COMMENTARY ON THE PRESCRIPTIVE METHOD FOR RESIDENTIAL COLD-FORMED STEEL FRAMING

Second Edition

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Foreword

For centuries homebuilders in the United States have made wood their material of choice because of its satisfactory performance, abundant supply, and relatively low cost. However, increases and unpredictable fluctuations in the price of framing lumber, as well as concerns with its quality, are causing builders and other providers of affordable housing to seek alternative building products.

Use of cold-formed steel framing in the residential market has increased over the past several years. Its price stability; consistent quality; similarity to conventional framing; success in the commercial market; and resistance to fire, rot, and termites have attracted the attention of many builders and designers. But lack of prescriptive construction requirements has prevented this alternative material from gaining wider acceptance among homebuilders and code officials.

Commentary on the Prescriptive Method for Residential Cold-Formed Steel Framing (Commentary) provides the background, supplemental information, engineering assumptions and methods, and detailed calculations that support its companion volume the *Prescriptive Method for Residential Cold-Formed Steel Framing*.

Both volumes were developed under the sponsorship of the U.S. Department of Housing and Urban Development (HUD) through a cooperative agreement with the National Association of Home Builders (NAHB) and the American Iron and Steel Institute (AISI). The program was conducted by the NAHB Research Center with assistance from steering, advisory, and engineering committees. These committees represented the interests and expertise of steel manufacturers, steel producers, code officials, academics, researchers, professional engineers, and builders experienced in cold-formed steel framing.

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COMMENTARY ON THE *PRESCRIPTIVE METHOD* FOR RESIDENTIAL COLD-FORMED STEEL FRAMING

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COMMENTARY ON THE *PRESCRIPTIVE METHOD* FOR RESIDENTIAL COLD-FORMED STEEL FRAMING

PART I

COMMENTARY

COMMENTARY ON THE *PRESCRIPTIVE METHOD* FOR RESIDENTIAL COLD-FORMED STEEL FRAMING

INTRODUCTION

The *Commentary on the Prescriptive Method for Residential Cold-Formed Steel Framing (Commentary)* is provided to facilitate the use of and provide background information for the second edition of the *Prescriptive Method For Residential Cold-Formed Steel Framing (Prescriptive Method)* [1]. The *Prescriptive Method* was developed as an interim guideline for the construction of one-and two-family residential dwellings using cold-formed steel framing. In this C*ommentary*, the individual sections, figures, and tables are presented in the same sequence found in the *Prescriptive Method*. Part I of the *Commentary* documents the assumptions and the derivation of the requirements contained in the *Prescriptive Method*. Part II of the Commentary contains an illustrative example on how to use the provisions of the *Prescriptive Method*. Part III of the *Commentary* contains design examples illustrating the proper application of the different standards and specifications in the *Prescriptive Method*.

C1.0 GENERAL

C1.1 Purpose

The goal of the *Prescriptive Method* is to present prescriptive criteria (tables, figures, guidelines) for the construction of one-and two-story dwellings framed with cold-formed steel members. Prior to this document, there were no prescriptive standards available to builders and code officials for the purpose of constructing cold-formed steel houses without the added expense of a design professional and other costs associated with using a "non-standard" material for residential construction.

The second edition of the *Prescriptive Method* presents the minimum requirements to provide basic residential construction that is consistent with the safety levels provided in the current U.S. building codes governing residential construction.

The *Prescriptive Method* is not applicable to all possible conditions of use and is subject to the applicability limits set forth in Table 1.1. The applicability limits should be carefully understood as they define important constraints on the use of the *Prescriptive Method*.

C1.2 Approach

The requirements, figures, and tables provided in the *Prescriptive Method* are based on the AISI *Specification for the Design of Cold-Formed Steel Structural Members* [2], the American Society of Civil Engineers' (ASCE) *Minimum Design Loads for Buildings and Other Structures* [3], and the pertinent requirements of the CABO *One- and Two-Family Dwelling Code* [4], the *BOCA National Building Code* [5], the *Uniform Building Code* [6], and the *Standard Building Code* [7]. Engineering decisions requiring interpretations or judgments in applying these references are documented in this *Commentary*.

C1.3 Scope

It is unrealistic to develop an easy to use document that provides prescriptive requirements to cover all types and styles of steel-framed houses. Therefore, the second edition of the *Prescriptive Method* [1] is limited in its applicability to light residential construction. The requirements set forth in the *Prescriptive Method* only apply to the construction of cold-formed steel-framed houses that meet the limits set forth in Table 1.1. The applicability limits are necessary to define reasonable boundaries to the conditions that must be considered in developing prescriptive construction requirements. The *Prescriptive Method*, however, does not limit the application of alternative methods or materials through engineering design.

The basic applicability limits were established from industry convention and experience. Detailed applicability limits were documented in the process of developing prescriptive design requirements for various elements of the structure. In some cases, engineering sensitivity analyses were performed to help define appropriate limits.

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. Additional research, design or testing is needed to relax overly-restrictive constraints within the *Prescriptive Method*. Special consideration is directed toward the following items related to the applicability limits.

Building Geometry

The provisions in the *Prescriptive Method* apply to detached one- or two-family dwellings, townhouses, and other attached single-family dwellings not more than two stories in height. Its application to homes with complex architectural configurations is subject to careful interpretation of the user and design support may be required.

The most common house widths (or depths) range from 24 feet to 36 feet, with load bearing wall heights up to ten feet. The house width as used in the *Prescriptive Method* is the dimension measured along the length of the trusses or the joists (floor or ceilings) between the outmost structural walls. A house complying with this document can not be of a length greater than 60 feet. Length is measured in the direction parallel to the roof ridge or floor joists.

Site Conditions

The snow loads are typically given in a ground snow load map such as provided in ASCE 7-93 [3] or by local practice. The major building codes in the U.S. either adopt the ASCE 7 [3] snow load requirements or have a similar map published in the code. The 0 to 70 psf ground snow load used in the *Prescriptive Method* covers approximately 90 percent of the United States', which was deemed to include the majority of the houses that are expected to utilize this document. Houses in areas with higher snow loads should not use this document without consulting a design professional.

All areas of the U.S. fall within the 70 to 110 mph (fastest-mile) range of design wind speeds [3]. The wind exposure category in the *Prescriptive Method* is limited to Exposures A, B, and C. Wind speed and exposure are defined in the *Prescriptive Method*. Wind exposure categories A, B, and C cover the majority of residential site conditions. 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. Houses built along the immediate coastline (i.e. beach front property) are classified as Exposure D and therefore, can not use this document without consulting a design professional. Because of additional engineering concerns and complications in high wind conditions (i.e. greater than 90 mph exposure C), engineering design of wall bracing and building anchorage (i.e. uplift straps and hold-down brackets) is required in these regions (refer to Sections C6.8 and C6.9).

Buildings constructed per the *Prescriptive Method* are limited to regions designated as Seismic Zones 0, 1, 2, 3 and 4. However, because of additional engineering concerns, complications, and limitations in the wall bracing and hold-down requirements, buildings in Seismic Zones 3 and higher may require the services of a design professional to address these particular items.

Loads

Building codes and standards handle loads and load combinations differently. 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 the *Prescriptive Method*. 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.

C1.4 Definitions

The definitions in the *Prescriptive Method* are self explanatory. Additional definitions that warrant some technical explanation, are briefly discussed below.

Design Thickness: The design thickness of the steel members is the minimum uncoated (i.e. not galvanized) material thickness divided by 0.95. Design thickness is used in developing the requirements and provisions of the *Prescriptive Method* (i.e. design thickness is used in calculating section properties, member capacities, and connections capacities). This adjustment is approved in the current AISI Design Specification [2] and it conservatively accounts for the normal variations of material thickness above the minimum required thickness.

***Torsional Rigidity*:** Relatively thin cold-formed C-shaped members are very susceptible to torsional buckling. Due to this concern, bending members (i.e. floor joists or roof rafters) should always be used in a manner consistent with the provisions in the *Prescriptive Method* and the loads associated with conventional, repetitive member framing practices.

Local Buckling and Post Buckling Strength: Cold-formed steel C-shaped members consist of thin elements compared to their overall dimensions, and therefore, tend to buckle or distort at stresses well below the yield point when they are subjected to high compression, bending, shear, or bearing loads. This concern is addressed in the engineering analysis; however, this could become a problem if steel members (studs, headers, etc.) are used in a manner inconsistent with the provisions in the *Prescriptive Method* and the conventional practices of repetitive member-framing.

Tests at the NAHB Research Center [25] [31] have shown that cold-formed elements fail (local buckling) at loads higher than predicted when used in a manner consistent with the *Prescriptive Method*. In fact, the elements tend to carry loads above their individual local buckling strength because of the load re-distribution experienced in repetitive member framing. Therefore, the post buckling strength is utilized in the design of cold-formed members for use in the *Prescriptive Method*.

C2.0 MATERIALS, SHAPES AND STANDARD SIZES

C2.1 Types of Cold-Formed Steel

C2.1.1 Structural Members

In the *Prescriptive Method*, acceptable structural members are standard C-shapes produced by roll forming hot-dipped metallic coated sheet steel conforming to one of the four categories of steel presented in the *Prescriptive Method*, and in accordance with the American Iron and Steel Institute (AISI) "Specification for the Design of Cold-Formed Structural Steel Members" [2] requirements. These four categories are: American Society for Testing and Materials (ASTM) A 653 [8], A 792 [9], A 875 [10], and steels conforming to A 653 [8] with the exception of certain elongation and tensile strength to yield strength ratio requirements. Tracks are commonly used in conjunction with the standard C-shaped members specified in the *Prescriptive Method.* Steel tracks consist of channels (i.e. U-shaped) that are also rolled or brake formed from hot-dipped metallic coated sheet steel.

C2.1.2 Non-Structural Members

ASTM C 645 [11] is currently referenced in the three model building codes and is used in the *Prescriptive Method* as a resource for construction of non-load bearing walls (i.e. partitions). By definition, non-structural members may only be used in non-load bearing applications.

C2.2 Physical Dimensions

Because of significant variation in the industry related to dimensions of cold-formed steel members, dimensions were standardized (i.e. web depth, flange width, lip size, uncoated steel thickness, bend radius, and yield strength) to allow for the development of the *Prescriptive Method.* Dimensional standardization was focused on the C-shaped member because of its widespread use in the industry.

Web Depth: The actual web sizes in the *Prescriptive Method* are limited to 3-1/2 inches, 5-1/2 inches, 8 inches, 10 inches, and 12 inches. The 3-1/2 and 5-1/2 inch 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. The size of the web for 8, 10, and 12-inch members are not of great significance because they are typically used for horizontal framing members (i.e. headers and joists).

Flange Width: The *Prescriptive Method* requires the standard C-shape have a minimum of 1-5/8 inch flange with a maximum flange dimension of 2 inches. A review of current producers of cold formed steel members showed that 1-5/8 inch flanges are widely used. The range of flange widths, from 1-5/8 to 2 inches, encompasses the majority of C-shaped products in the roll forming industry, when combined with the above web depths. An increase in flange size above the 2 inch maximum limit may result in decreased capacity for certain members.

Lip Size: The *Prescriptive Method* also provides a minimum size for the stiffening lip of 1/2 inch. This dimension is also common in the industry. Sensitivity analyses performed to optimize the lip dimension showed that a slight increase in capacity results from a 5/8 inch or 3/4 inch lip size in certain conditions. However, decreasing the lip size had a detrimental effects in many circumstances. Therefore, the 1/2 inch lip dimension is considered an optimal minimum requirement and basis for engineering analysis.

The industry designation column shown in Table 2.1 of the *Prescriptive Method* provides the terminology that the steel industry has agreed upon for designating the different steel members. For example a 350S162-33 designation indicates a steel stud (S) with 3.500 inch (3-1/2 inch) web, 1.625 inch (1-5/8 inch) flanges, and 33 mil thickness. A 350T162-33 indicates a track (T) with 3.500 inch (3-1/2 inch) web, 1.625 inch (1-5/8 inch) flanges, and 33 mil thickness. This industry designator is slightly different from the labeling section requirement because the intent for the industry designation is one method of meeting the labeling (or identification) requirements in the *Prescriptive Method*.

For engineering purposes, the section properties and an example on how to calculate these properties for the standard C-shapes are shown in PART III. These section properties were used in the engineering calculations to develop the *Prescriptive Method*.

The *Prescriptive Method* requires steel tracks to have a minimum flange dimension of 1-1/4 inches. This dimension is widely used by the steel industry and 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 properly nest into the track sections. Steel tracks are also available in thicknesses matching those required for the standards C-shapes. In the *Prescriptive Method*, tracks are always required to have the same thickness as the wall, floor, or roof structural members to which they are attached.

C2.3 Uncoated Material Thickness

The steel thickness required by the *Prescriptive Method* is the minimum uncoated material 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 current practice which uses a gauge designation for thickness. The gauge represents a range of thicknesses and is therefore, a vague unit of measure when specifying minimums. In order to achieve consistency, the "mil" designation was adopted. Therefore, the 33 mils (20 gauge), 43 mils (18 gauge), 54 mils (16 gauge), 68 mils (14 gauge), and 97 mils (12 gauge) are specified for the thicknesses.

The design thickness (shown in Table C2.3 below) is the value used in the computation of member capacities. The reduction in thickness that occurs at corner bends is purposefully ignored, and the design thickness of the flat steel stock, exclusive of coatings, is used in the structural calculations. The design thickness is defined as the minimum delivered thickness divided by 0.95 which follows the provisions of the AISI Design Specification [2]. This adjustment reasonably accounts for the normal variation in material thickness above the minimum delivered material thickness required. The reader is referred to Part III of this document for engineering calculations illustrating the use of the design thickness in the computation of section properties.

C2.4 Bend Radius

The bend radius is measured from the inside of bends in cold-formed steel members. It has an impact on the capacity of structural steel members. As the inside bend radius increases, the effective flat width of the element decreases and thus, the member's capacity decreases. Conversely, strength increases are realized in the regions of bends due to a phenomena known as cold-working which locally increases the yield strength of the steel. The steering committee established the maximum bend radius to be the greater of 3/32-inch or twice the material thickness. This requirement was established to provide manufacturers with a reasonable tolerance without adversely affecting the strength of the member. The maximum bend radii (shown in Table C2.4 below) were used in the computations of member capacities for the *Prescriptive Method*. The reader is referred to (Part III) for the engineering calculations illustrating the use of the bend radius in the computation of section properties.

Table C2.3
Material Thickness

Designation (mils)	Minimum Delivered Thickness (inches)	Design Thickness (inches)	Reference Gauge Number
18	0.018	0.019	25
27	0.027	0.028	22
33	0.033	0.035	20
43	0.043	0.045	18
54	0.054	0.057	16
68	0.068	0.071	14
97	0.097	0.102	12

For SI: 1 inch = 25.4 mm.

Table C2.4
Maximum Bend Radius

Designation (mils)	Minimum Delivered Thickness (t) (inches)	Maximum Bend Radius	Maximum Bend Radius (inches)
18	0.018	3/32 in.	0.0937
27	0.027	3/32 in.	0.0937
33	0.033	3/32 in.	0.0937
43	0.043	3/32 in.	0.0937
54	0.054	2 x thickness	0.108
68	0.068	2 x thickness	0.136
97	0.097	2 x thickness	0.194

For SI: 1 inch = 25.4 mm.

C2.5 Yield Strength

The *Prescriptive Method* applies to steels with minimum yield strengths of 33 ksi or 50 ksi. The 33 ksi steels are the minimum required for all steel floors, roofs, and header components. Steel stud tables are provided for both 33 ksi and 50 ksi minimum yield strength. The 50 ksi yield strength steel was included as separate option for wall studs selection because of the notable economic benefit in this particular application.

Strength increase from cold working is utilized for the standardized C-shaped members in the *Prescriptive Method* concerning the calculated bending strength of flexural members, concentrically loaded compression members, and members with combined axial and bending loads. The reader is referred to Part III for engineering calculations illustrating the stress increase due to cold-working and it's use in calculating section properties.

C2.6 Corrosion Protection

In residential construction, it is <u>not</u> considered likely for any structural steel framing to be exposed to a severe corrosive environment, except for locations close to coastal areas. The degree of protection is determined by considering the use of the member, its exposure, climate, and other conditions. Metallic coating is required to ensure that the steel framing structure will perform its required function during its expected life.

The minimum galvanizing coatings of G-40 for interior non-structural (non-load bearing) members, and G-60 for structural members provides adequate protective coating for steel-framed members in normal environments. The user should be cautious in coastal areas and other severe climatic conditions, where greater corrosion protection may be required (for example G-90 may be recommended). The *Prescriptive Method* provides the minimum coating designation and references the applicable ASTM standards. The requirements for metallic coatings are given in ASTM A924 [12]. Users of cold-formed steel framing are encouraged to locate (store or construct) steel framing members within the protection of the building envelope and adequately shield steel materials from direct, long-term contact with moisture, from the ground or the outdoor climate.

The *Prescriptive Method* includes a provision permitting the usage of other metallic coatings that are equivalent to the galvanized coating. However, only coatings that provide sacrificial protection are permissible. Sacrificial protection is necessary to protect bare steel from corrosion at locations where the steel has been penetrated (e.g. holes).

Table C2.6 provides coating designation references that are obtained from the applicable ASTM Standards.

Table C2.6
Coating Designation Reference

Material / Coating	Minimum Triple Spot Total - Both Sides (oz/ft^2)	Minimum Single Spot Total- Both Sides (oz/ft^2)	Minimum Triple Spot Total - Both Sides (g/m^2)	Minimum Single Spot Total- Both Sides (g/m^2)	Coating Thickness- Both Sides (microns)
AZ50 galvalume	0.50	0.43			40.7
AZ55 galvalume	0.55	0.50			44.8
AZ60 galvalume	0.60	0.52			48.8
AZ150 galvalume			150	130	40.0
AZ165 galvalume			165	150	44.0
AZ180 galvalume			180	155	48.0
G40 galvanized	0.40	0.30			17.1
G60 galvanized	0.60	0.50			25.6
G90 galvanized	0.90	0.80			38.5
Z120 galvanized			120	90	16.8
Z180 galvanized			180	150	25.2
Z275 galvanized			275	235	38.5
GF45 galfan	0.45	0.35			20.5
GF60 galfan	0.60	0.50			27.3
GF75 galfan	0.75	0.65			34.2
ZGF135 galfan			135	113	20.1
ZGF180 galfan			180	150	26.9
ZGF225 galfan			225	195	33.6

Densities: galvalume - 3.75 g/cm^3; galvanized - 7.1 g/cm^3; galfan - 6.70 g/cm^3.
Reference ASTM Standards for galvanized , galvalume, and galfan: A653 [8], A792 [9], and A875 [10] respectively.

C2.7 Web Holes

The AISI Design Specification [2] does not address web holes (punchouts) beyond small circular holes. Web holes, typically 1-1/2 inch wide x 4 inch long are common in the cold-formed steel industry to accommodate plumbing, electrical, and mechanical materials. Floor joists typically require larger holes to accommodate sanitary, drain lines, vents, and other utilities.

Floor and Ceiling Joists:

Floor and ceiling joists used in the *Prescriptive Method* are all designed assuming maximum hole dimensions and minimum spacings as shown in Table 2.4 of the *Prescriptive Method*. The width of the hole is measured along the depth of the web and the length of the hole is measured parallel to the length of the member. The span tables are calculated based on the presence of these holes. The design procedure follows the “Design Guide For Cold-Formed Steel Beams With Web Perforations” [13] which provides a criteria for calculating the allowable axial and bending load of perforated members subject to bending and shear, such as floor joists. The tabulated hole dimensions and spacing shown in Table 2.4 are based on a web hole width-to-web depth ratio not exceeding 0.4 and a maximum hole length of 2.67 times the width of the hole, but not greater than 6 inches.

Studs, header members, rafters, and other structural members:

Studs, header members, rafters, and other structural members (except floor and ceiling joists) used in the *Prescriptive Method* are all assumed punched with a standard hole of 1-1/2 in. wide x 4 in. long (the width of the hole is along the depth of the web and the length of the hole is parallel to the member). The span tables are calculated based on the presence of standard holes with a minimum center-to-center spacing equal to three times the outside depth of the stud section, but need not exceed 24 inches. A minimum distance of 12 inches from center of hole to end of member is also required. The design procedure follows International Conference of Building Officials (ICBO) AC46 "Acceptance Criteria For Steel Studs And Joists" [14]. ICBO AC46 provides a criteria to calculate the allowable axial and bending load of a perforated wall stud or a structural member. The example in Part III of this document illustrates the use of this procedure in calculating the reduction in the allowable shear capacity due to punchouts.

The ICBO AC46 approach has the following limitations:

Center to center spacing of the perforations (holes) shall be at least three times the outside depth of the member, but need <u>not</u> exceed 24 inches.

Perforation width (across the web) shall not exceed half the member depth or 2-1/2 inches. Perforation length shall not exceed 2.67 times the hole width or 4.5 inches.

Minimum distance between the end of the stud and the near edge of the perforation shall be 10 inches.

All web perforations are located along the center line of the web.

C2.8 Cutting, Notching, and Hole Patching

As mentioned before, the webs of all structural members, in the *Prescriptive Method*, were assumed to have holes when calculating member capacities. Structural members having holes that do not conform to the tabulated holes shown in Tables 2.4 or 2.5 need to be reinforced to achieve the structural capacity used in the design. The hole patch detail is applicable only if the hole depth does not exceed half the depth of the web, and the hole length (along the web) does not exceed the depth of the web or 6 inches, whichever is greater. The hole patching detail results in a member that has an equivalent capacity to an un-patched member with standard holes.

Cutting or notching of load bearing members are not allowed in the *Prescriptive Method* because this condition will greatly reduce the structural capacity of the member

C2.9 Bearing Stiffeners

Webs of thin walled cold-formed steel members may cripple or buckle locally at concentrated load or bearing reaction. 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.

Bearing stiffeners (also called transverse or web stiffeners) are required at all support or bearing point locations for all floor joists and where required for ceiling joists. The stiffeners are specified to be a minimum of 43 mil track section or 33 mil C-shaped member. These sections were specified because their axial load carrying capacity exceed that of the concentrated load (from top floors , walls, and roofs) and the additional reaction (from the imposed load on the joists).

C2.10 Clip Angles

The clip angles used in the *Prescriptive Method* are commonly used in the residential steel framing market.

C2.11 Fasteners

This section is not intended to limit the fastening techniques to those listed. Other fastening methods are permitted to be used with this document provided that the connection's capacity is shown to exceed that implied in the *Prescriptive Method*. Testing, design, or code approvals may be necessary for alternate fastening techniques.

C2.11.1 Screws

Fastening of cold-formed steel framing members is limited to screws in the *Prescriptive Method*. Self-drilling tapping screws conforming to the requirements of the Society of Automotive Engineers (SAE) J78 [15] are used extensively in the *Prescriptive Method.* There is a need for additional standards for screws related to applications with cold-formed steel framing. Requirements for sharp point screws, connecting gypsum board and sheathing to steel studs, are found in ASTM C 1002 [16] and ASTM C 954 [17]. An edge distance and center-to-center spacing of three screw diameters follow industry recommendations, the AISI guidelines [18] and the University of Missouri-Rolla Center for Cold-Formed Steel Structures' (CCFSS) Technical Bulletin [19].

The minimum screw size specified in this document is #8 for structural applications, however, larger diameter screws may be required to provide drill capacities through steel thicknesses greater than 100 mils (refer to Table 2.6 in the *Prescriptive Method*). In certain applications, # 10 screws are specified in the *Prescriptive Method* for practical purposes and added capacity. The point style of the screw will affect the constructability in certain applications. 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. For these reasons, screw manufacturer recommendations should be consulted.

Driveability of screws in different thicknesses of steels have been an issue with end users. This issue is addressed in the *Prescriptive Method* by providing a table listing the maximum steel thickness into which a particular self-drilling, tapping screw can be driven without difficulty. The maximum steel thickness represents the total thickness of the connected parts (refer to Fig. C2.11). This information is provided in the *Prescriptive Method* because the driveability of screws vary. Again, the user should consult with screw manufacturers for proper fastener application.

Screw capacities given in the table below (Table C2.11.1) are calculated based on the design method given in the CCFSS document [19]. The CCFSS document provides the equations necessary to calculate the shear, pullover, and pullout capacity of a connection based on the thicknesses of the steel being fastened together. The equations are conservatively based on tests performed on thousands of screws of many different types and levels of quality. An example of engineering calculations to determine the capacity of a screw is provided in Part III of this document. Table C2.11.2 provides a list of typical fastener type and application used in steel construction.

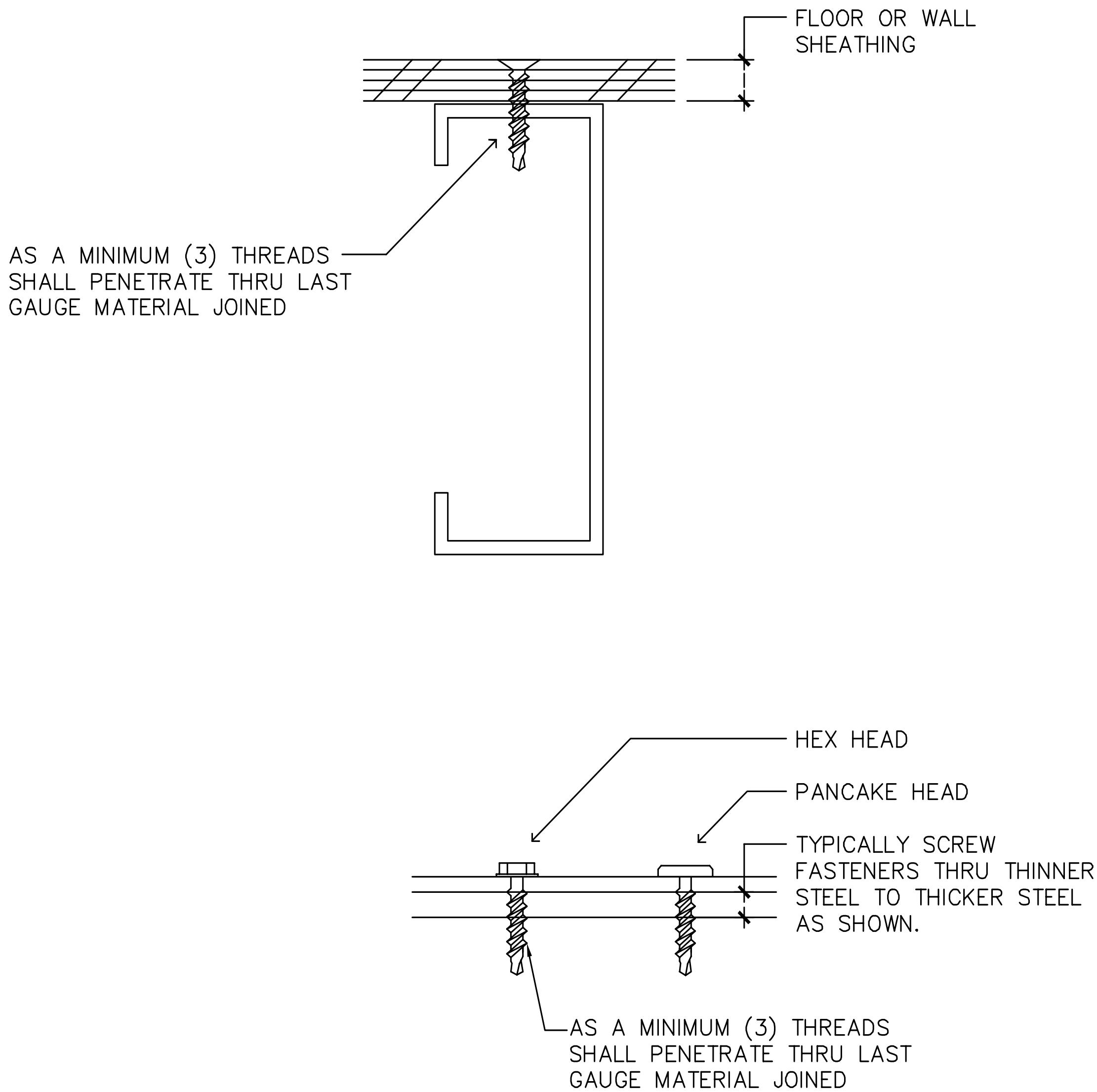


Figure C2.11 Screw Attachments

Table C2.11.1
Minimum Allowable Fastener Capacity for Steel to Steel Connections
[Safety factor = 3.0]

Screw Size	Minimum Shank Diameter (inches)	Minimum Head Diameter (inches)	Minimum Capacity (lbs.)			
			Shear Capacity		Pullout Capacity	
			43 mils [1]	33 mils [1]	43 mils [1]	33 mils [1]
#8	0.164	0.322	244	164	94	72
#10	0.190	0.384	263	177	109	84

For SI: 1 inch = 25.4 mm, 1 lb. = 4.5 N

1 The value represents the smaller thickness of two pieces of steel being connected.

Table C2.11.2
Typical Fasteners Used with Steel Framing

Application	Fastener
Steel to steel non-load bearing (less than 33 mils)	Minimum #6, sharp point, low-profile head.
Steel to steel load bearing	Minimum #8, self-drilling, low-profile head where gypsum board or sheathing is to be installed; otherwise, a hex head can be used.
Gypsum board	Minimum #6, sharp point/self drilling[1], bugle-head screws.
Interior trim	#6 minimum, sharp point/pilot point,[1] finish or trim-head screws. If wood blocking is installed use finishing nails.
Foam insulation	Roofing nails to structural sheathing, or minimum #6, self drilling, bugle head with a washer to prevent the screw from pulling through the foam.
OSB/Plywood	Minimum #8, sharp point/self drilling[1], bugle-head screw. Winged screws and pneumatic fasteners are also available.
Stucco lath	Nail lath to wood sheathing or screw through foam backing to stud with #8 minimum drill point, low profile.
Siding	Minimum #8, sharp point/self drilling[1], bugle head. Winged screws are also available.
Brick ties	Minimum #8, sharp point/self drilling[1], hex-head screws.
Exterior trim	Minimum #8, sharp point/self drilling[1], bugle-head or trim-head fastener.

[1] Use sharp point for studs up to 33 mils (0.84 mm) and self-drilling point for thicker steel.

C2.10.2 Bolts

Bolts used in cold-formed steel framing are specified to meet or exceed the requirements of ASTM A307 [20]. Similar to screws, bolt edge distance and center-to-center spacing are in accordance with industry standards.

C3.0 LABELING

The labeling (identification) system specified in the *Prescriptive Method* is a minimum requirement. Additional information may be added to the stamp, logo, stencil, or embossment. The requirements in this section closely follow what is currently proposed by the industry. The labeling requirements are necessary for building code enforcement, construction coordination, and quality control. The following is an example of a possible labeling stamp.

C4.0 FOUNDATION

The *Prescriptive Method* does not provide prescriptive requirements for foundation construction. It references the model building codes for the design and installation of the foundation. The *Prescriptive Method* provides prescriptive requirements for the connection of steel framing to the foundation in Sections 5.0 and 6.0. Anchor bolts (i.e. J-bolts) must be installed in concrete in accordance with current building code requirements.

C5.0 FLOORS

C5.1 Floor Construction

The *Prescriptive Method* shows a typical floor construction layout with the different elements of the floor identified. Floor construction in the *Prescriptive Method* is limited to design wind speeds less than or equal to 110 mph (fastest-mile) and to Seismic Zones 0, 1, 2, 3, and 4. However, as mentioned in the applicability limit table of the *Prescriptive Method* (Table 1.1), an approved design will be required for hold-down requirements, and/or uplift straps for steel floors in buildings subjected to wind speeds greater than 90 mph, Exposure C, and buildings located in Seismic Zone 3 or higher. This limitation is a result of the additional design required for connections of the floor to the foundation or bearing walls. In-line framing is also emphasized in the *Prescriptive Method.* The maximum tolerance between the centerlines of the floor joists and the studs is limited to a maximum of 3/4-inch. The tolerance given in is the maximum and at no time should be exceeded without consulting a design professional.

C5.2 Floor to Foundation or Bearing Wall Connection

The *Prescriptive Method* provides three different details for connecting floor joists to foundations or load bearing walls, three different details for connecting cantilevered floors to foundations or load bearing walls, one detail for lapped joists and one detail for continuous joist installation. The details are self explanatory and reflect a selection from current practice.

C5.3 Allowable Joist Spans

The *Prescriptive Method* provides floor joist tables with maximum allowable spans for two live load conditions: 30 psf and 40 psf. The spans shown in the *Prescriptive Method* assume bearing stiffeners are installed at each bearing point. Bearing (web) stiffener requirements are also provided in Section 2.9 of the *Prescriptive Method*. An example of the calculation of the maximum allowable joist span is shown in Part III of this document.

In the design of floor joists, any one of several engineering criteria may control the prescriptive requirements depending on the configuration of the section, thickness of material, and member length. The analysis used in the *Prescriptive Method* includes checks for:

- Yielding
- Flexural buckling
- Web crippling
- Shear
- Deflection
- Combined bending and shear (for multiple spans)

All joists are considered punched in accordance with Table 2.4. The compression flange (top flange) of the floor joists are also assumed to be continually braced by the subflooring, thus providing lateral restraint for the top flanges.

The joist span tables are calculated based on deflection limit of L/480 for live load and L/240 for total loads, where L is the span length. Building codes vary in requirements for deflection criteria. Also, deflection requirements which have worked well in the past for one type of material or method have created problems when applied to others. One particular serviceability problem is related to floor vibrations and many practitioners and standards use more stringent deflection criteria than the L/360 typically required for residential floors.

Deflection limits are primarily established with regard to serviceability concerns. The intent is to prevent excessive deflection which might result in cracking of finishes. The deflection criteria also affects the "feel" of the building in terms of rigidity and vibratory response to normal occupant loads. For a material like steel, which has a high material strength, longer spans are possible with members of lower apparent stiffness (i.e. E x I). In such cases, typical deflection criteria may not be appropriate. For example, industry experience indicates that an L/360 deflection limit often results in these floors being perceived to be "bouncy" by occupants. This condition may be misconstrued as a sign of weakness by the occupant.

While a deflection-to-span- ratio of L/360 may be adequate under static loading, it is suggested that a significantly tighter deflection-to-span ratio under the full design live load only may be appropriate to ensure adequate performance. The deflection-to-span ratio has always been a controversial issue when it is used to assess human comfort. A higher deflection limit is usually recommended to overcome the concern with nuisance vibrations. To ensure dynamic performance, the Australians [21] for example, consider a deflection-to-span ratio of L/750 (under full live loading) to be appropriate in the absence of full dynamic analysis. Furthermore, the Australians provide a criterion to determine what is an acceptable "house" system, based on critical damping. This method is not generic and requires the calculation of the first natural frequency and percent damping of the floor which depend on the physical dimensions, stiffness, and attachments of the house. Due to this fact, a more simplistic approach would be to tighten the deflection criteria. A floor deflection-to-span ratio of L/480 typically results in an increase in the percent of critical damping as suggested by the AISC, and thus ensures that vibration does not exceed a tolerable level. Many engineers apply a L/480 deflection criteria when designing steel floor joists.

