) = \underline{-23.0 \text{ psf}}, -14.2 \text{ psf} \quad \text{Uplift}$$

or

$$p_2 = p_3 = 24.3(0.85)(-0.18) - 24.3(\pm 0.18) = -8.1 \text{ psf}, \underline{+0.7 \text{ psf}} \quad \text{Downward}$$

Wind pressures for wind direction B ( $h/L = 23/32 = 0.72$ ):

Surface 1:  $C_p = -1.08, -0.18$

Figure 6-6

Surface 2:  $C_p = -0.54$

Figure 6-6

Surface 3:  $C_p = -0.83, -0.18$

Figure 6-6

$$p_1 = 24.3(0.85)(-1.08) - 24.3(\pm 0.18) = \underline{-26.7 \text{ psf}}, -17.9 \text{ psf} \quad \text{Uplift}$$

or

$$p_1 = 24.3(0.85)(-0.18) - 24.3(\pm 0.18) = -8.1 \text{ psf}, \underline{+0.7 \text{ psf}} \quad \text{Downward}$$

$$p_2 = 24.3(0.85)(-0.54) - 24.3(\pm 0.18) = \underline{-15.5 \text{ psf}}, -6.8 \text{ psf} \quad \text{Uplift}$$

$$p_3 = 24.3(0.85)(-0.83) - 24.3(\pm 0.18) = \underline{-21.5 \text{ psf}}, -12.8 \text{ psf} \quad \text{Uplift}$$

or

$$p_3 = 24.3(0.85)(-0.18) - 24.3(\pm 0.18) = -8.1 \text{ psf}, \underline{+0.7 \text{ psf}} \quad \text{Downward}$$

Wind direction A governs for downward pressure with  $P_1 = +2.5 \text{ psf}$  and  $P_2 = P_3 = +0.7 \text{ psf}$

Vertical component of downward wind load:

$$P_1 = 2.5 \text{ psf} \times \cos \theta = 2.4 \text{ psf}$$

$$P_2 = P_3 = 0.7 \text{ psf} \times \cos \theta = 0.7 \text{ psf}$$

Total downward wind load on the horizontal projected area:

$$W_{\text{down}} = \frac{(16 \text{ ft})^2}{4 \times \cos \theta} \cdot 2 \cdot \frac{2}{3} [2.4 \text{ psf} + (0.7 \text{ psf})] = 273 \text{ lbs}$$

Wind direction B governs for uplift with  $P_1 = -26.7 \text{ psf}$ ,  $P_2 = -15.5$  and  $P_3 = -21.5 \text{ psf}$

Vertical component of uplift wind load:

$$P_1 = -26.7 \text{ psf} \times \cos \theta = -25.9 \text{ psf}$$

$$P_2 = -15.5 \text{ psf} \times \cos \theta = -15.0 \text{ psf}$$

$$P_3 = -21.5 \text{ psf} \times \cos \theta = -20.9 \text{ psf}$$

Total uplift wind load on the horizontal projected area:

$$W_{\text{uplift}} = \frac{(16 \text{ ft})^2}{4 \times \cos \theta} \cdot 2 \cdot \frac{2}{3} \left[ -25.9 \text{ psf} + \frac{(-15.0 \text{ psf} - 20.9 \text{ psf})}{2} \right] = -3857 \text{ lbs}$$

### F8.3 Load Combinations

The load combinations listed below will be checked:

1.  $1.2D + 1.6(L_r \text{ or } S) + 0.8W_{\text{down}}$
2.  $1.2D + 1.6W_{\text{down}} + 0.5(L_r \text{ or } S)$
3.  $0.9D + 1.6W_{\text{uplift}}$

Uplift check

Controlling design loads:

Max. Downward (load case 1),  $W_u = 1.2(1231 \text{ lbs}) + 1.6(3584 \text{ lbs}) + 0.8(273 \text{ lbs}) = 7430 \text{ lbs}$

