

Cold-Formed Steel Gable End Wall Design Using the Prescriptive Method for One and Two Family Dwellings

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PREFACE

This report was developed by the University of Missouri-Rolla for the Steel Framing Alliance and the Prescriptive Methods Subcommittee of the AISI Committee on Framing Standards. The primary objective of this project was to create the documentation necessary to enable the expansion of the AISI *Standard for Cold-Formed Steel Framing – Prescriptive Method for One and Two Family Dwellings* for gable end walls.

Research Team
Steel Framing Alliance

**Civil Engineering Study 05-1
Cold-Formed Steel Series**

Final Report

**COLD-FORMED STEEL GABLE END WALL DESIGN USING THE
PRESCRIPTIVE METHOD FOR ONE AND TWO FAMILY
DWELLINGS**

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PREFACE

Cold-formed steel is gradually gaining acceptance in the residential construction sector. This is due in large part to the development of specifications and guidelines which aid in the design process. One such publication, the *Prescriptive Method*, is particularly attractive for residential projects, since it allows standard one and two story buildings to be designed without the aid of a design professional. One significant omission in the current *Prescriptive Method* is the design of gable end wall studs. As requested by the AISI Committee on Framing Standards (COFS), this report presents the tables and details necessary to incorporate gable end wall design into future editions of the *Prescriptive Method*. This report also serves as a commentary on these design aids, explaining the design methodologies and assumptions used in their creation. This study and report was limited to developing the necessary information to incorporate gable end walls in the *Prescriptive Method* and does not address design issues not presently incorporated in the *Prescriptive Method*. Also, where a design solution currently exists in the *Prescriptive Method* such design solution is not repeated herein.

Three primary gable end construction methods are explored: balloon framing, gable end truss framing, and cathedral ceiling framing. To resist wind forces, the balloon and gable end wall methods are provided with an out-of-plane bracing option using horizontal ceiling diaphragms. Additional design considerations including bottom track attachment to floors or foundations and requirements for openings in gable end walls are also addressed. Example calculations are included to support each of the design aids presented.

This report is based on a thesis submitted to the Faculty of the Graduate School of the Kansas State University in partial fulfillment of the requirements for the degree of Masters of Science in Architectural Engineering.

Technical guidance for this study was provided by the Steel Framing Alliance's Project Monitoring Task Group (W. Babich, S.R. Fox, K. Pagano, S.H. Walker), and the American Iron and Steel Institute's Prescriptive Method Subcommittee (S.R. Fox, Chairperson). The Project Monitoring Task Group and the Subcommittee's guidance is gratefully acknowledged. Thanks are also extended to J.W. Larson, AISI staff for his guidance and assistance.

1.0 Loads

1.1 Load Types

1.1.1 Dead Loads

According to Table A1.1-1 in the *Prescriptive Method* (AISI, 2002), roof/ceiling dead loads are taken as a total of 12 psf. This load is conservatively applied to the stud at the roof level, even though part of it occurs at the ceiling height. The tributary area for this load assumes a 2 ft. rafter or truss spacing and 1 ft. rake overhang. This represents the largest tributary area for any of the framing systems allowed. See Figure 2.3 for typical overhang details at the roof level. The floor dead load for lower walls in 2 story construction is 10 psf per Table A1.1-1 (AISI, 2002). A floor joist spacing of 2 ft. is used, yielding a tributary floor width of 1 ft.

1.1.2 Live Loads

Roof live loads are not considered in these tables, since it will be shown that snow loads will control design. Floor live loads for lower walls are conservatively taken as 40 psf, acting on the same area as floor dead loads.

1.1.3 Snow Loads

To include all design locations and maintain conservativeness, ground snow loads are considered to be 70 psf. The associated unbalanced roof snow load of 73.5 psf, calculated according to ASCE 7-02 Figure 7-5, is used to determine axial loads on the wall studs. This snow load acts on the same area described above for dead loads (ASCE, 2002).

1.1.4 Wind Loads

The final loads considered in stud design are the out-of-plane wind forces acting on the wall. There are two types of wind loads that are considered to act on building structural systems: Main Wind Force Resisting System (MWFRS), and Components

and Cladding (C&C) loads. Typically, structural elements that are part of the building's Primary Lateral Force Resisting System are designed using MWFRS loads. These components generally receive wind loads from more than one surface of the building simultaneously. Other exterior structural components which receive wind load directly and transfer it to the MWFRS must be designed for C&C loads.

Wall studs may be part of the Primary Lateral System (shear walls, etc.), yet clearly qualify as components and cladding. Therefore, according to Section 1609.6.2.3 of the 2000 IBC, "Members that act as both part of the main force resisting system and as components and cladding shall be designed for separate load cases" (ICC, 2000; AISI, 2003). This position is widely accepted in current codes; however, significant disagreement does exist as to how these two separate load types should be considered in conjunction with accepted load combinations. It is clear that MWFRS elements must be designed for all applicable load combinations, including both pure axial and combined axial and bending cases. It is less certain, however, as to what load combinations should be used to check wall studs for C&C loads. Although many argue that combined axial and bending combinations should be used for C&C loads, it is the current position of the AISI that "The component and cladding strength design should be a pure bending analysis independent of axial dead and live loads" (AISI, 2000).

In addition to considering the strength of wall studs under the above combinations, wall deflections must be controlled to prevent cracking, etc. of veneers and finishes. The IBC gives wall stud deflection criteria due to wind loads in Table 1604.3 (ICC, 2000). As a serviceability issue, it is accepted that these deflections may be calculated using either MWFRS loads or a percentage of the higher C&C loads. A 2000 AISI Position Paper states that walls should be evaluated for deflection based on the worst case of MWFRS loads or 70% of C&C loads (AISI, 2000). In the majority of cases, 70% of C&C loads will control deflection design. This 70% value is

conservatively based on the following statement found in the commentary to Appendix B of ASCE 7-98: “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 $D + 0.5L + 0.7W$ ” (ASCE, 1998).

To comply with these requirements, walls in the *Prescriptive Method* are designed for all applicable load combinations using MWFRS wind loads, and a pure bending combination using C&C loads. Deflections are checked using 70% of C&C loads against an $h/240$ deflection criteria, consistent with brittle veneers (ICC, 2000; AISI, 2003). All wind loads for studs in the *Prescriptive Method* are taken in the middle of the wall, away from the higher loads found at the corners. The *Prescriptive Method* does this because it assumes that the use of these higher wind loads for design of the entire wall would result in an overly conservative and uneconomical wall design (AISI, 2003).

1.2 Load Combination

The following load combinations from IBC 2000 must be considered in gable end wall design (ICC, 2000):

1.4D	Formula 16-1
$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$	Formula 16-2
$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (f_1L \text{ or } 0.8W)$	Formula 16-3
$1.2D + 1.6W + f_1L + 0.5(L_r \text{ or } S \text{ or } R)$	Formula 16-4
$1.2D + 1.0E + f_1L + f_2S$	Formula 16-5
$0.9D + (1.0E \text{ or } 1.6W)$	Formula 16-6

Where:

D = Roof/Ceiling or Floor Dead Load
L = Live Load
 L_r = Roof Live Load
S = Snow Load
R = Rain Load
W = Wind Load
E = Seismic Load

For gable end wall design, Formula 16-1 has been ignored due to the small amount of dead load on the studs. In Formulas 16-2 and 16-4, snow loads are assumed to control over roof live loads and rain loads. Formula 16-5 is neglected, since seismic effects rarely control out of plane wall design in the *Prescriptive Method*. Per IBC Section 1605, f_1 is taken as 0.5 in applicable combinations to coincide with typical live load conditions (ICC, 2000). With these assumptions, the combinations are adapted to gable end wall design as follows:

- (1) $1.2D + 1.6L + 0.5S$
- (2) $1.2D + 0.5L + 1.6S$
- (3) $1.2D + 0.8W + 1.6S$
- (4) $1.2D + 1.6W + 0.5L + 0.5S$
- (5) $0.9D + 1.6W$
- (6) $1.6W$
- (7) $0.7W$

For the pure axial load combinations (1) and (2), dead, live, and snow loads are applied as described above. In the combined axial and bending combinations (3) and (4), the wind load considered is the MWFRS load. Load combination (5) is considered for uplift design of roof overhangs, and stud to top track connections using C&C loads. It is also considered for combined shear and uplift for the bottom track connections to supporting structure using MWFRS loads. C&C loads are also used in the pure bending combination (6), which has been added to comply with current AISI requirements as previously discussed (AISI, 2000). Finally, load combination (7) is used to check mid-height wall deflections using C&C loads. In all of these combinations, design checks are made on the tallest stud at the middle of the gable end wall. Web crippling was also checked for the controlling load combination.

For full-height walls that carry only roof/ceiling axial loads, only load combinations (3), (4), (6), and (7) are considered, since it is assumed that the pure axial cases (1) and (2) will not control over the combined axial/bending combinations (3) and (4). For lower walls in 2-story structures, all combinations except (5) are used.

2.0 Wall Framing Methods

2.1 Balloon Gable Framing

Construction of full height balloon-framed stud walls is an efficient and economical method for framing gable end walls of residential structures. A typical elevation of this construction technique is shown in Figure 2.1. These stud walls resist both out-of-plane wind forces and axial roof loads. Two construction options were considered for this design:

1. Unbraced Full Height Wall
2. Wall Braced by Horizontal Ceiling Diaphragm.

The tables and details for these design options are presented below, along with pertinent design assumptions.

2.1.1 Unbraced Option

For many low-slope roofs and/or narrow buildings, it is possible to construct the gable end as a full height balloon framed wall without intermediate out-of-plane support for strong axis bending. This method is illustrated in Figure 2.2.

Typical construction details for the gable end overhang along the rake are shown in Figure 2.3. Figure 2.8 (*Prescriptive Method* Figure D2.4) shows typical blocking configurations and connections for overhangs according to the *Prescriptive Method*. Appendix A includes a calculation for the number of screws required in these connections. These screws must resist shear forces, uplift (tension) forces, and the combined effect of shear and tension.

The appropriate full height stud thickness for a given wall height, h (ft.), and wind speed can be selected from Table 2.1 or Table 2.2. These tables consider the following design checks: combined axial load and bending, pure strong axis bending, member deflection, and web crippling. The controlling check is determined from among these and used to establish the stud selections shown.

For combined axial load and bending, load combinations (3) and (4) are checked. Factored axial loads are applied at the roof level, and lateral stud bracing is assumed at 48 in. on center spacing. For the pure strong axis bending check of load combination (6), the stud is modeled as a full height simply supported beam between the structural tracks, with length h. Deflection is checked against a maximum mid-height deflection of h/240. The resulting stud thicknesses listed in the table are based on calculations for the wind load acting on an area equal to the height, h, times the stud spacing. Finally, web crippling at the top and bottom of the wall is considered using Equation C4.2-1 from the AISI Wall Stud Standard (AISI, 2004a):

$$\phi P_{nst} = Ct^2 F_y \left(1 - C_R \sqrt{\frac{R}{t}} \right) \left(1 + C_N \sqrt{\frac{N}{t}} \right) \left(1 - C_h \sqrt{\frac{h}{t}} \right) \quad (\text{Eq. C4.2-1})$$

Where:

P_{nst} = Nominal Crippling Strength

C = Web Crippling Coefficient = 3.7

C_R = Inside Bend Radius Coefficient = 0.19

C_N = Bearing Length Coefficient = 0.74

C_h = Web Slenderness Coefficient = 0.019

R = Stud Inside Bend Radius

N = Stud Bearing Length = 1.125 in. (For 1.25 in. Track)

h = Depth of Flat Portion of Stud Web Measured along Plane of Web

t = Stud Design Thickness

ϕ = 0.90

This capacity ϕP_{nst} is compared to load combination (6), with W taken as the C & C wind load on the interior part of the wall. A complete example calculation for Table 2.1 can be found in Appendix A.

2.1.2 Horizontal Ceiling Diaphragm Option

The second option for balloon framed gable end design is to construct a structural horizontal ceiling diaphragm to help resist the out-of-plane loads acting on the wall and reduce the effective stud height, h. Two diaphragm construction options are

considered for this project. The first is constructed of 3/8 in. (minimum thickness) C-D or C-C sheathing attached to the bottom of the ceiling joists using No. 8 screws at 6 in. on center at panel edges and in the field. This system has an excellent out-of-plane capacity based on Table D2-1 of the AISI Standard for Cold-Formed Steel Framing – Lateral Design (AISI, 2004); however, construction of a sheathed ceiling is cumbersome. Consequently, a gypsum board ceiling diaphragm was proposed by COFS at the September meeting. This diaphragm is made of 1/2 in. gypsum wall board secured to the bottom of the ceiling joists with No. 8 screws at 6 in. on center at panel edges, and 12 in. on center in the field. While simple to construct, this diaphragm has limited structural capacity, and is therefore only an option for regions with low wind loads. To adequately resist the out-of-plane loads at the gable end, these diaphragms must be constructed over a minimum length from the end wall, as indicated in Figure 2.4, and illustrated in the plan view of Figure 2.5. These minimum lengths are given in Tables 2.3 and 2.4 for gypsum diaphragms, and Tables 2.5 and 2.6 for sheathed diaphragms.

