



**American
Iron and Steel
Institute**

AISI STANDARD

**Commentary on the
Standard for
Cold-Formed Steel Framing –
Prescriptive Method for
One and Two Family Dwellings,
2001 Edition
with 2004 Supplement**

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

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

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

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PREFACE

The American Iron and Steel Institute (AISI) Committee on Framing Standards (COFS) has developed this *Commentary on the Standard for Cold-Formed Steel Framing - Prescriptive Method for One and Two Family Dwellings [Commentary]* to provide the background, supplemental information, engineering assumptions and methods, and detailed calculations for the provisions of the *Prescriptive Method* (AISI, 2001d). In 2004, this *Commentary* was expanded to address the 2004 *Supplement to the Standard for Cold-Formed Steel Framing - Prescriptive Method for One and Two Family Dwellings, 2001 Edition* (AISI, 2004).

The loads, load combinations, and other design parameters used to develop the provisions in the *Prescriptive Method* were based on the *International Residential Code* (ICC, 2000b), the *International Building Code* (ICC, 2000a) (where no provisions are included in the IRC) and ASCE 7 (ASCE, 1998).

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

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

Terms within the body of this *Commentary* that are shown in *italics* indicate that the italicized word is a defined term by the *Prescriptive Method* or by the *General Provisions* (AISI, 2001a).

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

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PART 1 – COMMENTARY

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

A. GENERAL

A1 Scope

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

A1.1 Limits of Applicability

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

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

The *Prescriptive Method*, however, does not limit the application of alternative methods or materials through engineering design.

Building Geometry: The provisions in the *Prescriptive Method* apply to detached one- and 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 by the user and therefore, engineering design support may be required. The most common building widths (or depths) range from 24 feet to 40 feet (7.3 to 12.2 m), with structural wall heights up to 10 feet (3.1 m). The building width as used in the *Prescriptive Method* is the dimension measured along the length of the *trusses* or *joists* (floor or ceiling) between the outmost structural walls. The maximum length of building is limited to 60 feet (18.3 m) where the length is measured in the direction parallel to the roof ridge or perpendicular to the floor joists or roof trusses.

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

Snow Loads: Snow load values are typically given in a ground snow load map such as provided in the building code, ASCE 7 (ASCE, 1998) or by local practice. The national model building codes in the U.S. either adopt the ASCE 7 snow load requirements or have a similar map published in the code. The 0 to 70 psf (0 to 3.35 kN/m²) 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 buildings that are expected to utilize this document. Buildings

in areas with greater snow loads than 70 psf (3.35 kN/m²) should not use this document without consulting a design professional.

Basic Wind Speed: All areas of the U.S. fall within the 90 to 130 mph (3-sec gust) (145 to 210 km/hr) range of design wind speeds, per ASCE 7 (ASCE, 1998). 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* is a critical determinant of the wind loads to be expected at a given site, and it should be determined by good judgment on a case-by-case basis. Buildings built along the immediate coastline (i.e. beach front property) are classified as Exposure D and therefore, cannot use this document without consulting a design professional. The three-second-gust wind speeds were used in the *Prescriptive Method* as identified in ASCE 7. Although ASCE 7 has wind contours up to 150 mph (241 km/hr), the 150 mph (241 km/hr) contour only occurs at the tip of southern Florida. Therefore, limiting the wind speed in the *Prescriptive Method* to 130 mph (210 km/hr) is appropriate.

Seismic Design Category: The *Prescriptive Method* covers all residential constructions in *Seismic Design Categories* A, B, C, D1, D2 and E (within the limits of applicability of Tables A1-1 and A1-2).

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. The load and resistance factor design (LRFD) load combinations as shown in ASCE 7 were used to develop the tables and other provisions in the *Prescriptive Method*.

A1.2 Limitations In High Seismic and High Wind Areas

A1.2.1 Irregular Buildings In High Seismic and High Wind Areas

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

A2 Definitions

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

A4 Limitations of Framing Members

A4.1 General

The *structural members* used in the *Prescriptive Method* are standard *C-shapes* produced by roll forming hot-dipped metallic coated sheet steel conforming to those specified in the *Standard for Cold-Formed Steel Framing – General Provisions* (AISI, 2001a).