Max. Uplift (load case 3),  $W_u = 0.9(1231 \text{ lbs}) + 1.6(-3857 \text{ lbs}) = -5063 \text{ lbs}$

#### F8.4 Material Properties

The calculated design flexure strength, design shear strength and section properties for a 2-550S162-68 column in accordance with the *Specification* are:

$$F_y = 50 \text{ ksi}$$

$$A = 1.3148 \text{ in}^2$$

$$A_{\text{net}} = 0.95826 \text{ in}^2$$

$$r_x = 2.0865 \text{ in}$$

$$r_y = 1.3148 \text{ in}$$

$$\phi P_n = 32.594 \text{ k compression}$$

$$\phi P_n = 46.715 \text{ k tension}$$

Unsupported column height:  $h = 16 \text{ ft} \times 3/12 = 4 \text{ ft}$

#### F8.5 Axial Compression

$\phi P_n = 32.594 \text{ k}$  for axial compression.

$$\phi P_n = 32.594 \text{ k} > P_u = 7.430 \text{ k} \quad \underline{ok}$$

#### F8.6 Axial Tension

$\phi P_n = 46.715 \text{ k}$  for axial tension.

$$\phi P_n = 46.715 \text{ k} > P_u = 5.063 \text{ k} \quad \underline{ok}$$

Therefore, a hip member support column composed of 2-550S162-68 is adequate for the conditions of this hip roof. This confirms the maximum span given in Table F4-2 of *AISI S230*.

#### F9 Roof Rafter or Truss to Wall Connection

Calculate the number of screws required for the uplift strap connection between a roof rafter and a wall for a 24-foot roof span, with roof and wall framing at 24" on center and two-story building subjected to 120 mph wind speed, Exposure Category C. The roof pitch is 9:12.

Based on ASCE 7, 9:12 roof pitch and roof corner pressures, wind load = 12.4 psf for 110 mph exposure C. To calculate the vertical component from this force used for uplift the following conversion is made:

$$\text{Wind Load} = 12.4 \text{ psf} (12/9) = 16.54 \text{ psf} \quad \text{at 110 mph}$$

$$\text{Wind Load} = 16.54 \left( \frac{120^2}{110^2} \right) = 19.68 \text{ psf} \quad \text{at 120 mph}$$

| Wind Load               | Building Width (ft)                                 |      |      |      |      |
|-------------------------|---|------|------|------|------|
|                         | 24'   | 28'  | 32'  | 36'  | 40'  |
|                         | Horizontal Length of Member Affected <sup>(1)</sup> |      |      |      |      |
|                         | 14'   | 16'  | 18'  | 20'  | 22'  |
|                         | Load Along Length of Affected Member (lb/ft)        |      |      |      |      |
| 16.54 psf (for 110 mph) | -232  | -265 | -298 | -331 | -364 |
| 19.68 psf (for 120 mph) | -276  | -315 | -354 | -394 | -433 |

<sup>(1)</sup> Half the width of the building plus a 2-ft overhang

Roof Dead Loads:

Ceiling Dead Load = 5 psf (24/2) = 60 lb/ft (for 24' wide building)

Roof Dead Load = 7 psf  $\left(\frac{15}{12}\right)\left(\frac{24+4}{2}\right)$  = 123 lb/ft (for 24' wide building)

Total Dead Load = 183 lb/ft

| Building Width (ft)    |     |     |     |     |
|------------------------|-----|-----|-----|-----|
| 24'                    | 28' | 32' | 36' | 40' |
| Roof Dead Load (lb/ft) |     |     |     |     |
| 183                    | 211 | 238 | 266 | 293 |

Load Combination: 0.9D - 1.6W

Uplift Load = 0.9(183) - 1.6(276) = 165 - 442 = -277 lb/ft (uplift)

At 24" framing spacing: Uplift Load, P = 277(2) = 553 lb

V<sub>all</sub> per No. 8 screw = 165 lb

No. of screws required = 553/165 = 3.36 screws, Use 4-No. 8 screws. This confirms the 4 screws required from *AISI S230* Table F8-2

#### F10 Ridge Tension Strap Connection Requirement

Calculate the number of screws required for the ridge tension strap for a 24-foot span, 6:12 pitched roof subjected to 120 mph wind speed, Exposure Category C and rafter spacing of 12 in o.c.