Figures 2.6 and 2.7 show the diaphragm attachment to the gable end walls and structural walls. Figure 2.8, taken from Figure D2-4 in the *Prescriptive Method*, shows proper blocking connection to the ceiling joists at the edges of the diaphragms (AISI, 2002).

As indicated, the required diaphragm lengths vary based on roof pitch, building width, and wind speed. A maximum aspect ratio of 3:1 for sheathed diaphragms and 2:1 for gypsum diaphragms is imposed to establish the given lengths. For ceiling joists with a designated thickness of 54 mils or less, No. 8 screws at the spacing indicated in the tables should be used. For joists thicker than 54 mils, No. 10 screws shall be used at the same spacing (AISI, 2004b). All calculations are based on unblocked diaphragm capacities however, a multiplication factor is provided to determine the diaphragm length if blocking is provided at all panel edges. Construction of diaphragms over 30 ft. from

each end wall would exceed the maximum overall building length of 60 ft. give in Table A1.1-1 in the *Prescriptive Method*; therefore, diaphragm lengths over 30 ft. have only been included in Tables 2.3 and 2.4 for use in diaphragm calculations for reduced fastener spacing and blocking options. Refer to Appendix A for typical diaphragm length calculations for both construction options.

Table 2.7 considers all of the same design checks as Tables 2.1 and 2.2 including combined axial and bending, pure bending, deflection ($h/240$ criteria), and web crippling. Table 2.7 may be also be used for stud walls on lower floors of 2-story construction, as shown in Figure 2.9. Studs in these walls resist greater axial forces than those under the gable end wall assembly. Therefore, stud sections in Table 2.7 are also checked for the pure axial combinations (1) and (2), in addition to the checks above. The factored axial load resisted by these sections is calculated using a floor dead load of 10 psf, and a floor live load of 40 psf, as described previously.

2.2 Gable End Framing with Roof Trusses

2.2.1 Pre-Manufactured Gable End Wall Assembly

For residential projects utilizing roof trusses, the gable end may be framed in with a pre-manufacturer gable end wall assembly as shown in Figure 2.9. In this design option, the triangular wall portion above the sidewall stud height is provided by the roof truss manufacturer. This assembly is essentially a structural wall with a stud wall framed in under it as shown in Figure 2.9. The studs supporting this pre-manufactured assembly are 8 ft., 9 ft., or 10 ft. tall and resist both out-of-plane wind forces and axial loads from the gable end wall assembly. The stud thickness may be selected from Table 2.7 for the given design conditions.

To resist the out-of-plane wind forces, a horizontal restraint must be provided at the connection of the wall and the gable end wall assembly. The principle mechanism of

providing this restraint is to utilize a horizontal structural diaphragm at ceiling level as shown in Figure 2.10.

Similar to balloon gable framing, this diaphragm may be composed of either gypsum board or wood structural panels, and must be constructed over the minimum lengths given in Tables 2.3 – 2.6, as illustrated in Figure 2.5. Refer to Figure 2.7 for attachment at the longitudinal shear walls. The connection at the gable end wall is detailed in Figure 2.11. Refer to the balloon framing option in section 2.1 for a discussion of the design methodology and assumptions used to develop the minimum diaphragm lengths in Tables 2.3 – 2.6.

2.2.2 Gable Infill Stud Wall

This option is an alternative to using the pre-manufactured gable end assembly as described in Section 2.2.1 (Figure 2.10). Rather than utilizing a triangular gable end assembly that is provided by the truss manufacturer, the wall can be built as a stud wall, selecting the thickness of studs from Table 2.1. Like the pre-manufactured gable end wall assembly, this wall system must be braced horizontally at the ceiling line where the two walls are attached as shown in Figure 2.11. The lower wall supporting the triangular shaped portion of the gable end wall will be selected as described in Section 2.2.1.

2.2.3 Full Height Balloon Gable Framing

Another option for framing the gable end wall is to use balloon gable framing similar to that described in Section 2.1.1. The wall studs are full height studs without bracing at the ceiling. Studs for this option are selected from Table 2.1 and overhangs are constructed according to Figure 2.3.

2.3 Cathedral Ceiling

For buildings with cathedral, or “vaulted” ceilings, gable end framing consists of a full height stud wall without any out-of-plane support, as shown in Figure 2.12. These studs are considered simply supported between the top and bottom tracks, and are

designed as previously discussed. A typical section of this construction is shown in Figure 2.13. Figure 2.3 details proper overhang design at the top of the wall.

The proper stud size and thickness for a given stud height, h , can be selected from Tables 2.1 or 2.2. To comply with the building width and roof slope limitations set forth in Table A1.1-1 in the *Prescriptive Method*, these studs may have heights up to 30 ft. at the center of the wall. However, it is clear from Tables 2.1 and 2.2 that construction of walls over 20 ft. tall is not possible with 3.5 in. or 5.5 in. studs.

3.0 Openings in Gable End Walls

3.1 General

Many residential structures have openings for windows and/or doors in the gable end walls. Care must be given to ensure that members around openings are designed for the redistributed axial and wind forces. This involves design of both the king and jack studs on either side of the opening, and of the header tracks at the top (and bottom for windows) of the opening. A typical opening detail is shown in Figure 3.1.

3.2 Studs Around Openings

King and jack studs are used on both sides of wall openings to resist the loads transferred around the opening. In gable end walls, these loads include both axial dead and snow loads from the roof, and out-of-plane wind loads. These wind loads consist of both uniform loads from the exposed wall surface, and a point load at the header track location. King studs run the full height of the wall. Jack studs, on the other hand, provide support the header as shown in Figure 3.1. Per Table 3.1 (Table E7-24 in the current *Prescriptive Method*), the total number of king studs required on each side of the opening is equal to the number of studs eliminated in the opening divided by two (AISI, 2002), rounded up to the nearest stud. The number of jack studs depends on the opening width, as shown in Table 3.1. These studs should be the same thickness as the wall studs. For example, if an opening interrupts three wall studs, two king studs and one jack stud should be placed on each side of the opening to compensate.

3.3 Headers

Due to the limited axial load on gable end walls, use of the bearing wall header tables in the current *Prescriptive Method* results in overly conservative designs. Consequently, 2-600S162-43 sections shall be used for less than 6 ft. wide in gable end walls. Openings over 6 ft. require 2-800S162-54 sections. These sections may be arranged in either back to back or box configurations, as shown in Figure 3.1

Though the axial loads on gable end walls are minimal, the out-of-plane loads due to wind are quite significant. Consequently, the head tracks over the opening must be designed to resist these forces. The track is designed to resist a tributary width equal to $h/2$, where h is the height of the wall at the center of the opening. Tables 3.2 - 3.6 give the maximum allowable head track spans, L , for various track sections, wind speeds, and building heights. Stud heights shall be rounded up to the nearest given height to determine the appropriate track spans. These spans are based on an unbraced length of 24 in., corresponding with a stud spacing of 24 in. on center. To maintain consistency with the current head track span table in the *Prescriptive Method* (Table E7-25), only 33 ksi material and 33, 43, and 54 mil thicknesses are considered (AISI, 2002). Spans given in tables consider both pure bending combination (6), and deflection combination (7). A sample calculation for these tables is included in Appendix A.

4.0 Bottom Wall Track Connection

4.1 Connection to Floor Joist or Track

For gable end walls in two story buildings, it is likely that the wall bottom track may be connected to either a floor joist or another track through the floor sheathing. This connection is generally made with screws evenly spaced along the wall. Design of this connection includes consideration of both the shear strength of the screw(s), and bearing strengths of the metal sections and floor sheathing. The minimum of these capacities determines the maximum connection shear force. This shear capacity is checked against the out-of-plane wind force of load combination (6). Tension capacities for both screw pullout and pullover in either the metal or floor sheathing are checked for the wind uplift condition in load combination (5). The resistance to the combined effect of shear and tension are also checked using an interaction equation. The required number and spacing of No. 8 screws for different wind speeds and wall heights, h , is shown in Table 4.1. Refer to Appendix A for a sample calculation of this connection.

4.2 Connection to Concrete Foundation

Gable end walls may also bear directly on a concrete foundation. Connection of the bottom track to the foundation is made with headed steel anchor bolts evenly spaced along the length of the wall. These bolts must be embedded at least 7 inches in the concrete, and should be connected to the track as indicated in Figures 4.1 and 4.2 (Figures E2-1 and E2-2 from *Prescriptive Method*). Figure 4.1 applies for connections away from stud locations, while Figure 4.2 may be used if the bolt is located adjacent to a wall stud.

Tensile capacity of this connection is governed by either the headed anchor bolt strength or by the breakout or pullout capacity of the concrete. This resistance is checked against the wind uplift in combination (5). The shear strength of this connection is based on either the breakout of the concrete, spacing/edge distance of the bolts,

bearing strength of the track material, or shear strength of the bolt. Load combination (6) is used to verify the connection adequacy for out-of-plane shear forces due to wind force. Finally, the combined effect of tension and shear on the bolts and concrete are considered using an interaction equation. The required bolt diameters and spacing for given design conditions are found in Table 4.2. Appendix A includes design calculations for this table.

4.3 Connection to Wood Sill

Finally, the bottom track connection may include a wood sill placed between the concrete foundation and the track. Proper construction for this case is detailed in Figure 4.3 (Figure E2-3 in the 2002 *Prescriptive Method*).

Calculations for this connection are similar for those for connection directly to the concrete slab. Once again, both tensile and shear capacities are developed based on the bolt embedment depth/concrete strength, the steel track, and the bolt itself. In addition, the bearing strength of the wood sill loaded perpendicular to the grain must be considered for overall shear capacity. For calculations in this report, a 2x Douglas Fir Larch sill is assumed. The connections in Table 4.2 consider the capacity of a wood sill, as indicated in the sample calculation; consequently, Table 4.2 may be used for tracks bearing directly on a foundation or on a wood sill.