A recent research program initiated an effort to develop an accurate method to predict steel floor vibration performance [22]. A method was proposed to determine the vibration acceptability of cold-formed steel, C-joist supported residential floor systems which depends only on the dimensions and cross sectional properties of the floor. While appears to provide a relatively consistent vibration performance criterion it is not adopted in the *Prescriptive Method* at this time. The reader is referred to reference 22 for a detailed description of the method and its application. Part III of this document summarizes the recommended method and provides an example on its application.

***Multiple Spans*:**

Multiple spans are commonly used in the residential steel building market. When multiple spans are used certain measures are necessary to address the reactions of the loaded members. The first reaction to address is the magnitude of the reaction at the middle support which is greater than the end reactions. This may create web crippling failure at the middle support. This is resolved by requiring web stiffeners at all bearing points. The second reaction is the presence of negative moments (i.e. reversed bending) at the middle reaction region, causing the compression flanges to be at the bottom rather than the top of the joists. If left unbraced, this would cause lateral instability and may cause premature failure of the joists under extreme loading conditions. Furthermore, shear and bending interaction are checked for multiple spans due to the presence of high shear and bending stresses at the middle reactions, creating greater susceptibility of web buckling.

Bottom flange bracing at interior supports is provided by ceiling finishes (when present) and by positive connection to the interior bearing wall. Possible benefits from composite action with the floor diaphragm were not utilized in the development of the *Prescriptive Method*.

C5.4 Joist Bracing

Steel floors have long been designed by considering the joists as simple beams acting independently without consideration of composite action from floor sheathing. For typical residential floors, it has been assumed that the floor sheathing's only function is to transfer the loads to the joists, and to provide continuous lateral bracing to the compression flanges neglecting many factors that affect the strength and stiffness of a floor. Testing has indicated that using a single joist for strength calculation agrees with actual behavior when uniform loads are applied [23].

Bracing the bottom flanges of joists as specified in the *Prescriptive Method* is based on industry practice and engineering judgment. Steel strapping as well as finished ceilings are considered to be adequate bracing for the tension flanges. It is necessary, however, for steel strapping to have blocking (or bridging) installed at a maximum spacing of 12 feet 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 or blocking (i.e. to concrete wall or foundation).

C5.5 Floor Cantilevers

Cantilevered floor joists are used often. In many cases, cantilevers support load bearing walls which create special loading conditions that require separate engineering analysis. In the *Prescriptive Method*, floor cantilevers are limited to a maximum of 24 inches 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, holes are not permitted in cantilevered portions.

Floor cantilevers supporting two stories are permitted provided that the floor joists are doubled (back to back C-shaped members screwed with two # 8 screws at minimum of 24 inches on center along the centerline of the webs) and the cantilevered section does not exceed 24 inches.

C5.6 Splicing

Splicing of structural members is not a typical application, but some situations may arise where splicing requirements would be useful. Applications may include repair of damaged joists, simplified details for dropped floors, and others.

Splices, generally, are required to transfer shear, bending moments, and axial loads. Some splices may occur over points of bearing, and splicing may only be required to transmit nominal axial loads. The Floor joist spans provided in the *Prescriptive Method* are based on the assumption that the joists are continuous, with no splices Therefore, splicing of joist members in the *Prescriptive Method* requires an approved design except when lapped joists occur at interior bearing points.

C5.7 Framing of Openings

Openings in floors are needed for several reasons (e.g. chimneys). The *Prescriptive Method* limits the width of the floor opening width to 8 feet 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 a C-shaped joist into a track and fastening them together along the top and bottom flanges. These built-up members are required to have a minimum size and thickness not less than the floor joists. Each header joist is required to be connected to the trimmer joist with a clip angle on either side of each connection. The clip angle is required to be of a thickness equivalent to the floor joists.

C5.8 Floor Trusses

This section is included so that pre-engineered floor trusses can be used in conjunction with this document. The American Iron and Steel Institute (AISI) has developed a Design Guide For Cold-Formed Steel Trusses [24] to assist in truss design.

C6.0 STRUCTURAL WALLS

C6.1 Wall Construction

The *Prescriptive Method* covers wall studs in buildings within the applicability limits of Table 1.1. The maximum design conditions are Seismic Zone 4, ground snow load of 70 psf, and design wind speed of 110 mph fastest-mile in Exposure C conditions (i.e. open terrain).

C6.2 Wall to Foundation or Floor Connection

The *Prescriptive Method* provides commonly used details to connect wall studs to foundations or bearing walls. The details are self explanatory and reflect current practice in residential steel framing.

C6.3 Load Bearing Walls

The *Prescriptive Method* provides the minimum required thicknesses of steel studs for different wind speeds, exposure categories, wall heights, building widths, and ground snow loads. Stud selection tables are limited to one- and two-story buildings with load bearing wall heights up to 10 feet.

The 8-foot walls are widely used in residential construction, however, steel framed homes often take advantage of higher ceilings such as 9- and 10-foot walls. The 50 ksi (steel's yield strength) stud tables were developed to take advantage of the higher yield strength which allows thinner studs in many cases.

The wall studs are grouped in two categories:

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

For walls sheathed with a wood structural panel (7/16-inch OSB or 15/32-inch plywood), a reduction in thickness of the stud is allowed by footnote in the stud selection tables for 33 ksi steel. These reductions have been investigated in recent testing of wall assemblies under combined bending and axial loads conducted at the NAHB Research Center [25]. All studs in exterior walls are treated as load bearing members for the purpose of simplifying the *Prescriptive Method*. The following design assumptions were made in analyzing the wall stud selection tables.

- Studs are simply supported beam - columns.
- 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.)
- Maximum roof overhang of 24 inches.
- Roof slopes limited to a range of 3:12 to 12:12.
- Deflection Criteria: L/240
- Ceilings, roofs, attics, and floors span the full width of the house (no interior load bearing walls).

Deflection limits are primarily established with regard to serviceability concerns. The intent is to prevent excessive deflection which might result in cracking of finishes. For walls, most codes generally agree that L/240 represents an acceptable serviceability limit for deflection. For walls with flexible finishes, such as steel siding or roofing, L/180 is often used. The reader is referred to Part III for an example calculation of wall studs.

Tables C6.3 and C6.4 (following pages) provide the maximum allowable height of curtain wall studs (i.e. gable end wall studs) for different wind speeds and exposures. These tables were developed based on the following assumptions:

- No composite action is considered for doubled-up members.
- All studs are considered to be mechanically braced at 48 inches on center.
- All steels have a minimum yield strength of 33 ksi.

Table C6.5 provides the minimum allowable stud thicknesses that can be used for 20 foot wall studs supporting roof and ceiling only. This table is also developed based on the following assumptions:

- No composite action is considered for doubled-up members or sheathed assemblies.
- All studs are considered to be mechanically braced at 48 inches on center.
- All steels have a minimum yield strength of 33 ksi.

It is to be noted that lower stud thicknesses can be achieved with 50 ksi steels and/or with fully sheathed walls.

Table C6.3
Maximum Allowable Heights For 2 x 4 Curtain Wall Studs
33 ksi
Mechanical Bracing Every 48 inches or Fully Sheathed Wall

Wind Speed Exp. A/B	Wind Speed Exp. C	Member Size[3]	Member Spacing (inches)	Allowable Height (feet-inches) Stud Thickness 33 mil	43 mil	54 mil	68 mil	97 mil
70 mph		2x4	16	11'-9"	12'-10"	13'-9"	14'-8"	16'-2"
			24	10'-4"	11'-2"	12'-0"	12'-10"	14'-1"
		(2) 2x4s	16	14'-10"	16'-2"	17'-4"	18'-6"	20'-4"
			24	13'-0"	14'-1"	15'-1"	16'-2"	17'-10"
80 mph	70 mph	2x4	16	10'-8"	11'-8"	12'-6"	13'-4"	14'-8"
			24	9'-4"	10'-2"	10'-11"	11'-8"	12'-10"
		(2) 2x4s	16	13'-6"	14'-8"	15'-9"	16'-10"	18'-6"
			24	11'-9"	12'-10"	13'-9"	14'-8"	16'-2"
90 mph	80 mph	2x4	16	9'-11"	10'-10"	11'-7"	12'-5"	13'-7"
			24	8'-8"	9'-5"	10'-1"	10'-10"	11'-11"
		(2) 2x4s	16	12'-6"	13'-8"	14'-7"	15'-7"	17'-2"
			24	10'-11"	11'-11"	12'-9"	13'-8"	15'-0"
100 mph	90 mph	2x4	16	9'-2"	9'-11"	10'-8"	11'-5"	12'-7"
			24	7'-0"	8'-8"	9'-4"	9'-11"	10'-11"
		(2) 2x4s	16	11'-6"	12'-7"	13'-5"	14'-4"	15'-10"
			24	12'-4"	15'-7"	16'-9"	17'-11"	19'-10"
110 mph	100 mph	2x4	16	7'-10"	9'-0"	9'-8"	10'-4"	11'-4"
			24	5'-3"	7'-10"	8'-5"	9'-0"	9'-11"
		(2) 2x4s	16	10'-5"	11'-4"	12'-2"	13'-0"	14'-4"
			24	9'-1"	9'-11"	10'-8"	11'-4"	12'-6"
	110 mph	2x4	16	6'-6"	8'-5"	9'-1"	9'-8"	10'-8"
			24	4'-4"	7'-5"	7'-11"	8'-5"	9'-4"
		(2) 2x4s	16	9'-10"	10'-8"	11'-5"	12'-3"	13'-5"
			24	8'-7"	9'-4"	10'-0"	10'-8"	11'-9"

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m^2, 1 mph = 1.61 km/hr, 1 foot = 0.3 m.

1 Deflection criteria: L/240
2 Design load assumptions: Roof dead load is 12 psf (0.575 kN/m^2)
Attic live load is 10 psf (0.479 kN/m^2)
3 For actual sizes of members, refer to Table 2.1 in the *Prescriptive Method*.

Table C6.4
Maximum Allowable Heights For 2 x 6 Curtain Wall Studs
33 ksi
Mechanical Bracing Every 48 inches or Fully Sheathed Wall

Wind Exp. A/B	Speed Exp. C	Member Size[3]	Member Spacing (inches)	Allowable Height (feet-inches) Stud Thickness 33 mil	43 mil	54 mil	68 mil	97 mil
70 mph		2x6	16	16'-9"	18'-3"	19'-7"	21'-0"	23'-2"
			24	13'-1"	15'-11'	17'-1"	18'-4"	20'-3"
		(2) 2x6s	16	21'-2"	23'-0"	24'-8"	26'-5"	29'-3"
			24	18'-6"	20'-1"	21'-7"	23'-1"	25'-6"
80 mph	70 mph	2x6	16	14'-9"	16'-7"	17'-10"	19'-1"	21'-1"
			24	9'-10"	14'-6"	15'-6"	16'-8"	18'-5"
		(2) 2x6s	16	19'-2"	20'-11"	22-5"	24'-0"	26'-7"
			24	16'-9"	18'-3"	19'-7"	21'-0'	23'-2"
90 mph	80 mph	2x6	16	11'-10"	15'-5"	16'-6"	17'-8"	19'-7"
			24	7'-10'	13'-5"	14'-5"	15'-5"	17'-1"
		(2) 2x6s	16	17'-10"	19'-5	20'-10"	22'-4"	24'-8"
			24	15'-7'	16'-11"	18'-2"	19'-6"	21'-6"
100 mph	90 mph	2x6	16	9'-3"	14'-2"	15'-2"	16'-3"	18'-0"
			24	6'-2"	12'-2"	13'-3"	14'-3"	15'-9"
		(2) 2x6s	16	16'-5"	17'-10"	19'-2'	20'-6"	22'-8"
			24	12'-4"	15'-7"	16'-9"	17'-11"	19'-10"
110 mph	100 mph	2x6	16	6'-10"	12'-10"	13'-9"	14'-9"	16'-4"
			24	4'-7"	9'-1"	12'-0"	12'-11"	14'-3"
		(2) 2x6s	16	13'-9"	16'-2"	17'-4"	18'-7"	20'-7"
			24	9'-2"	14'-2"	15'-2"	16'-3'	18'-0"
	110 mph	2x6	16	5'-8"	11'-3"	12'-11"	13'-10"	15'-4"
			24	3'-9"	7'-6"	11'-1"	12'-1"	13'-4"
		(2) 2x6s	16	11'-4"	15'-2"	16'-4"	17'-6"	19'-4"
			24	7'-7"	13'-3"	14'-3'	15'-3"	16'-10"

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m^2, 1 mph = 1.61 km/hr, 1 foot = 0.3 m.

1 Deflection criteria: L/240
2 Design load assumptions: Roof dead load is 12 psf (0.575 kN/m^2)
Attic live load is 10 psf (0.479 kN/m^2)
3 For actual sizes of members, refer to Table 2.1 in the *Prescriptive Method*.

Table C6.5
Steel Stud Thickness for 20-foot Walls Supporting Roof and Ceiling Only
(33 ksi Steel)

Wind Speed		Member Size[3]	Member Spacing (inches)	Stud Thickness (mils)[1,2]															
				Building Width (feet)[4,5]															
				24				28				32				36			
Exp. A/B	Exp. C			Ground Snow Load (psf)				Ground Snow Load (psf)				Ground Snow Load (psf)				Ground Snow Load (psf)			
				20	30	50	70	20	30	50	70	20	30	50	70	20	30	50	70
70 mph		2x6	16	43	43	43	54	43	43	43	54	43	43	54	54	43	43	54	54
			24	68	68	68	68	68	68	68	97	68	68	68	97	68	68	97	97
		(2) 2x6	16	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
			24	43	43	43	43	43	43	43	43	43	43	43	43	43	43	43	54
80 mph	70 mph	2x6	16	54	54	54	68	54	54	54	68	54	54	68	68	54	54	68	68
			24	97	97	97	97	97	97	97	97	97	97	97	97	97	97	97	97
		(2) 2x6	16	33	33	33	43	33	33	33	43	33	33	33	43	33	33	43	43
			24	43	43	54	54	43	54	54	54	54	54	54	54	54	54	54	68
90 mph	80 mph	2x6	16	68	68	68	68	68	68	68	97	68	68	68	97	68	68	97	97
			24	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
		(2) 2x6	16	43	43	43	43	43	43	43	43	43	43	43	43	43	43	43	43
			24	54	54	68	68	54	54	68	68	54	54	68	68	54	68	68	68
100 mph	90 mph	2x6	16	97	97	97	97	97	97	97	97	97	97	97	97	97	97	97	97
			24	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
		(2) 2x6	16	43	43	54	54	43	43	54	54	43	54	54	54	54	54	54	54
			24	68	68	68	97	68	68	97	97	68	68	97	97	68	97	97	97
110 mph	100 mph	2x6	16	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
			24	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
		(2) 2x6	16	54	68	68	68	68	68	68	68	68	68	68	68	68	68	68	68
			24	97	97	97	97	97	97	97	97	97	97	97	---	97	97	97	---
	110 mph	2x6	16	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
			24	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
		(2) 2x6	16	68	68	68	97	68	68	97	97	68	68	97	97	68	97	97	97
			24	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m^2, 1 mph = 1.61 km/hr, 1 foot = 0.3 m.

1 Deflection criteria: L/240

2 Design load assumptions: Roof dead load is 12 psf (0.575 kN/m^2)
Attic live load is 10 psf (0.479 kN/m^2)

3 For actual sizes of members, refer to Table 2.1 in the *Prescriptive Method*.

4 Building width is in the direction of horizontal framing members supported by the wall studs.

5 Exterior load bearing walls with a minimum of 1/2 inch (13 mm) gypsum board on the inside and 7/16 inch (11 mm) OSB or plywood on the outside, and interior load bearing walls with a minimum of 1/2 inch (13 mm) gypsum board on both sides may use the next thinner stud but not less than 33 mils (0.84 mm).

C6.4 Stud Bracing (Buckling Resistance)

Studs in load bearing walls are laterally braced on each flange by either a continuous 1-1/2 inch x 33 mil (minimum) strap at mid-height (or third points for 9-foot and 10-foot studs) or by direct attachment of structural sheathing or rigid wall finishes (i.e. APA rated plywood or OSB, gypsum wall board) according to the requirements of the *Prescriptive Method*. Therefore, all studs were assumed to be braced at mid-height (or third points for 9-foot and 10-foot studs) for engineering analysis of the stud tables. As previously noted, the benefit of structurally sheathed walls on the required stud thickness and the composite wall strength are recognized in the footnotes to the stud tables.

Temporary bracing is 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, a wall stud is free to twist, thus making the stud subject to premature failure under heavy construction loads (i.e. stack of gypsum wallboard or roof shingles). In such cases, adequate temporary bracing must be provided.

C6.5 Splicing

The stud tables provided in the *Prescriptive Method* are based on the assumption that the studs are continuous, with no splices. Therefore, load bearing steel studs shall not be spliced without an approved design. Non-structural members such as wall tracks are spliced according to the requirements and details in the *Prescriptive Method*.

C6.6 Corner Framing

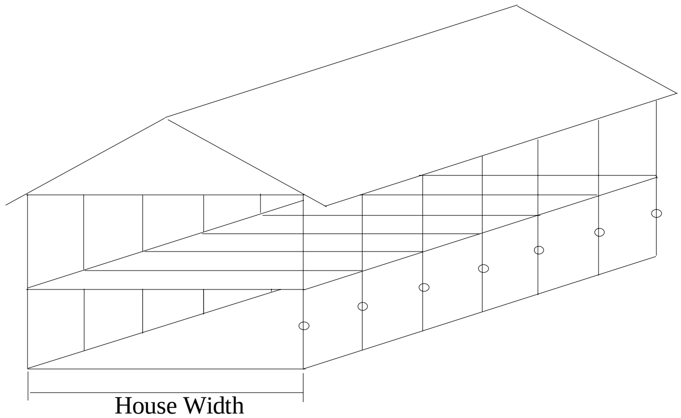
The *Prescriptive Method* utilizes a traditional three-stud practice for framing corners.

C6.7 Headers

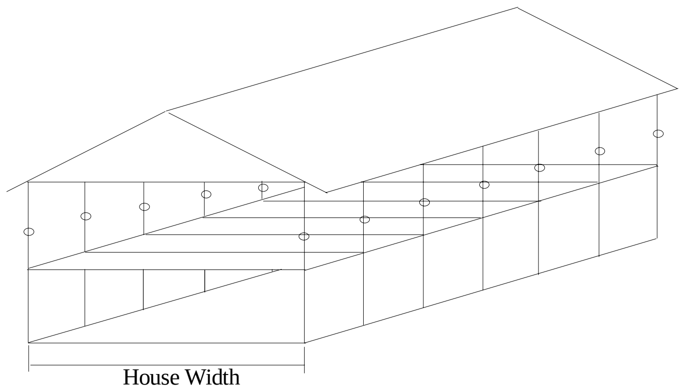
Headers are horizontal members used to transfer loads around openings in load bearing walls. Headers specified in the *Prescriptive Method* are allowed only above the opening immediately below the wall top track (i.e. high headers). Headers are formed from two equal sized C-shaped members in a back-to-back or box type configuration. Cripple studs, installed below headers or window sills, are required to have a size and thickness equivalent to or grater than the adjacent full height wall studs. Tracks used to frame around openings are also required to be of a thickness equivalent to or greater than the wall studs.

Headers were divided into three categories: headers supporting roof and ceiling only (i.e. one story house or upper story of two story house); headers supporting one story, roof and ceiling (i.e. bottom floor of a two story house); and headers supporting one story, roof and ceiling, with center load bearing beam (i.e. bottom floor of a two story house). These categories are illustrated in Figure C6.7.

Headers Supporting Roof and Ceiling Only.



Header Supporting One Floor, Roof and Ceiling.



Headers Supporting One Floor, Roof and Ceiling with Center Load Bearing Beam.

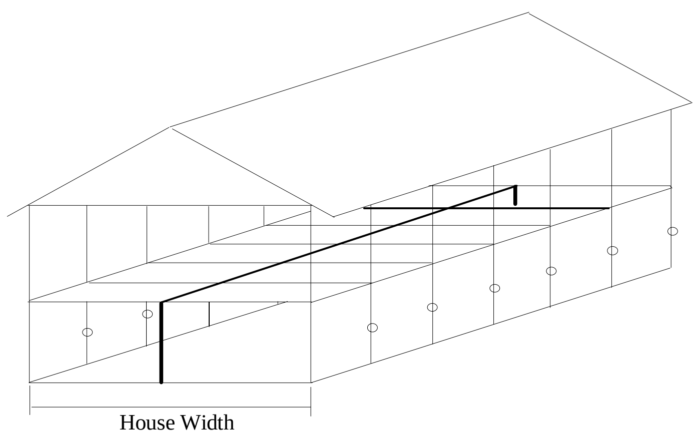


Figure C6.7 Header Category

The following design assumptions were made in analyzing header spans:

- Headers are simply supported beams.
- Maximum roof overhang of 12 inches.
- 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.
- The allowable capacity of each header is calculated as twice the allowable capacity of a single member (i.e. no composite action).
- Deflection criteria: L/240
- Reduction in tributary area of the attic is not considered. This reduction could reduce the attic design live load by a minimum of 25 percent.

The reader is referred to Part III for a detailed calculation of header spans.

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 the decimal to the next higher 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.

Figure C6.8 shows typical details of connecting the sill plate to the studs at the location of openings in a wall.

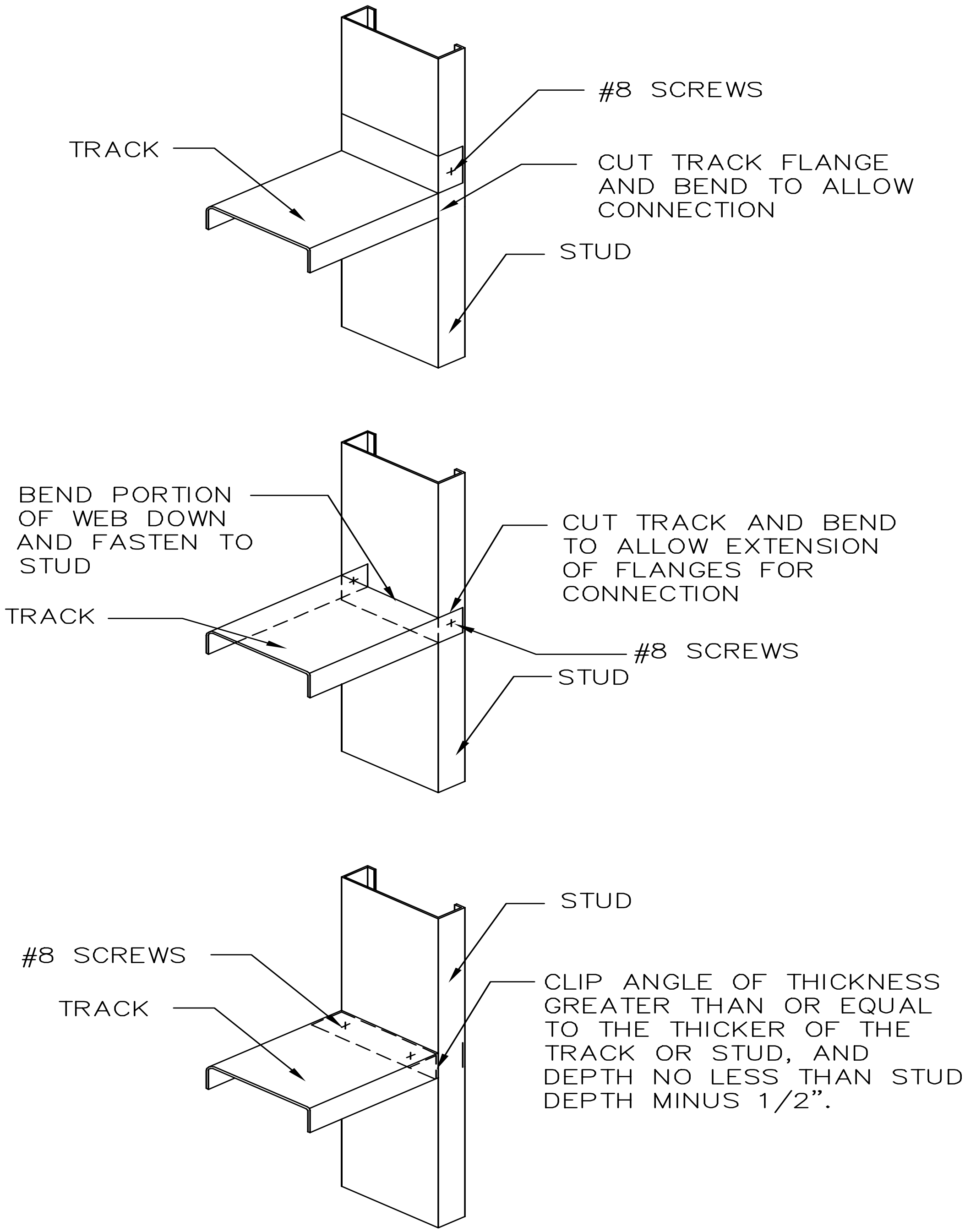


Figure C6.8 Typical Sill Plate Connection Detail

C6.8 Wall Bracing

The wall bracing requirements in the *Prescriptive Method* are limited to wind loads less than 90 C (or 100 A/B) and Seismic Zones 0, 1, and 2 (excluding Zones 3 and 4). These limits were established because of the additional concerns for building geometry constraints and design issues such as hold-down requirements for shearwalls. Homes built in conditions that exceed these loading limits must have the wall bracing engineered by a design professional. The reader is referred to Section C6.8.2.1 for an alternate shearwall bracing method for high wind and seismic regions.

The development of wall bracing requirements for the *Prescriptive Method* involved several activities including:

- a review of applicable wall bracing tests and data;
- development of the engineering approach to apply loads and determine lateral resistance;
- selection of wall bracing methods and materials for the *Prescriptive Method*;
- determination of appropriate safety factors and allowable capacities;
- sensitivity studies of the effects of lateral loads (wind and seismic) relative to building geometry (i.e.
- roof slope, width, length, and wall height);
- comparison of earthquake induced lateral loads to those from wind based on the applicability limits for the *Prescriptive Method* (i.e. dead loads, geometry, etc.);
- sensitivity (parametric) studies to guide simplification of the prescriptive requirements; and,
- testing of wall bracing requirements utilized in the *Prescriptive Method*.

Wall bracing in cold-formed steel structures has typically relied on case-by-case design of diagonal steel strapping, its anchorage, and associated details. Shearwall testing of steel-framed walls using monotonic and cyclic loads has recently provided data for applications using structural sheathing (15/32-inch plywood or 7/16-inch oriented strand board) as the bracing method [26].

In wood-frame homes, structural sheathing (e.g. plywood or OSB) is a common bracing method, particularly in conditions of high lateral loads. However, many conventionally built homes in normal load conditions use wood let-in braces (diagonal 1x4s) or steel straps for racking resistance. Alternatively, 4'x8' bracing panels of structural sheathing may be substituted for the let-in braces forming a "partially" or "intermittently" sheathed wall. While these methods are non-engineered and have been established by experience, they have performed well in regions not subject to high winds. For one- and two-story detached homes with simple geometries and limited wall openings, these conventional bracing practices have also performed satisfactorily in past seismic events. The trend in high seismic and high wind regions is toward engineered designs using structural sheathing and numerous hold-down brackets.

The wall bracing requirements in the *Prescriptive Method* were based on an engineered approach that utilized available technical knowledge. Two wall bracing methods were investigated based on engineering test data or rational analysis to provide wall bracing requirements for the *Prescriptive Method*. These methods are diagonal steel straps and structural (wood) sheathing (e.g. OSB or plywood). Verification tests were performed on the diagonal steel strap requirements. These tests indicated that additional testing or design work was needed to develop simple diagonal strap bracing requirements that may be used efficiently in the *Prescriptive Method*. This concern relates to the limited capability of the "non-structural" top and bottom wall tracks to adequately resist compressive or bending loads that result when strap bracing or non-continuous sheathing ("partially sheathed" walls) are used for lateral support. For this reason, the wall bracing in the *Prescriptive Method* was conservatively limited to the use of continuously sheathed walls with the previously described limitations on loading conditions and building geometry.

C6.8.1 Strap Bracing

Diagonal steel strap braces or 'X-braces' must be designed in accordance with approved engineering practices for the reasons mentioned previously.

C6.8.2 Structural Sheathing

The allowable shear capacities for plywood sheathing and oriented strand board sheathing are based on recent racking tests [26]. The test set-up was similar to that required by the ASTM E564 [27] standard for static load tests of framed shearwalls. A cyclic loading protocol was also performed on the test specimens. The monotonic load test results relevant to this document are summarized as follows (Table C6.8.1):

Table C6.8.1
Monotonic Load Test Results [26]

Description of Wall	Average Ultimate Capacity (plf)	Average Load at 1/2 inch Deflection (plf)	Allowable Shear Load Capacity (plf) S.F. = 2.5
15/32" Plywood APA rated sheathing w/ panels on one side	1063	508	425
7/16" OSB APA rated sheathing w/ panels on one side	911	593	364

For SI: 1 inch = 25.4 mm, I plf = 1.488 Kg/m

1 Framing at 24 inches on center.
2 Studs and track are 3-1/2 in. 33 mils. -- 33ksi steel.
3 Studs flange: 1-5/8 in.; stud lip: 3/8 in.; track flange: 1-1/4 in.
4 Framing screws -- No. 8 x 5/8 in. wafer head self-drilling tapping screws; Sheathing screws -- No. 8 x 1 in. flat head (coarse thread) @ spacing of 6" edge / 12 in. field of panels.
5 1/2" gypsum wall board on interior of wall.

An allowable shear capacity of 364 plf was used for the determination of the wall bracing requirements. The allowable capacity is less than the load recorded at 1/2 in. deflection of the wall test specimens.

For determining lateral forces, wind loads were calculated for the various building surfaces using the orthogonal wind loading approach of ASCE 7-93 [3]. A load combination of D + 0.75W (Dead load + 0.75 x Wind load) was used. The 0.75 factor was used to account for systematic biases in wind loads associated with wind direction, building siting, and correlation of windward and leeward maximum surface pressure coefficients. Standard tributary areas consisting of the leeward and windward building surfaces, were assigned to each exterior shearwall (i.e. sidewalls and endwalls) to determine the lateral, in-plane shear loads to be resisted by these walls, depending on the two worst-case orthogonal wind directions. No interior walls or alternate shear pathways were considered. A computer spreadsheet was designed to perform these calculations so that a wide range of building geometries and loading conditions could be investigated.

Based on the loads calculated as described above, the amount of full-height structural sheathing required was determined using the allowable sheathing capacity of 364 plf. 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 the *Prescriptive Method's* 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 openings). Further, the individual wall segments must be 48 inches 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 *Prescriptive Method*. The only building geometry parameter retained was roof slope due to a significant impact on the wind loads transferred to the shearwalls. Footnotes to the shearwall table provide additional information related to the proper applications of the requirements.

C6.8.2.1 Wall Bracing: Alternate Approach and Bracing for High Wind and Seismic Regions

The following is an alternate wall bracing method that is also applicable to high wind and seismic regions. This method is based on recent pilot tests conducted at the NAHB Research Center [28] to develop a "perforated" shearwall design approach for steel-framed walls. Basic shearwall capacity values are used from the tests performed at the University of Santa Clara [26]. These alternate provisions are considered preliminary and should be approved by an engineer prior to use.

<u>Wall Bracing</u>:

The minimum length of full-height sheathing on 8-foot high exterior walls shall be the greater length determined in accordance with Tables C6.8.2, C6.8.3, C6.8.4, or C6.8.5 depending on wind and seismic conditions and whether or not hold-downs are used. The length of full-height sheathing on a given wall line shall be determined by the sum of all individual full height sheathed wall sections, which are a minimum of 24 inches wide and which are uninterrupted by openings. Offset wall segments in a given wall line shall be permitted provided the total length of full-height structural sheathing meets or exceeds the minimum required length for the entire wall line, including all offset segments that are parallel to the main wall line. A minimum of 24 inches of full-height structural sheathing shall be located at each end of the exterior wall lines and at the ends of segments of those portions of walls that are offset from the main wall line by more than 4 feet.

> **Exception:** Panels to either side of garage openings shall be a minimum of 16 inches provided that living spaces are not supported by the garage walls.

For walls greater than 8 feet in height, the minimum lengths of full-height structural sheathing in Tables C6.8.2 and C6.8.4 shall be multiplied by 1.10 for 9-foot high walls and multiplied by 1.20 for 10-foot high walls. These multipliers shall not be used to increase minimum percent lengths of full-height sheathing required by Tables C6.8.4 and C6.8.5 for seismic conditions. A minimum of 27 inch width or 30 inch width of full-height structural sheathing shall be located at each end of the exterior wall lines for 9-foot high or 10-foot high walls, respectively.

> **Exception:** Panels to either side of garage openings shall be a minimum of 24 inches wide or 28 inches wide for 9-foot and 10-foot walls, respectively, provided that living spaces are not supported by the garage walls.
>
> All other provisions for 9-foot and 10-foot high walls are the same as required for 8-foot high walls except when hold-downs are used in accordance with the Hold-down Alternate Approach (see Section C6.9)

Table C6.8.2
Wall Bracing for Wind Conditions:
Minimum Length of Full-Height Structural Sheathing
on Exterior Walls with a Single Hold-down at Each Corner[1,2,3]

Wall Supporting:	Roof Slope	Wind Speed (mph-fastest mile) and Exposure						
		70 A/B	**80 A/B**	**90 A/B**	**100 A/B**	**110 A/B**		
				70 C	**80 C**	**90 C**	**100 C**	**110 C**
Endwalls (walls at gable end or perpendicular to ridge)								
Roof & Ceiling Only	3:12	4	4	4	4	4	4	5
	6:12	4	4	4	4	6	7	8
	9:12	4	5	6	8	10	12	14
	12:12	5	7	9	11	13	16	19
One Story, Roof & Ceiling	3:12	4	5	7	8	11	13	15
	6:12	5	6	8	10	12	15	17
	9:12	6	8	10	12	16	19	22
	12:12	8	10	12	15	19	22	25
Sidewalls (walls parallel to ridge)								
Roof & Ceiling Only	3:12	4	4	4	4	6	7	8
	6:12	4	4	4	6	7	9	11
	9:12	4	4	5	7	8	11	13
	12:12	4	5	6	8	10	12	15
One Story, Roof & Ceiling	3:12	4	5	6	8	10	12	14
	6:12	4	5	7	9	11	14	16
	9:12	5	6	8	10	13	15	18
	12:12	5	7	9	11	14	17	20

For SI: 1 mph = 49 m/sec; 1 foot = 0.3 m; 1 inch = 25.4 mm

1 All exterior wall surfaces shall be completely sheathed, including areas above and below openings.
2 Linear interpolation shall be permitted.
3 A single hold-down shall be located at each building corner in accordance with Section C6.9.