##### Calculate Wind Load

In accordance with ASCE 7 (ASCE, 1998):

Directionality Factor ( $K_d$ ) = 0.85

(Table 6-6)

Importance Factor = 1.0

(Section 6.5.5)

Topographic Factor ( $K_{zt}$ ) = 1.0

(Figure 6-2)

Velocity Exposure ( $K_z$ ) = 0.98

(Table 6-5)

Velocity Pressure =  $q_z = 0.00256 \times K_z \times K_{zt} \times K_d (V^2 \times I) = 30.7077$

(Eq. 6.5.10)

$$\text{Pressure} = p = q_z[(GC_{pf}) - (GC_{pi})] \quad (\text{Eq. 6.5.12.2.1})$$

**GC<sub>pf</sub> (Figure 6-4) CASE A**

Roof Slope = 26.56 degrees

| Area 1 | Area 2 | Area 3 | Area 4 | Area 1E | Area 2E | Area 3E | Area 4E |
|--------|--------|--------|--------|---------|---------|---------|---------|
| 0.550  | -0.103 | -0.447 | -0.391 | 0.727   | -0.196  | -0.586  | -0.536  |

**GC<sub>pi</sub> (Figure 6-4) CASE A**

| Enclosed Buildings |       |
|--------------------|-------|
| +0.18              | -0.18 |

**GC<sub>pf</sub> (Figure 6-4) CASE B**

Roof Slope = 26.56 degrees

| Area 1 | Area 2 | Area 3 | Area 4 | Area 5 | Area 6 | Area 1E | Area 2E | Area 3E | Area 4E | Area 5E | Area 6E |
|--------|--------|--------|--------|--------|--------|---------|---------|---------|---------|---------|---------|
| -0.45  | -0.69  | -0.37  | -0.45  | 0.4    | -0.29  | -0.48   | -1.07   | -0.53   | -0.48   | 0.61    | -0.43   |

**p (psf) - CASE A**

Roof Slope = 26.56 degrees

| Area 1 | Area 2 | Area 3 | Area 4 | Area 1E | Area 2E | Area 3E | Area 4E |
|--------|--------|--------|--------|---------|---------|---------|---------|
| 22.42  | -8.70  | -19.25 | -17.53 | 27.85   | -11.55  | -23.52  | -21.99  |

**p (psf) - CASE B**

Roof Slope = 26.56 degrees

| Area 1 | Area 2 | Area 3 | Area 4 | Area 5 | Area 6 | Area 1E | Area 2E | Area 3E | Area 4E | Area 5E | Area 6E |
|--------|--------|--------|--------|--------|--------|---------|---------|---------|---------|---------|---------|
| -19.35 | -26.72 | -16.89 | -19.35 | 17.81  | -14.43 | -20.27  | -38.39  | -21.80  | -20.27  | 24.26   | -18.73  |

The above are based on corner length as follows:

|                         |          |                     |
|-------------------------|----------|---------------------|
| 0.4 x Building Height = | 12.00 ft | (height = 30.00 ft) |
| 0.1 x min. Width =      | 2.40 ft  |                     |
| Not less than 3' =      | 3.00 ft  |                     |
| a =                     | 3.00 ft  |                     |
| Corner Length = 2a =    | 6.00 ft  |                     |