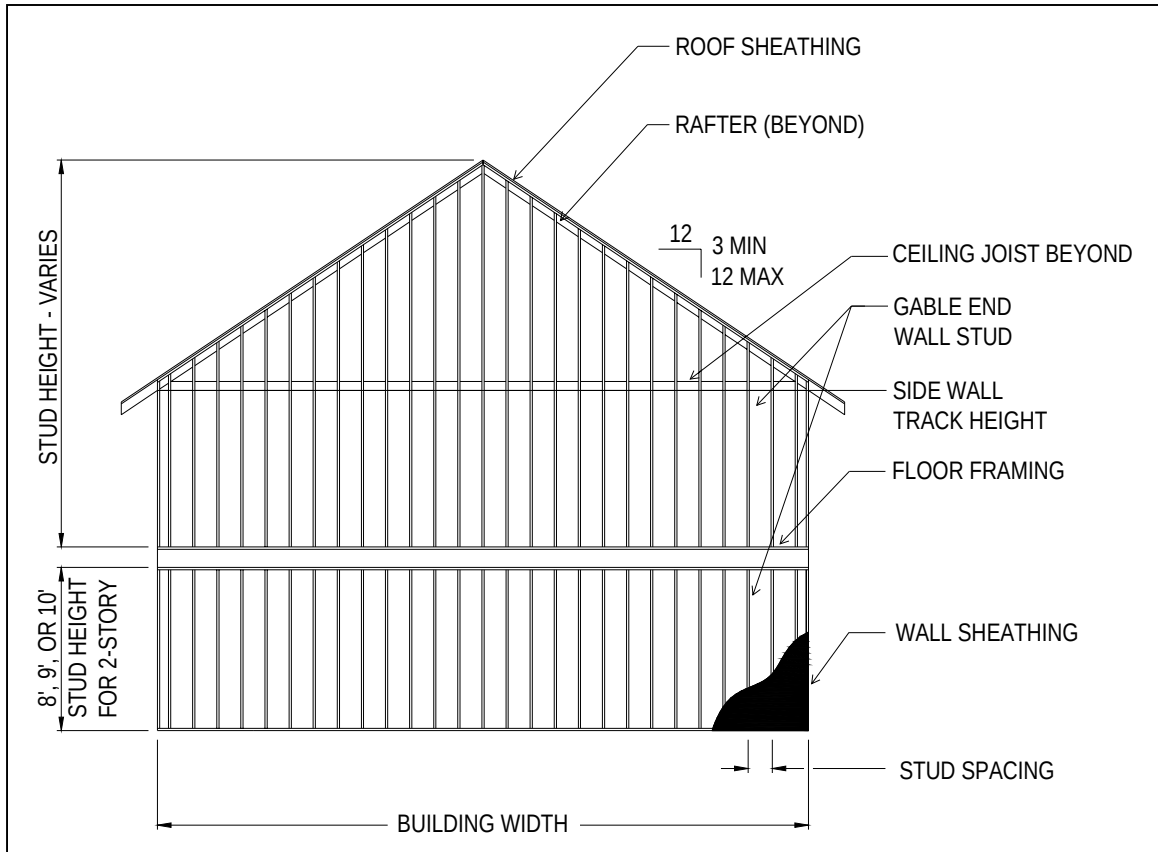


Figure 2.1 - Gable End Wall Elevation for Balloon Gable Framing Option

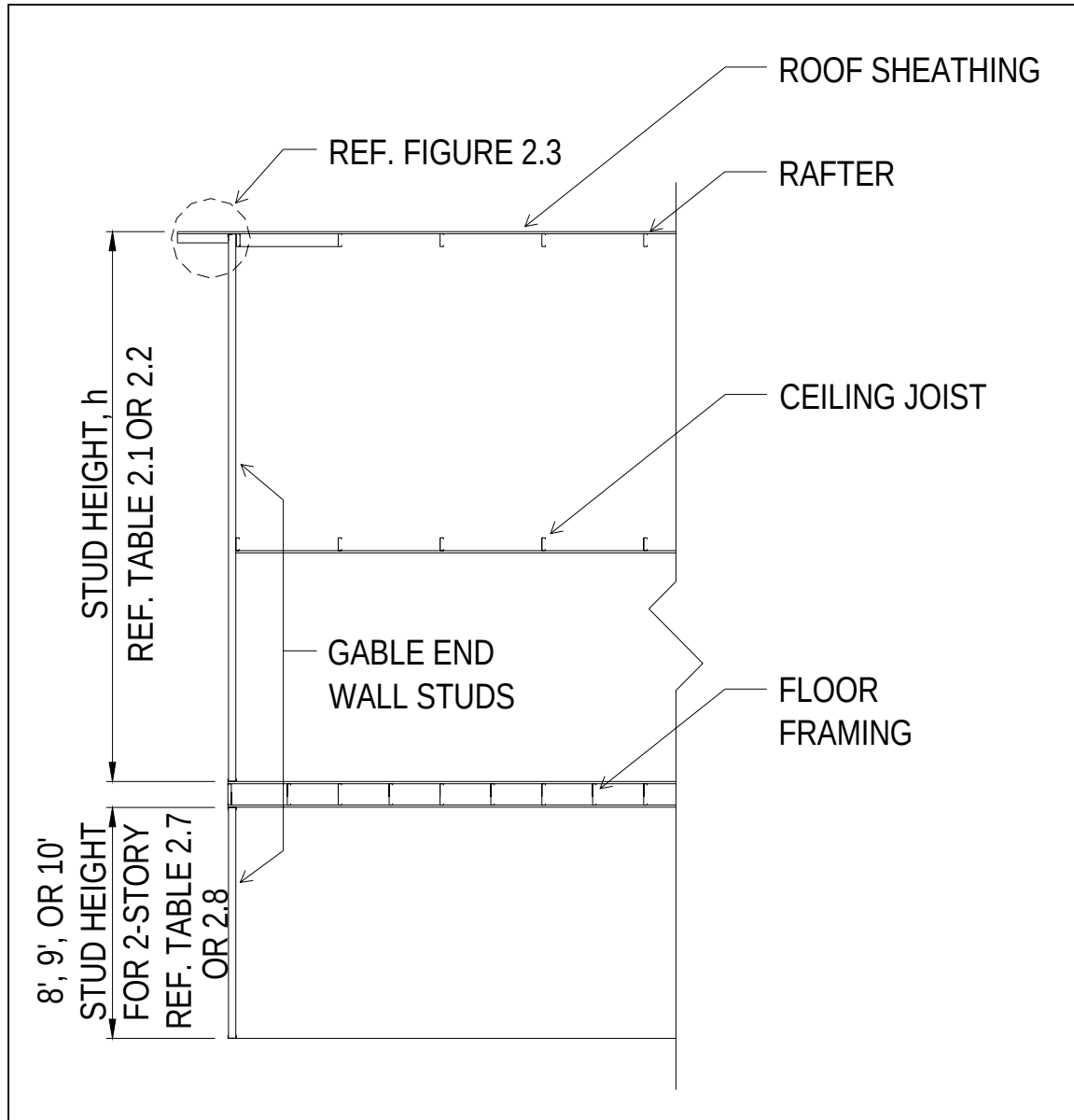


Figure 2.2 - Wall Section for Balloon Gable Framing – Unbraced Option

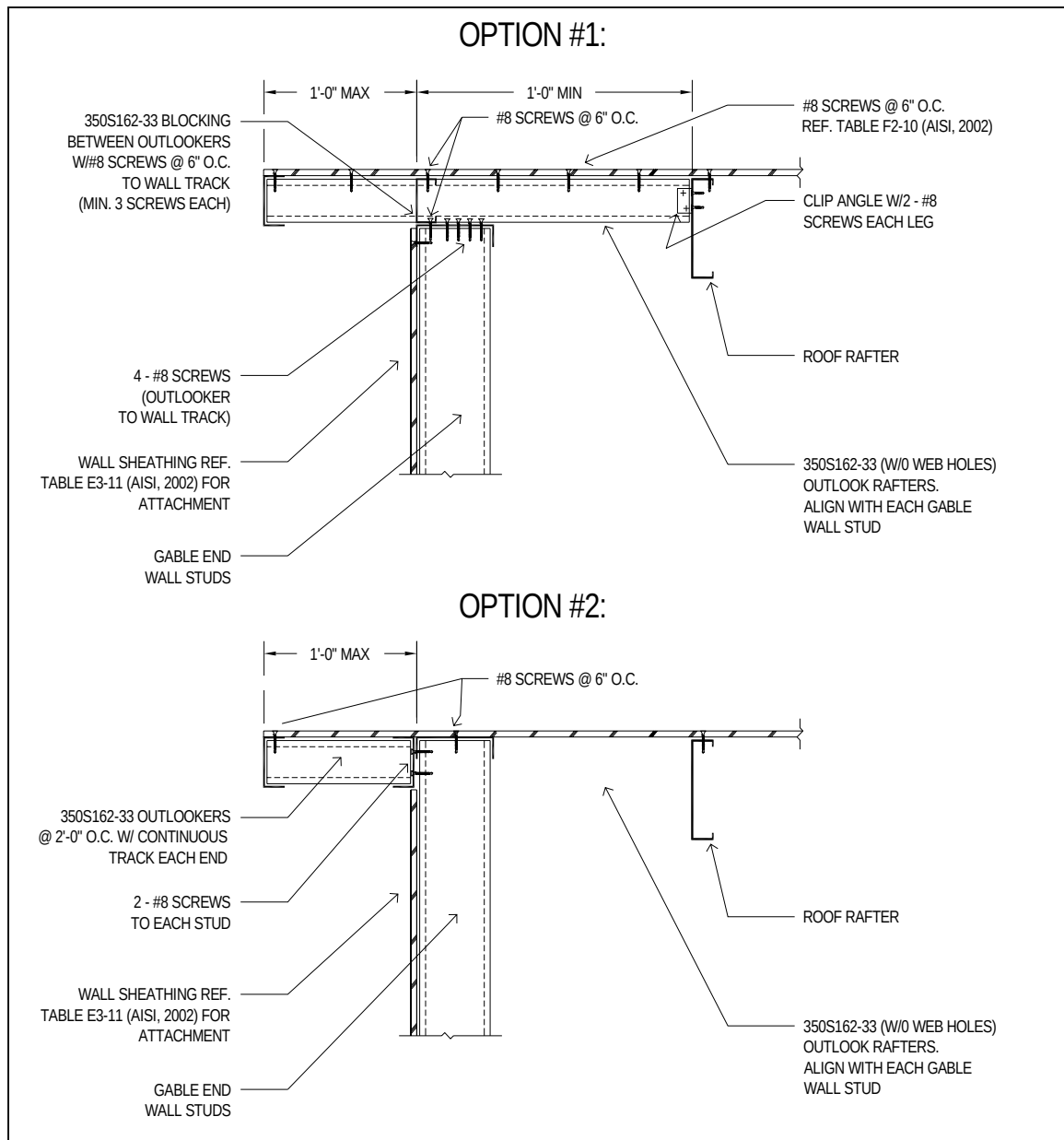


Figure 2.3 - Gable End Overhang Details

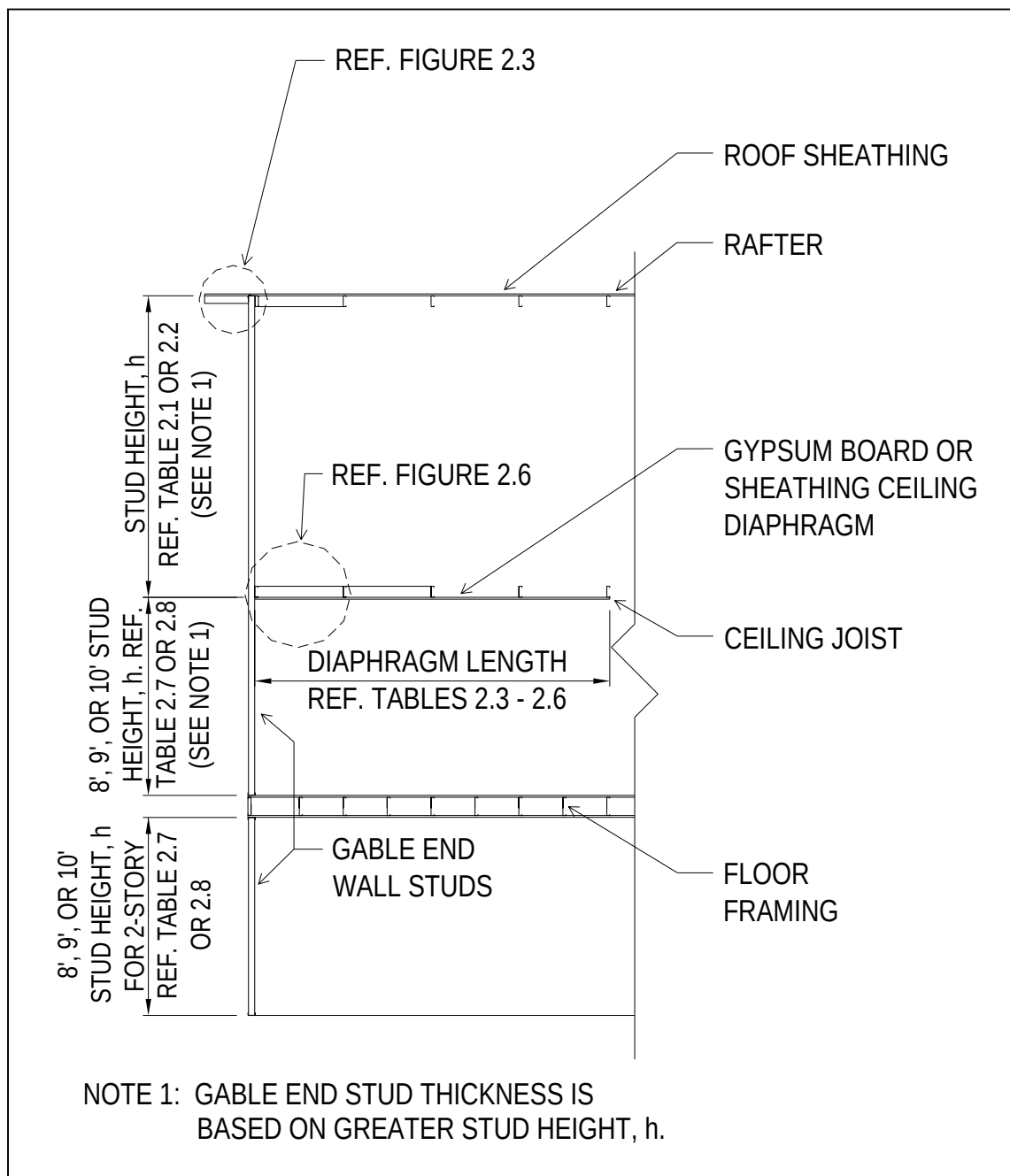


Figure 2.4 - Wall Section for Balloon Gable Framing – Ceiling Diaphragm Option