A4.2 Physical Dimensions

Member section designations, in accordance with the *General Provisions* (AISI, 2001a), are used through out the *Prescriptive Method*. The designation system was developed in 1996 in order to standardize the identification of cold-formed steel framing based on specific shapes and material thickness. The designator is consists of four parts, the first value represents the *web* depth, the second value represents the type of steel framing member, the third value represents the *flange* width, and the fourth value represents the minimum base metal thickness.

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

Flange Width: The *Prescriptive Method* requires the standard *C-shape* have a minimum of 1-5/8 inch (41 mm) *flange* with a maximum *flange* dimension of 2 inches (51 mm). An increase in *flange* size above the 2 inch (51 mm) 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 (12.7 mm). This dimension is also common in the industry. Decreasing the *lip* size has a detrimental effect on the structural capacity of structural members in many circumstances.

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

The steel thickness required by the *Prescriptive Method* is the minimum uncoated steel thickness (excluding the thickness of the metallic coating) and is given in mils (1/1000 of an inch). This unit is a deviation from the historic practice, which uses a gauge designation for thickness. The “gauge” is an outdated reference that represents a range of thickness and is, therefore, a vague unit of measure when specifying minimums. The practice of using “gauge” as a basis for measurement has been discontinued in the industry. In order to achieve consistency, the *mil* designation was adopted. For example, the 33 mils (i.e., 0.033 inches or 0.94 mm), 43 mils (i.e., 0.043 inches or 1.09 mm), 54 mils (i.e., 0.054 inches or 1.37 mm), 68 mils (i.e., 0.068 inches or 1.73 mm), and 97 mils (i.e., 0.097 inches or 2.46 mm) are specified for the thickness.

The *design thickness* is defined as the minimum delivered thickness divided by 0.95, which follows the provisions of the *AISI Specification* (AISI, 1999). 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. This adjustment reasonably accounts for the normal variation in material thickness above the minimum delivered material thickness required.

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

A4.3 Material Properties

The *Prescriptive Method* applies to steel with minimum *yield strength* of 33 ksi (230 MPa) or 50 ksi (345 MPa). The 33 ksi (230 MPa) steels are the minimum required for all steel floors, roofs, and *header* components. Steel *stud* tables are provided for both 33 ksi (230 MPa) and 50 ksi (345 MPa) minimum *yield strength*. The 50 ksi (345 MPa) *yield strength* steel was included as separate option for wall *studs* selection because of the notable economic benefit in this particular application.

The user is advised to check the availability of specific framing material in the region in which the dwelling is being constructed. Not all material specified in the *Prescriptive Method* is expected to be available in all locations. The user is advised to check with suppliers for availability.

Strength increase from the cold work of forming (where allowed by the *AISI Specification*) 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 Section 2 of this document for engineering calculations illustrating the stress increase due to cold work of forming and its use in calculating section properties.

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

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

A4.4 Web Holes

All *structural members* (i.e., floor and ceiling *joists*, wall *studs* and *headers*), except cantilevered portions of framing members, used in the *Prescriptive Method* are designed assuming maximum hole dimensions as shown in Figure A4-1 and A4-2 of the *Prescriptive Method*. The design procedure follows the *AISI Specification* (AISI, 1999).

A4.5 Hole Patching

This section provides fixes for holes violating the requirements of Section A4.4. The hole patch details are not applicable when (a) the depth of the hole, measured across the width of the web, exceeds 70% of the depth of the web; and/or, (b) the length of the hole measured along the length of the web, exceeds 10 inches (254 mm) or exceeds the depth of the web, whichever is greater.

B. CONNECTIONS

B1 Fastening Requirements

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 *General Provisions* (AISI, 2001a) are specified. Requirements for sharp point screws connecting gypsum board and sheathing to steel *studs* are found in ASTM C1002 (ASTM, 2001) and ASTM C954 (ASTM, 2000). The edge distance and center-to-center spacing of these screws follow industry recommendations and the *AISI Specification* (AISI, 1999). This section in the *Prescriptive Method* is not intended to limit the fastening techniques to screws. Other fastening methods are permitted to be used, provided that the connection capacity is shown to exceed that implied in the *Prescriptive Method*. Testing, design, or code approvals may be necessary for alternate fastening techniques.

In certain applications, No. 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 (0.84 mm). For these reasons, screw manufacturer recommendations should be consulted.

Screw capacities given in Table C-B1 are calculated based on the design method given in the *AISI Specification* (AISI, 1999). The *Specification* 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.