Table C6.8.3
Wall Bracing for Seismic Conditions:
Minimum Percent Length of Full-Height Structural Sheathing on Exterior Walls with a Single Hold-down at Each Corner[1,2,3]

Wall Supporting:	**Seismic Zone 1 (A_a = 0.1g)**		**Seismic Zone 2 (A_a = 0.2g)**		**Seismic Zone 3 (A_a = 0.3g)**		**Seismic Zone 4 (A_a = 0.4g)**	
	Endwall	**Sidewall**	**Endwall**	**Sidewall**	**Endwall**	**Sidewall**	**Endwall**	**Sidewall**
Roof & Ceiling Only	10%	10%	20%	15%	30%	20%	35%	25%
One Story, Roof & Ceiling	20%	15%	30%	25%	45%	30%	55%	40%

For SI: 1 mph = 49 m/sec; 1 foot = 0.3 m

1 All exterior wall surfaces shall be completely sheathed, including areas above and below openings.

2 A single hold-down shall be located at each building corner in accordance with the alternate approach in Section C6.9.

3 Percentages are given as a percentage of the total sidewall or endwall length. For example, a 28 foot long endwall supporting one story plus a roof and ceiling in Seismic Zone 3 is required to have 45% of its length in full-height structural sheathing (i.e. 45 x 28/100 = 12.6 ft).

Table C6.8.4
Wall Bracing for Wind Conditions:
Minimum Length (Feet) of Full-Height Structural Sheathing
on Exterior Walls Without Hold-downs at Corners[1,2,3]

Wall Supporting:	Roof Slope	Wind Speed (mph-fastest mile) and Exposure						
		70 A/B	80 A/B	90 A/B	100 A/B	110 A/B		
				70 C	80 C	90 C	100 C	110 C
Endwalls (walls at gable end or perpendicular to ridge)								
Roof & Ceiling Only	3:12	6	6	6	6	6	6	8
	6:12	6	6	6	6	9	11	12
	9:12	6	8	9	12	15	18	21
	12:12	8	11	14	17	20	24	29
One Story, Roof & Ceiling	3:12	6	8	11	12	17	20	23
	6:12	8	9	12	15	18	23	26
	9:12	9	12	15	18	24	29	33
	12:12	12	15	18	23	29	33	N/A
Sidewalls (walls parallel to ridge)								
Roof & Ceiling Only	3:12	6	6	6	6	9	11	12
	6:12	6	6	6	9	11	14	17
	9:12	6	6	8	11	12	17	20
	12:12	6	8	9	12	15	18	23
One Story, Roof & Ceiling	3:12	6	8	9	12	15	18	21
	6:12	6	8	11	14	17	21	24
	9:12	8	9	12	15	20	23	27
	12:12	8	11	14	17	21	25	30

For SI: 1 mph = 49 m/sec; 1 foot = 0.3 m

[1] All exterior wall surfaces shall be completely sheathed, including areas above and below openings.
[2] Linear interpolation shall be permitted.
[3] N/A = design required or use Table C6.8.2 which requires corner hold-downs.

Table C6.8.5
Wall Bracing for Seismic Conditions:
Minimum Percent Length of Full-Height Structural Sheathing on Exterior Walls Without Hold-downs at Corners[1,2]

Wall Supporting:	**Seismic Zone 1**		**Seismic Zone 2**		**Seismic Zone 3**		**Seismic Zone 4**	
	Endwall	**Sidewall**	**Endwall**	**Sidewall**	**Endwall**	**Sidewall**	**Endwall**	**Sidewall**
Roof & Ceiling Only	15%	15%	30%	20%	45%	30%	55%	35%
One Story, Roof & Ceiling	25%	20%	45%	35%	65%	45%	85%	60%

For SI: 1 foot = 0.3 m

1 All exterior wall surfaces shall be completely sheathed, including areas above and below openings.

2 Percentages are given as a percentage of the total sidewall or endwall length. For example, a 28 foot long endwall supporting one story plus a roof and ceiling in Seismic Zone 3 is required to have 65% of its length in full-height structural sheathing (i.e. 0.65 x 28 ft. = 18 ft.).

Explanation and Derivation of the Wall Bracing Alternate Approach

Shearwall tests performed on steel-framed wall specimens (i.e. 4 foot and 8 foot lengths) and assemblies up to 40 foot in length with varying degrees of openings made it possible to determine relatively efficient shearwall bracing requirements for high wind and seismic conditions [26] [28]. However, judgment is still required when applying the wall bracing requirements to homes with very complex layouts in areas where high seismic and wind risks occur (i.e. seismic zone 4 and the hurricane coastline). Complex homes may be generally defined by walls with numerous corners and offsets, particularly when coupled with large rooms having ceilings that extend vertically across more than one story. In such cases, an engineered design should be sought.

A growing resource of testing of steel-framed walls using monotonic and cyclic loads has recently provided data for applications using structural sheathing (15/32-inch plywood or 7/16-inch OSB) as the bracing method. This testing has also led to recent building code approvals providing data to a broader community of designers and code officials. The alternate approach provided in this section is based on engineered approach that utilized new design methodology known as perforated shearwalls [28] and shearwall capacity data [26].

Two structural sheathing materials are permitted based on test data and rational engineering analysis for shearwalls following the perforated shearwall design method and using ASCE 7-93 [3] wind and seismic loads. These materials are 7/16 inch OSB and 15/32 inch plywood with 1/2 inch gypsum wallboard on the interior wall surface.

The allowable shear capacities for plywood sheathing and oriented strand board sheathing was taken as 455 plf based on available test data conducted in conformance with the ASTM E 564 [27] standard. The results relevant to this document are summarized in Table C6.8.6:

Table C6.8.6
Monotonic Load Test Results [26]

Description of Wall	Average Ultimate Capacity (plf)	Average Load at 1/2 inch Deflection (plf)	Allowable Shear Load Capacity (plf) S.F. = 2.0
15/32" Plywood APA rated sheathing w/ panels on one side	1063	508	531
7/16" OSB APA rated sheathing w/ panels on one side	911	593	455

For SI: 1 inch = 25.4 mm, I plf = 1.488 Kg/m

1 Framing at 24 inches on center.
2 Studs and track are 3-1/2 in. 33 mils. -- 33ksi steel.
3 Studs flange: 1-5/8 in.; stud lip: 3/8 in.; track flange: 1-1/4 in.
4 Framing screws -- No. 8 x 5/8 in. wafer head self-drilling tapping screws; sheathing screws -- No. 8 x 1 in. flat head (coarse thread) @ spacing of 6" edge / 12 in. field of panels.
5 1/2" gypsum wall board on interior of wall.

An allowable shear capacity of 455 plf was used for the determination of the wall bracing requirements. The allowable capacity is less than the load recorded at 1/2 in. deflection of the wall test specimens.

For determining lateral forces, wind loads and seismic loads were calculated using the appropriate provisions of ASCE 7-93 [3]. A load combination of D + 0.75W (Dead load + 0.75 x Wind load) was used. The 0.75 factor was used to account for systematic biases in wind loads associated with wind direction, building siting, and correlation of windward and leeward maximum surface pressure coefficients. This adjustment is more conservative than the practice of multiplying the entire load combination by 0.75 as is commonly done for steel framing when wind loads are considered. For seismic loads, a combination of D+L+S+E was used with a ductility factor (R) of 6.0 instead of 6.5 as is commonly used for wood framing. This was done because of the slightly less ductile behavior of steel walls and is a judgment based on limited test results [26] [28]. No interior walls or alternate shear pathways were considered. Computer spreadsheets were designed to perform these calculations so that a wide range of building geometries and loading conditions could be investigated.

Based on the loads calculated as described above, the amount of full-height structural sheathing required was determined using the smaller (OSB) allowable sheathing capacity of 455 plf. The length of full-height sheathing required was then tabulated over the range of building geometries defined in the *Prescriptive Method's* applicability limits. This length is reported as a percentage of the total wall length for seismic loads because the seismic load is proportionate to the wall length for a given building width. For wind, actual sheathing lengths are used because the wind load is governed by a relationship between roof slope and the building plan dimensions. A maximum tributary length or width of 16 feet was also assumed for both wind and seismic loads on exterior shearwalls only. The length of wall with full-height sheathing is defined as the sum of wall segments that have sheathing extending from the base (bottom track) to the top track, without interruption due to openings (i.e., the total of lengths of wall between openings). Further, the individual wall segments must be 24 inches in length or greater to contribute to the required length of full-height sheathing for a given wall line. Separate tables are provided for seismic and wind conditions and the more stringent requirement for a give site applies. Also, the user may opt to include hold-down brackets in exchange for the amount of full-height sheathing required. Footnotes to the shearwall table provide additional information related to the proper application of the requirements.

C6.9 Hold-down Requirements

In wind conditions greater than 90 mph exposure C and in Seismic Zones 3 and 4, hold-down brackets to stabilize shearwalls must be designed by a design professional.

Alternate Hold-down Requirements

This alternate method is provided in conjunction with Tables C6.8.2 through C6.8.5 in Section C6.8 of the Commentary.

In conditions where wind speeds are in excess of 90 mph (144 km/hr) Exposure C or 100 mph Exposure A/B, and in Seismic Zones 3 and 4, hold-down brackets shall be provided at the ends of each major wall line and at the ends of any parallel wall segment offset from the main wall lines by greater than 4 feet. A hold-down bracket shall consist of an approved strap or bracket adequately attached to the base of the corner stud and anchored to the foundation, floor, or wall below to form a continuous load path to the foundation. Approved hold-down brackets or straps shall have a minimum allowable capacity of 2,000 lb. for walls supporting a roof and ceiling only and 3,000 lb. for walls supporting one story plus a roof and ceiling. These hold-down capacities shall be multiplied by 1.15 for 9-foot high walls and 1.25 for 10-foot high walls.

Minimum full-height sheathing lengths or percentages in Tables C6.8.2 and C6.8.3 require a single hold-down located at each building corner (i.e. each end of a wall). Hold-downs shall be provided at the ends of each major wall line and at the ends of any parallel wall segment offset from the main wall lines by greater than 4 feet when Tables C6.8.2 and C6.8.3 are used.

As an alternative approach to Tables C6.8.2 and C6.8.3 Tables C6.8.4 and C6.8.5 provide the minimum required lengths of full-height sheathing when hold-downs are not used. In higher wind conditions, wind uplift straps shall still be provided in accordance with Section 6.10.

Wall bracing tables (C6.8.2 through C6.8.5) are provided for the option of either including or not including hold-down brackets in exchange for greater amounts of full-height sheathing. When hold-downs are specified by preference, they are only necessary at the ends of the “perforated” shearwalls in accordance with verification tests and the design methodology developed for efficient shearwall design [28]. In such cases, the hold-downs shall be provided in accordance with the capacities stated. Specific engineered designs using hold-downs at both ends of each wall segment in a given wall line may provide some additional capacity or allow for greater amounts of window and door openings.

The hold-down resistance for shearwalls (rotational restraint) for the range of wind and seismic loads permitted is provided by standard connection details (screws and anchor bolts) for walls to foundations and floors, the available dead load restraining forces, and system effects at the corners of light-frame construction. For higher load conditions, hold-downs may be included at the ends of the walls to reduce the amount of full-height sheathing required or to increase the shear (racking) strength of the wall. The required capacity of the hold-down brackets are based on treatment of the wall as an assembly in accordance with the perforated shearwall approach referred to in Section C6.8 of the commentary.

C6.10 Wind Uplift

In wind conditions greater than 90 mph exposure C, uplift straps to offset potential wind uplift loads must be included as provided for in Table 6.1. Wind uplift loads are calculated in accordance with the wind load provisions of ASCE 7-93 [3]. Values for intermediate wind uplift loads in Table 6.1 may be interpolated. It should be noted that uplift straps are only required on walls directly connected to the roof. They are not required on lower story walls because the uplift loads are offset by floor dead loads and the conventional wall-to-floor connections given in Table 6.1, even in the higher wind conditions. Also, a specified uplift strap is required for each king stud supporting headers. The king stud strap or connector shall have the same capacity as the layout studs spaced at 16 inches or 24 inches on center. Building overturning (uplift) forces due to lateral wind loads are resisted by hold-down requirements (see Section C6.9).

C7.0 NON-STRUCTURAL WALLS

C7.1 Non-Load Bearing Studs

For interior non-load bearing partition walls, a minimum of 18 mil studs are specified in the *Prescriptive Method*. Consideration should be given for acceptable serviceability (rigidity) at interior wall openings for doors, shelving, cabinets, and other purposes. ASTM C 645 [11] is specified as the reference standard for non-load bearing walls. The *Prescriptive Method* provides several details commonly used in non-load bearing applications. Tables C7.1 and C7.2 (below) provide limiting heights for selected non-load bearing studs subjected to a lateral load of 5 psf. These tables are provided for guidance only and should only be used where instructions and/or recommendations are not available from the manufacturer.

Table C7.1
Maximum Allowable Clear Non-Load Bearing Stud Height [1,2,3,4]
Mid-Height Bracing

Industry Designation	Actual Stud Size (Web x Flange x Thickness) [5] (inches)	Stud Spacing (inches)	
		16" oc	24" oc
350S125-18	3.5 x 1.25 x 0.018	9'-10"	6'-6"
350S125-27	3.5 x 1.25 x 0.027	12'-4"	10'-11"
350S125-33	3.5 x 1.25 x 0.033	13'-0"	11'-8"

For SI: 1 inch = 25.4 mm, 1 psf = 0.048 kN/m^2, 1 foot = 0.3 m

1 Stud height based on stud capacity alone (i.e. no composite action).
2 Maximum lateral load is 5 psf (0.24 kN/m^2)
3 Minimum yield strength is 33 ksi (228 kPa)
4 Applicable to deflection limits of L/120, L/180, and L/240
5 All sections have a minimum lip size of 3/16 inch (5 mm)

Table C7.2
Maximum Allowable Clear Non-Load Bearing Stud Height [1,2,3]
Fully Braced Walls

Industry Designation	**Actual Stud Size (Web x Flange x Thickness) [4]** (inches)	**Stud Spacing (inches)** **Deflection Criteria**					
		16" oc L/120	**24" oc L/120**	**16" oc L/180**	**24" oc L/180**	**16" oc L/240**	**24" oc L/240**
350S125-18	3.5 x 1.25 x 0.018	9'-10"	6'-6"	9'-10"	6'-6"	9'-10"	6'-6"
350S125-27	3.5 x 1.25 x 0.027	16'-11"	13'-10"	15'-10"	13'-10"	14'-5"	12'-7"
350S125-33	3.5 x 1.25 x 0.033	19'-4"	15'-9"	17'-0"	14'-10"	15'-6"	13'-6"

For SI: 1 inch = 25.4 mm, 1 psf = 0.048 kN/m^2, 1 foot = 0.3 m,

1 Stud height based on stud capacity fully braced (continuously supported by gypsum wall board).
2 Maximum lateral load is 5 psf (0.24 kN/m^2)
3 Minimum yield strength is 33 ksi (228 kPa)
4 All sections have a minimum lip size of 3/16 inch (5 mm)

C7.2 Construction Details

Details C7.2.1, C7.2.2, C7.2.3, and C7.2.4 are typical door and window framing details that are widely used in the residential steel framing market.

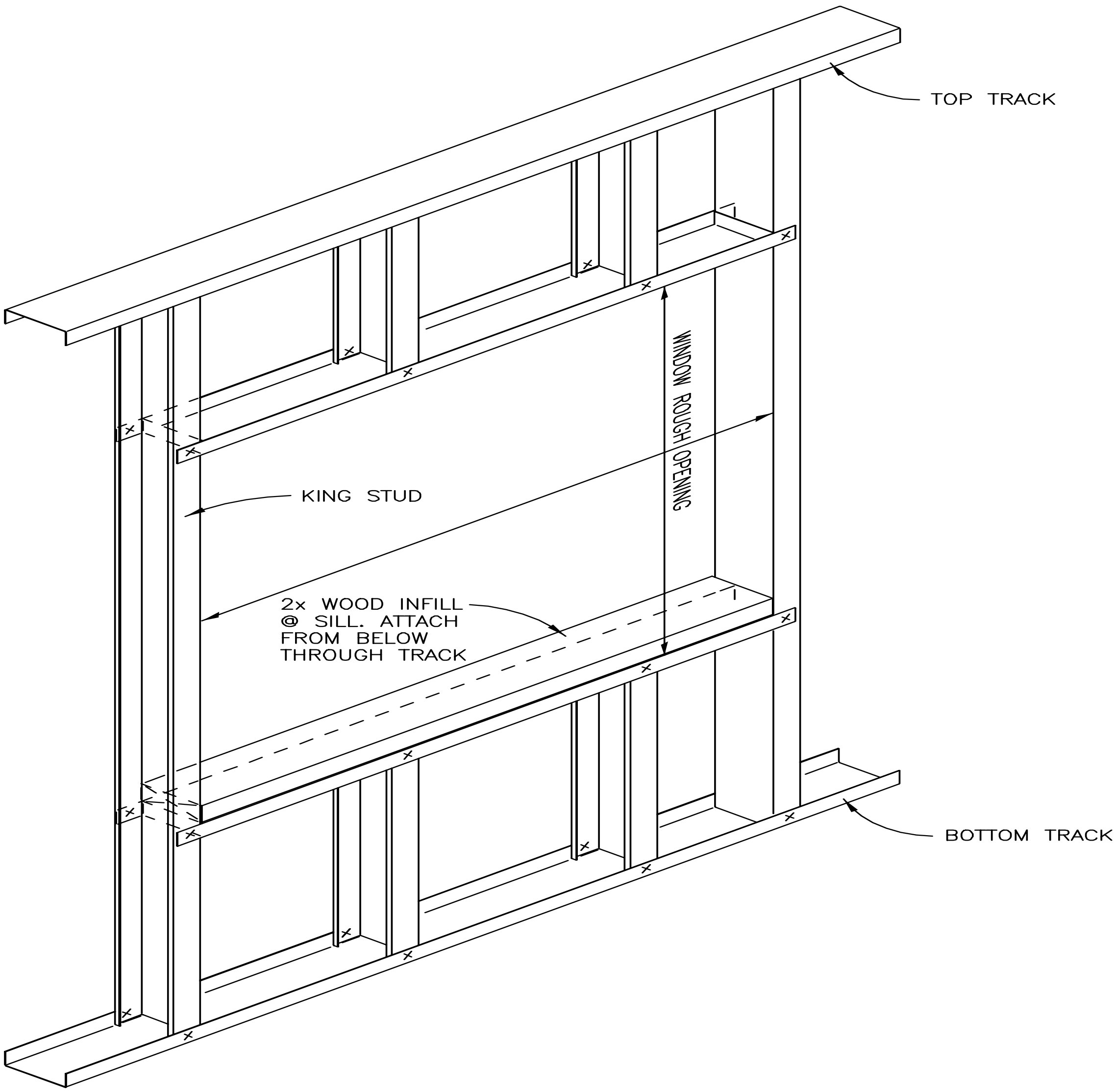


Figure C7.2.1 Window Framing Detail -1

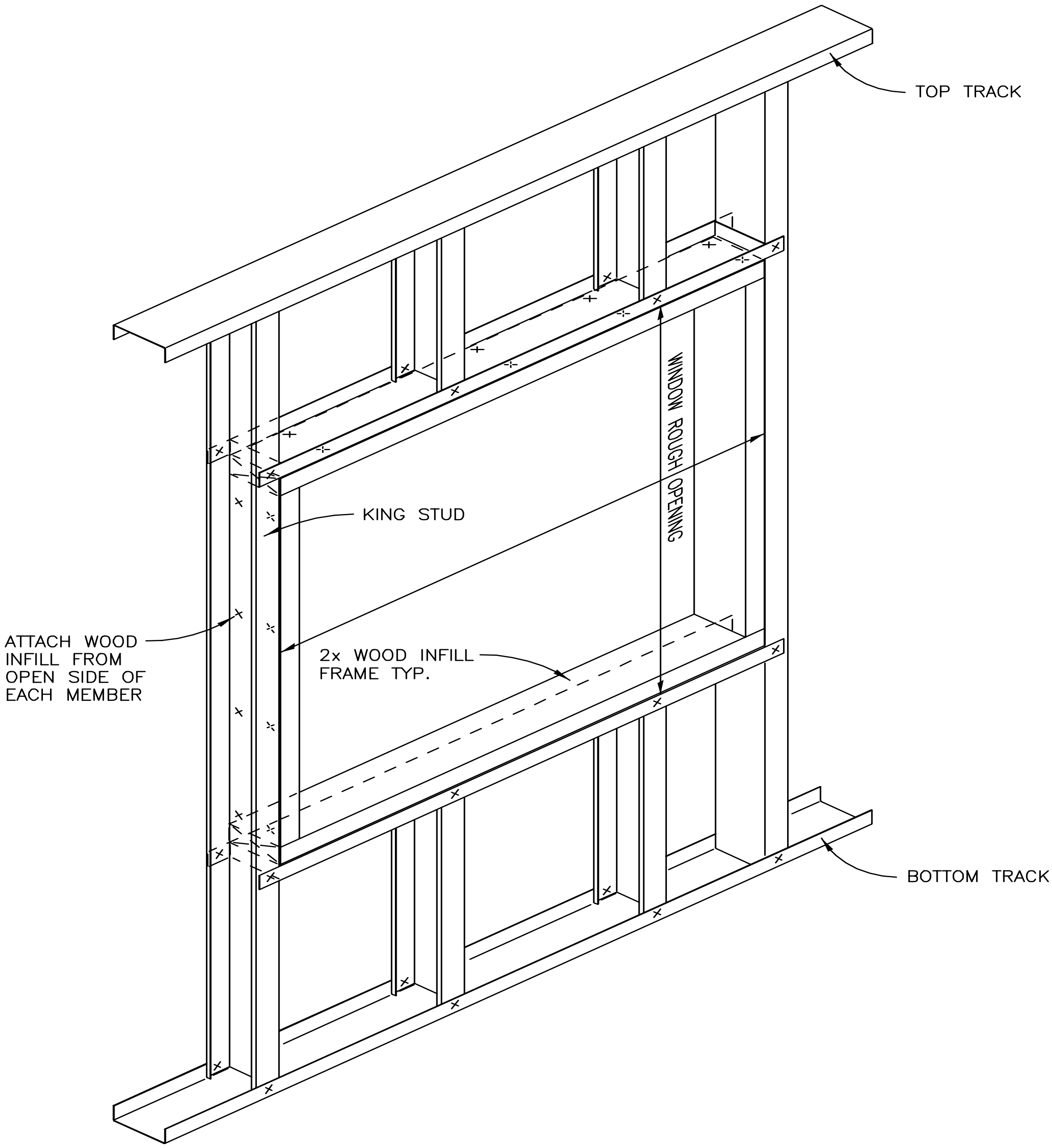


Figure C7.2.2 Window Framing Detail - 2

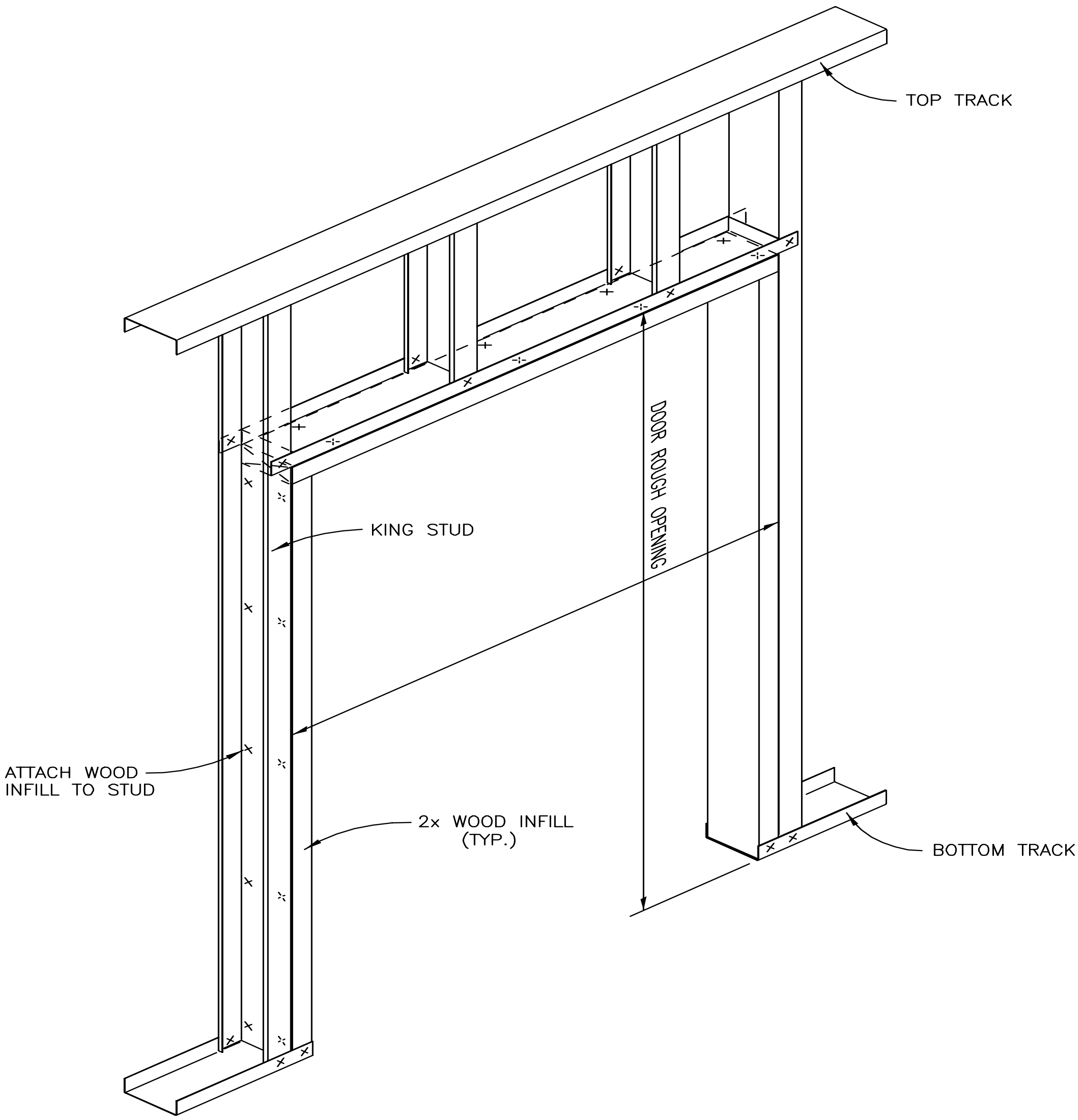


Figure C7.2.3 Door Framing Detail - 1

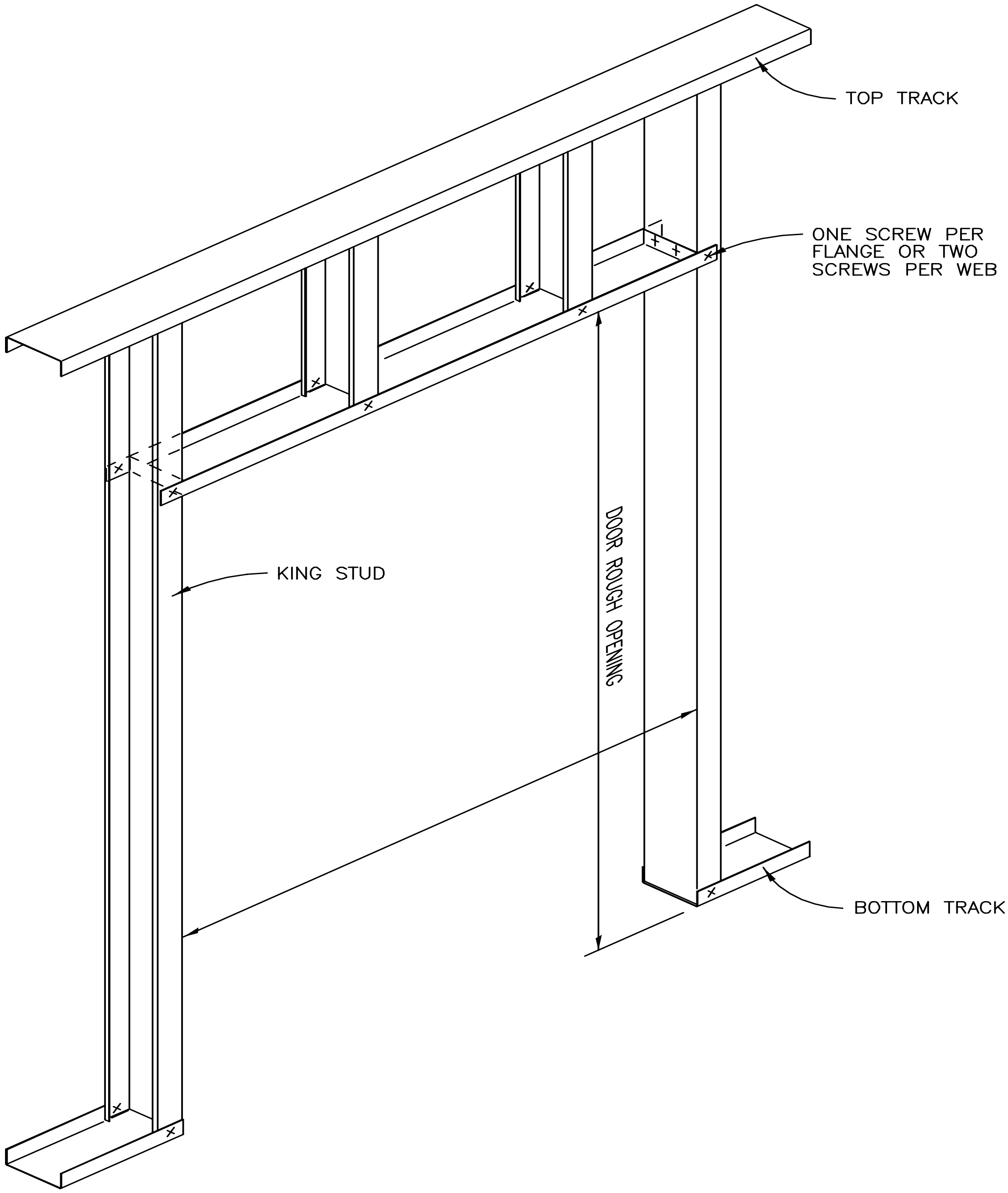


Figure C7.2.4 Door Framing Detail - 2

C8.0 ROOF SYSTEMS

C8.1 Roof Construction

Roof construction in the *Prescriptive Method* is limited to design wind speeds less than or equal to 110 mph (fastest mile) and to Seismic Zones 0, 1, 2, 3, and 4. Uplift straps should be provided for wind conditions in excess of 90 mph exposure C.

C8.2 Allowable Ceiling Joist Span

Ceiling joist tables in the *Prescriptive Method* provide the maximum allowable ceiling joist spans for two loading conditions: 10 psf and 20 psf attic live loads. Ceiling joists require stiffeners at all support locations.

In the design of ceiling joists, any one of several engineering criteria may control the prescriptive requirements depending on the configuration of the section, thickness of material, and member length. The analysis must include checks for:

- Yielding
- Flexural buckling
- Web crippling (not required if web stiffeners are specified)
- Shear
- Deflection

The engineering approach used to develop ceiling joist span tables for *Prescriptive Method* is similar to that used for floor joists with the exception of the magnitude of dead and live loads. The reader is referred to Part III for a detailed example of the engineering calculations.

C8.3 Ceiling Joist Bracing

Gypsum board (i.e. finished ceilings) is considered to be adequate bracing for the bottom (tension) flanges of the ceiling joists. Ceiling joist tables provide spans for braced, as well as unbraced, top flanges. For braced top (compression) flanges it is necessary for steel strapping to have blocking (or bridging) installed at a maximum spacing of 12 feet and at the termination of all straps. Moreover, the ends of steel straps are designed to be fastened to a stable component of the building.

C8.4 Rafters

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 10 psf dead load and the greater of a minimum 16 psf live load or the applied roof snow load. As mentioned previously, applied snow loads are calculated by multiplying the ground snow load by 0.7 (no further reductions or increases are made for special cases). Ground snow load intervals are as follows: 20 psf, 30 psf, 50 psf, and 70 psf.

Wind load effects are correlated to equivalent snow loads as shown in the *Prescriptive Method*. Wind pressures were calculated using the ASCE 7-93 [3] Components and Cladding method. 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.

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 rafters. The *Prescriptive Method* does not currently address cathedral ceilings because a prescriptive ridge beam and post design is not provided.

The heel joint connection, which connects the ceiling joist to the rafter, uses # 10 screws. Lapped ceiling joists shall 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.

C8.5 Rafter Bottom Flange Bracing

The bracing requirements provided in the *Prescriptive Method* are self explanatory and are commonly used in residential steel construction. The requirements are similar to those found in the floor framing section when gypsum wall board is not applied to the bottom of the joists.

C8.6 Splicing

Splices are usually required to transfer shear, bending moments, and axial loads. Splices or lapped ceiling joists may occur over points of bearing as previously described. The rafter spans provided in the *Prescriptive Method* are based on the assumption that the members are continuous, with no splices. Therefore, rafters shall not be spliced without an approved design. Splicing details for non-structural members (i.e. tracks) are shown in the *Prescriptive Method*.

C8.7 Roof Trusses

The *Prescriptive Method* allows for the usage of pre-engineered truss systems. Any pre-engineered truss member can not be cut, notched, or altered unless so designed.

C8.8 Roof Tie Downs

In wind conditions greater than 70 mph exposure C (90 mph exposure A/B), uplift straps or connectors shall be provided in accordance with Table 8.14. Wind uplift loads are calculated in accordance with the wind load provisions of ASCE 7-93 [3]. Values for intermediate wind uplift loads in Table 6.1 may be interpolated. Straps or connectors should be selected and installed such that the required capacities are achieved and in-line framing between the roof members and the walls studs is maintained.

C9.0 MECHANICAL, UTILITIES, AND INSULATION

This section provides references to the applicable building codes that must be followed when installing mechanical and utility equipment or services in residential steel-framed houses. The following recommendations reflect typical methods of construction.

Hangers for plumbing Pipes

Hangers for plumbing pipes in steel framing should be secured with a 3/4-inch, #6, sharp-point screw in 18- and 27-mil studs and a #8, self-drilling screw in thicker studs.