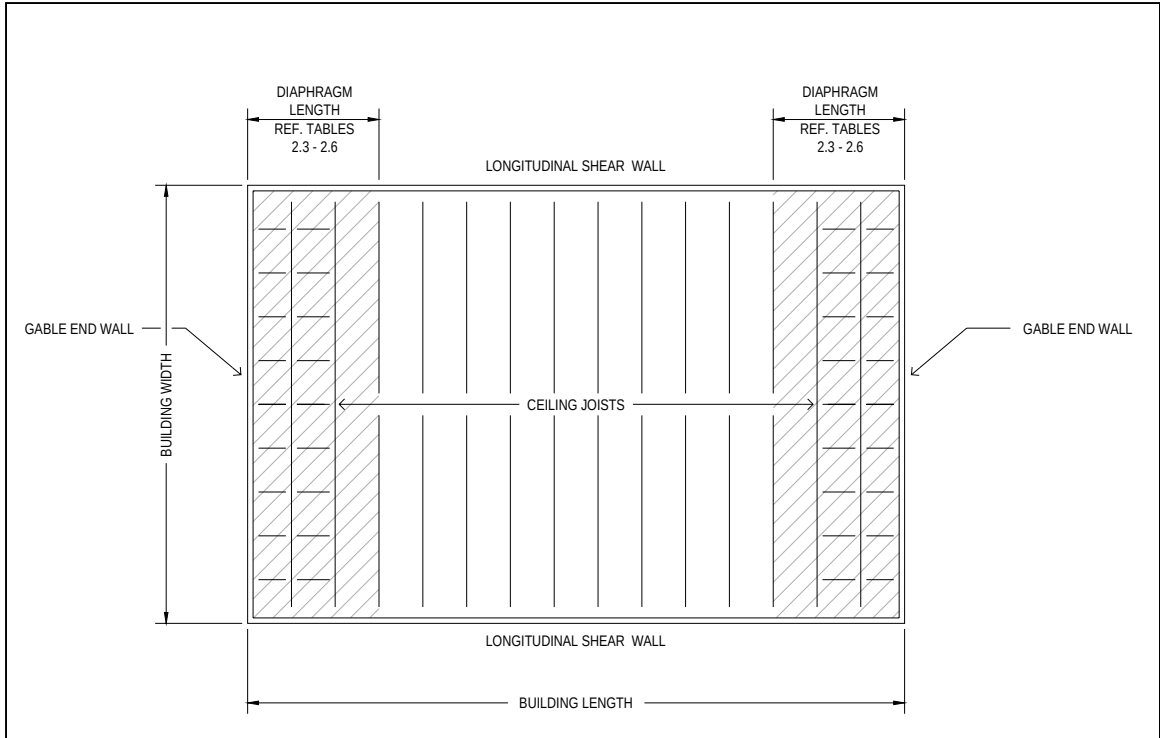


Figure 2.5 - Ceiling Diaphragm Plan for Balloon Gable Framing / Truss Framing

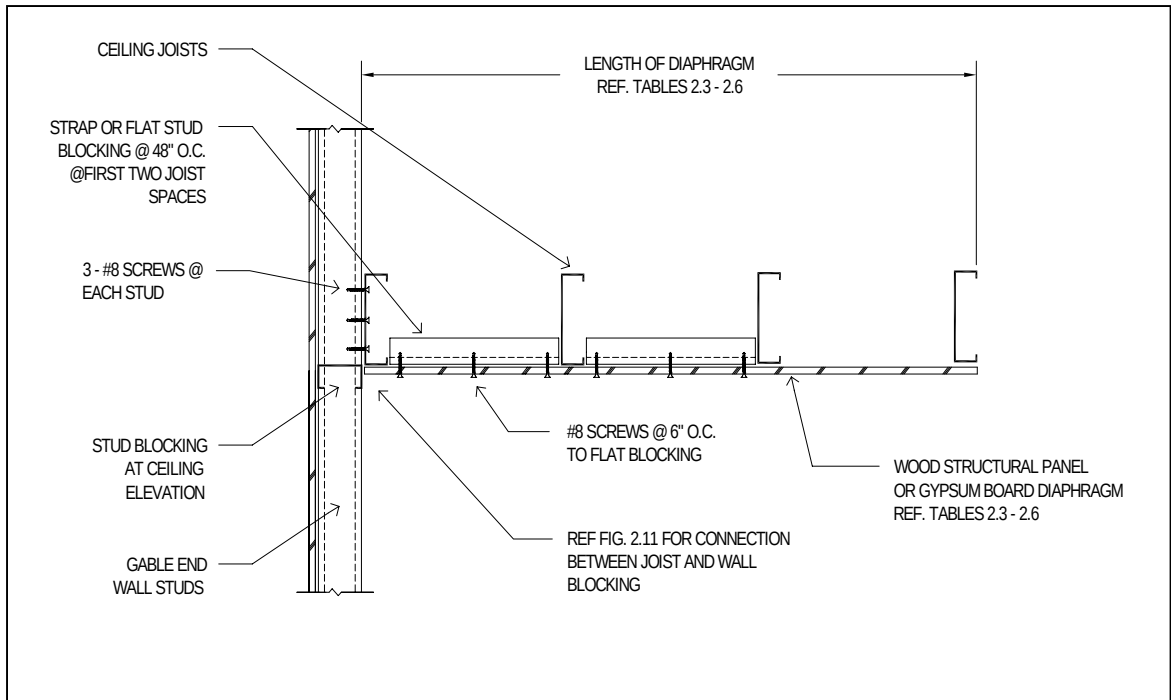


Figure 2.6 - Ceiling Diaphragm to Gable End Wall for Balloon Gable Framing

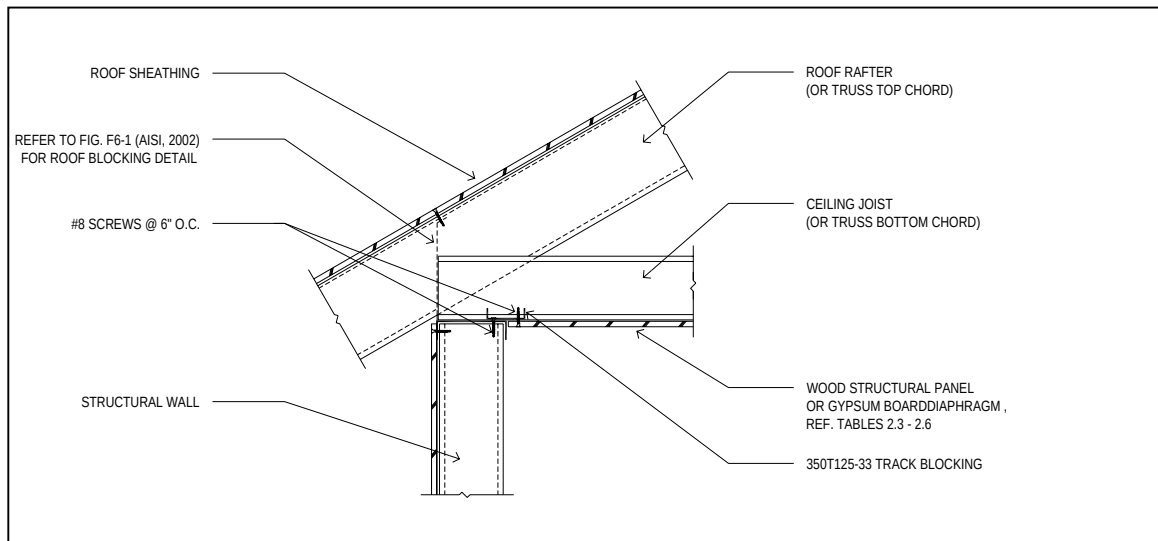


Figure 2.7 - Ceiling Diaphragm to Structural Wall for Balloon Gable Framing / Truss Framing

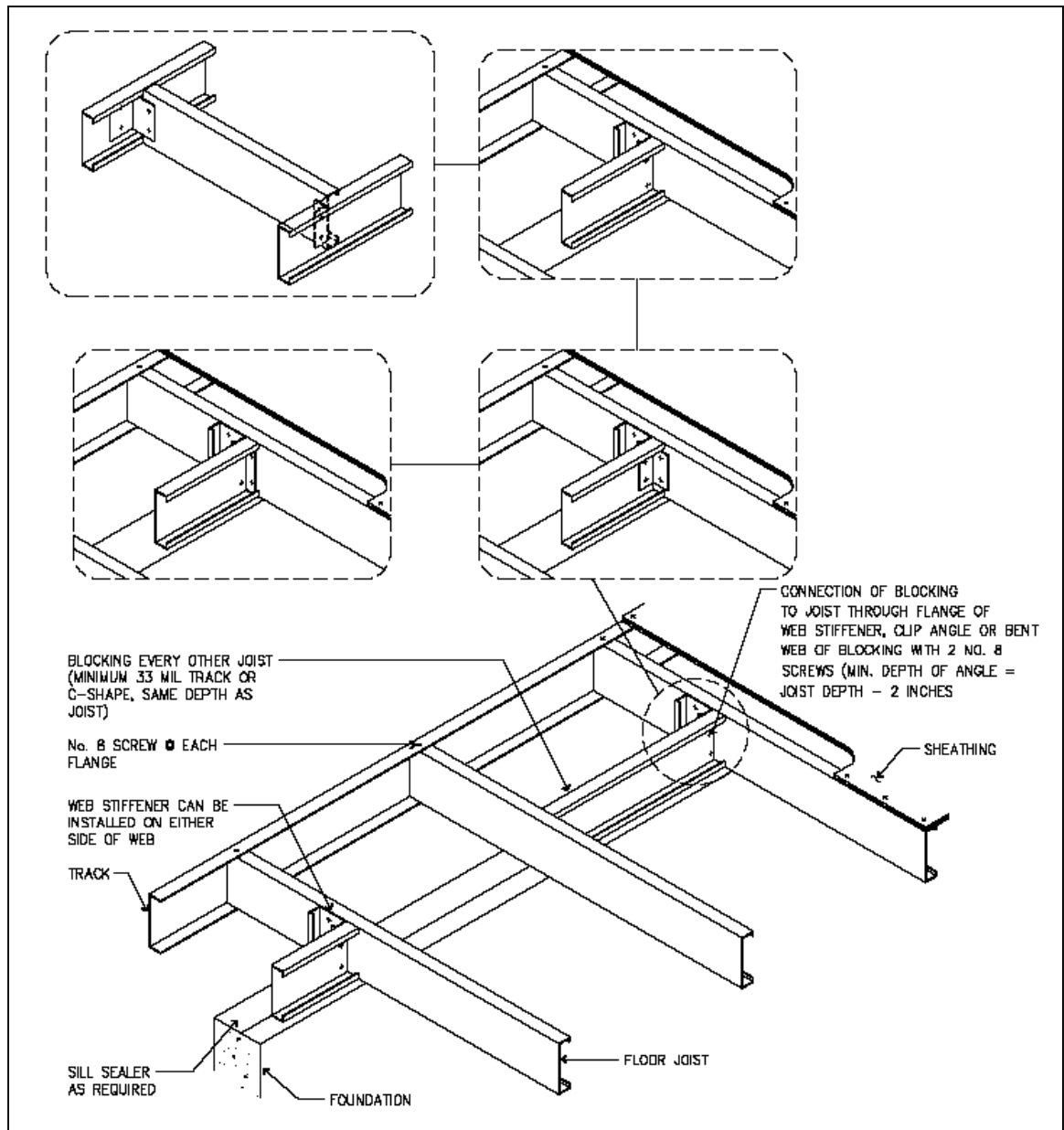


Figure 2.8 – Blocking Connections (AISI, 2002)