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

Table C-B1
Minimum Allowable Fastener Capacity for Steel-to-Steel Connections
[Safety factor = 3.0]

Screw Size	Minimum Shank Diameter (inch)	Minimum Head Diameter (inch)	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 = 4.448 N.

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

B2 Bearing Stiffeners

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

The floor *joist* spans in the *Prescriptive Method* were derived assuming *bearing stiffeners* (also called transverse or *web* stiffeners) are located at all support or bearing point locations. Ceiling *joist* span tables were developed for two cases, 1) assuming *bearing stiffeners* are located at all support or bearing point locations and 2) *bearing stiffeners* are not installed at support or bearing point locations. Where specified, *bearing stiffeners* are to be a minimum of 43 mil (1.09 mm) *track* section or 33 mil (0.84 mm) *C-shaped* member.

B3 Clip Angles

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

D. FLOOR FRAMING

D2 Floor to Foundation or Structural Wall Connection

The *Prescriptive Method* provides several details for connecting floor assemblies to foundations or structural walls. The details are self-explanatory and reflect a selection from current practice. In areas where wind speeds exceed 110 mph (177 km/hr) (exposure C) or in *Seismic Design Category* D1, D2 or E, additional requirements for hold-downs and anchors are specified in Sections E11, E12 and E13.

D3 Minimum Floor Joist Sizes

The *Prescriptive Method* provides floor joist tables with maximum allowable spans for two live load conditions: 30 psf and 40 psf (1.44 and 1.92 kN/m²). The two live load conditions are specified in major building codes such as the BOCA (BOCA, 1997) and the IRC (ICC, 2000b). The 30 psf (1.44 kN/m²) is typically specified for sleeping areas, while the 40 psf (1.92 kN/m²) is specified for living areas. The spans shown in the *Prescriptive Method* assume bearing stiffeners are installed at each bearing point. Bearing stiffener requirements are provided in Section B2 of the *Prescriptive Method*.

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 to have web holes (a.k.a. "penetrations", "utility holes", "punchouts"), in accordance with Section A4.4. The compression flanges (top flanges) of the floor joists are 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. This typically exceeds the minimum established in building codes, but the stricter limit (i.e., L/480) was selected in order to provide more satisfactory floor designs.

Deflection limits are primarily established with regard to serviceability concerns. 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. The intent is to prevent excessive deflections that might result in cracking of finishes. The deflection criteria also affects the "feel" (e.g. perception) 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. $E \times I$). 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. Occupants may misconstrue this condition as a sign of weakness.

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. A higher deflection limit is usually recommended to overcome the concern with nuisance vibrations as it relates to human comfort. To ensure dynamic performance, the Australians (AISC, 1991) 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 $L/480$ (with 40 psf live load) 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 an $L/480$ deflection criterion when designing steel floor joists.

Multiple spans are commonly used in the residential steel building market. With multiple spans, certain measures are necessary to address the responses of the loaded members. The magnitude of the reaction at the middle support will be greater than the end reactions, and may cause a web crippling failure at this location, which is controlled by requiring *bearing stiffeners* at all bearing points. The second issue is the presence of negative moments (i.e. reversed bending) at the middle support 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 maximum loading conditions. Furthermore, shear and bending interaction need to be 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*.

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

D3.1 Floor Cantilevers

In many cases, cantilevers support structural 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 (610 mm) for floors supporting one wall and roof only (one story). This limitation is imposed to minimize the impact of the added load on the floor joists. To fully utilize the strength of the joist, web holes are not permitted in cantilevered portions. The *Prescriptive Method* provides details for first and second story cantilevered floors. It is essential that *blocking* be installed between cantilevered joists at the bearing locations to adequately transfer floor *diaphragm* or *shear wall* loads (refer to Section D5.4).

D4 Bearing Stiffeners

The floor spans in the *Prescriptive Method* were calculated assuming *bearing stiffeners* (also



called transverse or web stiffeners) are located at all support or bearing point locations. The *bearing stiffeners* are specified to be a minimum of 43 mil (1.09 mm) *track* section or 33 mil (0.84 mm) *C-shaped* member. It is possible that a *bearing stiffener* may not be required for certain floor spans that are controlled by failure modes other than web crippling (such as deflection or bending). The web-crippling equations of the *AISI Specification* (AISI, 1999) may be checked to determine if a *bearing stiffener* is not required.