Protection of Plumbing Pipes

Plastic pipes need not be protected from corrosion when in contact with steel studs. However corrosion is a possibility where copper comes in direct contact with the steel. Copper shall be separated from the steel framing by either of the following methods:

- Plastic insulators, foam insulators or grommets should be used where copper passes through a steel stud.
- Copper piping shall be wrapped with pipe insulation when the copper piping run(s) are parallel to the steel framing member. The insulation is intended to separate the copper piping from the steel member.

Attachment of Plumbing Fixtures

Plumbing fixtures can be attached with #10, low-profile screws with a sharp point for 18- and 27-mil studs and self-drilling point screws for 33-mil and thicker studs.

Electrical Boxes

Electrical boxes that have mounting brackets that attach to the side of the studs rather than to the front should be used to prevent bulges in the gypsum board. Electrical boxes can be attached with 3/4-inch, #6, sharp-point screws for 18- and 27-mil studs and #8, self-drilling screws for thicker studs.

Securing Wiring

Two 1/4 inch holes and a zip tie are used to secure multiple wires such as at a receptacle installation. Duct tape should not be used without consulting the electrical inspector. Where multiple wires exiting a box, such as in a double or triple gang box, standoff clips are typically used for securing the wiring. These clips can be installed with a single 3/4-inch, #6, sharp-point screw for 18- and 27-mil studs and #8, self-drilling screws for thicker studs.

Hangers for Ducts

Hangers for ducts are typically attached to steel framing with 3/4-inch, #8, self-drilling screws.

Bulkhead Framing

Ducts are normally run in attic space, interior walls, or drop down ceilings, however, bulkheads will occasionally be needed. Where this is desired, this framing is typically non-load bearing, and can be framed using 33 mil or thinner, C-shaped studs and tracks. Sections are assembled the same as walls with #8 self-drilling, low-profile screws.

Batt Insulation

When batt type insulation is to be installed in the cavity of the framing member a “full width” type batt will be required in order to span from framing member web to framing member web. While batt insulation may friction fit between studs, duct tape may be used to hold the insulation in place until the gypsum board is installed.

Exterior-Foam Insulation

The installation of foam type insulation may require some additional lateral support for the framing members due to the forces of the wet foam product expanding during its curing process, therefore, it is recommended that the manufacturer be contacted for the installation preparation and installation instructions. Foam sheathing are typically installed on steel framed walls using any of the following:

- Self-drilling screws with washers to prevent the screws from pulling through.
- Construction adhesive applied to the studs to hold the foam in place before the siding material is applied.
- Double-sided tape applied to the studs to hold the foam until the siding is applied.

Where plywood or oriented strand board (OSB) sheathing is used, roofing nails, screws, or adhesive may be used to attach the foam.

In many climates the use of board type insulation may be required to provide a thermal break between the steel framing and the exterior temperatures. Suggested solutions may be found in the AISI publication RG-9405 [29] entitled “Thermal Design Guide For Exterior Walls”. For a complete energy analysis a design professional that is familiar with thermal analysis of steel framing may be consulted. Table C9.0 provides a partial list of fiber glass insulation properties. Table C9.1 provides a partial list of foam sheathing R values at 75^{o}F mean temperature. Both Tables C9.0 and C9.1 are provided here for reference only. Manufacturers data, publications, and technical catalogs shall be consulted.

Table C9.0
Fiber Glass Insulation

Product	R-Value	Thickness	Width[1]	Length	Face
CertainTeed	R-19	6¼"	24	48"	Unfaced
CertainTeed	R-19	6¼"	16 & 24	96"	Unfaced
CertainTeed	R-19	6¼"	16 & 24	96"	Kraft
CertainTeed	R-11	3½"	16 & 24	96"	Kraft
CertainTeed	R-19	6¼"	16 & 24	48"	Flame Foil
CertainTeed	R-13	3½"	16 & 24	48"	Flame Foil
CertainTeed	R-11	3½"	16 & 24	48"	Flame Foil
Owens Corning	R-11	3½"	16 & 24	96"	Unfaced, Kraft, or Foil
Owens Corning	R-13	3½"	16 & 24	96"	Unfaced, Kraft, or Foil
Owens Corning	R-19	6¼"	16 & 24	96"	Unfaced, Kraft, or Foil

For SI: 1 inch = 2.54 mm, $^oF = 1.8(^oC) + 32$

1 Width is “full width” type batt insulation.

Table C9.1
Foam Sheathing
R-Values at 75°F Mean Temperature

Product	Nominal Board Thickness (inches)				
	1/2"	3/4"	1"	1-1/2"	2"
Celotex Tuff-R	4.0	5.6	8.0	12.0	16.0
Celotex Thermax	3.6	5.4	7.2	10.8	14.4
Dow Syrofoam	3.0	4.0	5.0		

For SI: 1 inch = 25.4 mm, $^oF = 1.8(^oC) + 32$

Baseboard and Other Interior Trim Attachments

Several methods are used to secure trim to steel studs. The following is a list of current methods of attaching interior trim, cabinets and vanities, closet shelving, and gypsum board installation. The information is provided for guidance only.

- Construction adhesive. Finishing nails driven at criss-cross angles into the track in pairs hold the trim firmly in place while the adhesive dries (see Figure C9.1 below).

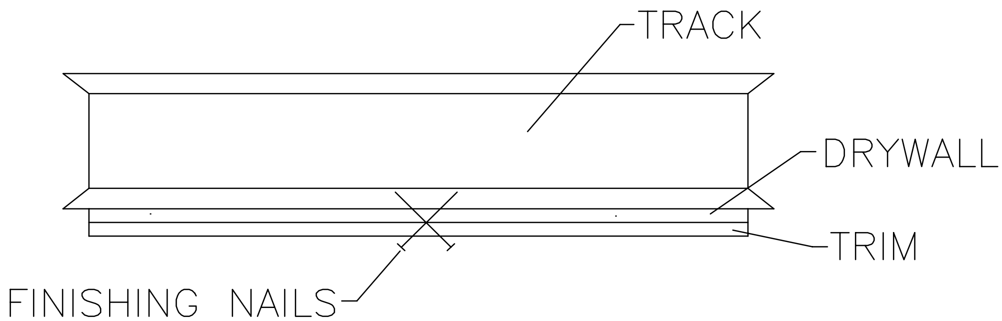


Figure C9.1 Plan View of Trim Attached With Finishing Nails

- Self-drilling finishing screws.
- A wood nailer may be installed during framing or 2 x 4 blocks may be placed in the track after the wall is framed. This will allow the trim to be attached with finishing nails.

Cabinets and Vanities

Special consideration has to be made to provide for the installation of cabinetry. Use one of the three methods listed below (see Figure C9.2).

- Wood blocking may be used between the studs. The blocking should be notched on one end at the lip of the stud.
- A 33 mil minimum thickness track may be used between the studs. The flanges at each stud should be notched. The track should be fastened with two screws.

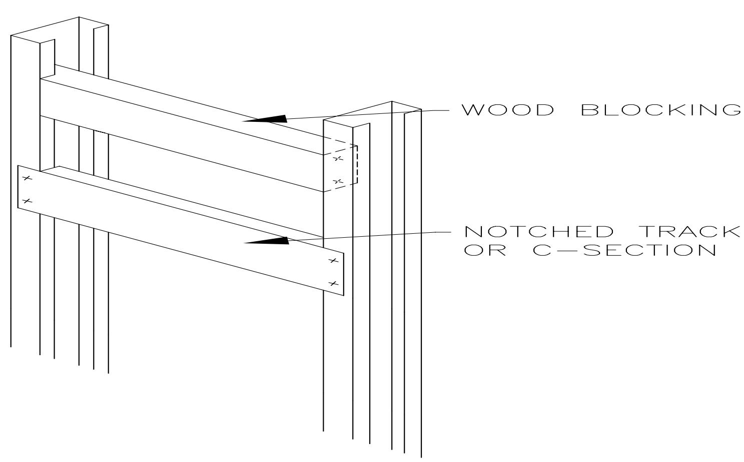


Figure C9.2 Cabinet Blocking Detail

Closet Shelving

Bugle-head fasteners are commonly used to attach wood supports for closet shelving. Sharp-point screws are used for 18- and 27-mil thick studs and wing-tips screws for 33-mil and thicker studs. Hex-head or low-profile screws can be used for the attachment of shelving brackets.

Gypsum Board Installation

Number 6, 1-1/4-inch bugle-head screws with sharp-point tips shall be used to hang gypsum board on 18- to 33-mil thick steel studs. Self-drilling #6, 1-1/4 inch, bugle-head screws are used to hang gypsum board on thicker studs. Construction adhesive may also be used. Refer to ASTM C955 [30] and ASTM C1002 [16].

C10.0 CONSTRUCTION GUIDELINES

The construction guidelines are provided to supplement the requirements of the *Prescriptive Method* and are considered good construction practices. These guidelines should not be considered comprehensive. Manufacturer's catalogs, recommendations, and other technical literature should also be consulted. Additional construction guidelines are provided below.

General

- Installation tolerances are very critical in achieving an acceptable floor or roof line in establishing effective bracing. User is encouraged to use string line, plum bob, level or transit in order to achieve acceptable installation tolerances.

Tools and Fasteners

Proper tools and techniques are very essential to any trade. Table C10.0 provides a list of tools that are commonly used in residential steel framing.

- Cutting methods which cause significant heating of the steel or damage to the coatings shall only be used when the galvanized coating is repaired.

- Welding, in lieu of fastening, is permitted provided that weld capacity is shown to exceed screw(s) capacity. All shop and field welds shall be brushed clean and provided with a corrosion protective metallic coating. Typically, steel members that are 33 mil in thickness or thinner should not be welded without the approval of a design professional. All welding shall be performed using certified welders specializing in cold-formed steel.

- Screws shall typically be driven through the thinner material into the thicker material.

- Installation drilling tools shall drive screws at lower speeds of 2500 RPM for screw sizes up to #10 and 1800 RPM for #12 (2500 RPM is acceptable provided that care is taken to minimize heat buildup).

- Pre-drilled holes for preset bolts shall not be oversized more than 1/16 inch for bolt sizes up to 1/2 inch and no more than 1/8 inch for bolt sizes larger than 1/2 inch in diameter.

Floor Joists

- Stiffeners are required at all joist bearing locations (i.e. where the joists sit on the bearing wall or beam). It is equally acceptable to install the stiffener on either face of the joist web. The required length (or height) of the stiffener need not be the full depth of the joist but rather may be allowed a 1/4 inch tolerance on each end of the stiffener. For example an 8 inch joist may have a 7-1/2 inch minimum stiffener length equally positioned over the height of the joist web. The end of a stiffener should not be more than 1/2 inch from either the top or bottom of the joist.

Walls

- Wall bridging will use the same pattern of blocked bay at the end of each run with additional intermediate blocked bays at 12 foot on center for lengths of walls greater than 12 feet. Wall bridging is not necessary if appropriate sheathing is placed on both flanges of the stud prior to loading the wall.

- Verify that interior non-load bearing partitions located under horizontal load carrying members (i.e. joists, rafters, and trusses) are attached in a manner that will allow for vertical deflection of the framing above without inducing load on the partition.

- A sill sealer, or equivalent, should be provided between the underside of the wall when fastened directly to concrete.

Trusses

- The *Prescriptive Method* calls for pre-engineered (pre-designed) trusses to be designed by others.

- Dimensions and proper bearing locations as shown on truss design drawing shall always be verified before starting installation of the truss.

- Temporary construction bracing should remain in place as long as necessary for the safe and acceptable completion of the roof or floor and may also remain in place after permanent bracing is installed.

- Trusses are laterally unstable until bracing is properly installed; necessary caution should be employed during the installation process.

- Overloading of roof trusses before permanent bracing and/or sheathing is installed shall not be permitted.

- Heavy construction loads such as stacks of plywood, gypsum board, bricks, HVAC units, etc. should never be placed on trusses before they are properly braced. Trusses are not typically designed for dynamic loads (moving loads). Sleepers for mechanical equipment should be located at panel points or over main supporting members, or on trusses which have been designed to carry such loads.

- Trusses shall not be placed over loose lintels, shelf angles, headers, beams, or other supporting structures not securely attached to the building.

- Trusses which do not meet interior load bearing walls should be shimmed for adequate bearing. Trusses shall not be pulled down to any interior partition.

Table C10.0
Tools Recommended for Use with Residential Steel Framing

Cutting

- Aviation snips - cuts up to 33-mil material and makes cuts for coping track flanges.
- 14-gauge swivel head electric shear - cuts up to 68-mil material.
- 14-inch chop saw - for cutting multiple sections simultaneously, especially partition studs.
- Step drill bit, 1 inch - for drilling holes in studs and track.
- Hole punch, 1¼ inch - for field punching holes for the installation of electrical and plumbing systems.

Fastening

- Adjustable clutch screw gun - with industrial motor (5.4 amps), 0-2500 rpm variable speed, reversible, bit tip holder release, adjustable torque control for framing.
- Magnetic bit tip holder and #2 Phillips bit tips.
- 5/16-inch magnetic hex driver for hex-head screws.
- Two pair of 3-inch, two pair of 6-inch, and one pair of 12-inch locking C-clamps with regular tips for clamping steel together while fastening.
- Deep-throat bar clamp - for clamping headers in wall sections while fastening.
- Gypsum board screw gun - with industrial motor (5.4 amps), 0-4000 rpm variable speed, reversible, with depth locating nose piece for sheathing and gypsum board installation.

Miscellaneous

- 3½-inch and 5-inch hand seamers - for bending and coping track.
- Bull-nose pliers - for removing screws.
- Magnetic level
- Felt markers - for layout and cuts (black and red).
- Other miscellaneous tools include: tape measure, speed square, utility knife, wallboard ax, and 50' grounded extension cords.

C11.0 REFERENCES

[1] *Prescriptive Method for Residential Cold-Formed Steel Framing,* Second Edition. Prepared for the US Department of Housing and Urban Development, the American Iron and Steel Institute, and the National Association of Home Builders, by the NAHB Research Center, Inc., Upper Marlboro, MD. August 1997.

[2] *Specification For The Design Of Cold Formed Steel Structural Members* (August 19, 1986 Edition with December 11, 1989 Addendum). American Iron and Steel Institute (AISI), Washington, DC. 1989.

[3] *Minimum Design Load for Buildings and Other Structures (*ASCE 7-93). American Society of Civil Engineers, New York, NY. 1993.

[4] CABO (1995), *One and Two Family Dwelling Code*, 1995 Edition, Council of American Building Officials.

[5] *The BOCA National Building Code.* Building Officials & Code Administrators International, Inc. (BOCA). Country Club Hills, Illinois. 1993.

[6] *Uniform Building Code*. International Conference of Building Officials (ICBO). Whittier, California. 1994.

[7] *Standard Building Code*. Southern Building Code Congress International, Inc. (SBCCI). Birmingham, Alabama. 1994.

[8] ASTM A 653 / A 653M - 1996 *Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process*, American Society for Testing and Materials (ASTM), West Conshohocken, PA. 1996.

[9] ASTM A 792 / A 792M *Standard Specification for Steel Sheet, 55% Aluminum-Zinc Alloy-Coated by the Hot-Dip Process*, American Society for Testing and Materials (ASTM), West Conshohocken, PA. 1996.

[10] ASTM A 875 / A 875M *Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy Metallic-Coated by the Hot-Dip Process*, American Society for Testing and Materials (ASTM), West Conshohocken, PA. 1996.

[11] ASTM C 645 - 96a *Standard Specification for Nonstructural Steel Framing Members*, American Society for Testing and Materials (ASTM), West Conshohocken, PA. 1996.

[12] ASTM A 924 / A 924M - 96a *Standard Specification for General Requirement for Steel Sheet, Metallic-Coated by the Hot-Dip Process*, American Society for Testing and Materials (ASTM), ASTM, West Conshohocken, PA. 1996.

[13] LaBoube, R. (1996), *Design Guide For Cold-Formed Steel Beams With Web Perforations*. University of Missouri-Rolla, Rolla, MO. November 1996.

[14] ICBO Evaluation Service, Inc. No. AC 46, *Acceptance Criteria for Steel Studs and Joists*, International Conference of Building Officials (ICBO). Approved July, 1992; Effective April, 1993. Whittier, California.

C11.0 REFERENCES (continued)

[15] SAE J-78-1979, *Steel Self Drilling Tapping Screws*, Society of Automotive Engineers (SAE), 1979.

[16] ASTM C 1002-96ae1 *Standard Specification for Steel Drill Screws for the Application of Gypsum Board or Metal Plaster Bases*, American Society for Testing and Materials (ASTM), West Conshohocken, PA. 1996.

[17] ASTM C 954-96ae1 *Standard Specification for Steel Drill Screws for the Application of Gypsum Panel Products or Metal Plaster Bases to Steel Studs From 0.033 in. (0.84 mm) to 0.112 in. (2.84 mm) in Thickness*, American Society for Testing and Materials (ASTM), West Conshohocken, PA. 1996.

[18] AISI Publication RG-933, *Fasteners for Residential Steel Framing*, American Iron and Steel Institute, Washington DC, June 1993.

[19] CCFSS Technical Bulletin Vol. 2, No. 1, *AISI Specification Provisions for Screw Connections*, Center for Cold-Formed Steel Structures, University of Missouri-Rolla, Rolla, MO; February 1993.

[20] ASTM Standard A 307 - 94 *Standard Specification for Carbon Steel Bolts and Studs, 60000 PSI Tensile Strength*, American Society for Testing and Materials (ASTM), ASTM, West Conshohocken, PA. 1994.

[21] *Structural Performance Requirements For Domestic Steel Framing,* Australian Institute of Steel Construction (AISC). Milsons Point, Australia. 1991.

[22] Kraus, C. and Murray, T. (1997), *Floor Vibration Criterion For Cold- Formed C-Shaped Supported Residential Floor Systems*; Report No. CE/VPI-ST 97/04, Virginia Tech, Blacksburg, VA, February 1997.

[23] WJE Report Number 73345, *Report of Laboratory Tests and Analytical Studies of Structural Characteristics of Cold-Formed Steel-Joists Floor Systems*, Wiss, Janney, Elstner Associates, Inc. San Francisco, California. December 1977.

[24] AISI Publication RG-9518, *Design Guide For Cold-Formed Steel Trusses,* American Iron and Steel Institute (AISI), Washington, DC. December 1995.

[25] NAHB Research Center C*ombined Axial and Bending Load Tests of Fully-Sheathed Cold-Formed Steel Wall Assemblies*, Prepared for the US Department of Housing and Urban Development, The American Iron and Steel Institute, and the National Association of Home Builders, by the NAHB Research Center, Inc., Upper Marlboro, MD. July 1997.

[26] Serrette, Reynaud, et al., “Shear Wall Values for Light Weight Steel framing,” Prepared for the American Iron and Steel Institute, Washington, DC. January 1996.

[27] ASTM E 564 *Standard Method of Static Load Test for Shear Resistance of Framed Walls for Buildings.* American Society for Testing and Materials (ASTM), West Conshohocken, PA. 1995.

[28] NAHB Research Center *Monotonic Tests of Cold-Formed Steel Shear Walls with Openings.* Prepared for the US Department of Housing and Urban Development, the American Iron and Steel Institute, and the National Association of Home Builders, by the NAHB Research Center, Inc., Upper Marlboro, MD. July 1997.

C11.0 REFERENCES (continued)

[29] AISI Publication RG-9405, *Thermal Design Guide For Exterior Walls*, American Iron and Steel Institute *(AISI)*, Washington, DC. December 1995.

[30] ASTM C 955-96a *Standard Specification for Load-Bearing (Transverse and Axial) Steel Studs, Runners (Tracks), and Bracing or Bridging for Screw Application of Gypsum Board and Metal Plaster Bases,* American Society for Testing and Materials (ASTM), West Conshohocken, PA. 1996.

PART II

HOW TO USE THE *PRESRIPTIVE METHOD*

COMMENTARY ON THE *PRESCRIPTIVE METHOD* FOR RESIDENTIAL COLD-FORMED STEEL FRAMING

1. INTRODUCTION

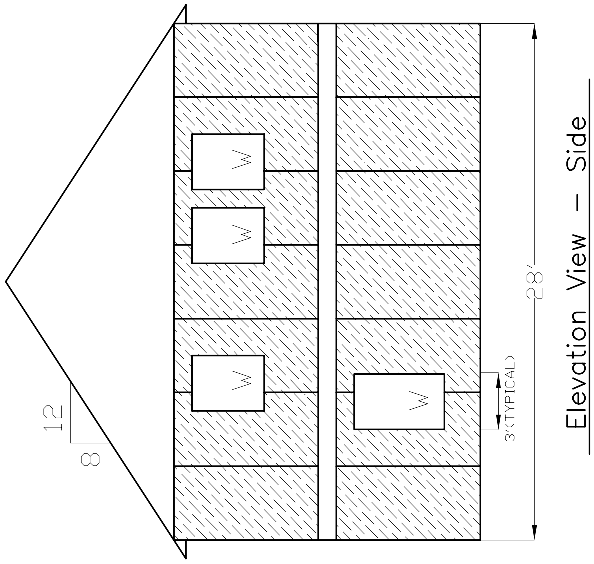
This part provides a step by step procedure on how to apply the requirements of the *Prescriptive Method* when designing a home. A typical residential building is used to demonstrate the application of steel framing requirements. The example building has the following physical characteristics:

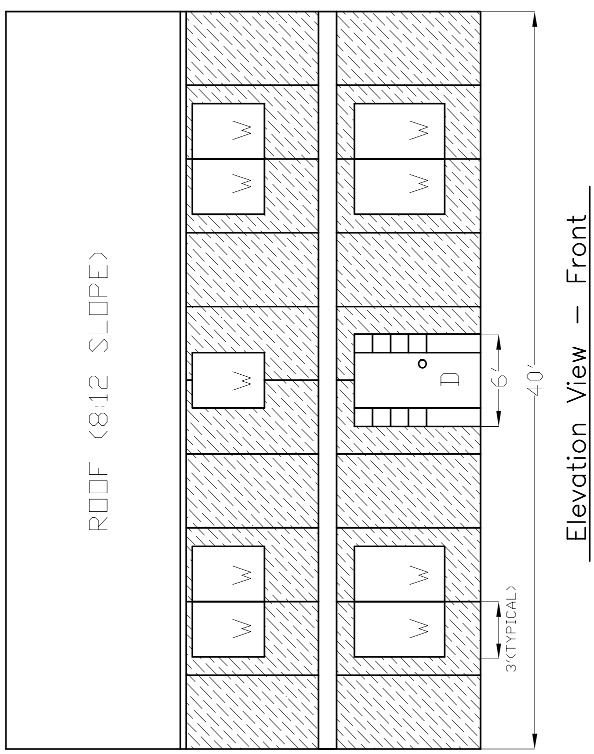
Building type:	Two story house (over an unfinished basement) with a center load bearing beam supporting the first floor and center load bearing wall supporting the second floor.
Building width:	28 feet
Building length:	40 feet.
First story wall height:	8 feet.
Second story wall height:	8 feet.
Wall stud spacing:	24" oc.
Floor joist spacing:	24" oc.
Roof framing:	Ceiling joists with rafters
Roof slope:	8:12
Roof soffit overhang:	1 foot

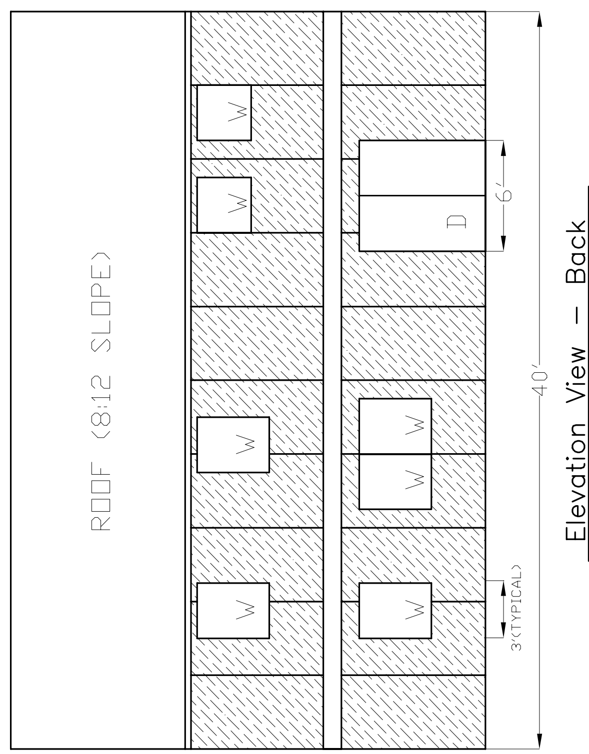
The following design criteria is applicable to the example home:

Ground Snow Load (psf)	Wind Speed (mph) and Exposure	Seismic Condition by Zone	Bottom Floor Live Load	Top Floor Live Load	Floor Dead Load
50	70 mph, Exp. C	Zone 0	40 psf	30 psf	10 psf

Building elevations are shown in the following page.


Elevation View – Side
28'
12
8
3'(TYPICAL)
W
W
W
W


Elevation View – Front
ROOF (8:12 SLOPE)
40'
6'
3'(TYPICAL)
D
W
W
W
W
W
W
W
W
W


Elevation View – Back
ROOF (8:12 SLOPE)
40'
6'
3'(TYPICAL)
D
W
W
W
W
W
W
W

2. FRAMING MEMBERS

The list below summarizes the framing member selection which result from applying the *Prescriptive Method* to the example building. A detailed description of the process is given in the following section. Connection requirements are not highlighted in this example since adequate details and tables are provided in the *Prescriptive Method*. Repeating this information would only detract from the introduction purpose of this example. The omission of detailed connection requirements from this example is not intended to diminish the importance of proper fastening. The novice user should devote substantial effort to this area, which cannot be adequately conveyed in a simple example. Equally important, the user must recognize that in-line framing is a requirement (i.e. floor joists, studs, and ceiling joists are aligned with one another).

Note: References to tables are in accordance with the *Prescriptive Method* sections

Member Location	Member Type		Reference
First Floor:			
Floor joists	2 x 8 x 54 mils	(@ 24” oc)	(Table 5.3)
Wall studs	2 x 4 x 54 mils	(@ 24” oc)	(Table 6.3)
Headers	2 - 2 x 4 x 54 mils	(3’-0” opening)	(Table 6.15b)
	2 - 2 x 8 x 68 mils	(6-foot opening)	(Table 6.17b)
Jack studs	2 x 4 x 54 mils.	(1 for 3’ & 6’ opening)	(Table 6.18)
King studs	2 x 4 x 54 mils.	(1 for 3’-0”opening) (2 for 6’-0”opening)	(Table 6.18)
Subflooring	23/32" TNG APA rated sheathing.		
Wall sheathing	7/16" OSB or 15/32" plywood, APA rated		
Second Floor:			
Floor joists	2 x 8 x 43 mils	(@ 24” oc)	(Table 5.3)
Wall studs	2 x 4 x 33 mils	(@ 24” oc)	(Table 6.2)
Headers	2 - 2 x 4 x 43 mils	(3’-6” opening)	(Table 6.15b)
Jack studs	2 x 4 x 33 mils.	(1 for 3’ & 6’ opening)	(Table 6.18)
King studs	2 x 4 x 33 mils.	(1 for 3’-0”opening)	(Table 6.18)
Subflooring	23/32" TNG APA rated sheathing.		
Wall sheathing	7/16" OSB or 15/32" plywood, APA rated		
Roof Framing			
Ceiling joists	2 x 8 x 43 mils	(@ 24” oc)	(Table 8.9)
Rafters	2 x 4 x 33 mils	(@ 24” oc)	(Table 8.12)
Roof Sheathing	15/32" plywood, APA rated		

3. FLOOR FRAMING SELECTION

Floor Joist Selection:

The design house is 28’-0” wide, and by taking into account the depth of the end wall supports and a beam or bearing wall at the middle span, the first floor clear span is 13’-3” and second floor span is 13’-6”. A 13’-3” clear span for the first floor framing members is used in conjunction with the desired 24 inch on-center spacing of joists and 40 psf live load in order to select a 2 x 8 x 54 mil joist member from Table 5.3 as shown below (Table 5.4 could have been used if a continuous joist is to be used). The 2 x 8 x 54 mil member happens to have an allowable span of exactly 13’-3”.

Applying the same sequence for the second floor framing, in this case the 13’-6” clear span in conjunction with the 24 inch on-center spanning and the 30 psf live load, Table 5.3 allows the use of a 2 x 8 x 43 mils joist member. The 2 x 8 x 43 mils member has an allowable span of 13’-7”.

In addition to the selection of joist members, Table 5.2 requires one #8 screw per flange of the joist member for connection to the joist track or two #8 screws per bearing stiffener. Bearing stiffeners shall be located, for purpose of this exercise, as illustrated in Figures 2.7, and 5.1.

Lateral bracing for the example building shall be provided in the form of subfloor sheathing panels, steel straps or gypsum board. For the application of plywood sheathing to the top flanges of the joists, Section 5.4 of the *Prescriptive Method* shall be employed with Table 5.2 which states that subflooring shall be fastened with #8 screws at six inches on-center at the edges and ten inches on-center at intermediate supports.

Table 5.3
Allowable Spans For Cold-Formed Steel Floor Joists
Single Span
33 ksi Steel

Nominal Joist Size [5]	30 psf Live Load			40 psf Live Load		
	Spacing (inches)			Spacing (inches)		
	12	16	24	12	16	24
2 x 6 x 54	13'-7"	12'-4"	10'-9"	12'-4"	11'-2"	9'-9"
2 x 8 x 33	15'-8"	13'-3"	8'-10"	14'-0"	10'-7"	7'-1"
2 x 8 x 43	17'-1"	15'-6"	13'-7"	15'-6"	14'-1"	12'-3"
2 x 8 x 54	18'-4"	16'-8"	14'-7"	16'-8"	15'-2"	13'-3"
2 x 8 x 68	19'-8"	17'-11"	15'-7"	17'-11"	16'-3"	14'-2"

Recognizing that the first floor framing is constructed over an unfinished basement, steel strapping may be used as bottom flange lateral support in lieu of gypsum ceiling board. The strapping shall be attached to the bottom of each flange of the joist members with #8 screws. In addition, blocking or bridging shall be spaced at 12-foot perpendicular to the framing direction, which complies with Section 5.4 and Figures 5.1 and 5.2. Since the second floor joists are framed over a finished space the use of gypsum board sheathing will be employed pursuant to Section 5.4.

4. WALL FRAMING SELECTION

Wall Studs Selection:

As part of the selection process for wall studs it had been assumed that the building is 28 feet in width, is subject to a wind speed of 70 mph /Exposure C (or 90 mph, exposure B) and 50 psf ground snow load, has walls 8 feet high per floor and a stud spacing of 24 inches on-center. For the second floor wall framing, Table 6.2 allows a 2 x 4 x 33 mil stud to be used as shown below. For the first floor stud wall framing (i.e. first story of a two story building), we find that Table 6.3 results in a 2 x 4 x 54 mil stud member as shown below. Since the exterior is structurally sheathed and the interior is finished with 1/2 in. gypsum wall board, the thickness of steel may be reduced to 43 mil in accordance with footnote 5 in Table 6.3. A #8 screw through each flange is used to fasten studs below to the top and bottom track pursuant to Table 6.14.

As currently required in the *Prescriptive Method*, all exterior portions of the walls are fully sheathed. The interior face of the wall is assumed to have a gypsum wall board finish. Section 6.4 of the *Prescriptive Method* provides the proper fastening requirements of the gypsum board. Interior load bearing walls(s) are sheathed with gypsum board on each side in accordance with Section 6.4.

Table 6.2
Steel Stud Thickness for 8' Walls Supporting Roof and Ceiling Only
(One Story or Second Floor of a Two Story Building)
33 ksi Steel

Wind Speed		Member Size[3]	Member Spacing (inches)	Stud Thickness (mils)[1,2]															
				Building Width (feet)[4,5]															
				24				28				32				36			
Exp. B	Exp. C			Ground Snow Load (psf)				Ground Snow Load (psf)				Ground Snow Load (psf)				Ground Snow Load (psf)			
				20	30	50	70	20	30	50	70	20	30	50	70	20	30	50	70
70 mph		2x4	16	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
			24	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	43
		2x6	16	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
			24	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
80 mph	70 mph	2x4	16	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
			24	33	33	33	33	33	33	33	33	33	33	33	43	33	33	43	43
		2x6	16	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33

Table 6.3
Steel Stud Thickness for 8' Walls Supporting One Floor, Roof and Ceiling
(First Story of a Two Story Building)
33 ksi Steel

Wind Exposure		Member Size[3]	Member Spacing (inches)	Required Stud Thickness (mils)[1,2] Building Width (feet)[4,5,6]															
				24				28				32				36			
				Ground Snow Load (psf)				Ground Snow Load (psf)				Ground Snow Load (psf)				Ground Snow Load (psf)			
Exp. B	Exp. C			20	30	50	70	20	30	50	70	20	30	50	70	20	30	50	70
70 mph		2x4	16	33	33	33	33	33	33	33	33	33	33	33	43	33	33	33	43
			24	43	43	43	43	43	43	43	43	43	43	43	54	43	43	54	54
		2x6	16	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
			24	33	33	33	33	33	33	33	43	33	33	43	43	33	43	43	54
80 mph	70 mph	2x4	16	33	33	33	33	33	33	33	33	33	33	33	43	33	33	43	43
			24	43	43	43	54	43	43	54	54	54	54	54	54	54	54	54	54
		2x6	16	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	43
			24	33	33	33	33	33	33	33	43	33	33	43	43	43	43	43	54
90 mph	80 mph	2x4	16	33	33	33	33	33	33	43	43	43	43	43	43	43	43	43	43
			24	54	54	54	54	54	54	54	54	54	54	54	68	54	54	68	68
		2x6	16	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
			24	33	33	33	33	33	33	43	43	43	43	43	43	43	43	43	54

Header Selection:

In this example there are two header sizes, the 6’-0” wide headers for the entrance doorway, slider door, and large window opening at the first floor level, and the 3’-0” wide window openings throughout the remainder of the building. Three tables will be employed in this exercise to determine the header types and the number of jack and king stud members to use in supporting the header. They are Tables 6.15b, 6.17b, and 6.18.

Both 6’-0” openings are located at the first floor of the building, therefore Table 6.17b shall be used to determine the header member sizes. To recap, the building is 28’-0” wide, is located in an area with a ground snow load of 50 psf, and the headers are supporting one floor, roof and ceiling with center load bearing wall. Table 6.17b illustrates that a 2-2 x 8 x 68 mil header will suffice as it can span up to 6’-10”. Supporting this header will be 1-jack stud and 2-king studs in accordance with Table 6.18 (Opening category 5’-6” to 8’-0”). Fastening requirements of the jack and king stud members dictate that 4-#8 screws are to be used for a header spanning between 4’-0” and 8’-0” in accordance with Table 6.19 of the *Prescriptive Method*. Figures 6.9 and 6.10 have been provided to clearly illustrate the configuration and fastening layout for the framing components.