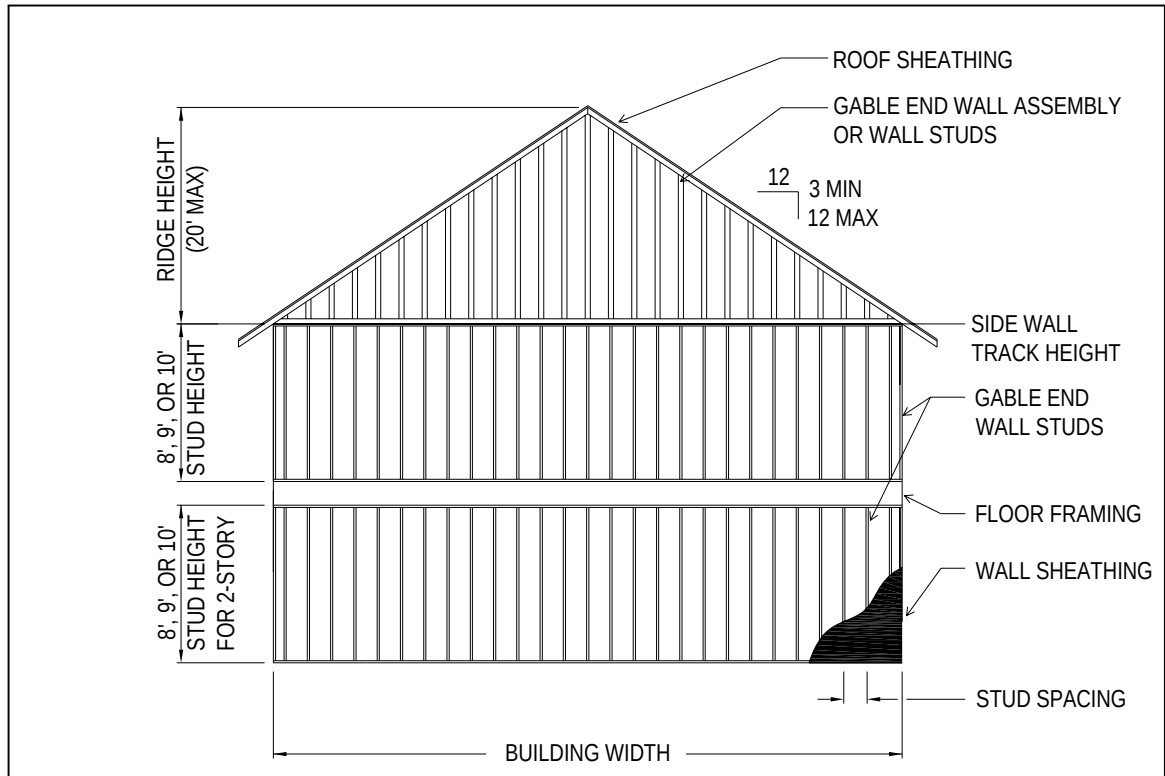


Figure 2.9 - Gable End Wall Elevation for Gable End Wall Assembly Option

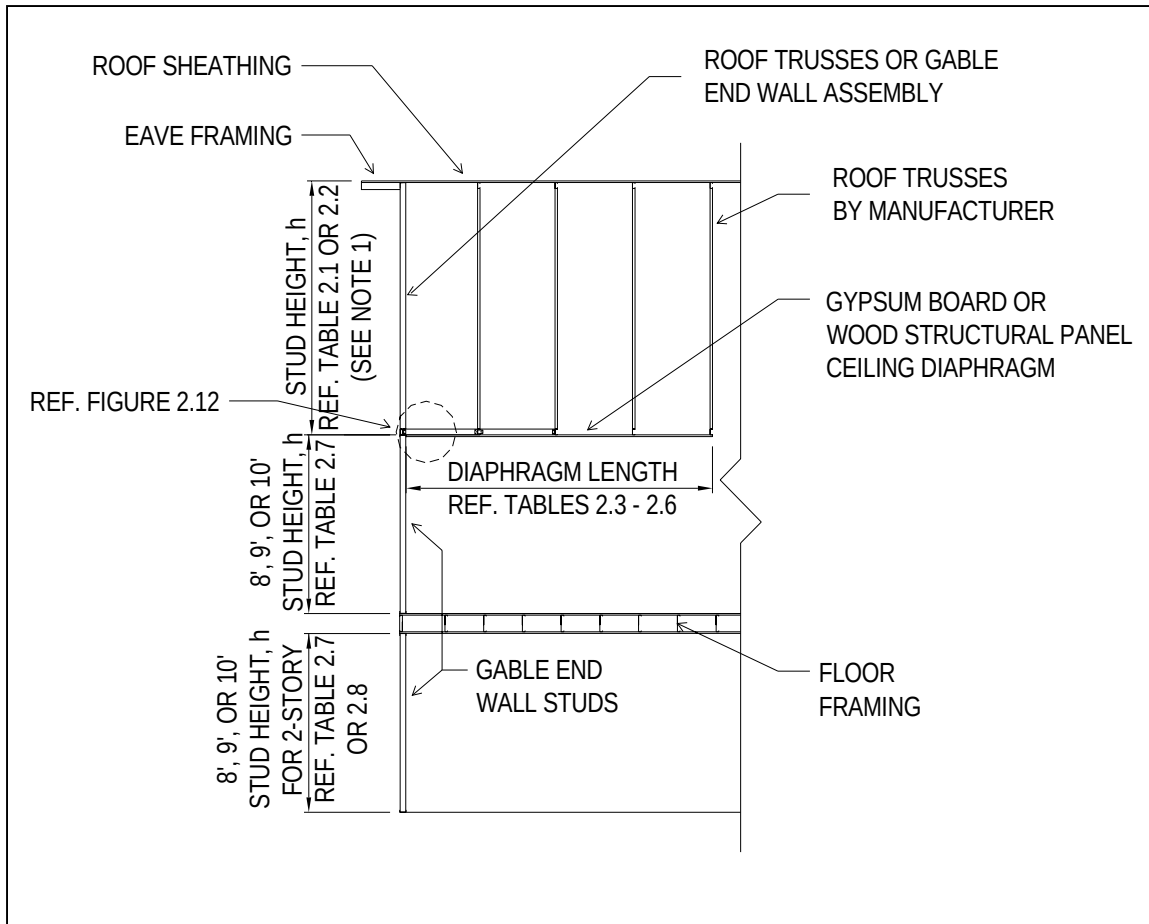


Figure 2.10 – Wall Section for Gable End Wall Assembly

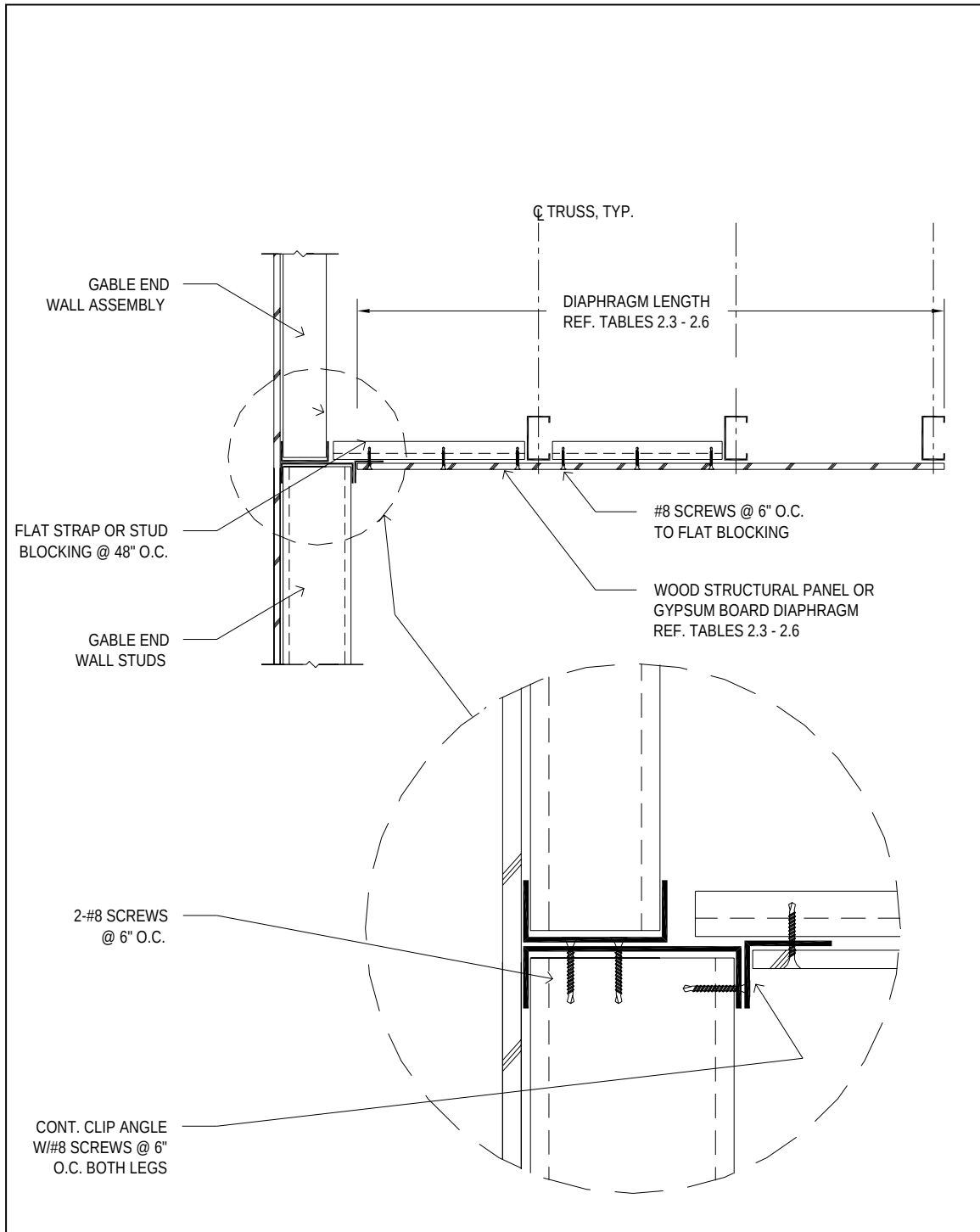


Figure 2.11 - Ceiling Diaphragm to Gable End Wall for Roof Trusses

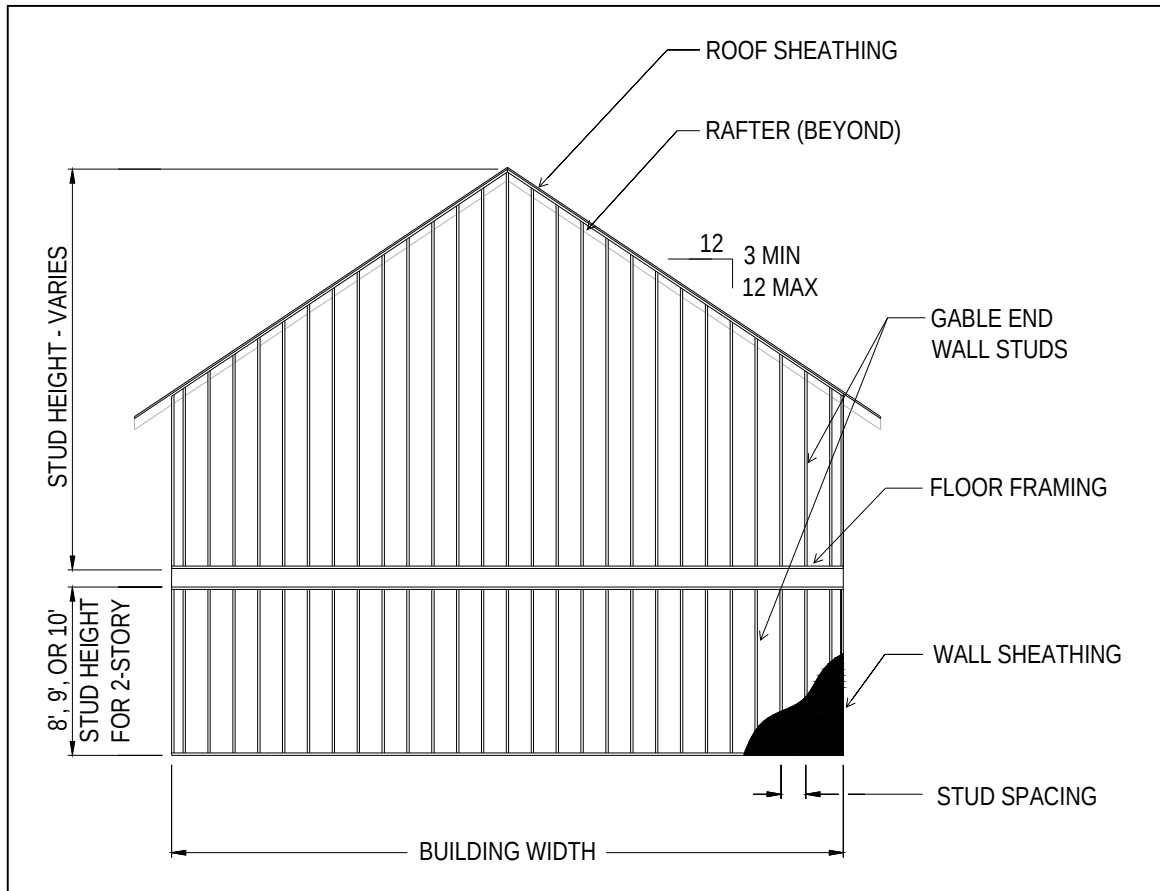


Figure 2.12 - Gable End Wall Elevation for Cathedral Ceiling Option