D5 Joist Bracing and Blocking

D5.1 Joist Top Flange 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 function of the floor sheathing 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 (WJE, 1977).

D5.2 Joist Bottom Flange Bracing/Blocking

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

D5.3 Blocking at Interior Supports

Single floor *joists* that are lapped over interior supports do not require *blocking* as the lapped sections provide adequate lateral strength to prevent lateral movements. Continuous *joists* over interior supports, on the other hand, require *blocking* at 12 feet (3.7 m) intervals to provide adequate support to prevent lateral movement.

D5.4 Blocking at Cantilevers

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

D6 Splicing

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

Splices, generally, are required to transfer shear, bending moments, and axial loads. Some splices may occur over points of bearing and 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.

D7 Framing of Floor Openings

Openings in floors are needed for several reasons (such as at stairs, chases, chimneys). The *Prescriptive Method* limits the maximum width of the floor opening to 8 feet (2.4 m) and provides a provision for reinforcing the members around floor openings. All members around floor openings (i.e. *header* and *trimmer joists*) are required to be box-type members made by nesting a *C-shaped joist* into a *track* and fastening them together along the top and bottom *flanges*. These built-up members are required to be equal to or a greater in size and steel thickness than the floor *joists*, which they are connecting to. Each *header joist* is required to be connected to the *trimmer joist* with a *clip angle* on either side of each side of each connection. The *clip angle* is required to be of a thickness equivalent to the floor *joists*.

D8 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 has developed a *Standard for Cold-Formed Steel Framing – Truss Design* (AISI, 2001b) to assist in *truss* design.

D9 Diaphragms

Floor *diaphragms* are required to adequately transfer shear loads to the foundation. In steel floors, this is typically accomplished by sheathing the top *flanges* of the *joists* with wood *structural sheathing* (such as OSB or plywood). Shear values used in verifying the adequacy of the floor *diaphragms* were taken from LGSEA Technical Note No. 558b-1 (LGSEA, 1998) for oriented-strand-board (OSB) panels fastened to steel members with No. 8 screws at 6 inch (152 mm) on center spacing at panel edges and 12 inch (305 mm) on center spacing at intermediate supports. Additional requirements for steel floors constructed in *high wind* (110 mph (177 km/hr) or greater) or *high seismic areas* (*Seismic Design Category* D1, D2 and E) are specified in Section D9.1.

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

Shear values used in verifying the adequacy of the floor *diaphragms* were taken from LGSEA Technical Note No. 558b-1 (LGSEA, 1998) for oriented-strand-board (OSB) panels fastened to steel members with No. 8 screws at 6 inch (152 mm) on center spacing at panel edges and 6 inch (152 mm) on center spacing at intermediate supports. The reduced fastener spacing from 12 inches (305 mm) to 6 inches (152 mm) is to ensure that the diaphragm adequately transfers shear loads to the foundation.

E. WALL FRAMING

E2 Wall to Foundation or Floor Connection

In 2004, Table E2-1 was revised to allow direct connection of wall *track* to the floor sheathing rather than to require connection only through the floor sheathing to the floor *joist* or *track*. This revision was based on research by the NAHB Research Center (NAHBRC, 2003) in which five shear tests and six withdrawal tests were conducted where 33-mil *track* was connected to 23/32-inch-thick OSB sheathing using #8 screws. The average ultimate shear capacity was 412.2 lb and the average ultimate pullout capacity was 350.2 lb. Considering that the minimum allowable fastener capacities for steel-to-steel connections for #8 screws and 33 mil material of 164 lb for shear and 72 lb for pullout were used to calculate the requirements for the *Prescriptive Method*, the Committee deemed that it would not be necessary to require that every fastener connect to a floor *joist* or *track* member.

E3 Minimum Stud Sizes

This section dictates the minimum required thickness of steel *studs* for different wind speeds, *wind exposure* categories, wall heights, building widths, live loads, and ground snow loads. *Stud* selection tables are limited to one- and two-story buildings with *structural wall* heights up to 10 feet (3.05 m).