Applying the same process to the 3’-0” window openings both Tables 6.15b (Header supporting roof and ceiling only) and 6.17b (Header supporting one floor, roof, and ceiling) must be used. For the first floor application 2-2 x 4 x 43 mils type header will be required with support provided by one jack and one king stud at each end of the header. The second floor windows require a minimum 2-2 x 4 x 43 mil header which will be supported by one jack and one king stud at each end of the header in accordance with the Table 6.18 of the *Prescriptive Method*.

Table 6.17b
Allowable Header Spans for
Headers Supporting One Floor, Roof and Ceiling [1,2]
First story of a two-story building with center load bearing Beam
33 ksi

Nominal Member Size [3]	50 psf Ground Snow Load				70 psf Ground Snow Load			
	Building Width [4]				Building Width [4]			
	24'	28'	32'	36'	24'	28'	32'	36'
2-2 x 4 x 33	2'-2"	-	-	-	-	-	-	-
2-2 x 4 x 43	3'-3"	3'-1"	2'-11"	2'-8"	3'-0"	2'-9"	2'-6"	2'-3"
2-2 x 4 x 54	3'-8"	3'-5"	3'-3"	3'-1"	3'-5"	3'-2"	3'-0"	2'-10"
2-2 x 4 x 68	4'-1"	3'-10"	3'-8"	3'-6"	3'-9"	3'-7"	3'-4"	3'-2"
2-2x 4 x 97	4'-10"	4'-7"	4'-4"	4'-1"	4'-6"	4'-3"	4'-0"	3'-10"
2-2x 6 x 33	-	-	-	-	-	-	-	-
2-2 x 6 x 43	4'-0"	3'-7"	3'-2"	2'-11"	3'-5"	3'-1"	2'-9"	2'-6"
2-2x 6 x 54	4'-11"	4'-8"	4'-5"	4'-2"	4'-7"	4'-4"	4'-1"	3'-11"
2-2 x 6 x 68	5'-7"	5'-3"	4'-11"	4'-9"	5'-2"	4'-10"	4'-7"	4'-4"
2-2 x 6 x 97	6'-8"	6'-3"	5'-11"	5'-7"	6'-2"	5'-9"	5'-6"	5'-2"
2-2 x 8 x 33	-	-	-	-	-	-	-	-
2-2 x 8 x 43	3'-1"	2'-9"	2'-5"	2'-3"	2'-8"	2'-4"	-	-
2-2x 8 x 54	6'-2"	5'-5"	4'-11"	4'-5"	5'-3"	4'-8"	4'-2"	3'-9"
2-2 x 8 x 68	7'-3"	6'-10"	6'-6"	6'-2"	6'-9"	6'-4"	6'-0"	5'-8"
2-2 x 8 x 97	8'-8"	8'-2"	7'-9"	7'-5"	8'-1"	7'-7"	7'-2"	6'-10"
2-2 x 10 x 43	2'-7"	2'-3"	2'-1"	-	2'-3"	-	-	-
2-2 x 10 x 54	5'-1"	4'-6"	4'-1"	3'-8"	4'-5"	3'-11"	3'-6"	3'-2"
2-2 x 10 x 68	8'-7"	8'-1"	7'-8"	7'-3"	7'-11"	7'-6"	7'-0"	6'-4"
2-2 x 10 x 97	10'-3"	9'-8"	9'-2"	8'-9"	9'-6"	9'-0"	8'-6"	8'-1"

Table 6.15b
Allowable Header Spans for
Headers Supporting Roof and Ceiling Only[1,2]
33 ksi

Nominal Member Size[3]	50 psf Ground Snow Load				70 psf Ground Snow Load			
	Building Width[4]				Building Width[4]			
	24'	28'	32'	36'	24'	28'	32'	36'
2-2 x 4 x 33	3'-0"	2'-7"	2'-4"	2'-1"	2'-4"	2'-1"	--	--
2-2 x 4 x 43	3'-10"	3'-7"	3'-4"	3'-2"	3'-5"	3'-2"	3'-0"	2'-9"
2-2 x 4 x 54	4'-3"	4'-0"	3'-9"	3'-7"	3'-10"	3'-7"	3'-4"	3'-2"
2-2 x 4 x 68	4'-10"	4'-6"	4'-3"	4'-0"	4'-3"	4'-0"	3'-9"	3'-7"
2-2x 4 x 97	5'-8"	5'-4"	5'-0"	4'-9"	5'-1"	4'-9"	4'-5"	4'-3"
2-2x 6 x 33	2'-6"	2'-2"	--	--	--	--		
2-2 x 6 x 43	5'-2"	4'-10"	4'-4"	3'-11"	4'-5"	3'-10"	3'-5"	3'-1"
2-2x 6 x 54	5'-10"	5'-5"	5'-1"	4'-10"	5'-2"	4'-10"	4'-7"	4'-4"

Table 6.18
Total Number of Jack and King Studs Required at Each End of an Opening

Size of Opening	24" o.c. stud spacing		16" o.c. stud spacing	
	No. of Jack studs	No. of King studs	No. of Jack studs	No. of King studs
Up to 3'-6"	1	1	1	1
> 3'-6" to 5'-0"	1	2	1	2
> 5'-0" to 5'-6"	1	2	2	2
> 5'-6" to 8'-0"	1	2	2	2
> 8'-0" to 10'-6"	2	2	2	3

Shearwall/Structural Sheathing:

To determine the minimum amount of structural sheathing required for our example house, Sections 6.8.2 and 6.8.3 and Table 6.20 must be employed with the design example information. The design house is two story house, is 28’-0” wide (endwalls) by 40’-0” long (sidewalls), has a roof slope of 8:12, and is located in a 70 mph / Exposure C area. At the beginning of the example, the entire house was assumed to be covered with structural sheathing. In fact, this is a requirement to meet the wall bracing requirements of Section 6.8.2.

Addressing only the first floor of this two story house, we see that the closest roof slope category is 9:12. Under the appropriate wind speed/exposure category the minimum percentage of full-height sheathing on the endwall is 75%. Similarly the side wall percentage is 50%. The design roof slope is 8:12, therefore the following:

FIRST FLOOR ENDWALL:

a. 9:12 (75%) minus 6:12 (55%) equals three equal percentage increments at 6.66%; (75% - 55%)/3 = 6.66
9:12 minus 8:12 (Design house slope) equal one increment at 6.66%.

b. 9:12 (75%) minus 1:12 (6.66%) indicates the endwall will require 68.3% of coverage

c. Therefore, a 28’-0” wide house requires 68.3% coverage which equals 19’-2”. Rounding to a figure more appropriate to plywood dimension, say 20’-0” or five 48-inch-wide panels

d. Remember that Section 6.8.2 requires a minimum of one 48-inch-wide panel at each corner

FIRST FLOOR SIDEWALL:

a. 9:12 (50%) minus 6:12 (40%) equals three equal percentage increments at 3.33%; (50% - 40%)/3 = 3.33
9:12 minus 8:12 (Design house slope) equals one increment at 3.33%.

b. 9:12 (50%) minus 1:12 (3.33%) indicates the sidewall will require 46.6% of coverage

c. Therefore, 40’-0” long house requires 46.6% which equals 18’-8”
Rounding to a figure more appropriate to plywood dimensions, say 20’-0” or five 48 inch wide panels

d. Table 6.20, Note #2, since the sidewall linear coverage is less than the end walls, the minimum 19’-2” figure must be applied to the sidewall coverage. If the five 48-inch-wide panels are used on each wall, the sheathing complies with the Table and note #1.

e. Reminder, that Section 6.8.2 requires a minimum of one 48-inch-wide panel at each corner.

SECOND FLOOR ENDWALL:

Applying the method above to the second story level of the example building the following results are obtained:

28’-0” multiplied by 40% requires 11’-3” of full-height sheathing, (the 40% value was interpolated between 45% and 30% similar to the first floor calculation)

SECOND FLOOR SIDEWALL:

40’-0” multiplied by 30% requires 12’-0” of full-height sheathing.

No adjustment to the lengths of full-height sheathing is necessary in either case.

Table 6.20
Minimum Percentage of Full Height
Structural Sheathing Along Exterior Wall Lines[1,2,3,4]

Wall Condition	Roof Slope	Wind Speed (mph) and Exposure			
		Up To 70 C or 90 A/B or Seismic Zones 0, 1, & 2		Up To 90 C or 100 A/B	
		Endwall	Sidewall	Endwall	Sidewall
One-Story or Second Floor of Two-Story Construction	3:12	30%	30%	30%	30%
	6:12	30%	30%	40%	30%
	9:12	45%	30%	75%	50%
	12:12	60%	40%	100%	70%
First Floor of Two-Story Construction	3:12	50%	35%	80%	55%
	6:12	55%	40%	90%	60%
	9:12	75%	50%	Design Required	
	12:12	95%	65%		

5. ROOF FRAMING SELECTION

Ceiling Joists Selection

The design house is 28'-0" wide, and by taking into account the depth of the end wall supports and a beam or bearing wall at the middle span, the ceiling joist clear span is 13'-9". A 13'-9" desired ceiling joist span is used in conjunction with the 24 inch on-center spacing of members and the 20 psf attic live load in order to select a 2 x 8 x 54 mil joist member from Table 8.11 as shown below. Moreover, no stiffeners are required for the ceiling joists.

Table 8.11
Allowable Spans For Cold-Formed Steel Ceiling Joists (33 ksi)
Two Equal Spans Without Bearing Stiffeners
20 Lbs. per Sq. Ft. Live Load (Limited Attic Storage) [1,2,3,4]

Nominal Joist Size [5]	Lateral Support of Top (Compression Flange)					
	Unbraced		Mid-Span Bracing		Third-Point Bracing	
	Spacing (inches)		Spacing (inches)		Spacing (inches)	
	16	24	16	24	16	24
2 x 4 x 68	14'-0"	12'-4"	15'-9"	12'-5"	15'-9"	12'-5"
2 x 4 x 97	16'-7"	14'-5"	18'-3"	15'-2"	18'-3"	15'-2"
2 x 6 x 33	9'-3"	6'-9"	9'-3"	6'-9"	9'-3"	6'-9"
2 x 6 x 43	13'-0"	9'-8"	13'-0"	9'-8"	13'-0"	9'-8"
2 x 6 x 54	14'-3"	12'-4"	16'-4"	12'-4"	16'-4"	12'-4"
2 x 6 x 68	15'-6"	13'-9"	20'-3"	15'-7"	20'-3"	15'-7"
2 x 6 x 97	18'-0"	15'-11"	23'-7"	20'-6"	25'-10"	21'-0"
2 x 8 x 33	-	-	-	-	-	-
2 x 8 x 43	14'-0"	10'-2"	14'-0"	10'-2"	14'-0"	10'-2"
2 x 8 x 54	15'-9"	13'-10"	18'-9"	13'-10"	18'-9"	13'-10"
2 x 8 x 68	17'-0"	15'-2"	23'-3"	18'-3"	24'-2"	18'-3"
2 x 8 x 97	19'-6"	17'-3"	26'-0"	23'-2"	30'-11"	25'-11"

In addition to the selection of joist members, Table 8.1 requires two #10 screws per joist for connection to the top track of a load bearing wall. Bearing stiffeners are not required for the ceiling joists selected.

Lateral bracing for the selected ceiling joists shall be provided in the form of sheathing, steel straps or gypsum board. A 2 x 4 x 33 C-shape or track member or steel strapping may be used as bracing of the top flange. The strapping should be fastened to each top flange of the ceiling joists with a minimum of one #8 screw. Blocking or bridging shall also be installed in-line with the strapping at a maximum spacing of 12'-0" and at the termination of all straps. The blocking and/or bridging details are similar to those used for the steel floor assembly. Gypsum board shall be installed on the bottom flanges (tension flanges) of ceiling joists with #6 screws in accordance with Section 8.3.

Rafters Selection

The design house is 28’-0” wide, and, therefore, the horizontal rafter span is 14’-0”. The ground snow load is 50 psf and the design wind speed is 70 mph, exposure C.

The first step is to convert the design wind speed into an equivalent snow load using Table 8.13 as shown below. From that table, a 70 mph wind exposure C with roof slope of 8:12 is equivalent to a 30 psf ground snow load.

Table 8.13
Wind Speed to Equivalent Snow Load Conversion

Roof Slope	**Equivalent Ground Snow Load (psf)**				
	Wind Speed (mph) Exposure C				
	70	**80**	**90**	**100**	**110**
3 / 12	20	20	20	30	50
4 / 12	20	20	30	50	50
5 / 12	20	20	30	50	50
6 / 12	20	20	30	50	70
7 / 12	30	30	50	70	70
8 / 12	30	30	50	70	
9 / 12	30	50	50	70	
10 / 12	30	50	50	Design Required	
11 / 12	30	50	70		

The second step is to select the rafter size using the maximum of the ground snow load (50 psf) or the equivalent ground snow load converted from the wind speed (30 psf). In this case, the ground snow load of 50 psf controls. Using Table 8.12, a 2 x 8 x 54 mil rafter is selected with a maximum allowable span of 14’-6” and a 24” on center rafter spacing.

Ceiling joist shall be connected to the rafter (heel joint) with 5-#10 screws evenly spaced as shown in Table 8.2 below.

Table 8.12
Allowable Horizontal Rafter Spans[1,2,3]
33 ksi

Nominal Joist Size [4]	Ground Snow Loads							
	20 psf Ground		30 psf Ground		50 psf Ground		70 psf Ground	
	Spacing (inches)		Spacing (inches)		Spacing (inches)		Spacing (inches)	
	16	24	16	24	16	24	16	24
2 x 6 x 33	12'-8"	10'-4"	11'-9"	9'-7"	9'-11"	8'-1"	8'-10"	7'-2"
2 x 6 x 43	15'-5"	12' -7"	14'-3"	11'-8"	12'-1"	9'-10"	10'-8"	8'-9"
2 x 6 x 54	13'-0"	14'-2"	16'-1"	13'-1"	13'-8"	11'-2"	12'-1"	9'-10"
2 x 6 x 68	18'-1"	15'-10"	17'-3"	14'-9"	15'-4"	12'-6"	13'-6"	11'-1"
2 x 6 x 97	20'-1"	17'-6"	19'-1"	16'-8"	17'-1"	14'	15'-7"	13'-2"
2 x 8 x 33	15'-5"	11'-5"	14'-4"	9'-10"	10'-7"	7'-1"	8'-3"	5'-6"
2 x 8 x 43	19'-1"	15'-7'	17'-9"	14'-6"	15'-1"	12'-3"	13'-3"	10'-9"
2 x 8 x 54	22'-7"	18'-5"	21'-0"	17'-1"	17'-9"	14'-6"	15'-9"	12-10"
2 x 8 x 68	24'-7"	20'-9"	23'-4"	19'-3"	20'-0"	16'-4"	17'-8"	14'-5"

Table 8.2
Number of Screws Required For Heel Joint Connections[1]

Roof Slope	Building Width (feet)															
	24'				28'				32'				36'			
	Ground Snow Load (psf)				Ground Snow Load (psf)				Ground Snow Load (psf)				Ground Snow Load (psf)			
	20	30	50	70	20	30	50	70	20	30	50	70	20	30	50	70
3/12	5	6	9	12	6	7	10	13	7	8	12	15	8	9	13	17
4/12	4	5	7	9	5	6	8	10	6	6	9	12	6	7	10	13
5/12	4	4	6	7	4	5	7	9	5	5	8	10	5	6	9	11
6/12	3	4	5	7	4	4	6	8	4	5	7	9	4	5	7	10
7/12	3	3	5	6	3	4	5	7	4	4	6	8	4	5	7	9
8/12	3	3	4	5	3	3	5	6	3	4	5	7	4	4	6	8
9/12	2	3	4	5	3	3	4	6	3	4	5	6	3	4	6	7

The rafter to ridge member connection shall use four #10 screws at each leg of the angle as shown in Table 8.3 below.

Table 8.3
Number of Screws Required at Each Leg of Clip Angle
For Rafter to Ridge Member Connection[1]

Building Width (feet)	Ground Snow Load (psf)			
	0 to 20	21 to 30	31 to 50	51 to 70
24	2	3	4	4
28	2	3	4	5
32	3	3	4	5
36	3	4	5	6

Bottom flange rafter bracing can be accomplished using a 1-1/2” x 33 mil strap, 33 mil C-shape member, or 33 mil track at a maximum spacing of 8 feet along the rafter span. A 2 x 4 x 33 mil C-shape member at mid span is selected here. The C-shape member is fastened to each rafter with a minimum of one #8 screw.

Permanent roof bracing for the example building is provided by structural (roof) sheathing fastened to the rafters with #8 screws at 6 inches on center along the edges and 12 inches on center at interior supports in accordance with Table 8.1. Adequate temporary bracing of the roof framing is required for stability during construction until the roof sheathing is applied.

PART III

ENGINEERING CALCULATIONS

COMMENTARY ON THE *PRESCRIPTIVE METHOD* FOR RESIDENTIAL COLD-FORMED STEEL FRAMING

COMMENTARY ON THE *PRESCRIPTIVE METHOD* FOR RESIDENTIAL COLD-FORMED STEEL FRAMING

INTRODUCTION

This section of the document contains example calculations for the different requirements contained in the *Prescriptive Method*. The section starts out with the basic calculation of the different section properties of a typical C-shaped member, than the capacity of that member is calculated based on the requirements of the AISI Specification. The flow of the engineering calculations follow that of the *Prescriptive Method*: designing floor joists, wall studs, headers, shearwalls, and finally roof framing.

All references used in the design examples shown in this section are listed in Part I of this document.

SECTION PROPERTIES EXAMPLES

The following examples illustrate the calculation of various section properties for 2 x 8 x 54 mil and 2 x 4 x 43 mil C-shapes. The purpose of these examples is to show how the section properties are derived.

All properties are calculated in accordance with the AISI "*Specification For The Design Of Cold-Formed Steel Structural Members, August 19, 1986 Edition with December 11, 1989 Addendum*". The section properties are based on the requirements of Sections 2.1, 2.2, 2.3, 2.4, and 2.5 of the *Prescriptive Method.*

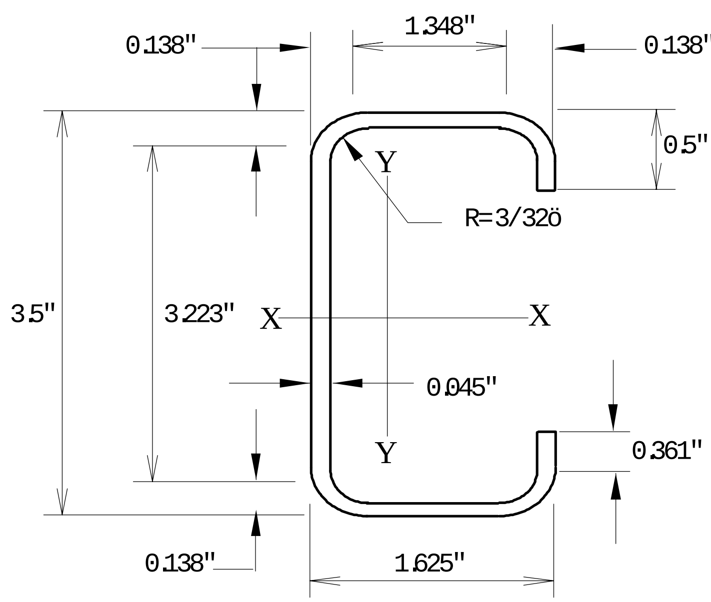
<u>Calculate the gross section properties of a 2 x 4 x 43 mil C-shape:</u>

The following information is either specified in the *Prescriptive Method* or is given in the AISI Specification:

Web size =	3.5 in.
Flange size =	1.625 in.
Lip size =	0.5 in.
Base metal thickness =	0.0423 in.
Inside bend radius =	0.09375 in.
Hole width =	1.5 in.
Hole length =	4 in.
Yield stress =	33,000 psi
Tensile strength =	45,000 psi

(AISI Section 5 of Supplementary Information)

Modulus of elasticity = 29,500,000 psi
(AISI Section 4 of Supplementary Information)
Shear modulus = 11,300,000 psi
(AISI Section 4 of Supplementary Information)



<u>Area:</u>

<u>Gross Area:</u>

Design thickness = Minimum uncoated thickness / 0.95 = 0.043 / 0.95 = 0.045 in.

a =	Web - 2(Radius + thickness)	=	3.5 - 2(0.0937 + 0.045) = 3.223 in.
b =	Flange - 2(Radius + thickness)	=	1.625 - 2(0.0937 + 0.045) = 1.348 in.
c =	Lip - (Radius + thickness)	=	0.5 - (0.0937 + 0.045) = 0.361 in.
u =	1.57 x (Radius + thickness/2)	=	1.57(0.0937 + 0.045/2) = 0.182 in.
r =	(Radius + thickness/2)	=	(0.0937 + 0.045/2) = 0.116 in.
α =	1		

The above properties are based on the equations given in the Supplementary Information Section of the AISI Specification:

Area = thickness [a + 2b + 2c + α (2u + 2c)] AISI Section 1.2
Area = 0.045[(3.228 + 2 x 1.347 + 2 x 0.361 + 1(2 x 0.183 + 2 x 0.361)] = **0.332 in^2**

<u>Net Area:</u>

Net area = Gross area - hole area

Net Area = 0.332 - 1.5 (0.045) = **0.265 in^2**

Weight per foot:

Gross weight per foot:

weight/foot = Gross area x density of steel = 0.322 (490/144) = **1.131 lb./ft.**

Net weight per foot:

weight/foot = Net area x density of steel = 0.265 (490/144) = **0.901 lb./ft.**

Gross Properties About X-Axis: (AISI Supplementary Information Section 1.2)

Gross moment of inertia, I_{xx}

$I_{xx} =$ $2t\{ 0.417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.0149r^3 + \alpha[0.0833c^3 + (c/4)(a - c)^2 + u(a/2 + 0.637r)^2 + 0.149r^3]\}$

$I_{xx} =$ 0.649 in.^{4}

Gross section modulus, S_{xx}

$S_{xx} =$ I_x / Y_{cg} $Y_{cg} = 3.5 / 2 = 1.75$ in.

$\boldsymbol{S_{xx}}$ = 0.649 / 1.75 = **0.371 in^3**

Radius of gyration, r_x

$r_x = [I_x / A]^{1/2}$
$\mathbf{r_x}$ = $[0.649 / 0.332]^{1/2}$ = **1.398 in.**

Gross Properties About Y-Axis:

Gross moment of inertia, I_{yy}

$I_{yy} =$ $2t\{ b(b/2 + r)^2 + 0.833b^3 + 0.356r^3 + \alpha[c(b + 2r)^2 + u(b + 1.637r)^2 + 0.149r^3]\} - \text{Area}\,(x')^2$

Where x' = Distance between centroid and web centerline.

$x' = (2t/\text{Area})\{ b(b/2 + r) + u(0.363r) + \alpha[u(b + 1.637r) + c(b + 2r)]\}$
x' = 0.522 in.

$\mathbf{I_{yy}}$ **= 0.124 in^4**

Gross section modulus, S_{yy}

$S_{yy} = I_y / X_{cg}$ $X_{cg} = x' + \text{thickness}/2 = 0.522 + 0.045/2 = 0.545$ in.

$\mathbf{S_{yy}}$ = 0.124 / 0.545 = **0.228 in^3**

Radius of gyration, r_y

$r_y = [I_y / A]^{1/2}$
$\mathbf{r_y} = [0.1240 / 0.332]^{1/2} = \mathbf{0.611\ in.}$

<u>Torsional Properties:</u>

<u>St. Venant torsion constant, J:</u>

$J = (t^3/3)[\ a + 2b + 2u + \alpha\ (\ 2c + 2u)]$ AISI Supplementary Information, Section 1.2.2
$J = [(0.0451)^3/3]\ [\ 3.222 + 2(1.347) + 2(0.183) + 1(2 \times 0.361 + 2 \times 0.183)]$

$\mathbf{J = 2.25 \times 10^{-4}\ in.^4}$

<u>Warping constant, C_w:</u>

The warping constant is calculated in accordance with equation 9 of Section 1.22 of the AISI Supplementary Information (Page III-9).

$\mathbf{C_w = 0.344\ in.^6}$

<u>Polar radii of gyration, r_o:</u>

The polar radius of gyration of a section is calculated about the centroidal principal axis.

$r_o = [r_x^2 + r_y^2 + x_o^2]^{1/2}$ Where $x_o = -\ (x' + m)$

m = distance between shear center and web centerline calculated in accordance with equation 6 of Section 1.2.2 of the Supplemental Information of the AISI Specification = 0.822 in..

$x_o = -\ (\ 0.522 + 0.822) = 1.344$
$r_o = [(1.398)^2 + (0.611)^2 + (1.344)^2\ ]^{1/2}$

$\mathbf{r_o = 2.033\ in.}$

<u>Torsional flexural constant, β:</u>

$\beta = 1 -\ (\ x_o / r_o\ )^2$

$\beta = 1 -\ (\ 1.344 / 2.033\ )^2$

$\boldsymbol{\beta} = \mathbf{0.563}$

<u>Web Crippling Strength:</u>

Calculate the web crippling strength of a 2 x 4 x 43 mil C-shape per Section C3.4 of the AISI Design Specification.

h/t = 3.5 / 0.045 = 77.78 < 200
R/t = 0.0937 / 0.045 = 2.082 < 6
N/t = 1.5 / 0.045 = 33.33 < 210
N/h = 1.5 / 3.5 = 0.429 < 3.5

Therefore, the equations in AISI Table C3.4-1 apply.

Equation C3.4-1, for stiffened flanges, end reactions, will be used:

$t^2kC_3C_4\,C_\theta\,[179 - 0.33(h/t)][1 + 0.01(N/t)]$ Eq. C3.4-1

$k = F_4 / 33 = 1$ Eq. C3.4-21
$C_3 = 1.33 - 0.33k = 1.0$ Eq. C3.4-12
$C_4 = 0.5 \le 1.15 - 0.15\ R/t \le 1.0 = 0.838$ Eq. C3.4-13
$C_\theta = 0.7 + 0.3\ (\theta / 90)^2 = 0.7 + 0.3(90/90)^2 = 1.0$ Eq. C3.4-20

Web crippling capacity = $(0.045)^2(1)(1)(0.838)(1)[179 - 0.33(77.78)][1 + 0.01(33.33)] \times 1000 =$ **347 lb.**

Calculate the web crippling strength of a back-to-back 2 x 4 x 43 mil C-shape per Section C3.4 of the AISI Design Specification.

Equation C3.4-3, shapes having multiple webs, will be used: $t^2F_yC_6[5.0 + 0.63(N/t)^{1/2}]$
$C_6 = 1 + (h/t)/750$ when $h/t \le 150$ = 1.095 Eq. C3.4-15

Web crippling capacity = $(0.045)^2(33{,}000)(1.095)[5 + 0.63(33.33)^{1/2}] =$ **632 lbs.**

2 x 8 x 54 mil C-Shape Section Properties:

The following section properties correspond to a 2 x 8 x 54 mil C-shape (From section property tables)
(Dimensions and configuration are in accordance with Section 2.0 of the *Prescriptive Method*)

Gross Area = 0.667 in^2	Net Area = 0.582 $in.^2$	X_o= - 0.935 in.
I_{xx} = 5.70 $in.^4$	S_{xx} = 1.425 $in.^3$	J = 71.3x10^{-5} $in.^4$
I_{yy} = 0.192 $in.^4$	S_{yy} = 0.150 $in.^3$ (gross section modulus)	C_w = 2.503 $in.^6$
r_x = 2.93 in.	r_y = 0.536 in.	r_o= 3.12 in.

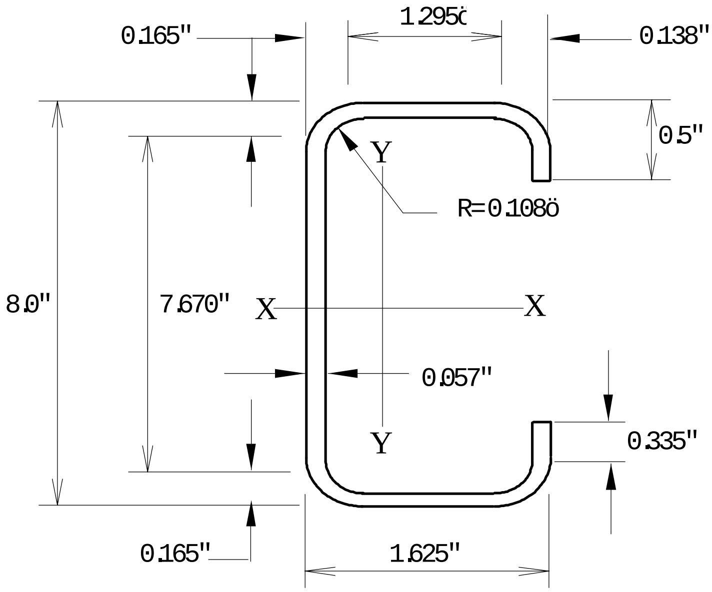
S_{yy} = 0.136 $in.^3$	(Effective section modulus)	β = 0.91
I_x = 5.70 $in.^4$	(Effective moment of inertia)	
S_x = 1.425 $in.^3$	(Effective section modulus)	
F_{ya} = 37.2 ksi		
M_a = 31,767 lb.-in. (2,647.5 ft-lb.)		
V_a = 1,9731 lbs.	(shear capacity for section with no holes)	
V_a = 1,566 lbs.	(shear capacity for section with holes)	
P_a = 1029 lbs.	(Allowable web crippling capacity, multiple webs)	
P_a = 471 lbs.	(Allowable web crippling capacity, single web)	

(All references to nomenclature, sections, and equation numbers are from the AISI Design Specification):

Design thickness = 0.057 in.

Minimum uncoated delivered thickness
= 0.057 x 0.95 = 0.054 in.
(AISI Section A3.4)
Inside bend radius = R
R = 2t = 2 x 0.0538 = 0.108 in.

Area = t[a + 2b + 2c + α (2u + 2c)]
(AISI Supplementary Information Section 1.22)



Where:

a = Web - 2(Radius + thickness) = 8.0 - 2(0.108 + 0.057) = 7.670 in.
b = Flange - 2(Radius + thickness) = 1.625 - 2(0.108 + 0.057) = 1.295 in.
c = Lip - (Radius + thickness) = 0.5 - (0.108 + 0.057) = 0.335 in.
u = 1.57 (Radius + thickness/2) = 1.57(0.108 + 0.057/2) = 0.214 in.
a = 1.0

Gross Area = 0.057(7.670 + 2 x 1.295 + 2 x 0.335 + 2 x 0.335 + 2 x 0.214) = **0.647 $in.^2$**
Net Area = Gross area - area of opening = 0.686 - 1.5 x 0.057 = **0.600 $in.^2$**

Weight per foot = Gross area x density of steel = 0.647 x 490 /144 = **2.20 lb./ft**.

Gross $\mathbf{Y_{cg}}$ = 8 / 2 = **4.00 in.**

Calculate gross moment of inertia, I_{xx}: (AISI Supplementary Information Section 1.2)

r = Radius + Thickness/2 = 0.136 in.

$I_{xx} = 2t\{ 0.417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.0149r^3 + \alpha[0.0833c^3 + (c/4)(a - c)^2 + u(a/2 + 0.637r)^2 + 0.149r^3]\}$

$\mathbf{I_{xx} = 5.70\ in^4}$

Gross section modulus, S_{xx}:

$\mathbf{S_{xx}} = I_{xx} / Y_{cg} = 5.70 / 4.00 =$ **1.425 in.**^{3}

Radius of gyration, r_x :

$\mathbf{r_x} = [(I_{xx}/\text{Area})]^{1/2} = [(5.70/0.647)]^{1/2} =$ **2.97 in.**

Calculate gross moment of inertia, I_{yy}:(AISI Supplementary Information Section 1.2)

$I_{yy} = 2t\{ b(b/2 + r)^2 + 0.833b^3 + 0.356r^3 + \alpha[c(b + 2r)^2 + u(b + 1.637r)^2 + 0.149r^3]\} - \text{Area } (x')^2$

Where x' = Distance between centroid and web centerline = $(2t/\text{Area})\{ b(b/2 + r) + u(0.363r) + \alpha[u(b + 1.637r) + c(b + 2r)]\} = 0.319$ in.

$\mathbf{I_{yy} = 0.192\ in.^4}$

Gross X_{cg} = x' + t/2 = **0.347 in.**

Gross section modulus, S_{yy}:

$\mathbf{S_{yy}} = I_{yy} / X_{cg} = 0.192 / 0.347 =$ **0.553 in.**^{3}

Radius of gyration, r_y :

$\mathbf{r_y} = [(I_{yy}/\text{Area})]^{1/2} = [(0.192/0.647)]^{1/2} =$ **0.545 in.**

Torsional Properties:

Calculate distance between centroid and shear center: (AISI Supplementary Information Section 1.2)

$X_o = -(x' + m)$

m = distance between shear center and web centerline calculated in accordance with equation 6 of Section 1.2.2 of the Supplemental Information of the AISI Specification.

m= 0.616 in.

$\mathbf{X_o}$= -(0.319 + 0.616) = **- 0.935 in**.

St. Venant torsional constant, J: (AISI Supplementary Information Section 1.2)

$J = (t^3/3)[\, a + 2b + 2u + \alpha\ (\, 2c + 2u)]$ AISI Supplementary Information, Section 1.2.2

$J = 7.13 \times 10^{-4}$ in.^{4}

<u>*Warping constant, C_w:*</u>

For calculation of C_w refer to AISI Supplementary Information Section 1.2, Section III, page 11:

The warping constant is calculated in accordance with equation 9 of Section 1.22 of the AISI Supplementary Information (Page III-9).

$C_w = 2.503$ in.^{6}

<u>*Polar radii of gyration, r_o:*</u>

The polar radii of gyration of a section is calculated about the centroidal principal axis.

$r_o = [(r_x^2 + r_y^2 + X_o^2)]^{1/2}$
$r_o = [(2.93^2 + 0.536^2 + 0.935^2)]^{1/2}$
$r_o = 3.12$ in.

<u>*Torsional flexural constant, β:*</u> (AISI Supplementary Information Section 1.2)

$\beta = 1 - (X_o / R_o)^2$
$\beta = 1 - (-0.935/3.12)^2$
$\beta = 0.910$

<u>Calculate the yield strength due to cold working, F_{ya}:</u>

In order to use equation A5.2.2-1 of the AISI Design Specification, the channel must have a compact compression flange, that is $\rho = 1$. For this section, assume the reduction factor ρ is 1.