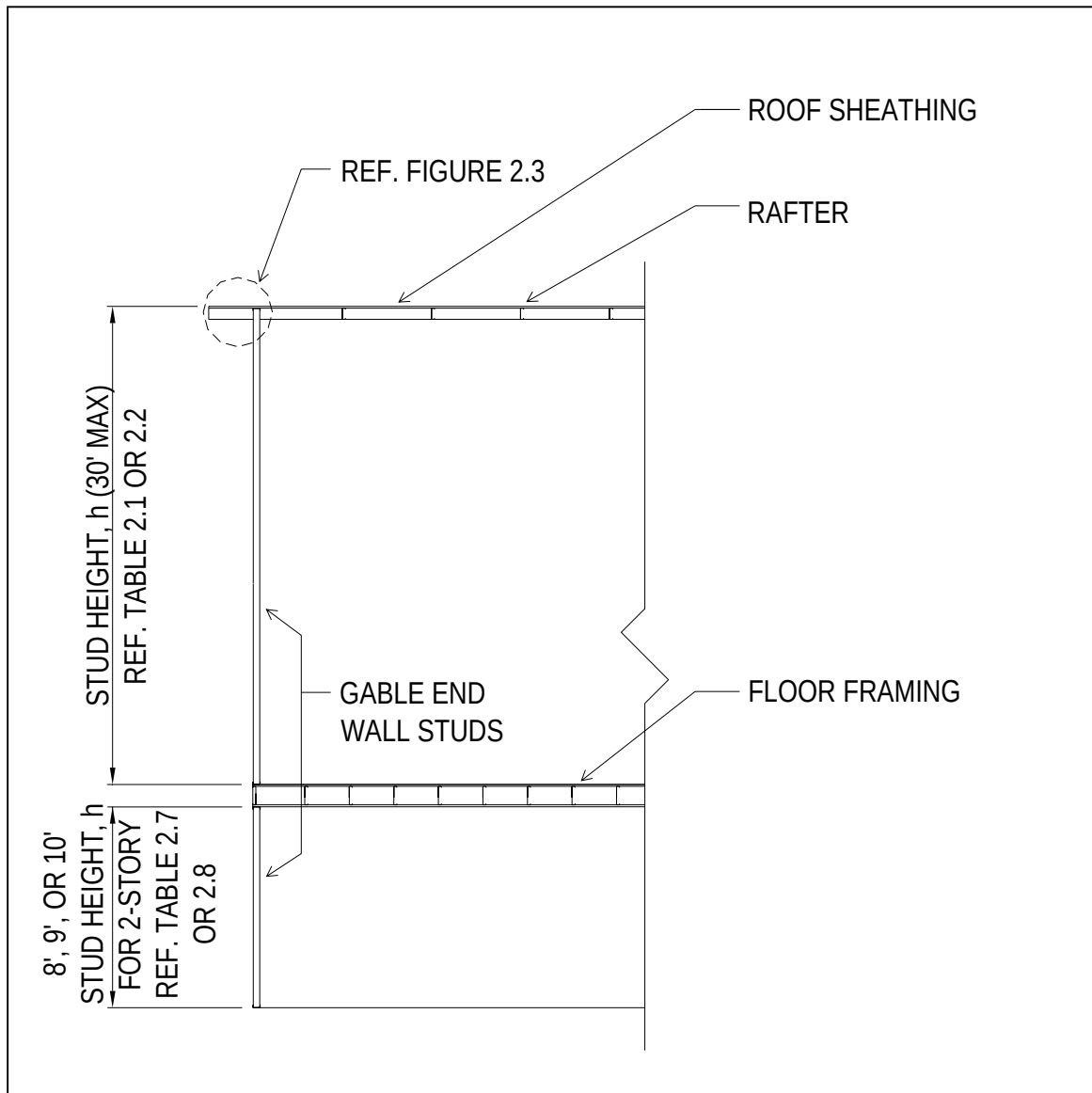


Figure 2.13 - Gable End Wall Section for Cathedral Ceiling Option

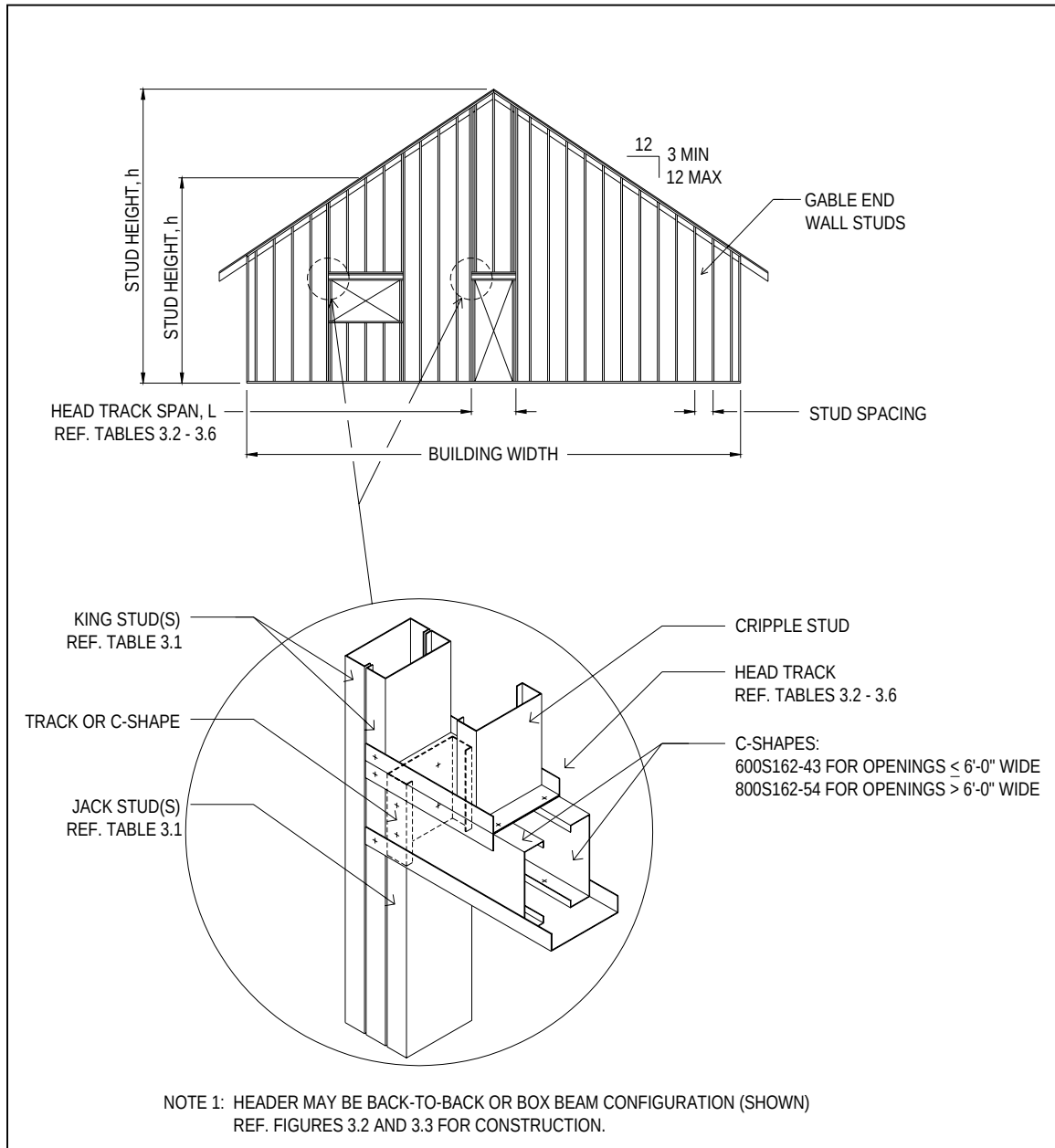


Figure 3.1 - Openings in Gable End Walls

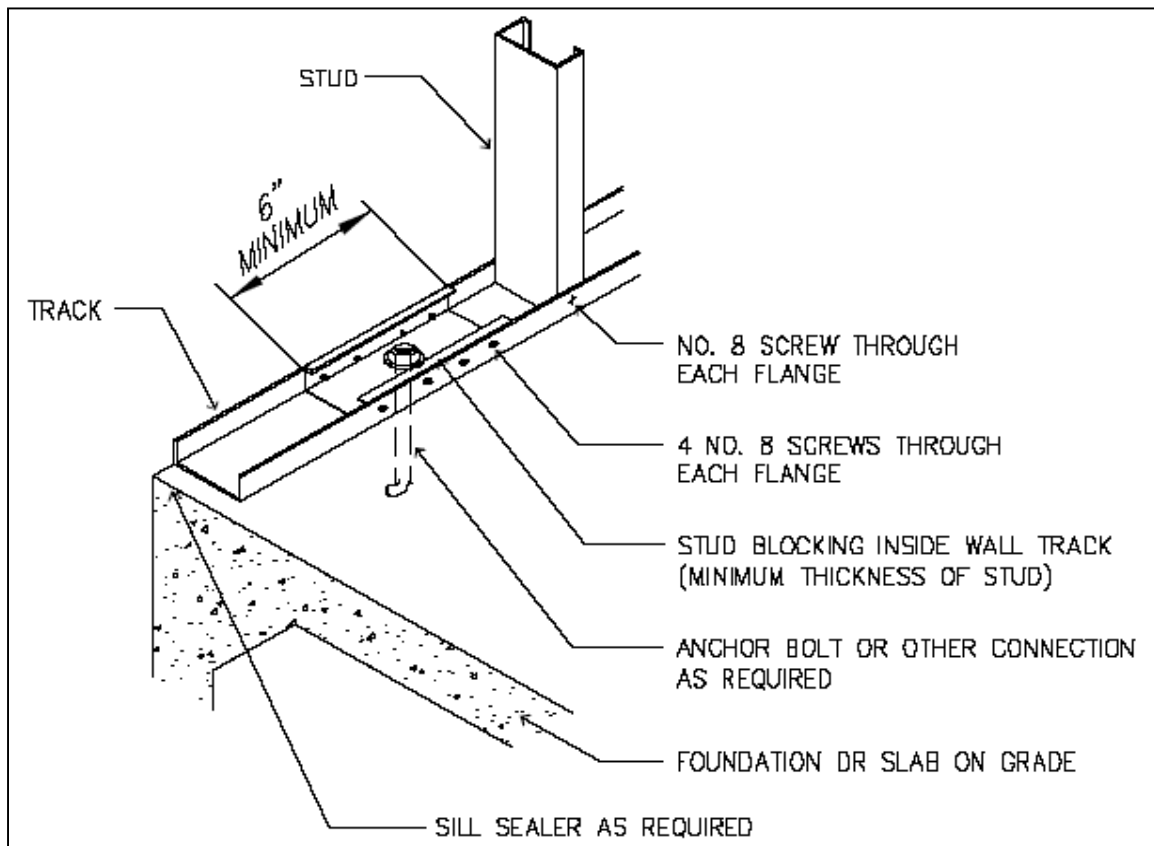


Figure 4.1 - Wall Bottom Track to Foundation – Away From Stud (AISI, 2002)

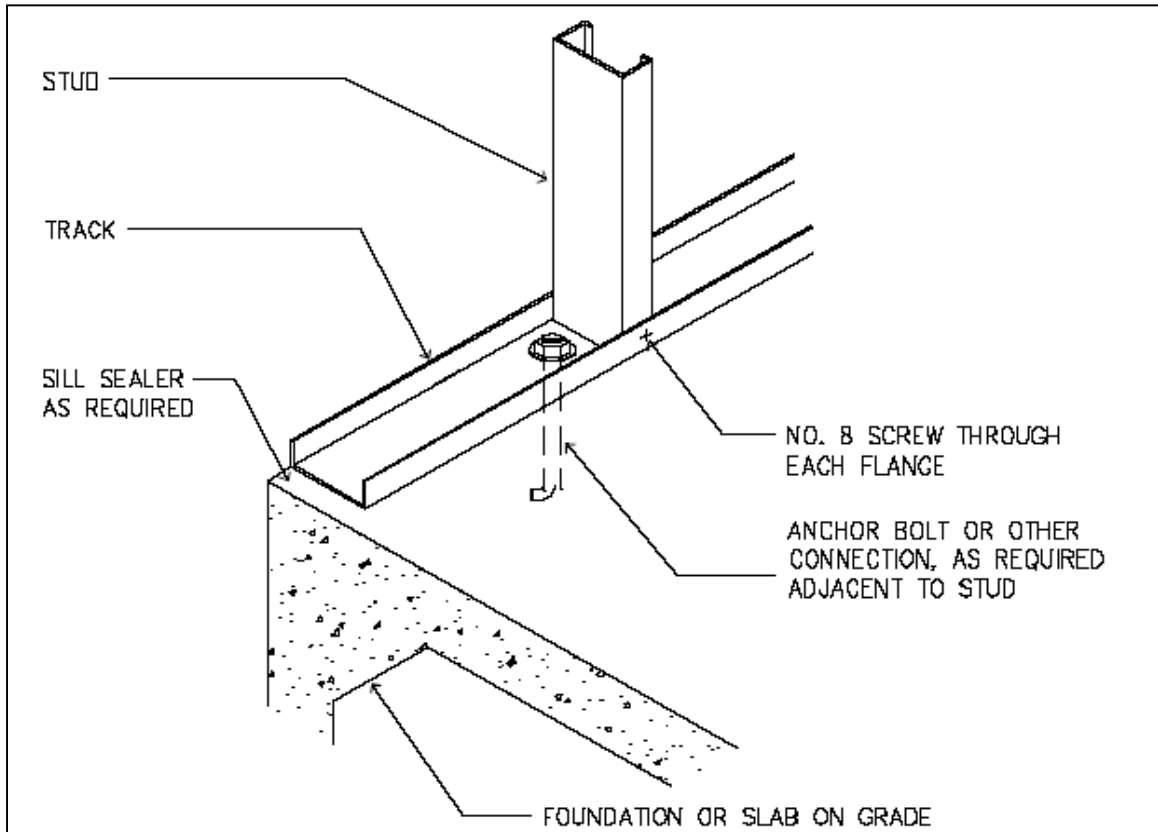


Figure 4.2 - Wall Bottom Track to Foundation – Adjacent to Stud (AISI, 2002)