The 8-foot (2.44 m) walls are widely used in residential construction; however, steel framed buildings often take advantage of higher ceilings such as 9- and 10-foot (2.74 and 3.05 m) walls. The 50 ksi (345 MPa) *yield strength stud* tables were developed to take advantage of the higher *yield strength*, which allows thinner *studs* in many cases. The user should look into the availability of certain steel sizes and thickness in 33 or 50 ksi (230 and 345 MPa) *yield strengths* as many steel manufacturers do not produce certain steels in both 33 or 50 ksi (230 and 345 MPa) *yield strength*.

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

- *Studs* are simply supported beam - columns
- *Bracing* of the interior and exterior *flanges* of the *studs* by *structural sheathing* or mechanical *bracing* (mechanical bracing at mid-height for 8-foot studs (2.4 m), 1/3 point for 9-foot (2.74 m) and 10-foot (3.05 m) studs)
- Maximum roof overhang of 24 inches (610 mm)
- Roof slopes limited to a range of 3:12 to 12:12
- Deflection limit of $L/240$
- Ceilings, roofs, attics, and floors span the full width of the house (no interior *bearing* walls)
- Permitted attic live load is limited to 10 psf (0.48 kN/m²), unless an adjustment in the snow loads is made

- Second floor live load is 30 psf (1.44 kN/m²) unless an adjustment is made to the snow loads

Stud Design

The design of the *studs* was based on the following design checks:

- Combined bending and axial stresses (strengths) using Main Wind Force Resisting System (MWFRS) wind loads
- Bending stresses (strengths) based on Components and Cladding (C&C) loads
- Deflection limits based upon 70% of Components and Cladding with no axial loads

These design checks are not specifically discussed in any of the national building codes, such as the IBC; however, Section 1609.6.2.3 of the 2000 IBC (ICC, 2000a) states that:

“Members that act as both part of the main force resisting system and as components and cladding shall be designed for separate load cases.”

The discussion in the Southern Building Code Commentary (SBCCI, 1999) sheds the most light on a reasonable approach to the design of wall *studs* for wind resistance. The Commentary states that:

“Some elements of a building will function as part of the main wind force resisting system and components and cladding also. Such members include but not limited to roof panels, rafters, and wall studs. These elements are required to be designed using the loads that would occur by considering the element as part main wind force resisting system, and also separately checked or designed for loads that would occur by considering the element as component and cladding. The use of this section can be demonstrated by considering, for example, the design of a wall stud. When designing the stud for main wind force resisting system loads, all loads such as bending from the lateral force with the wind on the wall in addition to any uplift in combinations with the dead load of the roof or a story above induced by the simultaneous action of roof forces should be considered together. When designing the stud for component and cladding loads, only the bending resulting from the wind force normal to the stud and the dead load associated with that member should be considered. The member should be sized according to the more critical loading condition.”

The wood industry has also investigated this condition and has adopted a similar policy as shown in the Wood Frame Construction Manual (AFPA, 1995) where Section 2.4 states that:

“Stud tables are based upon bending stresses induces by C&C loads. The bending stresses are computed independent of axial stresses. In addition, the case in which bending stresses from MWFRS loads act in combination with axial stresses from wind and gravity loads have been analyzed. For buildings limited to the conditions in the WFCM-SBC, the C&C loads control stud design.”

Design Loads

Both the Components and Cladding (C&C) and the Main Wind Force Resisting System (MWFRS) loads at the ends and corners of walls can be significantly higher than in the middle of the wall. Rather than design the entire wall for these increased loads, the loads in the middle of the wall were used to design the studs.

Deflection Criteria

Building codes (such as BOCA, 1997 and ICC, 2000a) specify the deflection limits for exterior walls subjected to wind loads. The limits are L/240 for brittle finishes and L/120 for flexible finishes. Furthermore, none of the national codes give any guidance on whether the deflection limits should be based on MWFRS or C&C loads. The commentary to Appendix B of ASCE 7 (ASCE, 1998) provides some guidance on the selection of loads for checking the serviceability

limit state of buildings and components thereof. Section B1.2 therein, states in part:

“Use of factored wind load in checking serviceability is exclusively conservative. The load combination with an annual probability of 0.05 of being exceeded, which can be used in checking short-term effects, is

$$D + 0.5L + 0.7W \text{ ”}$$

Thus using 70% of the wind load from Components and Cladding (C&C) would conservatively satisfy the above.

AISC Design Guide No. 3 (AISC, 1990) also recommends reduced wind loads when checking serviceability of cladding based upon a 10-year return period on the wind or a probability of 0.10.