$F_{ya} = CF_{yc} + (1 - C)F_{yf}$ Equation A5.2.2-1
Limitation of this equation: $F_u/F_y \geq 1.2$
$R/t \leq 7$
$\theta \leq 120°$
$F_y = 33$ ksi, $F_u = 45$ ksi, $t = 0.057$ in., $R = 0.108$ in., $\theta = 90$ degrees,
$F_{yf} = 33$ ksi (Virgin yield point)

$F_u/F_y = 45 / 33 = 1.364 \geq 1.2$ OK
$R/t = 0.108 / 0.057 = 1.894 \leq 7$ OK
$\theta = 90° \leq 120°$ OK

$F_{yc} = [(B_c)(F_y)/(R/t)^m]$ Equation A5.2.2-2
$m = 0.192(F_u/F_y) - 0.068$ Equation A5.2.2-4
$m = 0.192(1.364) - 0.068 = 0.194$

$B_c = 3.69(F_u/F_y) - 0.819((F_u/F_y)^2 - 1.79$ Equation A5.2.2-3
$B_c = 3.69(1.364) - 0.819(1.364)^2 - 1.79 = 1.719$
C = (total cross sectional area of two corners) / (Full cross sectional area of flange)

Flange width = 1.625 -2(R + t) = 1.625 - 2(0.108 + 0.057) = 1.295 in.

Arc length of radius = 1.57 r (Where r = R + t/2)
1.57(0.108 + 0.057/2) = 0.214 in.

C = (2 x 0.214) / (2 x 0.214 + 1.295) = 0.248

F_{yc} = [(1.719)(33.0)/(1.894)$^{0.194}$ = 50.12 ksi

$\mathbf{F_{ya}}$ = 0.248(50.12) + (1 - 0.248)(33) = **37.2 ksi**

Effective Section Properties:

I_x and M_n are first calculated based on initiation of yielding, and then the effective moment and I_x are calculated based on Procedure I for deflection determination at the allowable moment.

Calculate I_{xx}:

Refer to Section B4.2 of the AISI Design Specification:
Flange width/thickness = w/t = 1.295 / 0.057 = 22.72 < 60 OK
Lip depth/thickness = d/t = 0.335 / 0.057 = 5.88 < 60 OK

S = (1.28)x[(E/f)]$^{1/2}$ Equation B4-1
Where E = 29,500,000 psi, and f = 37,200 psi (due to cold forming)

S = 1.28 x [(29,500,000 / 37,200)]$^{1/2}$ = 36.04
S/3 = 12.01
36.04 > w/t= 22.72 > 12.01 Section B4.2 (a)
Therefore, Case II applies
Therefore, I_a / t^4 = 399 [(w/t)/S - 0.330)]^3 Equation B4.2-6
I_a / t^4 = 10.91 $I_a = 11.2 \times 10^{-5}$ in.^{4}

The moment of inertia of the full edge stiffener (lip), I_s:
d = D - (R + t) = 0.5 - (0.107 + 0.057) = 0.336 in.
d/t = 0.335/0.057 = 5.88 < 14 (max. d/t)
$I_s = td^3/12 = (0.335)^3(0.057) / 12 = 18 \times 10^{-5}$ in.^{4}

$I_s/ I_a = 18 \times 10^{-5} / 11.2 \times 10^{-5} = 1.61$ This is greater than 1, therefore, use 1.0
C_2 = 1.0 Equation B4.2-7
C_1 = 2 - 1 = 1 Equation B4.2-8

D/w = 0.5/1.30 = 0.385

0.8 > D/w = 0.385 > 0.25
n = 0.5

k = [4.82 - 5D/w](I_s/ I_a)^n + 0.43 =< 5.25 -5D/w Equation B4.2-9
k = 3.3

Use k = 3.3 to calculate l the plate slenderness factor for the compression flange per section B2.1:

λ = [1.052/(k)$^{1/2}$](w / t)(f / E)$^{1/2}$ = 0.468 Equation B2.1-4

λ < 0.673, Therefore, **b = w = 1.297 in.** Equation B2.1-1

[Compression flange is fully effective]

The effective width of the edge stiffener can be computed in accordance with section B3.2 of the AISI Design Specification. Using k = 0.43, d/t = 5.89 and a conservative f = F_y, the slenderness factor is

$\lambda = [1.052/(0.43)^{1/2}](5.88)(f / E)^{1/2} = 0.336$ Equation B2.1-4
$\lambda < 0.673$, Therefore, $d'_s = d_s$ Equation B2.1-1
Where d'_s is the effective width of the edge stiffener.

$\mathbf{d_s}$ = $d's(I_s/I_a)$ = **0.335 in.**
Compression stiffener (lip) is fully effective.

<u>Check if web is fully effective:</u>

Locate neutral axis based on a full element. Assume the web is fully effective, and top fiber stress is F_{ya}:
(F_{ya} was previously calculated to be 37,200 ksi)

Element	Effective Length (L) (in.)	Distance. From Top Fiber, y (in.)	Ly (in^2)	Ly^2 (in^3)	I_1' About Own Axis (in^4)
Web	7.670	4.00	30.687	122.747	37.627
Tension Flange	1.295	7.972	10.337	82.404	
Compression Flange	1.295	0.0283	0.0367	0.001	
Upper Corners	0.427	0.078	0.033	0.003	
Lower Corners	0.427	7.9224	3.3792	26.771	
Upper Stiffener	0.335	0.332	0.112	0.037	0.003
Lower Stiffener	0.335	7.668	2.575	19.748	0.003
Sum	11.79		47.16	251.71	37.63

$y_{cg} = \Sigma (Ly) / \Sigma L = 47.16 / 11.79 = 4.00$ inches. (Distance from top fiber)

Since the distance from the compression fiber to the neutral axis is equal to one half the beam depth, a compression stress of F_{ya} will govern as assumed (i.e. initial yield is in compression)

Use section B2.3 of the AISI Design Specification to check the effectiveness of the web element.

$f_1 = F_{ya}$ (y_{cg} - thickness - radius) / y_{cg} = 37,200(4.0 - 0.057 - 0.108) / 4 = 35,665 psi (Compression)
f_2 = - F_{ya} (Web - y_{cg} - thickness - radius)/y_{cg} = -37,200 (8.0 - 4.00 - 0.057 - 0.108) / 4.00
f_2 = - 35,665 psi (Tension)
$\Psi = f_2/f_1 = -1$
$\Psi < -0.236$ Equation B2.3-1
$k = 4 + 2(1 - \Psi)^3 + 2(1 - \Psi) = 24$ Equation B2.3-4

$b_1 = b_e / (3 - \Psi)$ Equation B2.3-1

$b_2 = b_e / 2$ Equation B2.3-2

Where b_e is calculated per section B2.1 of the AISI Design Specification with f_1 substituted for f and k as determined above:

$\lambda = [1.052/(24)^{1/2}](7.670 / 0.057)(35{,}665 / 29{,}500{,}000)^{1/2} = 1.005$ Equation B2.1-4

$\lambda > 0.673$, Therefore, $b_e = \rho$ (web) Equation B2.1-2
$\rho = (1 - 0.22 / \lambda) / \lambda = (1 - 0.22/1.005) / 1.005 = 0.777$

$b_e = 0.777(7.670) = 5.96$ in.

$b_2 = 5.96 / 2 = 2.98$ in.

$b_1 = 5.96 / (3 + 1) = 1.49$ in.

$b_1 + b_2 = 2.98 + 1.49 = 4.47$ in.

Compression portion of the web calculated on the basis of the effective section =
$y_{cg} - (R + t) = 4.00 - (0.108 + 0.057) = 3.836$ in.

Since $b_1 + b_2 = 4.47$ in. > 3.836 in., the web element is fully effective as assumed.
$b_1 + b_2$ shall be taken as 3.836 in.

$\mathbf{y_{cg} = 4.00}$ **in.**

$I'_{(x)} = Ly^2 + I'_1 - Ly_{cg}^2 = 251.71 + 37.63 - 11.79(4.0)^2 = 100.7$ in.^{3}

$\mathbf{I_x} = I'_{(x)} * t = 100.7\,(0.057) =$ **5.74 in.^{4}**
(This number is slightly larger than the number generated by the computer program, as shown in the section property tables, due to the fact that the thickness used above is rounded up to 0.057 while the computer used all decimal places.)

$\mathbf{S_x} = I_x / y_{cg} = 5.74/4.0 =$ **1.43 in.^{3}** (Effective section modulus)
$M_n = S_e F_y = 1.43\,(37{,}200) = 53{,}196$ lb.-in.

Safety factor = 1.67

$\mathbf{M_a} = M_n / S.F. = 53{,}196 / 1.67 =$ **31,854 lb.-in.** (2,654 ft-lb.)

<u>Calculate Allowable Moment, M_y</u>

$M_y = (F_y)(S_y) / 1.67$ (where S_y is the effective section modulus)

$\mathbf{M_y} = 33{,}000\,(0.136) / 1.67 = 2687$ lb.-in. = **224 ft-lb.**
<u>Calculate Allowable Shear:</u>

Refer to section C3.2 of the AISI Design Specification, for unpunched web (web with no holes).

Compute the depth of the flat portion of the web (h)
$h = h - 2(R + t) = 8.00 - 2(0.108 + 0.057) = 7.67$ in.
$h/t = 7.67/0.057 = 135 < 200$ OK

Calculate $1.38[EK_v/F_y]^{1/2} = 1.38\ [29{,}500{,}000 \times 5.34 / 37{,}200]^{1/2} = 89.8$

$h/t > 89.8$, Therefore, $V_a = 0.53\ EK_v(t)^3 / h$ Equation C3.2-2

$\mathbf{V_a} = 0.53(29{,}500{,}000)(5.43)(0.057)^3 / 7.67 =$ **2,050 lbs**. (unpunched)

Calculate the allowable shear value for the 2 x 8 x 54 mil C-shaped member with punched web, per the ICBO AC46 method. Holes are 1.5 in. wide x 4 in. long, located along the centerline of the web.

The allowable shear capacity of a 2 x 8 x 54 mil C-shape was calculated above as 2,050 lb. for a section with no holes.

Calculate reduction factor: q_s
$q_s = 1 - 1.1\ (a/d) = 1 - 1.1(1.5 / 8) = 0.794$
where a = web size = 3.5 in. and d = width of hole = 1.5 in.

Allowable shear $\mathbf{V_a}$ = $2{,}050 * q_s = 2{,}050\ (0.794) =$ **1,628 lbs**. (with standard holes)

Calculate Allowable Web Crippling Strength:

Refer to section C3.4 of the AISI Design Specification, for web with no holes

The equations given in Table C3.4-1 of the AISI Design Specification are applicable to sections with $h/t < 200$ and $R/t \leq 6$. For the 2 x 8 x 54 mil C-shape, both provisions are met.

The length of bearing (N) is assumed to be a minimum of 1.5 in.
Use equation C3.4-1 for single webs, stiffened flanges:

$P_a = t^2\ k\ C_3\ C_4\ C_\theta\ [179 - 0.33(h/t)][1 + 0.01(N/t)]$ Equation C3.4-1
$k = F_y/33 = 33.0/33.0 = 1.0$ (Note F_{ya} can not be used for this section) Equation C3.4-21
$C_3 = 1.33 - 0.33(1.00) = 1.00$ Equation C3.4-12
$C_4 = 1.15 - 0.15R/t = 0.866$ Equation C3.4-13
$C_6 = 1 + (h/t) / 750 = 1.179$ when $h/t \leq 150$ Equation C3.4-15
$C_\theta = 0.7 + 0.3(\theta/90)^2 = 1.0$ Equation C3.4-20

$P_a = (0.057)^2\ (1.00)(1.00)(0.866)(1.0)[179 - 0.33(135)][1 + 0.01(1.5/0.057)]$

$\mathbf{P_a}$ = 0.477 kips = 477 lbs.

Calculate the web crippling capacity for a multiple web, stiffened flanges section using equation C3.4-3:

$P_a = (0.057)^2\ (33{,}000)(1.179)[5.0 + 0.63(1.5/0.057)^{0.5}]$

$\mathbf{P_a}$ = 1,028 lbs.

SCREW CAPACITY EXAMPLE

The following is an example for calculating the capacity of # 10 screw connecting two 43 mil steel C-shaped members. All referenced equations and sections are to the CCFSS Technical Bulletin Vol. 2, No. 1, February 1993 document.

Screw diameter = 0.19 inches (# 10 screw)
Ultimate capacity of steel = 45 ksi (tensile)
Steel minimum uncoated thickness = 0.045 inch (43 mils design thickness)
Factor of safety = 3.0

Calculate Shear capacity: (Section E4.3.1)

The shear force per screw shall not exceed P_{as}
$P_{as} = P_{ns} / 3$

$t_2/t_1 = 1.0 < 1.0$
$P_{ns1} = 4.2(t^3d)^{1/2} F_{u2}$ [Eq. E4.3.1]
$P_{ns1} = 4.2(0.045^3 \text{ x } 0.19)^{1/2} (45.0) = 0.786$ kips

$P_{ns2} = 2.7(t_1d) F_{u1}$ [Eq. E4.3.2]
$P_{ns2} = 2.7(0.045\text{x } 0.19)(45.0) = 1.039$ kips

$P_{ns3} = 2.7(t_2d) F_{u2}$ [Eq. E4.3.3]
$P_{ns1} = 2.7(0.045\text{x } 0.19)(45.0) = 1.039$ kips

Therefore, $P_{ns} =$ 0.786 kips = 786 lbs. $\Rightarrow$ $P_{as} = P_{ns} / 3 = 262$ lbs.

Pull out in Screw: (Section E4.4.1)

The pullout force shall not exceed P_{not}
where $P_{not} = 0.85 \text{ x } t_c \text{ x d x } F_{u2}$ [Eq. E4.4.1.1]
$P_{not} = 0.85 \text{ x } 0.045 \text{ x } 0.19 \text{ x } 45.0 = 0.327$ kips = 327 lbs.
$P_{aot} = 327 / 3 = 109$ lbs.

Pull-over in Screw: (Section E4.4.2)

The pull-over force shall not exceed P_{nov}
where $P_{nov} = 1.5 \text{ x } t_1 \text{ x } d_w \text{ x } F_{u1}$ [Eq. E4.4.2.1]
$P_{nov} = 1.5 \text{ x } 2 \text{ x } 0.045 \text{ x } 0.19 \text{ x } 45.0 = 1.154$ kips = 1,154 lbs.
(conservatively assuming the washer diameter is 2 x screw diameter)
$P_{aov} = 1{,}154 / 3 = 385$ lbs.

Therefore, the controlling capacity of the connection is: Shear = 262 lbs.

Pullout = 109 lbs.

FLOOR JOIST DESIGN AND EXAMPLE

The engineering approach used in the *Prescriptive Method* carefully considers floor joists dead and live load combinations as they apply to the design of joists. The objective was to maintain a simple table format without sacrificing the economies possible through case-by-case design.

Floor joists were designed using conservative engineering assumptions which neglect the benefits of composite action of floor assemblies. The composite strength of typical residential floor assemblies may provide higher resistance to bending and deflection. Web crippling check, and combined bending and web crippling check are not required because web stiffeners are specified at all bearing points.

The following applied loads were used in developing the joist tables:

Dead Load:	Floor Dead Load = 10 psf
Live Loads:	Sleeping Quarters = 30 psf Other Rooms = 40 psf

These loads are widely accepted by the engineering community (i.e. ASCE 7-93), and are specified in the major building codes: CABO, SBCCI, UBC, and BOCA (UBC does not permit the 30 psf live load). The section properties and moment and shear capacities were determined in accordance with the AISI Design Specification requirement.

The allowable joist spans were based on the minimum spans resulting from the following equations based on a simply supported span:

Bending:
$$S_b = \sqrt{\frac{F_b \times S_x \times 8000}{w \times \text{Spacing}}}$$

Deflection
$$S_d = \sqrt[3]{\frac{I_x \times 188800000}{w * \text{Spacing} \times \text{Deflection limit}}}$$

Web Crippling
$$S_w = \frac{P_{allow} \times 24}{w \times \text{Spacing}}$$

Shear
$$S_s = \frac{P_{allow} \times 24}{w \times \text{Spacing}}$$

Where: S = Single span, feet
F_b = Allowable bending (compressive) stress = 33/1.67 = 19.76 ksi
I_x = Moment of inertia, inches^4.
w = Load per square foot, psf
S_x = Section modulus, inches^3.
Spacing = Spacing of joists, inches.
Deflection limit = Allowable deflection limit (S/480 for live loads and S/240 for total load)
P_{allow} = Allowable end reaction for 1.5 in. bearing length, lbs.

The total load was considered to be uniformly distributed along a simply supported joist span. The above equations are derived from the following equations:

Bending moment, $M = \frac{wL^2}{8}$, Shear, $V = \frac{wL}{2}$

Live load deflection, $\Delta = \frac{5w_1L^4}{384EI}$, Total load deflection, $\Delta = \frac{5w_tL^4}{384EI}$

The following is an example for calculating the maximum allowable single span for 2 x 8 x 54 mil C-shape and a 2 x 8 x 43 mil C-shape floor joist members with 10 psf DL + 30 psf LL and 10 psf DL + 40 psf LL respectively. The joists are spaced at 24 inches on center.

The joist allowable span is controlled by bending, web crippling, shear, or deflection. Joists are installed with web stiffeners at all supports per Section 5.3 of the *Prescriptive Method*. Therefore, web crippling does not control the design (an example for calculating the web crippling strength is shown in the section properties calculation).

i *Maximum span due to bending:*

Bending stress, $F_b = \frac{M}{S_x}$

$M = \frac{wL^2}{8}$ (simply supported beam)

Where w = Applied load in pounds per linear foot

F_b = 33,000/ 1.67 = 19,760 psi = 19.76 ksi

Note: A yield stress increase due to cold forming could have been used but was neglected since bending does not control the design.

$S_x = 1.425$ in^3 (for a 2 x 8 x 54 mil joist, see section property tables)
$S_x = 1.151$ in^3 (for a 2 x 8 x 43 mil joist, see section property tables)
$I_x = 5.70$ in^4 (for a 2 x 8 x 54 mil joist, see section property tables)
$I_x = 4.605$ in^4 (for a 2 x 8 x 43 mil joist, see section property tables)

Therefore, $L_{max} = \sqrt{\frac{\text{Fb * Sx * 8000}}{\text{w x spacing}}}$

For the 2 x 8 x 43 mil joist, $L_{max} = \sqrt{\frac{19.76 * 1.151 * 8000}{40 \text{ x } 24}}$ = <u>13’-9"</u>

For the 2 x 8 x 54mil joist, $L_{max} = \sqrt{\frac{19.76 * 1.425 * 8000}{50 \text{ x } 24}}$ = <u>13’-8</u>"

ii *Maximum span due to deflection:*

For a simply supported beam with a uniformly distributed load, the maximum deflection is:

$$\delta_{\max} = \frac{5wL^4}{384EI}$$

δ_{max} = L/360 for w_t and L/480 for w_l
w_t = Total load (psf)
w_l = Live load (psf)
E = 29,500,000 psi

For the 2 x 8 x 43 mil joist:

Due to total load: $L_{\max} = \sqrt[3]{\frac{4.605 \ * \ 188800000}{40 * 24 * 360}}$ = 13'-7"

Due to live load: $L_{\max} = \sqrt[3]{\frac{4.605 \text{ x } 188800000}{30 * \ 24 \text{ x } 480}}$ = 13'-7"

Therefore, the maximum allowable span is 13'-7"

For the 2 x 8 x 54 mil joist:

Due to total load: $L_{\max} = \sqrt[3]{\frac{5.70 \ * \ 188800000}{50 * 24 * 360}}$ = 13'-7"

Due to live load: $L_{\max} = \sqrt[3]{\frac{5.70 \ * \ 188800000}{40 * 24 * 480}}$ = 13'-3"

Therefore, the maximum allowable span is 13'-3"

iii *Maximum span due to shear:*

Shear, $V = \frac{wL}{2}$ w (plf) = Applied load (psf) multiplied by the joist spacing (ft)

V_{allow} = 1,566.5 lbs. (for a 2 x 8 x 54 mil punched joist, see section property tables)
V_{allow} = 787.2 lbs. (for a 2 x 8 x 43 mil punched joist, see section property tables)

For the 2 x 8 x 43 mil joist:

$$L_{\max} = \frac{24V_{allow}}{w * spacing} \Rightarrow L_{\max} = \frac{24 * 787.2}{40 * 24}$$ = 19.68' = 19'-8"

For the 2 x 8 x 54 mil joist:

$$L_{\max} = \frac{24V_{allow}}{w * spacing} \Rightarrow L_{\max} = \frac{24*1566.6}{50*24} = 31.33' = 31'\text{-}4"$$

The allowable spans are the smaller of the maximum spans due to bending, shear, and deflection limits.

Maximum allowable span for 2 x 8 x 43 mil joist is 13'-7" for a 30 psf live load + 10 psf dead load.
Maximum allowable span for 2 x 8 x 54 mil joist is 13'-3" for a 40 psf live load + 10 psf dead load.

Floor Vibration Proposed Procedure

The proposed Virginia Tech criterion for determining vibration acceptability in cold-formed steel joist supported residential floor systems due to human activity is described below:

1. Calculate the critical central floor deflection from Onysko's Criterion as modified to represent expected in-situ performance:

   For L < 144 inches

   $$Y_{crit} = \frac{41}{L^{1.3}} \text{ inches}$$

   For 144 inches≤ L ≤ 288 inches

   $$Y_{crit} = \frac{41}{L^{1.3}}\left[0.00458L + 0.338\right] \text{ inches}$$

   Where L is the floor joist span in inches.

2. Calculate the predicted deflection of a single joist, A_{0t}, due to 225 lb. (1.0 kN) concentrated load at mid-span:

   $$A_{ot} = \frac{225L^3}{48EI} \text{ in.}$$

   Where L = floor span, in.
   E = modulus of elasticity of the joists, psi
   I = moment of inertia of joist alone, in.^{4}

3. Calculate the number of effective joists, N_{eff}, from the SJI Equation:

   $$N_{eff} = 1 + 2\sum \cos\frac{X\pi}{2X_o}$$

   Where: X = distance from the center joist to the joist under consideration, in.
   X_0 = distance from center joist to the edge of the effective floor = 1.06εL, in.
   L = joist span, in.
   $\varepsilon = (D_x/D_y)^{0.25}$
   D_x = flexural stiffness perpendicular to the joist = $E_c t^3/12$, lb.-in.
   D_y = flexural stiffness parallel to the joist = EI_t/S
   E_c = modulus of elasticity of the sub-flooring, psi
   E = modulus of elasticity of the joists, psi
   t = sub-flooring thickness, in.
   I_t = moment of inertia of joist alone, in.^{4}
   S = joist spacing, in.

4. Calculate the predicted center floor deflection, A_o:

$$A_o = \frac{A_{ot}}{A_{eff}}$$

Where A_0 = deflection of floor at mid-bay, in.
A_{ot} = deflection of a single joist due to a 225 lb. concentrated load at mid-span, in.
N_{eff} = number of effective joists in the floor system

5. Compare the value of A_0 to the critical deflection, y_{crit}

If $A_o < y_{crit}$	Acceptable
If $A_o > y_{crit}$	Unacceptable
If $y_{crit} < A_o \le 1.1(y_{crit})$	Marginal

Example1: A floor is to be built using 2x12x54 mil C-shaped joists (I = 15.65 in.^{4})spaced at 24 inches on center. The span is 19'-0" single span. The sub-flooring is 23/32" APA rated STURD-I-FLOOR. The floor is supported on concrete masonry units (CMU) foundation.

$$Y_{crit} = \frac{41}{228^{1.3}}\left[0.00458(228) + 0.338\right] = 0.049 \text{ in.}$$

$$A_{ot} = \frac{225 \;*\; 228^3}{48 \;*\; 29{,}500{,}000 \;*\; 15.65} = 0.120 \text{ in.}$$

$$D_x = \frac{580{,}000 \;*\; (23/32)^3}{12} = 1.79E + 04 \text{ lb. - in.}$$

$$D_y = \frac{29{,}500{,}000 \;*\; 15.65}{24} = 1.92E + 07 \text{ lb. - in.}$$

$$\varepsilon = \left(\frac{1.79E + 04}{1.92E + 07}\right)^{0.25} = 0.174$$

X_0 = 1.06 * 0.174 * 228 = 42.2 in.

$$\frac{X_o}{S} = \frac{42.2}{24} = 1.75$$

$$N_{eff} = 1 + 2\left(\cos\frac{24\pi}{2 * 42.2}\right) = 2.25$$

$$A_o = \frac{0.120}{2.25} = 0.053 \text{ in.} > 0.049 \text{ in.}$$

This floor is rated as UNACCEPTABLE.

Example2: A floor is to be built using 2x10x43 mil C-shaped joists (I = 7.98 in.^{4}) spaced at 24 inches on center. The span is 11'-10" single span. The sub-flooring is 23/32" APA rated STURD-I-FLOOR. The floor is supported on CMU foundation.

$$Y_{crit} = \frac{41}{142^{1.3}} = 0.065 \text{ in.}$$

$$A_{ot} = \frac{225 * 142^3}{48 * 29{,}500{,}000 * 7.98} = 0.057 \text{ in.}$$

$$D_x = \frac{580{,}000 * (23/32)^3}{12} = 1.79E + 04 \text{ lb. - in.}$$

$$D_y = \frac{29{,}500{,}000 * 7.98}{24} = 9.81E + 06 \text{ lb. - in.}$$

$$\varepsilon = \left(\frac{1.79E + 04}{9.81E + 06}\right)^{0.25} = 0.207$$

X_0 = 1.06 * 0.207 * 142 = 31.16 in.

$$\frac{X_o}{S} = \frac{31.16}{24} = 1.30$$

$$N_{eff} = 1 + 2\left(\cos\frac{24\pi}{2 * 31.16}\right) = 1.7$$

$$A_0 = \frac{0.057}{1.70} = 0.033 \text{ in.} < 0.065 \text{ in.}$$

This floor is rated as ACCEPTABLE.

WALL STUD DESIGN AND EXAMPLE

In the design of compression members, such as wall studs, any one of several engineering criteria may control the prescriptive requirements depending on the thickness and length of the stud. The engineering analysis includes checks for:

- Yielding,
- Overall column buckling,
- Flexural buckling,
- Torsional buckling,
- Torsional-flexural buckling, and
- Local buckling of an individual member.

The engineering approach used in the *Prescriptive Method* considers wind, snow, and live load combinations as they apply to the design of studs. The objective was to maintain a simple table format without sacrificing the economies possible through case-by-case design.

Wall studs were designed using conservative engineering assumptions which neglect the benefits of system effects of wall assemblies. The composite strength of typical residential wall assemblies provides higher resistance to out-of-plane bending and axial load conditions.

The following applied loads were used in developing the wall stud tables:

Dead Loads:	Floor Dead Load = 10 psf
	Wall Dead Load = 10 psf
	Ceiling Dead Load = 5 psf
	Roof Dead Load = 7 psf
Live Loads:	Second Floor Live Load = 30 psf
	First Floor Live Load = 40 psf
	Roof Live Load = Greater of 16 psf or Snow Load.

Snow Loads

The ASCE 7-93 ground snow map was divided into four regions as follows:

Region	Ground snow load
1	0 to 20 psf
2	21 to 30 psf
3	31 to 50 psf
4	51 to 70 psf

Applied roof snow loads are calculated by simply multiplying the ground snow load by a 0.7 conversion factor recognized in ASCE 7 and other snow load provisions (no further reductions were made for special cases). Roof slopes are assumed to range from 3:12 to 12:12.

Ground Snow Load	(psf)	20	30	50	70
Applied Roof Snow Load	(psf)	16	21	35	49

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

$$P_f = 0.7\, C_e\, C_t\, I\, P_g$$

I is the importance factor depending on building classification. Houses are typically class I structures, with an importance factor of 1.0.

P_g = Ground snow load from the ASCE 7-93 estimated ground snow map (psf). This map is also included in all major building codes.

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 is deemed reasonable for houses.

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 conditions which is likely to exist during the life of the structure. Houses are typically considered heated structures with C_t = 1.0. Although it is possible that a brief interruption of power will cause temporary cooling of a heated house, the joint probability of this with a peak snow load is highly unlikely. Houses that are unoccupied during cold seasons, may experience a higher thermal factor, however, for unoccupied buildings the importance factor drops to 0.8, thus reducing the design loads by 20%, which offsets the 20% increase in the thermal factor.

C_s is the roof slope factor ranging from approximately 0.1 to 1.0. For warm roofs (i.e. house roofs) the C_s curve is slightly smoother than that for cold roofs. Roofs with slopes up to 6:12 have a slope factor of 1.0, while roofs with slopes greater than 7:12 have a slope factor between 0.4 to 1.0. A slope factor of 1.0 is judged to be conservative for houses with roof slopes from 3:12 to 12:12.

Unbalanced snow loads, sliding snow loads, and snow drifts on lower roofs were not considered due to the lack of evidence for damage from unbalanced loads on homes and the lack of data to typify the statistical uncertainties associated with this load pattern on residential structures. Rain-on-snow surcharge load was also not considered in the calculations. Roof slopes in the *Prescriptive Method* exceed the 1/2 inch per foot requirement by ASCE 7-93 for the added load to be considered.

Therefore, the snow load can be computed as: $1.0 * 0.7 * 1.0 * 1.0 * P_g = 0.7\ P_g$

Wind Loads

Wind loads were based on wind speeds ranging from 70 to 110 mph for exposure categories A, B, and C in ASCE 7. Studs were designed using Components and Cladding wind pressure coefficients and a load combination of D + W. Studs were also designed for concurrent wind, snow, and live loads (multiple loading) using Main Wind Force Resisting System (MWFRS) pressure coefficients and the following load combinations: (D + L + ½ S + W) or (D + L + S + ½ W), based on the *Uniform Building Code* (UBC). The multiple load combination was found to control the design for the range of conditions encountered in the *Prescriptive Method*. The MWFRS wind loads are shown in Table C6.3

a. For Main Wind Force Resisting Systems (MWFRS):

Wind load = $W = 0.00256 K_z G_h C_p (VI)^2$

Where K_z = 0.87 (at 20 feet, for Exposure C)
G_h = 1.29 (Exposure C, 20 feet)
C_p = 0.8 (Windward wall)
I = 1 (for residential buildings in areas with wind speed < 100 mph)

b. For Components and Cladding:

Wind load = $W = 0.00256 K_z (GC_p + GC_{pi})(VI)^2$
Where: K_z = 0.87 (at 20 feet, for Exposure C)

GC_p = - 1.5 (Components and Cladding, wall interior, Zone 4)
GC_{pi} = ± 0.25 (Components and Cladding, interior pressure, enclosed buildings)
I = 1 (for residential buildings in areas with wind speed <90 mph)
I = 1.05 (for residential buildings in areas with wind speed ≥ 100 mph)

Exposure B Components and Cladding pressure coefficients GC_p and GC_{pi} were taken as 85 percent of the Exposure C coefficients based upon ASCE 7-95.

Applying the above equation, the design wind loads used in generating the stud tables are as follows (Table C6.3):

Table C6.3

Wind Exposure	**MWFRS Wind Loads (psf)**				
	70 mph	**80 mph**	**90 mph**	**100 mph**	**110 mph**
B	15	20	25	32	43
C	20	25	32	43	52

Load Combination:

The following loading combinations were investigated to determine the controlling design load on the studs:

A: Using MWFRS coefficients for wind loads with a 1/3 increase in allowable stresses for load combinations including wind (per AISI Cold-Formed Steel Design Specification):

1) 0.75(D + L + ½ S + W)
2) 0.75(D + L + S + ½ W)

B: Using Component and Cladding coefficients for wind loads with a 1/3 increase in allowable stresses.

D + W

Where:

D = Dead load;
S = Snow load;
L = Live load
W = Wind load

Example

The following is an example for calculating the applicable loads on wall studs and designing the wall studs to resist such loads. A hypothetical two story steel building is used to demonstrate the engineering calculations used in the *Prescriptive Method*. The building has the following physical dimensions:

Building type:	Two story house with basement.
Building width:	28 feet (with 2 foot roof overhang)
Building length:	40 feet.
First story wall height:	8 feet.

Second story wall height:	8 feet.
Wall stud spacing:	24" o.c.
Wind speed and Exposure:	70 mph Exposure C (or 80 mph Exposure B)
Ground snow load:	30 psf
Opening size:	8 feet.

The wall studs are subjected to a combined axial and lateral loads. Axial loads are due to roof live and dead loads, plus top floor live and dead loads. Lateral loads are due to wind pressure. The following loads are given:

Ground Snow load	=	50 psf
Roof dead load	=	7 psf
Wall dead load	=	10 psf
Ceiling dead load	=	5 psf
Floor dead load	=	10 psf
Top floor live load	=	30 psf
Attic live load	=	10 psf
Wind speed & Exposure		70 mph, Exposure C

i. *Roof Load:*

For wall studs, the roof load was based on a ground snow load of 50 psf.

Roof Snow load = S = 0.8 x 50 = 40 psf (0.8 is conservatively used)

ii. *Wind Loads:*

Wind loads were calculated using both main wind force resisting systems (MWFRS) and component and cladding (C&C) coefficients, as follows:

a. MWFRS: Wind load = $W = 0.00256K_zG_hC_pV^2$

$K_z =$ 0.87 (at 20 feet, Exposure C)
$G_h =$ 1.29 (Exposure C, 20 feet)
$C_p =$ 0.8 (Windward wall)
$V =$ 70 mph

$W = 0.00256(0.87)(1.29)(0.8)(70)^2 = 11.26$ psf use 12 psf.

b. Components and Cladding:

Wind load = $W = 0.00256K_z(GC_p + GC_{pi})(V)^2$
$K_z =$ 0.87 (at 20 feet, Exposure C)
$GC_p = \pm 1.5$ (for components and cladding, Zone 4, wall interior)
$GC_{pi} = \pm 0.25$ (for components and cladding interior pressure, enclosed buildings)
$V =$ 70 mph

$W = 0.00256(0.87)(1.5 + 0.25)(70)^2 = 19.10$ psf

The applied loads on the two story building are calculated as follows:

Roof live load = maximum of snow load or roof live load (16 psf minimum).

Snow load = 0.7 x 30 = 21 psf (calculated per ASCE 7-93 criteria, no other reduction is used)

Wind loads are calculated in accordance with ASCE 7-93.

Wind load = WL = $0.00256K_z(GC_p + GC_{pi})(VI)^2$

WL = $0.00256(0.87)(1.5 + 0.25)(70 \times 1)^2 = 19.10$ psf round up to 20 psf.