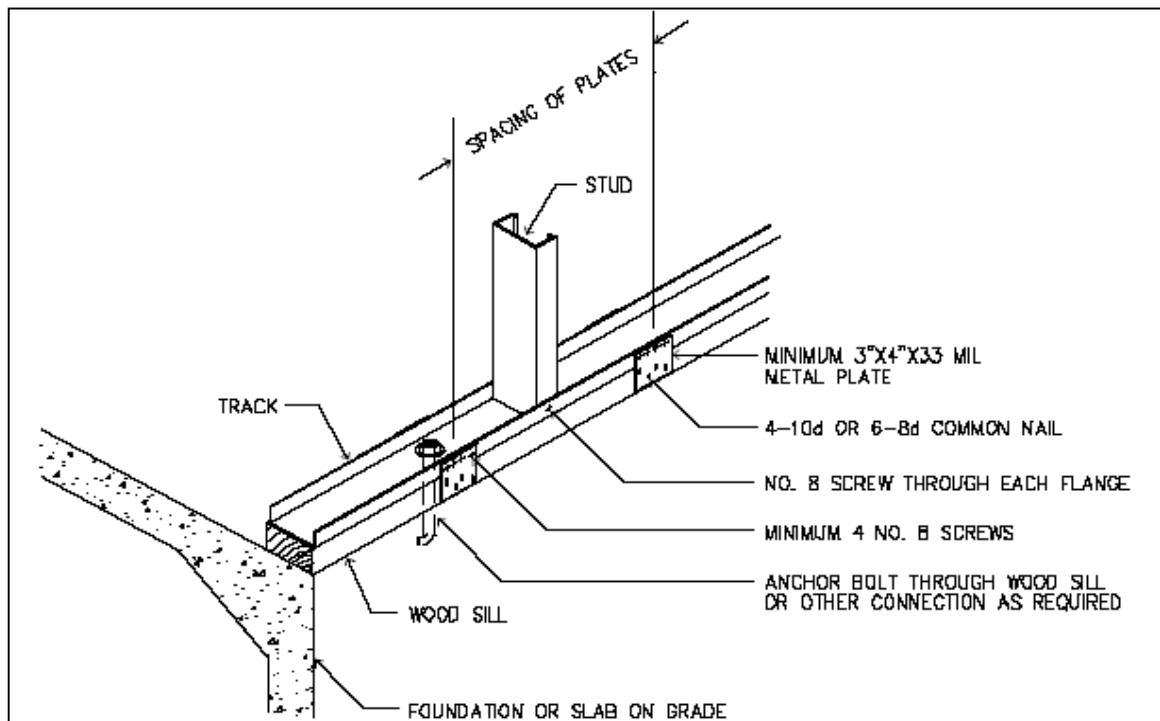


Figure 4.3 - Wall Bottom Track to Wood Sill (AISI, 2002)

33
KSI

Table 2.1
Stud Thickness
All Building Widths Supporting Roof Only – Gable End Walls of Variable
Height^{1,2,3}
F_y = 33 ksi

Wind Speed		Member Size	Stud Spacing (inches)	Minimum Stud Thickness (mils)				
Exp. A/B	Exp. C			Stud Height, h (feet)				
				10 < h ≤ 12	12 < h ≤ 14	14 < h ≤ 16	16 < h ≤ 18	18 < h ≤ 20
85 mph		350S162	16	33	43	54	97	-
			24	43	54	97	-	-
		550S162	16	33	33	33	33	43
			24	33	33	43	43	54
90 mph		350S162	16	33	43	68	97	-
			24	43	68	97	-	-
		550S162	16	33	33	33	43	43
			24	33	33	43	54	68
100 mph	85 mph	350S162	16	43	54	97	-	-
			24	54	97	-	-	-
		550S162	16	33	33	43	43	54
			24	33	43	54	68	97
110 mph	90 mph	350S162	16	43	68	-	-	-
			24	68	-	-	-	-
		550S162	16	33	33	43	54	68
			24	43	43	68	97	97
120 mph	100 mph	350S162	16	54	97	-	-	-
			24	97	-	-	-	-
		550S162	16	33	43	54	68	97
			24	43	54	68	97	-
130 mph	110 mph	350S162	16	68	97	-	-	-
			24	97	-	-	-	-
		550S162	16	43	43	54	68	97
			24	54	68	97	-	-
	120 mph	350S162	16	68	-	-	-	-
			24	-	-	-	-	-
		550S162	16	43	54	68	97	-
			24	54	97	97	-	-
	130 mph	350S162	16	97	-	-	-	-
			24	-	-	-	-	-
		550S162	16	43	68	97	97	-
			24	68	97	-	-	-

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Deflection criteria: h/240

² Design load assumptions:

Roof and ceiling dead load is 12 psf (0.58 kN/m²)

Ground snow load is 70 psf (3.35 kN/m²)

³ Building width is in the direction of horizontal framing members supported by the wall *studs*.

33
KSI

Table 2.1 (Continued)
Stud Thickness
All Building Widths Supporting Roof Only – Gable End Walls of Variable Height^{1,2,3}
F_y = 33 ksi

Wind Speed		Member Size	Stud Spacing (inches)	Minimum Stud Thickness (mils)				
Exp. A/B	Exp. C			Stud Height, h (feet)				
				20 < h ≤ 22	22 < h ≤ 24	24 < h ≤ 26	26 < h ≤ 28	28 < h ≤ 30
85 mph		350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	54	68	97	-	-
			24	97	97	-	-	-
90 mph		350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	54	97	97	-	-
			24	97	-	-	-	-
100 mph	85 mph	350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	68	97	-	-	-
			24	97	-	-	-	-
110 mph	90 mph	350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	97	-	-	-	-
			24	-	-	-	-	-
120 mph	100 mph	350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	-	-	-	-	-
			24	-	-	-	-	-
130 mph	110 mph	350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	-	-	-	-	-
			24	-	-	-	-	-
	120 mph	350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	-	-	-	-	-
			24	-	-	-	-	-
	130 mph	350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	-	-	-	-	-
			24	-	-	-	-	-

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Deflection criteria: h/240

² Design load assumptions:

Roof and ceiling dead load is 12 psf (0.58 kN/m²)

Ground snow load is 70 psf (3.35 kN/m²)

³ Building width is in the direction of horizontal framing members supported by the wall *studs*.

50
KSI

Table 2.2
Stud Thickness
All Building Widths Supporting Roof Only - Gable End Walls of Variable
Height^{1,2,3}
 $F_y = 50$ ksi

Wind Speed		Member Size	Stud Spacing (inches)	Minimum Stud Thickness (mils)				
Exp. A/B	Exp. C			Stud Height, h (feet)				
				10 < h ≤ 12	12 < h ≤ 14	14 < h ≤ 16	16 < h ≤ 18	18 < h ≤ 20
85 mph		350S162	16	33	43	54	97	-
			24	33	54	97	-	-
		550S162	16	33	33	33	33	43
			24	33	33	33	43	54
90 mph		350S162	16	33	43	68	97	-
			24	43	68	97	-	-
		550S162	16	33	33	33	33	43
			24	33	33	33	43	68
100 mph	85 mph	350S162	16	33	54	97	-	-
			24	54	97	-	-	-
		550S162	16	33	33	33	43	54
			24	33	33	43	54	97
110 mph	90 mph	350S162	16	43	68	-	-	-
			24	68	-	-	-	-
		550S162	16	33	33	33	43	68
			24	33	43	54	68	97
120 mph	100 mph	350S162	16	54	97	-	-	-
			24	97	-	-	-	-
		550S162	16	33	33	43	54	97
			24	33	43	54	97	-
130 mph	110 mph	350S162	16	54	97	-	-	-
			24	97	-	-	-	-
		550S162	16	33	43	54	68	97
			24	43	54	68	97	-
	120 mph	350S162	16	68	-	-	-	-
			24	-	-	-	-	-
		550S162	16	33	43	54	97	-
			24	54	54	97	-	-
	130 mph	350S162	16	97	-	-	-	-
			24	-	-	-	-	-
		550S162	16	43	54	68	97	-
			24	54	68	97	-	-

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Deflection criteria: h/240

² Design load assumptions:

Roof and ceiling dead load is 12 psf (0.58 kN/m²)

Ground snow load is 70 psf (3.35 kN/m²)

³ Building width is in the direction of horizontal framing members supported by the wall studs.

50
KSI

Table 2.2 (Continued)
Stud Thickness
All Building Widths Supporting Roof Only - Gable End Walls of Variable
Height^{1,2,3}
F_y = 50 ksi

Wind Speed		Member Size	Stud Spacing (inches)	Minimum Stud Thickness (mils)				
Exp. A/B	Exp. C			Stud Height, h (feet)				
				20 < h ≤ 22	22 < h ≤ 24	24 < h ≤ 26	26 < h ≤ 28	28 < h ≤ 30
85 mph		350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	54	68	97	-	-
			24	97	97	-	-	-
90 mph		350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	54	97	97	-	-
			24	97	-	-	-	-
100 mph	85 mph	350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	68	97	-	-	-
			24	97	-	-	-	-
110 mph	90 mph	350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	97	-	-	-	-
			24	-	-	-	-	-
120 mph	100 mph	350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	-	-	-	-	-
			24	-	-	-	-	-
130 mph	110 mph	350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	-	-	-	-	-
			24	-	-	-	-	-
	120 mph	350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	-	-	-	-	-
			24	-	-	-	-	-
	130 mph	350S162	16	-	-	-	-	-
			24	-	-	-	-	-
		550S162	16	-	-	-	-	-
			24	-	-	-	-	-

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Deflection criteria: h/240

² Design load assumptions:

Roof and ceiling dead load is 12 psf (0.58 kN/m²)

Ground snow load is 70 psf (3.35 kN/m²)

³ Building width is in the direction of horizontal framing members supported by the wall studs

Table 2.3
Required Lengths for Gypsum Wallboard Sheathed Ceiling Diaphragms –
Ceiling Height = 8 ft^{1,2,3,4,5,6}

		Basic Wind Speed (mph)							
Exposure A/B		85	90	100	110	120	130		
Exposure C				85	90	100	110	120	130
Roof Pitch	Building Endwall Width (ft)	Minimum Diaphragm Length (ft)							
3:12 to 6:12	24 - 28	14	16	20	24	28	32	38	44
	28 - 32	16	18	24	28	32	38	44	52
	32 - 36	20	22	26	32	38	44	52	60
	36 - 40	22	24	30	36	44	50	60	70
6:12 to 9:12	24 - 28	16	18	22	26	32	36	44	50
	28 - 32	20	22	26	32	38	44	52	60
	32 - 36	22	26	32	38	44	52	62	72
	36 - 40	26	30	36	44	52	60	72	84
9:12 to 12:12	24 - 28	18	20	26	30	36	42	48	56
	28 - 32	22	24	30	36	42	50	60	68
	32 - 36	26	30	36	42	50	60	70	82
	36 - 40	30	34	42	50	60	70	84	96

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Ceiling diaphragm composed of 1/2" gypsum wall board (min. thickness) secured with screws spaced at 6 in o.c. at panel edges and 12 o.c. in field. Use No. 8 screws (min.) when framing members have a designation thickness of 54 mils or less and No. 10 screws (min.) when framing members have a designation thickness greater than 54 mils.

² Maximum aspect ratio (length/width) of diaphragms is 2:1.

³ Building width is in the direction of horizontal framing members supported by the wall *studs*.

⁴ Required diaphragm lengths are to be provided at each end of the structure (refer to Figure 2.5).

⁵ Required diaphragm lengths may be multiplied by 0.35 if all panel edges are blocked.

⁶ Required diaphragm lengths may be multiplied by 0.9 if all panel edges secured with screws spaced at 4 in o.c.

Table 2.4
Required Lengths for Gypsum Wallboard Sheathed Ceiling Diaphragms –
Ceiling Height = 9 or 10 ft^{1,2,3,4,5,6}

		Basic Wind Speed (mph)							
Exposure A/B		85	90	100	110	120	130		
Exposure C				85	90	100	110	120	130
Roof Pitch	Building Endwall Width (ft)	Minimum Diaphragm Length (ft)							
3:12 to 6:12	24 - 28	16	18	22	26	32	38	44	52
	28 - 32	20	22	26	32	38	44	52	60
	32 - 36	22	24	30	36	44	50	60	70
	36 - 40	26	28	36	42	50	58	70	80
6:12 to 9:12	24 - 28	18	20	26	30	36	42	50	58
	28 - 32	22	24	30	36	42	50	60	68
	32 - 36	26	28	36	42	50	58	70	80
	36 - 40	30	34	42	48	58	68	80	94
9:12 to 12:12	24 - 28	20	22	28	34	40	46	56	64
	28 - 32	24	28	34	40	48	56	66	78
	32 - 36	28	32	40	48	56	66	78	92
	36 - 40	34	38	46	56	66	78	92	106

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Ceiling diaphragm composed of 1/2" gypsum wall board (min. thickness) secured with screws spaced at 6 in o.c. at panel edges and 12 o.c. in field. Use No. 8 screws (min.) when framing members have a designation thickness of 54 mils or less and No. 10 screws (min.) when framing members have a designation thickness greater than 54 mils.

² Maximum aspect ratio (length/width) of diaphragms is 2:1.

³ Building width is in the direction of horizontal framing members supported by the wall *studs*.

⁴ Required diaphragm lengths are to be provided at each end of the structure (refer to Figure 2.5).

⁵ Required diaphragm lengths may be multiplied by 0.35 if all panel edges are blocked.

⁶ Required diaphragm lengths may be multiplied by 0.9 if all panel edges secured with screws spaced at 4 in o.c.