The Wood Frame Construction Manual (AFPA, 1995) is based slightly on a different criterion. The deflection limit is set at $L/120$ using MWFRS loads. The use of MWFRS appears to be within the intent of the recommended provisions of ASCE 7 (ASCE, 1998) Appendix B, since MWFRS loads are often about 70% of the C&C at least in the middle of the walls. This is an acceptable approach for *studs* supporting flexible finishes however, it is questioned if appropriate for use in *studs* supporting brittle finishes such as stucco. Therefore, *stud* deflections were checked against 70% of the C&C loads in the middle of the wall.

E4 Stud Bracing

Studs in *structural walls* are laterally braced on each *flange* by either a continuous 1-1/2 inch x 33 mil (38.1 x 0.84 mm) (minimum) *strap* at mid-height (or third points for 9-foot (2.74 m) and 10-foot (3.05 m) studs) or by direct attachment of *structural sheathing* or rigid wall finishes (i.e. structural panels such as plywood, OSB or gypsum board) according to the requirements of the *Prescriptive Method*. Therefore, all *studs* are considered to be braced at mid-height (or third points for 9-foot (2.74 m) and 10-foot (3.05 m) 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 allowance in dropping down a *stud* thickness (but not less than 33 mil (0.84 mm)).

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 or bridging, 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, temporary *bracing* must be provided.

E5 Splicing

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

E6 Corner Framing

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

E7 Headers

Headers are horizontal members used to transfer loads around openings in *structural walls*. *Headers* specified in the *Prescriptive Method* are allowed only above the opening immediately below the wall top *track* (i.e. *high headers*). Historically, the two traditional ways of constructing *headers* was to put two *C-shaped* members back-to-back or in a box shape. However, recent testing of L-shaped *headers* has proven that they could be an economical alternative to traditional *headers* in lightly loaded situations.

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

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

The design of *headers* is based on the *Standard for Cold-Formed Steel Framing – Header Design* (AISI, 2001c).

E7.1 Box-Beam Headers

Box-beam *headers* are formed from two equal sized *C-shaped* members placed toe-to-toe in a box type configuration and fastened to the wall top *track* and the bottom *track* spanning the width of the opening to create the box. *Tracks* used to frame around openings are required have a steel thickness equivalent to or greater than the wall *studs*. The bottom *track* can face towards the top or the bottom. Box-beam *headers* do require pre-insulation prior to installation, but have an advantage in *high wind* areas because *strapping* can be easily wrapped around them. Box-beam *headers* also provide a larger surface to apply interior and exterior finishing materials.

E7.2 Back-to-Back Headers

Back-to-back *headers* are formed from two equal sized *C-shaped* members in a back-to-back configuration creating an I-section. These *C-shaped* sections are fastened to the wall top *track* and the bottom *track* spanning the width of the opening. *Tracks* used to frame around openings are required to have a steel thickness equivalent to or greater than the wall *studs*. The bottom *track* can face towards the top or the bottom. Back-to-back *headers* are easier to construct than box-beam *headers* and do not require insulation to be installed prior to the installation of the *header*. However, it is more difficult to install *strapping* around back-to-back *headers* in *high wind* areas.

E7.3 Double L-Headers

A double L-header is shown in Figure E7-3 of the *Prescriptive Method*. Tables for gravity and uplift loads are provided for double L-headers. Double L-headers are typically the easiest *headers* to install. They can be installed during or after the wall has been framed. They do not require pre-insulation and provide a large surface to apply finishing materials. They also require less material (steel and screws) than back-to-back or box-beam *headers*. Double L-headers do not need to be cut to exact lengths; however, they need to lap over a minimum of one *king stud* at each end.

In 2004, the requirements in the *Prescriptive Method* for the *L-header* to *king stud* connection was revised to be consistent with the *Header Standard* (AISI, 2001c). This was unintentionally missed in the previous edition of the *Prescriptive Method*.

E7.4 Jack and King Studs and Head Track

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).

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

E8 Wall Bracing

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

Three different *bracing* methods are recognized in the *Prescriptive Method*:

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

E8.1 Strap Bracing (X-brace)

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

E8.2 Structural Sheathing

Shear wall testing of steel-framed walls provided the data necessary to develop the provisions and tables of values for the *structural sheathing* sections. All tests performed were based on sheathing panels, which consisted of either 15/32 inch (0.47 mm) plywood or 7/16 inch (0.44 mm) oriented strand board as the bracing method (AISI, 1998).