[or for 80 mph Exposure B: WL = $0.00256(0.87)(1.5 + 0.25)(80 \times 1)^2 \times 0.85 = 21$ psf]

Case 1: Dead load + live load + half snow load + full wind load (D + L + ½ S + W)

a. One story building or the top floor of a two story building:

Applied axial load = (roof dead load) x (house width + 2' overhang) / 2 + 1/2 (snow load) x (house width + 2')/2 + (attic live load + ceiling dead load) x (house width)/2

Applied axial load = (7)(28 + 2)/2 + ½ (40) x (28 + 2) / 2 + (10 + 5)(28/2) = **615 lb./ft**

Applied axial load = **1230 lbs.** Per stud for 24 inch on center spacing.
Applied lateral load = **12 psf** (MWFRS)

b. Bottom floor of a two story building:

Applied axial load = (roof dead load) x (house width + 2' overhang) / 2 + (second floor + ceiling dead load) x (28'/2) + second story wall dead load + (attic live load + second floor live load)x(house width)/2 + ½ (roof live load) x (house width + 2' overhang)/2}

Applied axial load = (7)(28 + 2)/2 + (10 + 5) x (28/2) + (10 x 8) + (10 + 30) x (28) / 2 + 1/2 (40)(28 + 2) / 2 = **1255 lb./ft**

Applied axial load = 1255 x 24"/12" = **2510 lbs.** Per stud (24" spacing)
Applied lateral load = **12 psf** (MWFRS)

Case 2: Dead load + live load + half wind load + full snow load (D + L + S + ½ W)

a. One story building or the top floor of a two story building:
Applied axial load = (roof dead load) x (house width + 2' overhang) / 2 + (snow load) x (house width + 2') / 2 + (attic live load + ceiling dead load) x (house width)/2

Applied axial load = (7)(28 + 2)/2 + (40) x (28 + 2) / 2 + (10 + 5)(28/2) = **915 lb./ft**

Applied axial load = 915 x 24"/12" = **1830 lbs.** per stud (24" spacing)
Applied lateral load = Half MWFRS load = 12 / 2 = **6 psf** (MWFRS)

b. Bottom floor of a two story building:

Applied axial load = (roof dead load) x (house width + 2' overhang) / 2 + (second floor + ceiling dead load)*(28'/2) + second floor wall dead load + (attic live load + second floor live load) x (house width) / 2 + (snow) x (house width + 2' overhang) / 2}

Applied axial load = (7)(28 + 2)/2 + (10 + 5) x (28/2) + (8 x 10) + (10 + 30) x (28) / 2 +(40) x (28 + 2)/2 = **1555 lb./ft**

Applied axial load = 1555 x 24"/12" = **3110 lbs.** per stud (24" spacing)

Applied lateral load = Half MWFRS load = 12 / 2 = **6 psf** (MWFRS)

Case 3: Dead load + Wind load (D + W)

a. One story building or the top floor of a two story building:

Applied axial load = (roof dead load) x (house width + 2' overhang) / 2 + ceiling dead load) x (house width) / 2
Applied axial load = (7)(28 + 2)/2 + (5)(28/2) = **175 lb./ft**

Applied axial load = (175)(24"/12") = **350 lbs.** per stud (24" o.c. spacing)

Applied lateral load = **19.10 psf** (components and cladding)

b. Bottom floor of a two story building:

Applied axial load = (roof dead load) x (house width + 2' overhang) / 2 + (second floor dead load + ceiling dead load) x (28'/2) + second floor wall dead load

Applied axial load = (7)(28 + 2)/2 + (10 + 5) x (28/2) + (10 x 8) = **395 lb./ft**

Applied axial load = 395(24"/12") = **790 lbs.** per stud (24" o.c. spacing)

Applied lateral load = **19.10 psf** (components and cladding)

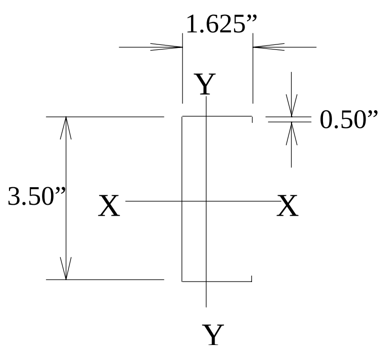
Therefore, the load combinations to be checked are:

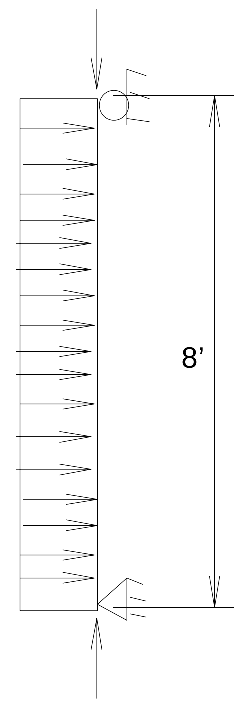
a) One story building:
1830 lbs. axial with 6 psf lateral
1230 lbs. axial with 12 psf lateral
350 lbs. axial with 19.1 psf lateral

b) Bottom story:
3110 lbs. axial with 6 psf lateral
2510 lbs. axial with 12 psf lateral
790 lbs. axial with 19.1 psf lateral

Design Wall Studs

Select a stud size and check its capacity to see if it can resist the applied loads. A 2 x 4 x 43 mil C-shaped stud is initially selected. Calculate the capacity of a 2 x 4 x 43:





The allowable moment is calculated as the smallest nominal moment calculated per Sections C3.1.1, C3.1.2, and C3.1.3 divided by a factor of safety of 1.67. All referenced equations and sections here are those in the AISI Design Specification.

Nominal section strength per AISI Section C3.1.1:

Use procedure (a) based on initiation of yielding.

$M_n = S_e F_y$ Eq. C3.1.1-1

Where F_y is the design yield strength.

S_e = Elastic section modulus of the effective section calculated with the extreme compression or tension fiber at F_y

Calculate strength increase due to cold working per Section A5.2.2 of the AISI Specification:
(assuming that the quantity r is unity as determined according to AISI Section B2 for each of the component elements of the section.)
$F_{ya} = CF_{yc} + (1 - C)F_{yf}$ Eq. A5.2.2-1

F_{yf} = 33,000 psi

C = corner cross sectional area of flange/full cross sectional area of flange

u = 1.57 (0.0937 + 0.045/2) = 0.182 in.
Area of corners = 2(0.045)(0.182) = 0.016 $in.^2$
Area of flanges = 2(0.045)(0.182) + 1.347(0.045) = 0.077 $in.^2$

C = 0.016 / 0.077 = 0.208

$F_{yc} = B_c F_{yv} / (R/t)^m$ Eq. A5.2.2-2

F_{uv} / F_{yv} = 45,000/33,000 = 1.364 > 1.2 OK

R/t = 0.0937/0.045 = 2.08 < 7 OK

∅ < 120 degrees OK

$B_c = 3.69(F_{uv} / F_{yv}) - 0.819 (F_{uv} / F_{yv})^2 - 1.79$ Eq. A5.2.2-3

$B_c = 3.69(1.364) - 0.819 (1.36\ 4)^2 - 1.79 = 1.719$

$m = 0.192(F_{uv} / F_{yv}) - 0.068$ Eq. A5.2.2-4

m = 0.192(1.364) - 0.068 = 0.194

$F_{yc} = 1.719(33{,}000) / (0.0937/0.045)^{0.194} = 49{,}203$ psi

$\mathbf{F_{ya}}$ = 0.208(49,203) + (1 - 0.208)33,000 = **36,370 psi**

Calculate the elastic section modulus of the effective section calculated with the extreme compression or tension fiber at F_y = 36,3700 psi

w = Flange width - 2(radius + thickness) = 1.347 in.
w/t = 1.347 / 0.045 = 29.93
d = Lip - radius - thickness = 0.5 - 0.0937 - 0.045 = 0.361 in.

$S = 1.28(E/f)^{1/2} = 1.28(29{,}500{,}000/36{,}370)^{1/2} = 36.45$ Eq. B4-1
S/3 = 12.15 < w/t = 29.93 < S = 36.45

$I_a = 399t^4\{[(w/t)/S] - 0.33\}^3$ Eq. B4.2-6
$I_a = 399(0.045)^4\{(29.93/36.45) - 0.33\}^3 = 19.4\text{x}10^{-5}$ $in.^4$
n = 1/2
D/w = 0.5 / 1.347 = 0.371 ⇒ 0.25 < D/w = 0.371 < 0.8

$k = [4.82 - 5(D/w)](I_s / I_a)^n + 0.43 \leq 5.25 - 5(D/w)$ Eq. B4.2-9

$I_s = d^3t/12 = (0.361)^3 (0.045)/12 = 17.6\text{x}10^{-5}$ $in.^4$
$k = [4.82 - 5(0.371)](17.6\text{x}10^{-5} / 19.4\text{x}10^{-5})^{1/2} + 0.43 \leq 5.25 - 5(0.371)$
k = 3.25

Effective width of the flange shall be calculated according to Section B2.1 using k as calculated above, and $f = M_c / S_f$.

w/t = 29.93 < 90 OK Section B1.1-(a)-(3)
$\lambda = (1.052 / k^{1/2})(w/t)(f / E)^{1/2}$ Eq. B2.1-4

Where $f = M_c / S_f = 36{,}370$ psi
$\lambda = [1.052/ (3.25)^{1/2}](29.93)(36{,}370/29{,}500{,}000)^{1/2} = 0.613$
$\lambda \leq 0.613 \Rightarrow b = w$ Eq. B2.1-1

Effective width **b = 1.347 inches** (i.e. compression flange is fully effective)

<u>Compression Stiffener:</u>

Determine the effective width of the compression stiffener (lip):

D = 0.5 inches
d = 0.361 inches.

d/t = 0.361/0.045 = 8.02 < 60 OK Section B1.1-(a)-(3)

D/w = 0.5 / 1.347 = 0.371

0.25 < D/w = 0.371 < 0.8

Calculate the effective width for the lip per AISI Section B2.1 :

$\lambda = [1.052/ (k)^{1/2}](w/t)[f / E]^{1/2}$ Eq. B2.1-4

Where $f = M_c / S_f = 36{,}370$ psi and k = 0.43 per AISI Section B3.1

$\lambda = [1.052 / (0.43)^{1/2}](8.02)(36{,}370/29{,}500{,}000)^{1/2} = 0.452$
$\lambda \leq 0.673 \Rightarrow d'_s = d = 0.361$ in. Eq. B2.1-1

$d_s = d'_s (I_s/I_a)$ £ d'_s Eq. B4.2-11
$d_s = 0.361 (17.6\text{x}10^{-5} /19.4\text{x}10^{-5}) \leq 0.361$

$d_s = 0.33$ in.
Effective width of lip **d_s = 0.33 inches** (i.e. compression stiffener is not fully effective)

Assume the web is fully effective.

Element	Effective length (in)	Distance from top fiber, y (in.)	Ly in.^{2}	Ly^2 in.^{3}	I_1' About own axis in.^{3}
Web	3.222	1.75	5.639	9.868	2.788
Tension flange	1.347	3.478	4.685	16.292	-
Comp. Flange	1.347	0.023	0.030	0.001	-
Upper corners	0.365[1]	0.065	0.024	0.002	-
Lower corners	0.365[1]	3.435	1.255	4.309	-
Upper stiffener	0.33	0.303	0.099	0.030	0.003
Lower stiffener	0.361	3.181	1.149	3.653	0.004
Sum	7.337		12.881	34.156	2.795

[1] Where corner length = Length of arc = 1.57(R + t/2) = 0.183 in.. Two corners = 0.365 in.

Distance from top fiber to x-axis is $Y_{cg} = 12.881 / 7.337 = 1.756$ inches

Since distance from the top compression fiber to the neutral axis is greater than one half the beam depth, the compression stress of 36,440 psi will govern as assumed (i.e. initial yielding is in compression).
Check if web is fully effective:

$f_1 = +f\,(Y_{cg} - t - R)/Y_{cg} = 36{,}370(1.756 - 0.045 - 0.0937)/1.756 = 33{,}497$ psi

$f_2 = -f\,(\text{Web size} - Y_{cg} - t - R)/Y_{cg} = 36{,}370(3.5 - 1.756 - 0.045 - 0.0937)/1.756$

$f_2 = -33{,}248$ psi (tension)

The effective widths are calculated per AISI Section B2.3 (a) for load capacity determination.

$\Psi = f_2 / f_1 = -0.993 \leq -0.236$ Section B2.3
$b_1 = b_e / (3-\Psi)$ Eq. B2.3-1
$b_2 = b_e / 2$ Eq. B2.3-2
$b_1 + b_2$ shall not exceed the compression portion of the web calculated on the basis of effective section.

b_e = Effective width b determined in accordance with Section B2.1 with f_1 substituted for f and k determined as follows:
$k = 4 + 2(1 - \Psi)^3 + 2(1 - \Psi)$ Eq. B2.3-4

$k = 4 + 2(1 + 0.993)^3 + 2(1 + 0.993) = 23.818$
$h = w = \text{Web} - 2(\text{Radius} + \text{thickness}) = 3.5 - 2(0.0937 + 0.045) = 3.223$ in.
$h/t = 3.223 / 0.045 = 71.62 < 200$ OK Section B1.2-(a)

$\lambda_{(web)} = [1.052 / (23.818)^{1/2}](71.62)(33{,}497/29{,}500{,}000)^{1/2} = 0.52 < 0.673$ Eq. B2.1-4

$b_e = w = 3.223$ in. Eq. B2.1-1
$b_2 = 3.223 / 2 = 1.611$ in.
$b_1 = 3.223 / (3 + 0.993) = 0.807$ in.
$b_1 + b_2 = 1.611 + 0.807 = 2.418$ in.

Compression portion of the web $= y_{cg} - (\text{Radius} + \text{thickness})$
$= 1.756 - (0.0937 + 0.045) = 1.617$ inches.

Since $b_1 + b_2 = 2.418$ in. > 1.617 in., $b_1 + b_2$ shall be taken as 1.617 in.
This verifies the assumption that the web is fully effective.

$I'_x = Ly^2 + I'_1 - L(y_{cg})^2$
$I'_x = 34.156 + 2.795 - 7.337(1.756)^2 = 14.33$ in.^{3}

Actual $I_x = I'_x\,t = (14.33)(0.045) = 0.645$ in^4

$\mathbf{S_e} = I_x / y_{cg} = 0.645 / 1.756 =$ **0.367 in^3**
$M_n = S_e F_y$ Eq. C3.1.1-1
$\mathbf{M_n} = (0.367)(36{,}370) =$ **13,348 lb.-in. = 1,112 ft-lb.**

<u>Nominal section strength per AISI section C3.1.2:</u>

$M_n = S_c (M_c / S_f)$ Eq. C3.1.2-1
$S_f = I_x / Y_{cg} = 0.645 / 1.756 = 0.367 \text{ in}^4$
Where I_x is obtained from the section property table.

$M_c = M_y (1 - M_y / 4M_e)$ Eq. C3.1.2-2
$M_y = S_f F_y$ Eq. C3.1.2-4

$M_y = 0.367 \times 36{,}370 = 13{,}348$ in-lb.
$C_b = 1$ (Combined axial and bending, Section C3.1.2)

$M_e = C_b r_o A[\sigma_{ey}\sigma_t]^{1/2}$ Eq. C3.1.2-5
$\sigma_{ey} = (\pi^2 E)/(K_y L_y/r_y)^2$ Eq. C3.1.2-8
$\sigma_t = (1/Ar_o^2)[GJ + \pi^2 EC_w / (K_t L_t)^2]$ Eq. C3.1.2-9

Section properties are taken from the section property table of this document.
$\sigma_{ey} = (\pi^2 * 29{,}500{,}000)/(1 * 48 / 0.611)^2 = 47{,}176$ psi

$\sigma_t = [1 / \{0.332 \times (2.033)^2\}]\times[11{,}300{,}000 \times 0.000225 + \{\pi^2 \times 29{,}500{,}000 \times 0.344\} / (1 * 48)^2]$
$\sigma_t = 33{,}634$ psi
$M_e = 1\times 0.332 \times 2.033\, [(47{,}176)(33{,}634]^{1/2} = 26{,}886$ lb.-in. $> 0.5\, M_y = 6{,}674$ in-lb.
$M_c = 13{,}348\, (1 - 13{,}348 / 4 \times 26{,}886) = 11{,}691$ in-lb.

Determine S_c, the elastic section modulus of the effective section calculated at a stress of M_c / S_f in the extreme compression fiber. $M_c / S_f = 11{,}691 / 0.367 = 31{,}856$ psi

<u>Compression Flange:</u>

Determine the effective width of the compression flange:

w = Flange width - 2(radius + Thickness) = 1.347 inches
w/t = 1.347 / 0.045 = 29.93

$S = 1.28[E/f]^{1/2} = 1.28[(29{,}500{,}000/31856)]^{1/2} = 38.95$ Eq. B4-1

$S/3 = 12.98 < w/t = 29.9 < S = 38.95$

$I_a = 399t^4\{[(w/t)/S] - 0.33\}^3$ Eq. B4.2-6
$I_a = 399(0.045)^4\{(29.93/38.95) - 0.33\}^3 = 13.8\times10^{-5} \text{ in}^4$
$n = 1/2$
$D/w = 0.5 / 1.347 = 0.371, 0.25 < D/w = 0.371 < 0.8$

$k = [4.82 - 5(D/w)](I_s / I_a)^n + 0.43 \leq 5.25 - 5(D/w)$ Eq. B4.2-9

$I_s = d^3t/12 = (0.361)^3(0.045)/12 = 17.6\times10^{-5} \text{ in}^4$
Since $I_s / I_a = 1.27 > 1.0$ use $I_s / I_a = 1.0$
$k = [4.82 - 5(0.371)](1.0)^{1/2} + 0.43 \leq 5.25 - 5(0.371)$
$k = 3.40$

Effective width of the flange shall be calculated according to Section B2.1 using k as calculated above, and $f = M_c/S_f$.

$w/t = 29.93 < 90$ OK Section B1.1-(a)-(3)

$\lambda = [1.052 / (k)^{1/2}](w / t)(f / E)^{1/2}$ Eq. B2.1-4

Where $f = M_c / S_f = 31{,}856$ psi

$\lambda = [1.052 / (3.40)^{1/2}](29.93)(31{,}856 / 29{,}500{,}000)^{1/2} = 0.561$

$\lambda \leq 0.673 \Rightarrow b = w$ Eq. B2.1-1

Effective width **b = 1.347 inches** (i.e. compression flange is fully effective)

<u>Compression Stiffener:</u>

Determine the effective width of the compression stiffener (lip):
d = Lip size - radius - thickness = 0.5 - 0.0937 - 0.045 = 0.361 in.

$d/t = 0.361/0.045 = 8.02 < 60$ OK Section B1.1-(a)-(3)

$D/w = 0.5 / 1.347 = 0.371, 0.25 < D/w = 0.371 < 0.8$

Calculate the effective width for the lip per AISI Section B2.1 :

$\lambda = [1.052/(k)^{1/2}](w/t)(f/E)^{1/2}$ Eq. B2.1-4

Where $f = M_c / S_f = 31{,}856$ psi and $k = 0.43$ per AISI Section B3.1

$\lambda = [1.052/(0.43)^{1/2}](8.02)(31{,}856 / 29{,}500{,}000)^{1/2} = 0.423$

$\lambda \leq 0.673 \Rightarrow d'_s = d = 0.361$ in. Eq. B2.1-1

$d_s = d'_s (I_s/I_a) \leq d'_s$ Eq. B4.2-11

Effective width of lip **d_s = 0.361 inches** (i.e. compression stiffener is fully effective)

Thus, one concludes that the section is fully effective. $Y_{cg} = 3.5 / 2 = 1.75$ in. (from symmetry)

Web:

Check if web is fully effective: (Section B2.3): Assume the web is fully effective, and top fiber stress is 31,907 psi:

Element	Effective length L (in)	Distance from top fiber, y (in.)	Ly in.^{2}	Ly^2 in.^{3}	I_1' About own axis in.^{3}
Web	3.222	1.75	5.639	9.868	2.788
Tension flange	1.347	3.478	4.685	16.292	-
Comp. Flange	1.347	0.023	0.030	0.001	-
Upper corners	0.365[1]	0.065	0.024	0.002	-
Lower corners	0.365[1]	3.435	1.255	4.309	-
Upper stiffener	0.361	0.319	0.115	0.037	0.004
Lower stiffener	0.361	3.181	1.149	3.653	0.004
Sum	7.37	12.25	12.897	34.163	2.796

[1] Where corner length = Length of arc = 1.57(R + t/2) = 0.183 in.. Two corners = 0.365 in.

Distance from top fiber to x-axis is Y_{cg} = 12.897 / 7.367 = 1.75 inches

$f_1 = +f\,(Y_{cg} - t - R)/Y_{cg}$ = 31,856(1.75 - 0.045 - 0.0937)/1.75 = 29,331 psi
$f_2 = -f\,(\text{Web size} - Y_{cg} - t - R)/Y_{cg}$ = 31,856(3.5 - 1.75 - 0.045 - 0.0937)/1.75 =- 29,331 psi

The effective widths are calculated per Section B2.3 (a) for load capacity determination.
$\Psi = f_2 / f_1 = -1 \leq -0.236$ Section B2.3
$b_1 = b_e / (3-\Psi)$ Eq. B2.3-1
$b_2 = b_e / 2$ Eq. B2.3-2
$b_1 + b_2$ shall not exceed the compression portion of the web calculated on the basis of effective section.

b_e = Effective width b determined in accordance with Section B2.1 with f_1 substituted for f and k determined as follows:
$k = 4 + 2(1 - \Psi)^3 + 2(1 - \Psi)$ Eq. B2.3-4

$k = 4 + 2(1 + 1)^3 + 2(1 + 1) = 24$
h = w = Web - 2(Radius + thickness) = 3.5 - 2(0.0937 + 0.045) = 3.222 in.
h/t = 3.222 / 0.045 = 71.45 < 200 OK Section B1.2-(a)

$\lambda_{(web)} = [1.052 / (24)^{1/2}](71.62)[29,331/29,500,000]^{1/2} = 0.485 < 0.673$ Eq. B2.1-4

b_e = w = 3.222 inches Eq. B2.1-1
b_2 = 3.222 / 2 = 1.611 inches
b_1 = 3.222 / (3 + 1) = 0.805 inches
$b_1 + b_2$ = 1.611 + 0.805 = 2.416 inches

Compression portion of the web = y_{cg} - (Radius + thickness) = 1.75 - (0.0937 + 0.045) = 1.611in.
Since $b_1 + b_2$ = 2.416 in. > 1.611 in., $b_1 + b_2$ shall be taken as 1.611in.
This verifies the assumption that the web is fully effective.

$I'_x = Ly^2 + I'_1 - Ly^2_{cg}$
$I'_x = 34.163 + 2.796 - 7.37(1.75)^2 = 14.388 \text{ in}^3$

Actual $I_x = I'_x\, t = (14.388)(0.045) = 0.647 \text{ in}^4$

$S_e = I_x / y_{cg} = 0.647 / 1.75 = 0.370 \text{ in}^3$

$M_n = S_e F_y$ Eq. C3.1.1-1

M_n = 0.370 x 31,856 = **11787 in-lb. = 982 ft-lb.**

Nominal section strength per AISI Section C3.1.3:

This section is not applicable to the wall studs in this example.

Allowable moment:

The allowable moment is the smallest nominal moment calculated per Sections C3.1.1, C3.1.2, and C3.1.3 divided by a factor of safety of 1.67.

$\Omega = 1.67$

$M_a = M_n / 1.67$ Eq. C3.1-1

M_a = 11,787/1.67 = 7,058 in-lb. = **588 ft-lb.**

ii. Allowable Shear V_a

Shear for unpunched web:

The allowable shear V_a is calculated in accordance with Section C3.2 of the AISI Specification.

Calculate the depth of the flat portion of the web, h, measured along the plane of the web.

h = Web size - 2(radius + thickness) = 3.5 - 2(0.0937 + 0.045) = 3.223 in.

h/t = 3.223/0.045 = 71.62 < 200

Calculate $1.38 (Ek_v / F_y)^{1/2} = 1.38(29{,}500{,}000 \times 5.34 / 33{,}000)^{1/2} = 95.35$

Where k_v is shear buckling coefficient = 5.34

F_y = 33,000 psi (stress increase due to cold-forming can not be used here)

$h/t = 71.62 < 95.35 \Rightarrow V_a = 0.38t^2 (Ek_vF_y)^{1/2} \leq 0.4(h)(t)(F_y)$ Eq. C3.2-1
$V_a = 0.38(0.045)^2 (29{,}500{,}000 \times 5.34 \times 33{,}000)^{1/2} \leq (0.4 \times 3.223 \times 0.045 \times 33{,}000)$
V_a = 1,754 lbs. ≤ 1914 lbs. OK.

V_a = 1,754 lbs.

<u>*Shear for punched web*</u>:

The allowable shear for webs with holes is calculated in accordance with the ICBO AC46 criteria.

Calculate the reduction factor q:
$q_s = 1 - 1.1(a/d)$ where a = web size = 3.5 in. and b = width of hole = 1.5 in.
$q_s = 0.529$

Allowable shear $V_a = (1754)(q_s)$

V_a = 928 lbs.

iii. Axial Capacity

The axial capacity of the member is calculated per AISI Section C4.
$P_a = P_n / \Omega$ Eq. C4-1
Where $P_n = A_e \times F_n$
A_e is the effective area at stress F_n. F_n is a function of F_e. F_e is calculated as the minimum of the elastic flexural buckling, torsional, or torsional-flexural buckling stress.

$\Omega = 1.92$ (factor of safety)

1. Calculate F_e for sections not subject to torsional or torsional-flexural buckling per Section C4.1 of the AISI Specification.

   $F_e = \pi^2 E / (KL/r)^2$ Eq. C4.1-1
   Check $K_xL_x/r_x = 1 \times 8 \times 12 / 1.397 = 69 < 200$
   $F_e = (\pi^2)(29{,}500{,}000) / (1 \times 8 \times 12 / 1.397)^2 = 61{,}655$ psi

   Check $K_yL_y/r_y = 1 \times 4 \times 12 / 0.611 = 79 < 200$
   $F_e = (\pi^2)(29{,}500{,}000) / (1 \times 4 \times 12 / 0.611)^2 =$ **47,176 psi**

2. Calculate F_e for sections subject to torsional or torsional-flexural buckling per section C4.2 of the AISI Specification.

   $F_e = 1/2b \{(\sigma_{ex} + \sigma_t) - [(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t]^{1/2}\}$ Eq. C4.2-1

   $\sigma_{ex} = (\pi^2 \times 29{,}500{,}000) / (1 \times 96 / 01.397)^2 = 61{,}655$ psi
   $\sigma_t = [1/ 0.332 \times (2.033)^2][11{,}300{,}000 \times 0.000225 + \pi^2 \times 29{,}500{,}000 \times 0.344 / (1 \times 48)^2]$
   $\sigma_t = 33{,}533$ psi
   $\beta = 1 - (x_o / r_o)^2$ Eq. C4.2-3
   $\beta = 1 - (-1.3445 / 2.033)^2 = 0.563$

   $F_e = 1 / (2 \times 0.563)\{[61{,}655 + 33{,}533]-[(61{,}655 + 33{,}533)^2 - 4(0.563)(61{,}655)(33{,}533)]^{1/2}\}$

   F_e = 25,594 psi < 47,176 psi

$F_y / 2 = 36{,}370 / 2 = 18{,}185$ psi Strength increase due to cold-forming is used.
$F_e = 25{,}594 > 18{,}185 \Rightarrow F_n = F_y (1 - F_y / 4F_e)$ Eq. C4-3

$F_n = 36{,}370 [1 - 36{,}370 / (4 \times 25{,}594] = 23{,}449$ psi

A_e can be easily calculated to be 0.243 in^2
$P_n = (0.243)(23{,}449) \cong 5{,}700$ lbs.
$P_a = 5{,}700 / 1.92$
P_a = 2,969 lbs.

iv. ***Combined Axial and Bending***

Check combined axial and bending in accordance with AISI Section C5.

The axial force and bending moments shall satisfy the following AISI interaction equations:

$P/P_a + (C_{mx}M_x)/(M_{ax}\alpha_x) + (C_{my}M_y)\,(M_{ay}\alpha_y) \leq 1.33$ Eq. C5-1
$P/P_{ao} + M_x / M_{ax} + M_y / M_{ay} < 1.33$ Eq. C5-2
In this example, $M_y = 0$ and the stresses are increased by 33% per AISI Section A4.4

$C_{mx} = 1$ conservatively taken as 1 per AISI Section C5
$1/\alpha_y$, $1/\alpha_x$ are magnification factors $= 1/[1 - (\Omega_c P/P_{cr})]$ Eq. C5-4

P = Applied axial load
M_x = Applied moment = $wL^2/8$
P_a = Allowable axial load determined in accordance with Section C4 = 2,969 lbs.
P_{ao} = Allowable axial load determined in accordance with Section C4, with $F_n = F_y$
$P_{ao} = F_y A_e / 1.92 = (36{,}370)(0.243) / 1.92 = 4{,}603$ lbs.
Ω_c = Factor of safety = 1.92
$P_{cr} = \pi^2 EI_b / (K_b/L_b)^2$ Eq. C5-5
I_b = Gross moment of inertia = 0.65 in^4

$P_{cr} = \pi^2 (29{,}500{,}000)(0.65) / (1 / 96)^2 = 20{,}535$ lbs.
M_{ax} = 588 ft-lbs. (previously calculated)
$\alpha_x = 1 - \Omega_c P/P_{cr} = 1 - 1.92\,(P / 20{,}535)$

Case 1: $D + L + (L_r$ or $S) + ½ W$

One story: 1230 lbs. axial load with 12 psf lateral load

Lateral load = 12 psf = 12 x 24" / 12" = 24 plf

$M_x = wL^2/ 8 = (24)(8)^2 / 8 = 192$ ft-lb.

Eq. C5-1: $1230/2{,}969 + (1\text{x } 192) / (588)[1 - 1.92\,(1{,}230/20{,}535] \leq 1.33$
0.78 < 1.33 OK

Eq. C5-2: $1{,}230 / 4{,}603 + 192 / 588 \leq 1.33$
0.59 < 1.33 OK

Bottom story: 2510 lbs. axial load with 12 psf lateral load

Lateral load = 12 psf = 12 * 24" / 12" = 24 plf

$M_x = wL^2/ 8 = (24)(8)^2 / 8 = 192$ ft-lb.

Eq. C5-1: $2{,}510 / 2{,}969 + (1\text{x } 192) / (588)[1 - 1.92\,(2{,}510 / 20{,}535] \leq 1.33$
1.27 < 1.33 OK

Eq. C5-2: 2,510 / 4,603 + 192 / 588 < 1.33
0.87 < 1.33 OK

Case 2: D + L + (L_r or S) + ½ W

One story: 1830 lbs. axial load with 6 psf lateral load

Lateral load = 6 psf = 6 x 24" / 12" = 12 plf

$M_x = wL^2/ 8 = (12)(8)^2 / 8 = 96$ ft-lb.

Eq. C5-1: 1,830/2,969 + (1x 96) / (588)[1 - 1.92 (1,830/20,535] ≤ 1.33
0.81 < 1.33 OK

Eq. C5-2: 1,830 / 4,603 + 96 / 588 ≤ 1.33
0.56 < 1.33 OK

Bottom story: 3110 lbs. axial load with 6 psf lateral load

Lateral load = 6 psf = 6 x 24" / 12" = 12 plf

$M_x = wL^2/8 = (12)(8)^2 / 8 = 96$ ft-lb.

Eq. C5-1: 3,110 / 2,969 + (1x 96) / (588)[1 - 1.92 (3,110 / 20,535] ≤ 1.33
1.26 < 1.33 OK

Eq. C5-2: 3,110 / 4,603 + 96 / 588 ≤ 1.33
0.84 < 1.33 OK

Case 3: D + W

One story: 350 lbs. axial load with 19.1 psf lateral load

Lateral load = 19.1 psf = 12 x 24" / 12" = 38.2 plf

$M_x = wL^2 / 8 = (38.2)(8)^2 / 8 = 305.6$ ft.-lbs.

Eq. C5-1: 350/2,969 + (1x 305.6) / (588)[1 - 1.92 (350/20,535] ≤ 1.33
0.65 < 1.33 OK

Eq. C5-2: 350 / 4,603 + 305.6 / 588 ≤ 1.33
0.59 < 1.33 OK

Bottom story: 790 lbs. axial load with 19.1 psf lateral load

Lateral load = 19.1 psf = 12 x 24" / 12" = 38.2 plf

$M_x = wL^2/ 8 = (38.2)(8)^2 / 8 = 305.6$ ft-lb.

Eq. C5-1: 790 / 2,969 + (1 x 305.6) / (588)[1 - 1.92 (790 / 20,535] ≤ 1.33
0.83 < 1.33 OK

Eq. C5-2: $790 / 4{,}603 + 305.6 / 588 \leq 1.33$

$0.69 < 1.33$ OK

Therefore, the 2 x 4 x 43 mil stud is adequate.

v. Deflection Check:

Check the stud deflection using the maximum lateral pressure of 19.1 psf:

$$\delta = \frac{5wL^4}{384EI}$$

$$\delta = \frac{5\frac{(19.1)}{(12)}\frac{(24)}{(12)}(8 \times 12)^4}{384(29{,}500{,}000)(0.649)} = 0.18 \text{ in.} < L/240 = 0.40 \text{ in.} \qquad \text{OK}$$

HEADER DESIGN AND EXAMPLE

The following loads were used in developing the header tables:

Ceiling dead load	=	5 psf
Roof dead load	=	7 psf
Second floor dead load	=	10 psf
Wall dead load	=	10 psf
Roof live load	=	Greater of snow load or 16 psf
Attic live load	=	10 psf
Snow load	=	Varies from 0 to 70 psf ground snow load
Second floor live load	=	30 psf
Attic live load	=	10 psf

Headers supporting roof and ceiling only:

Vertical load acting on a header supporting one floor only = Roof dead load + roof live load + ceiling dead load + attic live load

Headers supporting one floor, roof and ceiling:

Vertical load acting on a header supporting one floor and a roof = Roof dead load + roof live load + ceiling dead load + attic live load + second floor live and dead load + second floor wall dead load.

Headers supporting one floor, roof and ceiling, with center load bearing beam:

Vertical load acting on a header supporting one floor and a roof = Roof dead load + roof live load + ceiling dead load + attic live load + (1/2 second floor live and dead load + 1/2 second floor wall dead load)

Snow loads

Snow loads were calculated for 20, 30, and 50 psf ground snow loads as follows:

Design snow load = 0.7 x Ground snow load (No other adjustments were made for special cases)

Load Combination:

Several load combinations were also investigated to determine the controlling design load on the headers. The following load combination was found to control the design:

$$D + 0.75(L + S)$$

Example:

Design a header for an 6-foot opening and calculate the number of king and jack studs required. The 6-foot opening is located on the first floor of a two-story building, and the studs are installed 24 in. on center. The building width is 24 feet. The ground snow load is 40 psf and the attic live load is 10 psf.