Roof Area 2 Factored Wind Load = -26.72 (1.6) = -42.75 psf

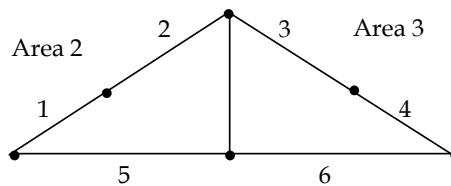
Roof Area 3 Factored Wind Load = -16.89 (1.6) = -27.02 psf

Factored Roof Dead Load = 0.9(7 psf) = 6.3 psf

A plane frame truss analysis yields the following axial loads in the members labeled in the truss model shown.

|          |            |
|----------|------------|
| Member 1 | -327.07 lb |
| Member 2 | -345.97 lb |
| Member 3 | -417.63 lb |
| Member 4 | -398.72 lb |
| Member 5 | 275.58 lb  |
| Member 6 | -1.06 lb   |

Maximum load is in member 3, P = 417.63 lb



$V_{all}$  per No. 8 screw = 165 lb

No. of screws required =  $417.63 = 2.54$  screws, Use 3-No. 8 screws at each end of strap.  
This confirms the 3 screws required from *AISI S230* Table F8-3.

#### **F11 Ridge Tension Strap Design**

Calculate the minimum thickness of 1.25" steel strap with 3-No. 8 screws.

$$T = A (0.6F_y)$$

$$T = 3(165) = 495 \text{ lb}$$

$$495 = [1.25(t)][33,000(0.6)(1.33)]$$

$$t = 0.015 \text{ in}$$

Use 33-mil strap.

This confirms a 33-mil strap required from *AISI S230* Table F8-5.

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## APPENDIX A METRIC CONVERSION

The following list provides the conversion relationship between U.S. customary units and the International System (SI) units. A complete guide to the SI system and its use can be found in ASTM E 380, Metric Practice.

| <b>To convert from</b>     | <b>to</b>                  | <b>multiply by</b> |
|----------------------------|----------------------------|--------------------|
| <b>Length:</b>             |                            |                    |
| inch (in)                  | millimeter (mm)            | 25.4               |
| inch (in)                  | centimeter (cm)            | 2.54               |
| inch (in)                  | meter (m)                  | 0.0254             |
| foot (ft)                  | meter (m)                  | 0.3048             |
| <b>Area:</b>               |                            |                    |
| square foot (sq. ft)       | square meter (sq. m)       | 0.0929             |
| square inch (sq. in)       | square centimeter (sq. cm) | 6.452              |
| <b>Force:</b>              |                            |                    |
| kip (1000 lb)              | Newton (N)                 | 4,448.222          |
| pound (lb)                 | Newton (N)                 | 4.448222           |
| <b>Stress or Pressure:</b> |                            |                    |
| kip/sq. inch (ksi)         | megapascal (MPa)           | 6.894757           |
| pound/sq. inch (psi)       | pascal (Pa) **             | 6,894.757          |
| pound/sq. inch (psi)       | megapascal (MPa)           | 0.00689476         |
| pound/sq. foot (psf)       | pascal (Pa)                | 47.88              |
| <b>Moment:</b>             |                            |                    |
| 1 foot-pound (ft-lb)       | Newton-meter (N-m)         | 1.356              |

\*\* A pascal equals 1000 Newton per square meter

The prefixes and symbols below are commonly used to form names and symbols of the decimal multiples and sub-multiples of the SI units.

| <b>Multiplication Factor</b> | <b>Prefix</b> | <b>Symbol</b> |
|------------------------------|---------------|---------------|
| 1,000,000,000 = $10^9$       | giga          | G             |
| 1,000,000 = $10^6$           | mega          | M             |
| 1,000 = $10^3$               | kilo          | k             |
| 0.01 = $10^{-2}$             | centi         | c             |
| 0.001 = $10^{-3}$            | milli         | m             |
| 0.000001 = $10^{-6}$         | micro         | μ             |
| 0.000000001 = $10^{-9}$      | nano          | n             |



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