The wall *bracing* requirements in the *Prescriptive Method* were based on an engineered approach that utilized available technical knowledge. The allowable shear capacities for plywood and oriented- strand-board (OSB) sheathing are based on test results summarized in

Table 1 of the *AISI Shear Wall Design Guide* (AISI, 1998). The load test results relevant to this document are summarized in Table C-E8.1.

Table C-E8.1
Cold-Formed Steel Framed Walls Nominal Shear Values (AISI, 1998)^{1,2,3,4}

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	1065	508	425
7/16" OSB APA rated sheathing w/ panels on one side	910	593	364

For SI: 1 inch = 25.4 mm, 1 plf = 0.0148 kN/m

- ¹ Framing studs spaced at 24 inches on center.
- ² Studs and tracks: 350S162-33 and 350T125-33 are both ASTM A653 Grade 33.
- ³ Framing screws: No. 8 x 5/8 inch wafer head self-drilling screws;
Sheathing screws: No. 8 x 1 inch flat head (coarse thread), sharp point, self-drilling
@ spacing of 6 inch at panel edges and 12 inches on intermediate members.
- ⁴ 1/2" gypsum wallboard on interior of wall.

The requirement in the *Prescriptive Method* is to have the entire building fully sheathed (except for door and window openings, as limited by the minimum length of full height sheathing). Therefore, in low wind regions, there is little or no wind uplift pressure and therefore, an allowable shear capacity of 364 plf (5.39 kN/m) was used for the determination of the wall *bracing* requirements. The allowable capacity is less than the load recorded at 1/2 inch (12.7 mm) 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 (ASCE, 1998). Standard tributary areas consisting of the leeward and windward building surfaces, were assigned to each exterior *shear wall* (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 developed 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 (5.39 kN/m). 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* applicability limits. The length of wall with full-height sheathing is defined as the sum of wall segments that have sheathing extending from the bottom track to the top track, without interruption due to openings (i.e., the total of lengths of wall between window and door openings). Further, the individual wall segments must be 48 inches (1.22 m) in length or greater to contribute to the required length of full-height sheathing for a given wall line, unless permitted otherwise.

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

E8.3 Structural Sheathing Fastening

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

E8.4 Hold-down Requirements

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

E9 Exterior Wall Covering

It is recommended that exterior coverings be installed in accordance with the recommendations of the manufacturer. The *Prescriptive Method* limits the total exterior envelope dead load (total load = stud framing plus wall coverings) to 10 psf (0.48 kN/m²). If the total exterior envelope dead load exceeds that value, then the walls must be engineered for that load (see Table A1-2 for maximum wall dead loads in high seismic areas).

E11 Braced Walls in High Wind and High Seismic Areas

E11.1 General

This section provides additional *shear wall* requirements for buildings located in *high seismic areas* (i.e., *Seismic Design Categories* D1, D2 and E) or *high wind areas* (i.e., wind speed between 100 to 130 mph (161 to 209 km/hr)). In *high seismic areas*, buildings are required to comply with Section E11 and E12; and in *high wind areas*, buildings are required to comply with the requirements in Section E11 and E13.

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

- Provisions and tables are limited to buildings no more than two-stories
- Provisions and tables are limited to buildings on slab-on-grade or spread footing with stemwall foundation system with a single top of slab/top of stemwall elevation
- Wall clear heights are limited to 8, 9, and 10 feet (2.44, 2.74 and 3.05 m)
- Maximum roof slope is limited to 6:9:12
- All ceilings are considered leveled (i.e., no offsets or cathedral ceilings)

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

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

- Roof/ceiling dead load = 25 psf (1.2 kN/m²) for heavy weight roofs
= 15 psf (0.72 kN/m²) for normal weight roofs
= 12 psf (0.57 kN/m²) for light weight roof systems
- Wall dead load = 14 psf (0.67 kN/m²) for heavy walls
= 7 psf (0.34 kN/m²) for light walls
- Floor/ceiling dead load = 10 psf (0.48 kN/m²)
- Interior wall dead load = 5 psf (0.24 kN/m²) (based on 10 foot (3.04 m) wall)
- Ground snow load = 30 psf (1.44 kN/m²) for normal or light weight roofs
= 70 psf (3.35 kN/m²) for heavy weight roof systems
- Roof weight includes a 2-foot (610 mm) overhang

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

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

For SI: 1 psf = 0.0479 kN/m².