Design loads for bottom floor headers are as follows:

Snow load	=	40 psf	Roof dead load	=	7 psf
Wall dead load	=	10 psf	Ceiling dead load	=	5 psf
Second floor dead load	=	10 psf	Second floor live load	=	30 psf
Attic live load	=	10 psf			

Headers were designed for vertical loads only. The load combination used to design the headers is:

$$D + 0.75\{L + (L_r \text{ or } S)\}$$

Vertical design load acting on the header = Roof dead load + ceiling dead load + second floor dead load + second story wall dead load + 0.75[snow load + attic live load + second floor live load]

W = (7 psf)(28’ + 2’)/2 + (5 psf)(28’/2) + (10 psf)(28’/2) + 10 psf x 10’
+ 0.75[(40 psf)(28’ + 2’)/2 + (10 psf)(28’/2) + 30 psf (28’/2)]

W = 1363 lbs./ft.
(a wall height of 10 feet is conservatively used to calculate second story wall dead load)

Select a 2 x 8 x 68 back-to-back header, and check its adequacy to resist the applied loads.

Allowable moment for a 2 x 8 x 68 mil section = 3,356 ft-lb. (From section property table)

Conservatively, multiply this moment capacity by two to obtain the capacity for a back-to-back header consisting of two 2 x 8 x 68 mil C-shaped members.

M_a = 3,356 x 2 = 6,712 lbs.-ft.

Allowable shear = 6333 lbs. (From section property table)

Applied moment on header, $M_a = \frac{WL^2}{8} \Rightarrow L = \sqrt{\frac{8M_a}{W}}$

Maximum span due to applied moment, $L = \sqrt{\frac{8(6{,}712)}{1{,}363}} = 6.28$ feet.

Applied shear on header, $V = \frac{WL}{2} \Rightarrow \quad L = \frac{2V}{W}$

Maximum span due to allowable shear, $L = \frac{2 \times 6{,}333}{1{,}363} = 9.29$ feet.

Web crippling is not a concern here because the header is connected with an angle to the king stud. The angle stiffens the web and acts as a web stiffener.

The maximum header span is, therefore, 6’-3" which is greater than the opening size.

Calculate number of jack and king studs:

The number of king studs is calculated based of the actual number of full height studs required if there was no opening. The opening size is 6 feet and the spacing is 24 in. on center, therefore 3 studs are required (i.e. 6/2). As for lateral loads, the king studs are considered to provide the main resistance to wind loads. The jack studs, cripples, lintels, and window framing will act as a system in resisting and transferring the lateral loads to the adjacent king studs. This framing system will resist a portion of the lateral load, thus reducing the total lateral load acting on the king studs.

SHEARWALL BRACING REQUIREMENTS CALCULATION

A. Capacity of wall sheathing materials

Two structural sheathing materials are permitted based on test data and rational engineering analysis for shearwalls. These materials are 7/16 in. OSB and 15/32 in. plywood. In these tests, 1/2" gypsum wallboard was shown to have an additive allowable shear capacity of 123 plf, but this additive strength was not considered in the development of the shearwall bracing table.

The allowable shear capacities for plywood sheathing and oriented strand board sheathing are based on recent racking tests performed at the University of Santa Clara (AISI Publication RG-9604, May 1996). These tests were conducted in conformance with the ASTM E 564 standard. The results relevant to this document are summarized as follows:

Description of Wall	Average Ultimate Capacity (plf)	Average Load at ½" Deflection (plf)	Allowable Load Capacity (plf)
15/32" Plywood APA rated sheathing w/ panels on one side	1065	508	426
7/16" OSB APA rated sheathing w/ panels on one side	910	593	**364**

Notes:
1. Framing at 24"oc
2. Studs and track are 3-1/2 in. 20-ga. -- (33ksi) steel
3. Studs flange: 1-5/8"; stud lip: 3/8"; track flange: 1-1/4".
4. Framing screws: No. 8 x 5/8" wafer head self-drilling
5. Sheathing screws: No. 8 x 1" self-drilling, flat head, coarse thread @ 6” oc edge/12” oc field spacing
6. A safety factor of 2.5 was used to determine allowable capacity

A value of 364 plf was used to determine shearwall requirements (percent length of full-height sheathed wall). The allowable load capacity is less than the load at 1/2" deflection of the wall.

B. Capacity of connections for shearwall hold-downs

The hold-down resistance for shearwalls (rotational restraint) for the range of wind and seismic loads permitted is provided by standard connection details (screws and anchor bolts) for walls to foundations and floors, the available dead load restraining forces, and system effects at the corners of light-frame construction.

C. Shear loads

Shear loads on walls of one- and two-story buildings were determined in accordance with ASCE 7-93 *Minimum Design Loads for Buildings and Other Structures* as modified for use for residential construction. Shear loads are a result of lateral forces due to wind or inertial reaction due to seismic ground accelerations. Several wind analysis spreadsheets were developed for multiple building geometries and loading conditions. The design did not consider the benefit of any other shear transfer pathways that exist in conventional homes (i.e. interior walls). The loads were then divided by the sheathing capacity (364 plf) in order to determine required lengths of full-height sheathing. These lengths were then converted to percentages of wall length as shown in Table 6.20 of the *Prescriptive Method*. The shear load values were compared to seismic loads through multiple runs of a similar seismic analysis spreadsheet using identical building geometries. As expected, lateral loads from wind dominate over seismic loads for light-frame steel structures in seismic Zones 2, 1, or 0. The wind spreadsheet was also used to determine adjustment factors for loads when 9' or 10' walls are used in lieu of a standard 8' wall.

D. Required full height sheathing

The required amounts of structural sheathing in Table 6.20 were determined based on the structural sheathing capacities and shear loads calculated as described above. For a 28’ x 40’, 2-story house (roof slope 8:12), and interpolating between values in Table 6.20, the following design is achieved:

	Percentage from Table 6.20 %	Wall Length L	Required Length of Full-height Sheathing % x L	Actual Capacity (L) x (%) x 364plf (lbs.)	Required Capacity (from specific analysis of building loads)
Upper Story					
endwall	40 %	28’	11’-0"	4,004 lb.	2,816 lb.
sidewall	30 %	40’	12’-0"	4,368 lb.	1,776 lb.
Lower Story endwall	68 %	28’	19’-0"	6,919 lb.	5,451 lb.
sidewall	47 %	40’	19’-0"	6,919 lb.	2,801 lb.

As shown in this example, the actual capacity is well above that required by a detailed analysis.

CEILING JOIST DESIGN AND EXAMPLE

The following applied loads were used in developing the ceiling joist tables:

Dead Load:	Ceiling Dead Load = 5 psf	
Live Loads:	With Attic Storage = 20 psf;	Without Attic Storage = 10 psf

The section properties, moment and shear capacities were determined per the AISI Specification. All joists are assumed punched with 1-1/2 in. wide x 4 in. long holes spaced at 24 inches on center. The compression flanges of the ceiling joists are assumed to be laterally braced at mid-point, third point, and unbraced. The provisions for the combined bending and web crippling and combined bending and shear were taken directly from the AISI Design Specification.

Deflection limits are primarily established with regard to serviceability concerns. The intent is to prevent excessive deflection which might result in cracking of finishes. For ceiling joists, most codes generally agree that L/240 represents an acceptable serviceability limit for deflection. Therefore, ceiling finishes in the *Prescriptive Method* are limited to L/240 deflection.

Example:

The following is an example for selecting and designing a ceiling joist member for the following building configuration and loading conditions:

Building width = 30 ft	Wind speed = 80 mph, exposure C	Attic dead load = 5 psf
Ground snow load = 30 psf	Spacing = 24 inches on center	Roof slope = 6:12
Roof dead load = 10 psf	Load bearing wall at mid-span	

First, select the proper ceiling joist from the ceiling joist tables in the *Prescriptive Method*:

With an interior bearing wall the clear span would be 14'-6" (30'/2 = 15' less 6", i.e. 4" for outside wall and 2" for inside wall), select a ceiling joist at 24 "on center with a dead load of 5 psf and a live load of 10 psf. One feasible solution is a twin span 2 x 6 x 33 mil joist with a maximum allowable span of 14'-10".

The section properties for this member can be calculated using the AISI Design Specification. The section properties listed below are taken from the section property table.

F_y = 33 ksi	S_x = 0.505 in^3	I_x = 1.451 $in.^4$	$I_{x\ gross}$ = 1.451 $in.^4$
F_{ya} = 33 ksi	r_y = 0.587 in.	x_o = -1.136 in.	r_x = 2.11 in.
G = 11,300 ksi	r_o = 2.467 in.	J = 0.00013 $in.^4$	Spacing = 24 in. on center
E = 29,500 ksi	C_W = 0.702 $in.^6$	A_{gross} = 0.326 $in.^2$	K = 1.0
$I_{y\ gross}$ = 0.1125 in^4	$Def._{LL}$= L/240	$Def._{TL}$= L/180	
L.L. = 10 psf	D.L. = 5 psf		

The allowable span shall be the smaller of:

1. maximum span due to bending
2. maximum span due to shear
3. maximum span due to deflection
4. maximum span due to bending and shear combined

1. Bending per AISI Design Specification Section C3, "Flexural Members"
(all referenced equations and sections are those of the AISI Design Specification)

$M_a = M_n/W_f$
M_n = smallest nominal moment of Sections C3.1.1, C3.1.2, and C3.1.3
$\Omega_f = 1.67$ (factor of safety)

C3.1.1 Nominal Section Strength

$M_n = S_eF_y$
$S_e = S_x$
$M_n = (0.505)x(33.0) = 16.67$ kip-in. $\cong$ 1390 lb. - ft.

C3.1.2 Lateral Buckling Strength

$M_n = S_c (M_c / S_f)$ Eq. C3.1.2-1
$S_f = I_x / Y_{cg} = 1.451 / 2.75 = 0.528$ in.^{4}

$M_c = M_y (1 - M_y / 4M_e)$ Eq. C3.1.2-2
$M_y = S_f F_y$ Eq. C3.1.2-4
$M_y = 0.528 * 33{,}000 = 17{,}424$ lb.-in.
$C_b = 1.75$ ($M_1 = 0$, Section C3.1.2)

$M_e = C_b r_o A[\sigma_{ey}\sigma_t]^{1/2}$ Eq. C3.1.2-5
$\sigma_{ey} = (\pi^2 E)/(K_yL_y/r_y)^2$ Eq. C3.1.2-8
$\sigma_t = (1/Ar_o^2)[GJ + \pi^2 EC_w / (K_t L_t)^2]$ Eq. C3.1.2-9

$\sigma_{ey} = (\pi^2 \text{ x } 29{,}500{,}000)/(1 \text{ x } 89 / 0.587)^2 = 12{,}665$ psi

$\sigma_t = [1 / 0.326 \text{ x } (2.467)^2]\text{x}[11{,}300{,}000 \text{ x } 0.00013 + \pi^2 \text{ x } 29{,}500{,}000 \text{ x } 0.702 / (1 \text{ x } 89)^2]$
$\sigma_t = 13{,}746$ psi

$M_e = 1.75 \text{ x } 2.467 \text{ x } 0.326 [(12{,}665)(13{,}746)]^{1/2} = 18{,}570$ lb.-in. $> 0.5 M_y = 8{,}712$ lb.-in.

$M_c = 17{,}424 (1 - 17{,}424 / 4 \text{ x } 18{,}570) = 13{,}337$ lb.-in.

Determine Sc, the elastic section modulus of the effective section, calculated at a stress of M_c / S_f in the extreme compression fiber. $M_c / S_f = 13{,}337 / 0.527 = 25{,}307$ psi

Compression Flange:

Determine the effective width of the compression flange:
w = Flange width - 2(radius + Thickness) = 1.368 in.
w/t = 1.368/0.035 = 39.09 <60 O.K.

$S = 1.28[(E/f)]^{1/2} = 1.28[(29{,}500{,}000/25{,}307)]^{1/2} = 43.70$ Eq. B4-1

$S/3 = 14.57 < w/t = 39.09 < S = 43.70$

$I_a = 399t^4\{[(w/t)/S] - 0.33\}^3$ Eq. B4.2-6
$I_a = 399(0.035)^4\{[39.09/43.70] - 0.33\}^3 = 10.7\text{x}10^{-5}$ in.^{4}
n = 1/2

$D/w = 0.5 / 1.367 = 0.366$, $0.25 < D/w = 0.366 < 0.8$

$k = [4.82 - 5(D/w)](Is/Ia)^n + 0.43 \leq 5.25 - 5(D/w)$ Eq. B4.2-9

d = lip-radius-thickness = 0.372 in.

$I_s = d^3t/12 = (0.372)^3 (0.035)/12 = 14.8x10^{-5}$ in.^{4}

$k = [4.82 - 5(0.365)]x[14.8x10^{-5}/ 10.7x10^{-5}]^{0.5} + 0.43 \leq 5.25 - 5(0.365)$
$k = 3.43$

Effective width of the flange shall be calculated according to AISI Design Specification Section B2.1 using k as calculated above, and $f = M_c / S_f$.

Since Is > Ia and D/w < 0.8, the stiffener is not considered as a simple lip.

$w/t = 39.09 < 90$ OK Section B1.1-(a)-(3)

$\lambda = [1.052/(k)^{1/2}](w/t)(f/E)^{1/2}$ Eq. B2.1-4
Where $f = M_c / S_f = 25{,}307$ psi
$\lambda = [1.052/(3.43)^{1/2}](39.09)(25{,}307 / 29{,}500{,}000)^{1/2} = 0.650$
$\lambda \leq 0.673 \Rightarrow b = w$ Eq. B2.1-1

Effective width **b = 1.368 inches** (i.e. compression flange is fully effective)

<u>Compression Stiffener:</u>

Determine the effective width of the compression stiffener (lip):
D = 0.5 inches
$d/t = 0.372/0.035 = 10.629 < 60$ OK Section B1.1-(a)-(3)
$D/w = 0.5 / 1.368 = 0.365$, $0.25 < D/w = 0.365 < 0.8$
Calculate the effective width for the lip per AISI Design Specification Section B2.1 :

$\lambda = [1.052/(k)^{1/2}](w/t)(f/E)^{1/2}$ Eq. B2.1-4
Where $f = M_c / S_f = 25{,}307$ psi and k = 0.43 per AISI Design Specification Section B3.1
$\lambda = [1.052/(0.43)^{1/2}](10.629)(25{,}307/29{,}500{,}000)^{1/2} = 0.50$
$\lambda \leq 0.673 \Rightarrow d'_s = d = 0.372$ in. Eq. B2.1-1
$d_s = d'_s (I_s/I_a) \leq d'_s$ Eq. B4.2-11

Effective width of lip $\mathbf{d_s = 0.372}$ **inches** (i.e. compression stiffener is fully effective)

Thus, one assumes that the section is fully effective. $Y_{cg} = 5.5 / 2 = 2.75$ in. (from symmetry)

<u>Web:</u>

Check if web is fully effective: (AISI Design Specification Section B2.3)

Assume the web is fully effective, and top fiber stress is 25,307 psi:

$f_1 = +f (Y_{cg} - t - R)/Y_{cg} = 25{,}307(2.75 - 0.035 - 0.0937)/2.75 = 24{,}123$ psi

$f_2 = -f (\text{Web size} - Y_{cg} - t - R)/Y_{cg} = 25{,}307(5.5 - 2.75 - 0.035 - 0.0937)/2.75$

f_2 = - 24,123 psi (tension)

The effective widths are calculated per AISI Design Specification Section B2.3 (a) for load capacity determination.

$\Psi = f_2 / f_1 = -1 \leq -0.236$ — Section B2.3
$b_1 = b_e / (3 - \Psi)$ — Eq. B2.3-1
$b_2 = b_e / 2$ — Eq. B2.3-2
$b_1 + b_2$ shall not exceed the compression portion of the web calculated on the basis of effective section.

b_e = Effective width b determined in accordance with Section B2.1 with f_1 substituted for f and k determined as follows:
$k = 4 + 2(1 - \Psi)^3 + 2(1 - \Psi)$ — Eq. B2.3-4

$k = 4 + 2(1 + 1)^3 + 2(1 + 1) = 24$
h = w = Web - 2(Radius + thickness) = 5.5 - 2(0.0937 + 0.035) = 5.243 inches.
h/t = 5.243 / 0.035 = 149.80 < 200 OK — Section B1.2-(a)

$\lambda_{(web)} = [1.052/(24)^{1/2}](149.80)(24{,}123/29{,}500{,}000)^{1/2} = 0.87 > 0.673$ — Eq. B2.1-4
$\rho_{(web)} = (1-.22/\lambda)/\lambda = (1-0.22/0.87)/0.87 = 0.859$

$b_e = \rho_{(web)}\, w = 4.503$ inches — Eq. B2.1-1
$b_2 = b_e / 2 = 2.25$ inches
$b_1 = b_e / (3 + 1) = 1.126$ inches.
$b_1 + b_2 = 1.126 + 2.25 = 3.376$ inches

Compression portion of the web = y_{cg} - (Radius + thickness)
= 2.75 - (0.0937 + 0.035) = 2.62 inches.

Since $b_1 + b_2 = 3.376 > 2.622$ in., $b_1 + b_2$ shall be taken as 2.62 in.
This verifies the assumption that the web is fully effective.

Element	Effective length (in)	Distance from top fiber, y (in.)	Ly in.^{2}	Ly^2 in.^{3}	I_1' About own axis in.^{3}
Web	5.243	2.75	14.419	39.652	12.012
Tension flange	1.368	5.483	7.502	41.131	-
Comp. Flange	1.368	0.018	0.024	0.0004	-
Comp. corners	0.349[1]	0.058	0.020	0.001	-
Tension corners	0.349[1]	5.442	1.898	10.328	-
Comp. stiffener	0.372	0.314	0.117	0.037	.004
Tension stiffener	0.372	5.186	1.927	9.995	.004
Sum	9.42		25.91	101.145	12.02

[1] Where corner length = Length of arc = 1.57(R + t/2) = 0.174. Two corners = 0.349 in

$I'_x = Ly^2 + I'_1 - Ly^2_{cg}$
$I'_x = 101.145 + 12.02 - 9.42(2.75)^2 = 42.23$ in.^{3}

Actual $I_x = I'_x\, t = (42.23)(0.035) = 1.478$ in.^{4}

$S_e = I_x / y_{cg} = 1.478 / 2.75 = 0.537$ in.^{3}
$M_n = S_e F_y$ Eq. C.1.1-1

M_n $= 0.537 \times 25{,}307 =$ **13,590 lb.-in**

C3.1.3 "Beams Having One Flange Through-Fastened to Deck or Sheathing" does not apply.
Allowable moment:

The allowable moment is the smallest nominal moment calculated per AISI Design Specification Sections C3.1.1, C3.1.2, and C3.1.3 divided by a factor of safety of 1.67.

W = 1.67
$M_a = M_n / 1.67$ Eq. C3.1-1
M_a $= 13{,}590/1.67 = 8{,}138$ lb.-in. = **678 ft.-lb.**

$M_{max} = (9/128)wL^2$ (twin span beam, with web stiffeners, at the mid span where bottom flange is unbraced)

w= 15 psf (10 psf Attic LL + 5 psf DL)
w= 15 psf x 2 ft o.c. = 30 plf

$$L = \sqrt{\frac{128x8{,}138}{9x\frac{30}{12}}}$$

L = 215 in
L = 17 ft - 11 in

2. <u>C3.2 Strength for shear (with ICBO AC46 method of calculating shear with holes)</u>

Calculate the depth of the flat portion of the web, h.
h = web depth - 2(r + t)
h = 5.5 - 2(0.0937 - 0.035) = 5.243 in

h/t = 151.54

Calculate $1.38(EK_v/F_y)^{1/2}$
$1.38(Ek_v / F_y)^{1/2} = 1.38(29{,}500{,}000x5.34 / 33{,}000)^{1/2} = 95.35$

Since $h/t > 1.38(Ek_v/F_y)^{1/2}$
$V_a = 0.53(Ek_v t^3/h)$
$V_a = 0.53(29{,}500{,}000 \times 5.34 \times 0.035^3 / 5.243) =$ **660 lb. (without holes)**

ICBO method for calculating shear capacity of member with holes.
q_s = 1 - 1.1 (hole depth / web depth) (q_s is the shear reduction factor)
$q_s = 1 - 1.1(1.5/5.5) = 0.7$
$V_{a(hole)} = V_a \times q_s = 462$ lb.

V = 5/8 wL (simply supported twin span with web stiffeners)
$L =(5/8) \times V_{a(hole)} / w$
$L = (8/5) \times 462 / 30$
L =24.6 ft
L= 24'-7"

3. Span limit due to deflection

$$\delta = \frac{wL^4}{185EI} \quad \text{(simply supported twin span)}$$

d= L/240
w = 15 psf = 0.104 psi

$$L = \sqrt[1/3]{\frac{185EI}{w * spacing * 240}}$$

L = 236 in.
L= 19'- 8"

4. Calculate the span limit due to combined bending and shear at the intermediate support.
(AISI Design Specification Section C3.3 Strength for Combined Bending and Shear)

$M/M_{ado} \leq 1$ and V/V_a £ 1

M_{axo} = Allowable moment about the centroidal axis per AISI Design Specification Section C3.1.1
M_{axo} = 1390/1.67 = 832 ft-lb. (as calculated before)

V_a = 660 lb. (V_a without holes because holes are not permitted within 12 in. of a bearing point)
V = (5/8) wL (maximum shear at the intermediate support)

$M = wL^2/8$ (Maximum moment at intermediate support considering the member fully braced at both flanges)

$[(wL^2/8) / 832] \leq 1$
$[0.625wL / 660] \leq 1$

Solve for L using worst case (shear and moment):

L= 14.90 ft when $M = M_{ado}$
L= 14'-11"

Therefore, at L = 14.90 ft, V = (5/8) wL = (5/8) x 30 x 14.90 = 279 lb.

Since V/V_a = (283/660) = 0.43 < 0.7 the interaction equation need not be checked.
Hence, L= 14'-11"

Allowable joist span is controlled by AISI Design Specification Section C3.3 = **L = 14'-11"**

RAFTER DESIGN AND EXAMPLE

The following is an example for selecting and calculating the capacity for the rafters required for the example building configuration and loading condition shown in the ceiling joist example above.

Rafter Selection:

The horizontal rafter projection is just under 15 feet. From the "Allowable Rafter Span" table, select a rafter at 24 inches on center and the worst case of a ground snow load of 30 psf or an equivalent load to a wind speed of 80 mph, exposure C. First, convert the 80 mph exposure C wind speed to an equivalent ground snow load. For a slope of 6:12, an 80 mph Exposure C converts to a 20 psf ground snow load. Therefore, the 30 psf ground snow load should be used because it is higher. Two feasible solutions are 2 x 8 x 54 mil C-shaped member spanning 17'-1" or a 2 x 10 x 43 mil C-shaped member spanning 16'-0".

Calculate the capacity of the members selected to prove their structural adequacy.

i. Calculate the wind load due to the 80 mph, Exposure C, wind speed:

Both uplift and inward acting wind loads must be examined and the worst case converted to an equivalent snow load effect. ASCE 7-93 components and cladding coefficients are used to calculate the wind pressures.

Downward load:

$q = 0.00256 \times K_z \times (GC_p + GC_{pi}) \times (V \times I)^2$
$K_z = 0.87$ for exposure C
$GC_p = 0$ (Zone 1, Tributary area = 75 ft^2, page 18 ASCE 7-93)
$GC_{pi} = 0.25$ (± .25 whichever is worst)
$I = 1.0$ (non-coastal)
$q = 0.00256 \times 0.87 \times (0 + 0.25)\ (80x1)^2$
$q = 3.56$ psf (acting perpendicular to the plane of the rafter)
$q_{y\text{-slope}} = q/\cos\theta$ (converts load to an equivalent load acting in the y direction along the rafter plane)
$q_{y\text{-slope}} = 3.56 / \cos(26.56) = 3.98$ psf
$q_{horizontal} = q_{y\text{-slope}}/\cos\theta$ (converts load to an equivalent load acting perpendicular to the horizontal projection)
$q_{horizontal} = 3.98 / \cos(26.56) = 4.45$ psf

Upward load:

$q = 0.00256 \times K_z \times (GC_p + GC_{pi}) \times (V \times I)^2$
$K_z = 0.87$ for exposure C
$GC_p = -1.2$ (Zone 1, Tributary area = 75 ft^2, page 18 ASCE 7-93)
$GC_{pi} = -0.25$ (± 0.25 whichever is worse)
$I = 1.0$ (non-coastal)
$q = 0.00256 \times 0.87 \times (-1.2-0.25)\ (80x1)^2$
$q = -20.67$ psf (acting perpendicular to the plane of the rafter)
$q_{y\text{-slope}} = q/\cos\theta$ (converts load to an equivalent load acting in the y direction along the rafter plane)
$q_{y\text{-slope}} = -20.67 / \cos(26.56) = -23.11$ psf
$q_{y\text{-slope}} + DL = -23.11 + 10 = 13.11$ (compensate for dead load while loads are in the same direction and plane)
$q_{horizontal} = (q_{y\text{-slope}} + D.L.) / \cos\theta$ (converts load to an equivalent load acting perpendicular to the horizontal projection)
$q_{horizontal} = -13.11 / \cos(26.56) = -14.66$ psf

ii. Calculate the maximum allowable rafter span

From the Section Property Table

F_y = 33 ksi	$S_x = 1.41$ in^3	$I_x = 7.98$ in^4	D.L. = 15 psf
S.L.$_{ground}$= 30 psf	Def.$_{TL}$= L/180	Def.$_{LL}$= L/240	Spacing = 24 in. on center

The allowable span shall be the shorter of:
1. maximum span due to bending
2. maximum span due to shear
3. maximum span due to deflection

1. <u>Bending per AISI Design Specification Section C, "Flexural Members"</u>
   (all referenced equations and sections are those of the AISI Design Specification)

$M_a = M_n/\Omega_f$
M_n = smallest nominal moment of C3.1.1, C3.1.2, C3.1.3
$\Omega_f = 1.67$ (factor of safety)

C3.1.1 Nominal Section Strength

$M_n = S_eF_y$
$S_e = S_x$
M_n = (1.41)x(33.0) = 46.5 kip-in = 46,500 lb.-in.

C3.1.2 Lateral Buckling Strength

This section does not apply because roof sheathing provides lateral support.

C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing

Does not apply.

M_a= (46,500)/1.67 = 27,844 lb.-in.

$M_{max} = wL^2 / 8$ (simply supported single span)

w = (15 + 30 x 0.8)psf = 39 psf
w = 36 psf x 2 ft oc = 78 plf
L^2 = (8 x 27,844) / (78/12 lb./in)
L = 185.12 in $\Rightarrow$ L = 15'-3"

2. <u>C3.2 Strength for Shear (with ICBO AC46 method of calculating shear with holes)</u>

Calculate the depth of the flat portion of the web, h.
h = web depth - 2(r + t)
h = 10.0 - 2(0.0937 + 0.045) = 9.723 in.

h/t = 216.07

Calculate $1.38(EK_v/F_y)^{.5}$
$1.38(Ek_v/F_y)^{.5} = 1.38(29{,}500{,}000 \times 5.34 / 33000)^{1/2} = 95.35$

Since $h/t > 1.38(EK_v/F_y)^{.5}$
$V_a = 0.53(EK_v t^3/h)$
$V_a = 0.53(29{,}500{,}000 \times 5.34 \times 0.045^3\ / 9.723) = 782$ lb. (without holes)

ICBO AC46 method for calculating shear capacity with holes
$q_s = 1 - 1.1$(hole depth / web depth) shear reduction factor
$q_s = 1 - 1.1(1.5/10) = 0.835$
$V_{a(hole)} = V_a \times q_s = 653$ lb.

$V = wL/2$ (simply supported single span)
$L = 2 \times V_{a(hole)} / w$
$L = 2 \times 653 / 78$
L = 16.74 ft = 16'-9"

<u>3. Span limit due to deflection</u>

$d = 5 \times w \times L^4 / (384 \times E \times I)$ (simply supported single span)

Total load (d= L/180, w = 39 psf = 3.25 psi)

$L = [384 \times 29{,}500{,}000 \times I_x / (3.25 \times 24 * 180)]^{1/3}$
L = 186 in. = 15-6”

Live load (d= L/240, w = 24 psf = 2.00 psi)
$L = [384 \times 29{,}500{,}000 \times I_x / (2.00 \times 24 \times 240)]^{1/3}$
L = 198.72 in. = 16'-7"

Bending controls the design, hence the allowable rafter span is **L = 15'-3"**

iii. Verify the adequacy of the ridge member shear connection.

Consider the horizontal projection of a simply supported rafter.
$V_{max} = wL/2$ (L=15 ft, w =78 plf)
$V_{max} = 78 \times 15 / 2 = 585$ lb.

The nominal shear capacity of a # 10 screw is 789 lb. Therefore the allowable screw capacity (S_{allow}) is:

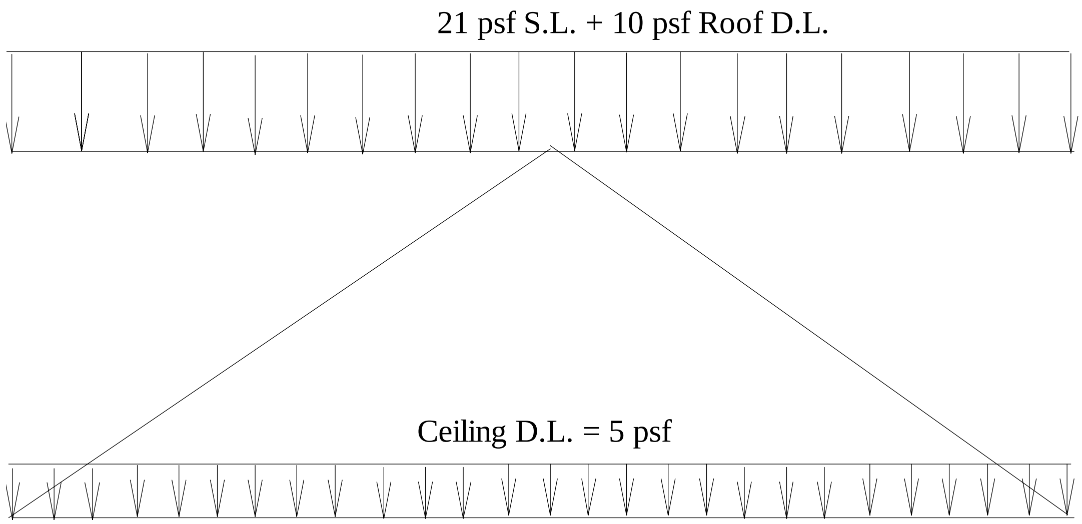
$S_{allow} = 789 / 3$ (use a safety factor of 3)
$S_{allow} = 263$ lb.

Number of screws needed = 585/263 = 2.22 screws

This number is rounded up to 3 screws.

iv. Verify the adequacy of the heel joist connection

SL = 24 psf (30 x 0.8)	House Width = 32 ft (use 32' to match table intervals)
Roof DL = 7 psf	Ceiling D.L. = 5 psf
Spacing = 24 in	Screw Capacity = 243 lb. (with a safety factor of 3)

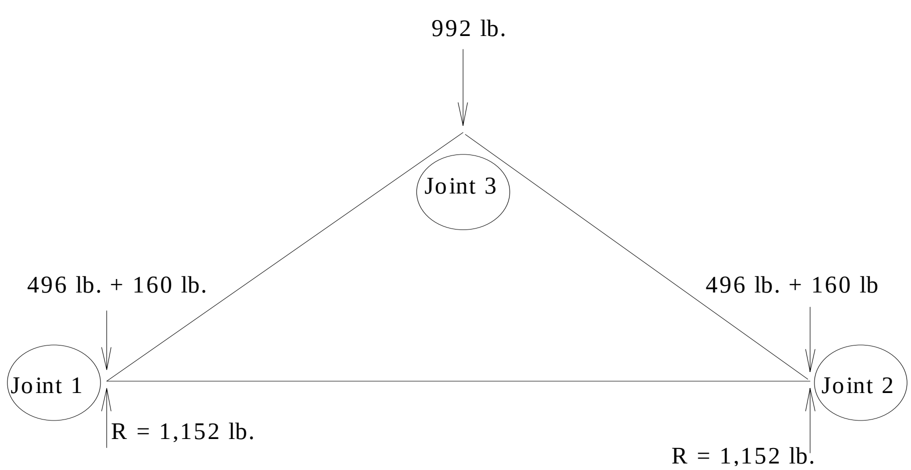


Roof Loading Diagram

Find the reactions.

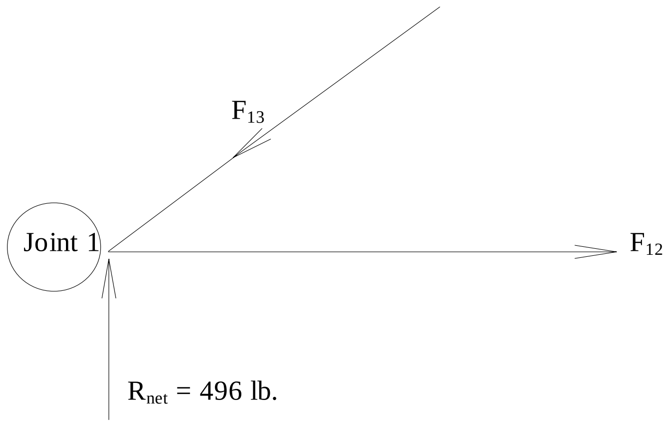
R = wL/2
R = ((24+7+5) x 2) x 32 / 2 = 1,152 lb.

A distributed load of [(24+7) x2] psf or 62 plf is spread across the length of the building. One fourth of this load will be concentrated at the end walls, joints 1 & 2, and half the load will be concentrated at the ridge beam connection, joint 3. Similarly, a distributed load of 5 psf or 10 plf is spread across 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).



Free Body Diagram

Joint 1 is examined to see which force has a greater magnitude (the compression in F_{13} or the tension in F_{12}).



Joint 1

q = tan^{-1} (6/12) = 26.565 degrees

F_{13} = 496 / sin(26.565) = 1109 lb.

F_{12} = 496 x cot(26.565) = 992 lb.

Heel joint connection shall be designed from the compression in F_{13} since it represents the worst case.

Number of screws = 1109 / 243 = 4.56

Round the number of screws up to 5

For both the heel joint and the ridge member connection analysis, a factor of safety of 3 is used.

COMMENTARY ON THE *PRESCRIPTIVE METHOD* FOR RESIDENTIAL COLD-FORMED STEEL FRAMING

APPENDIX A

Section Property Tables

APPENDIX B

Metric Conversion