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

For SI: 1 psf = 0.0479 kN/m².

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

For SI: 1 psf = 0.0479 kN/m².

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

- Nominal shear values are taken from Table 2211.1(3) of the *International Building Code* (IBC) (ICC, 2000a)
- *Seismic Design Category* (SDC) assignments in accordance with Table R301.2.2.1.1 of the *International Residential Code* (ICC, 2000b)
- Seismic base shears were calculated in accordance with the IBC (ICC, 2000a) using an $R = 5.5$ and $\Omega_0 = 2.5$. Upper end S_{DS} values are used for each SDC
- *Diaphragms* are considered to be flexible rather than rigid. No requirement for inclusion of accidental torsion and reduction of Ω_0 from 3 to 2.5 in accordance with the IBC (ICC, 2000a) was used
- ϕ factor was used in combination with the Ω_0 over strength factor to determine screw requirements for chord splices
- $F_u = 45$ ksi (310 MPa) and $F_y = 33$ ksi (230 MPa) were used in lieu of the IRC (ICC, 2000b) allowed 1.08 tensile/yield ratio in determining screw capacities. *Shear wall* test values are based on the 33 ksi/45 ksi (230/310 MPa) steels
- Chord splice screw requirements are based on the lesser of Ω_0 times $T_{seismic}$ or T_n , divided by ϕV_n . Both the 3-1/2 inch (88.9 mm) and the 5-1/2 inch (140 mm) members were considered, as well as both the 33 mil (0.84 mm) and the 43 mil (1.09 mm) thickness
- Where the ground snow load is 70 psf (3.35 kN/m²), the heavy roof system criteria applies

Example: Load due to 70 psf ground snow load for normal weight roofs =
 $15 \text{ psf} + 0.2 \times 0.7 \times 70 \text{ psf} = 24.8 \text{ psf}$ (1.19 kN/m²) (equals the heavy weight roof systems)

E11.2 Braced Wall Lines

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

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

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

E11.4 Type II (Perforated) Braced Wall Lines

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

E12 Braced Wall Design in High Seismic Areas

E12.2 Braced Wall Anchorage and Chord Stud Requirements

Hold-downs and the proper number and thickness of chord *studs* are essential to allow the building to resist uplift forces in *high seismic areas*.

E12.3 Wall Top Track

The top *track* splice shall be adequate to transfer *diaphragm* forces. Table E12-5 provides the number of screws required for each splice.

E13 Braced Wall Design in High Wind Areas

E13.3 Connections of Walls in High Wind Areas

E13.3.3 Header Uplift Connections

In 2004 a figure was added to the *Prescriptive Method* to illustrate a *header* uplift connection to a back-to-back *header* beam. For back-to-back *headers* supporting roof and ceiling only, these provisions require that uplift straps be installed on both sides of the *header* beam (inside and outside of the wall) in order to minimize any effect of torsion. The Committee felt this was appropriate since back-to-back *header* beams lack sufficient torsional strength and stiffness. For back-to-back *headers* supporting loads from one floor, roof and ceiling, and for any box and double L-headers, a single uplift strap is permitted and may be installed on either side of the *header* beam.

F. ROOF FRAMING

F1 Roof Construction

Roof construction in the *Prescriptive Method* is limited to roof *rafters* and ceiling *joists*. Roof *trusses* are not prescriptively addressed in the *Prescriptive Method*, but are permitted and must be designed by a design professional. Hips, valleys, and roof girders are also not addressed in the *Prescriptive Method*.

F2 Ceiling Joists

F2.1 Minimum Ceiling Joist Size

Ceiling *joist* tables in the *Prescriptive Method* provide the maximum allowable ceiling *joist* spans for two loading conditions: 10 psf (0.48 kN/m²) and 20 psf (0.96 kN/m²) attic live loads.

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 the *Prescriptive Method* is similar to that used for floor *joists* with the exception of the magnitude of dead and live loads.

F2.2 Ceiling Joist Bearing Stiffeners

Bearing stiffeners are required at the ends of ceiling *joists* unless the tables with no *bearing stiffeners* are used.

F2.3 Ceiling Joist Bottom Flange Bracing

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

F2.4 Ceiling Joist Top Flange Bracing

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