Table 2.5
Required Lengths for Wood Structural Panel Sheathed Ceiling Diaphragms –
Ceiling Height = 8 ft^{1,2,3,4,5}

		Basic Wind Speed (mph)							
Exposure A/B		85	90	100	110	120	130		
Exposure C				85	90	100	110	120	130
Roof Pitch	Building Endwall Width (ft)	Minimum Diaphragm Length (ft)							
3:12 to 6:12	24 - 28	10	10	10	10	10	10	10	10
	28 - 32	12	12	12	12	12	12	12	12
	32 - 36	12	12	12	12	12	12	12	12
	36 - 40	14	14	14	14	14	14	14	14
6:12 to 9:12	24 - 28	10	10	10	10	10	10	10	10
	28 - 32	12	12	12	12	12	12	12	12
	32 - 36	12	12	12	12	12	12	12	12
	36 - 40	14	14	14	14	14	14	14	14
9:12 to 12:12	24 - 28	10	10	10	10	10	10	10	10
	28 - 32	12	12	12	12	12	12	12	12
	32 - 36	12	12	12	12	12	12	12	14
	36 - 40	14	14	14	14	14	14	14	16

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Ceiling diaphragm composed of 3/8" C-D, C-C sheathing (min. thickness) secured with screws spaced at 6 in o.c. at panel edges and in field.

² Diaphragm is secured with No. 8 screws (min.) when framing members have a designation thickness of 54 mils or less and No. 10 screws (min.) when framing members have a designation thickness greater than 54 mils.

³ Maximum aspect ratio (length/width) of diaphragm is 2:1.

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

⁵ Required diaphragm lengths are to be provided at each end of the structure (refer to Figure 2.5).

Table 2.6
Required Lengths for Wood Structural Panel Sheathed Ceiling Diaphragms –
Ceiling Height = 9 or 10 ft^{1,2,3,4,5}

		Basic Wind Speed (mph)							
Exposure A/B		85	90	100	110	120	130		
Exposure C				85	90	100	110	120	130
Roof Pitch	Building Endwall Width (ft)	Minimum Diaphragm Length (ft)							
3:12 to 6:12	24 - 28	10	10	10	10	10	10	10	10
	28 - 32	12	12	12	12	12	12	12	12
	32 - 36	12	12	12	12	12	12	12	12
	36 - 40	14	14	14	14	14	14	14	14
6:12 to 9:12	24 - 28	10	10	10	10	10	10	10	10
	28 - 32	12	12	12	12	12	12	12	12
	32 - 36	12	12	12	12	12	12	12	12
	36 - 40	14	14	14	14	14	14	14	14
9:12 to 12:12	24 - 28	10	10	10	10	10	10	10	10
	28 - 32	12	12	12	12	12	12	12	12
	32 - 36	12	12	12	12	12	12	12	14
	36 - 40	14	14	14	14	14	14	14	16

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Ceiling diaphragm composed of 3/8" C-D, C-C sheathing (min. thickness) secured with screws spaced at 6 in o.c. at panel edges and in field.

² Diaphragm is secured with No. 8 screws (min.) when framing members have a designation thickness of 54 mils or less and No. 10 screws (min.) when framing members have a designation thickness greater than 54 mils.

³ Maximum aspect ratio (length/width) of diaphragm is 2:1.

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

⁵ Required diaphragm lengths are to be provided at each end of the structure (refer to Figure 2.5).

33 KSI

Table 2.7
Stud Thickness
All Building Widths – Gable End Stud Wall^{1,2,3}
 $F_y = 33 \text{ ksi}$

Wind Speed		Member Size	Stud Spacing (inches)	Minimum Stud Thickness (mils)		
Exp. A/B	Exp. C			Stud Height, h (feet)		
				8	9	10
85 mph		350S162	16	33	33	33
			24	33	33	33
		550S162	16	33	33	33
			24	33	33	33
90 mph		350S162	16	33	33	33
			24	33	33	33
		550S162	16	33	33	33
			24	33	33	33
100 mph	85 mph	350S162	16	33	33	33
			24	33	33	43
		550S162	16	33	33	33
			24	33	33	33
110 mph	90 mph	350S162	16	33	33	33
			24	33	33	33
		550S162	16	33	33	33
			24	33	33	33
120 mph	100 mph	350S162	16	33	33	33
			24	43	33	43
		550S162	16	33	33	33
			24	33	33	33
130 mph	110 mph	350S162	16	33	33	33
			24	33	43	54
		550S162	16	33	33	33
			24	33	33	33
	120 mph	350S162	16	43	54	68
			24	68	97	97
		550S162	16	33	33	43
			24	43	43	54
	130 mph	350S162	16	54	68	97
			24	54	97	-
		550S162	16	33	43	43
			24	43	43	43

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Deflection criteria: $h/240$

² Design load assumptions:

Roof and ceiling dead load is 12 psf (0.58 kN/m²)

Ground snow load is 70 psf (3.35 kN/m²)

Floor dead load is 10 psf (0.48 kN/m²)

Floor live load is 40 psf (1.92 kN/m²)

³ Building width is in the direction of horizontal framing members supported by the wall studs.

50 ksi

Table 2.8
Stud Thickness
All Building Widths – Gable End Stud Wall^{1,2,3}
 $F_y = 50$ ksi

Wind Speed		Member Size	Stud Spacing (inches)	Minimum Stud Thickness (mils)		
Exp. A/B	Exp. C			Stud Height, h (feet)		
				8	9	10
85 mph		350S162	16	33	33	33
			24	33	33	33
		550S162	16	33	33	33
			24	33	33	33
90 mph		350S162	16	33	33	33
			24	33	33	33
		550S162	16	33	33	33
			24	33	33	33
100 mph	85 mph	350S162	16	33	33	33
			24	33	33	33
		550S162	16	33	33	33
			24	33	33	33
110 mph	90 mph	350S162	16	33	33	33
			24	33	33	43
		550S162	16	33	33	33
			24	33	33	33
120 mph	100 mph	350S162	16	33	33	33
			24	33	33	43
		550S162	16	33	33	33
			24	33	33	33
130 mph	110 mph	350S162	16	33	33	33
			24	33	43	54
		550S162	16	33	33	33
			24	33	33	33
	120 mph	350S162	16	33	33	43
			24	43	43	68
		550S162	16	33	33	33
			24	33	33	33
	130 mph	350S162	16	33	43	54
			24	43	54	68
		550S162	16	33	33	33
			24	33	33	43

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Deflection criteria: $h/240$

² Design load assumptions:

Roof and ceiling dead load is 12 psf (0.58 kN/m²)

Ground snow load is 70 psf (3.35 kN/m²)

Floor dead load is 10 psf (0.48 kN/m²)

Floor live load is 40 psf (1.92 kN/m²)

³ Building width is in the direction of horizontal framing members supported by the wall studs.

Table 3.1 – Required Number of King and Jack Studs (AISI, 2002)

Size of Opening	24 inch o.c. Stud Spacing		16 inch 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
> 10'-6" to 12'-0"	2	2	3	3
> 12'-0" to 13'-0"	2	3	3	3
> 13'-0" to 14'-0"	2	3	3	4
> 14'-0" to 16'-0"	2	3	3	4
> 16'-0" to 18'-0"	3	3	4	4

For SI: 1 inch = 25.4 mm, 1 foot = 0.305 m

33
KSI

Table 3.2
Head Track Span for 14' Maximum Stud Height, $h^{1,2,3}$
 $F_y = 33$ ksi

Basic Wind Speed (mph)		Allowable Head Track Span, L (ft.-in.)					
Exposure		Track Designation					
A/B	C	350T125-33	350T125-43	350T125-54	550T125-33	550T125-43	550T125-54
85		4'-7"	5'-6"	6'-4"	5'-11"	7'-4"	8'-9"
90		4'-4"	5'-2"	6'-0"	5'-7"	7'-0"	8'-4"
100	85	3'-11"	4'-8"	5'-4"	5'-0"	6'-3"	7'-5"
110	90	3'-7"	4'-3"	4'-11"	4'-7"	5'-8"	6'-9"
120	100	3'-3"	3'-10"	4'-6"	4'-2"	5'-3"	6'-2"
130	110	3'-0"	3'-7"	4'-2"	3'-10"	4'-10"	5'-9"
	120	2'-9"	3'-3"	3'-9"	3'-6"	4'-5"	5'-3"
	130	2'-6"	3'-0"	3'-6"	3'-3"	4'-1"	4'-10"

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Deflection criteria: $L/240$

² Stud height, h , measured from wall bottom track to wall top track or brace connection height at center of opening.

³ Head track spans are based on components and cladding wind speeds and tributary span $h/2$.

**33
KSI**

Table 3.3
Head Track Span for 18' Maximum Stud Height, $h^{1,2,3}$
 $F_y = 33$ ksi

Basic Wind Speed (mph)		Allowable Head Track Span, L (ft-in.)					
Exposure		Track Designation					
A/B	C	350T125-33	350T125-43	350T125-54	550T125-33	550T125-43	550T125-54
85		4'-1"	4'-10"	5'-7"	5'-3"	6'-6"	7'-9"
90		3'-10"	4'-7"	5'-3"	4'-11"	6'-2"	7'-4"
100	85	3'-5"	4'-4"	4'-9"	4'-5"	5'-6"	6'-6"
110	90	3'-2"	3'-9"	4'-4"	4'-0"	5'-0"	6'-0"
120	100	2'-10"	3'-5"	3'-11"	3'-8"	4'-7"	5'-6"
130	110	2'-8"	3'-2"	3'-8"	3'-5"	4'-3"	5'-1"
	120	2'-5"	2'-10"	3'-4"	3'-1"	3'-10"	4'-7"
	130	2'-3"	2'-8"	3'-1"	2'-10"	3'-7"	4'-3"

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Deflection criteria: L/240

² Stud height, h , measured from wall bottom track to wall top track or brace connection height at center of opening.

³ Head track spans are based on components and cladding wind speeds and tributary span $h/2$.

**33
KSI**

Table 3.4
Head Track Span for 22' Maximum Stud Height, $h^{1,2,3}$
 $F_y = 33$ ksi

Basic Wind Speed (mph)		Allowable Head Track Span, L (ft-in.)					
Exposure		Track Designation					
A/B	C	350T125-33	350T125-43	350T125-54	550T125-33	550T125-43	550T125-54
85		3'-8"	4'-4"	5'-1"	4'-9"	5'-10"	7'-0"
90		3'-6"	4'-2"	4'-9"	4'-6"	5'-7"	6'-7"
100	85	3'-1"	3'-8"	4'-3"	4'-0"	4'-11"	5'-11"
110	90	2'-10"	3'-4"	3'-11"	3'-8"	4'-6"	5'-5"
120	100	2'-7"	3'-1"	3'-7"	3'-4"	4'-2"	4'-11"
130	110	2'-5"	2'-10"	3'-4"	3'-1"	3'-10"	4'-7"
	120	2'-2"	2'-7"	3'-0"	2'-10"	3'-6"	4'-2"
	130	2'-0"	2'-5"	2'-9"	2'-7"	3'-3"	3'-10"

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Deflection criteria: L/240

² Stud height, h , measured from wall bottom track to wall top track or brace connection height at center of opening.

³ Head track spans are based on components and cladding wind speeds and tributary span $h/2$.

**33
KSI**

Table 3.5
Head Track Span for 26' Maximum Stud Height, $h^{1,2,3}$
 $F_y = 33$ ksi

Basic Wind Speed (mph)		Allowable Head Track Span, L (ft-in.)					
Exposure		Track Designation					
A/B	C	350T125-33	350T125-43	350T125-54	550T125-33	550T125-43	550T125-54
85		3'-5"	4'-0"	4'-8"	4'-4"	5'-5"	6'-5"
90		3'-12"	3'-9"	4'-5"	4'-1"	5'-1"	6'-1"
100	85	2'-10"	3'-5"	3'-11"	3'-8"	4'-7"	5'-5"
110	90	2'-7"	3'-1"	3'-7"	3'-4"	4'-2"	5'-0"
120	100	2'-5"	2'-10"	3'-3"	3'-1"	3'-10"	4'-7"
130	110	2'-2"	2'-7"	3'-0"	2'-10"	3'-6"	4'-2"
	120	2'-0"	2'-5"	2'-9"	2'-7"	3'-3"	3'-10"
	130	1'-10"	2'-2"	2'-7"	2'-5"	3'-0"	3'-6"

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Deflection criteria: $L/240$

² Stud height, h , measured from wall bottom track to wall top track or brace connection height at center of opening.

³ Head track spans are based on components and cladding wind speeds and tributary span $h/2$.

**33
KSI**

Table 3.6
Head Track Span for 30' Maximum Stud Height, $h^{1,2,3}$
 $F_y = 33$ ksi

Basic Wind Speed (mph)		Allowable Head Track Span, L (ft-in.)					
Exposure		Track Designation					
A/B	C	350T125-33	350T125-43	350T125-54	550T125-33	550T125-43	550T125-54
85		3'-2"	3'-9"	4'-4"	4'-1"	5'-0"	6'-0"
90		3'-0"	3'-6"	4'-1"	3'-10"	4'-9"	5'-8"
100	85	2'-8"	3'-2"	3'-8"	3'-5"	4'-3"	5'-1"
110	90	2'-5"	2'-11"	3'-4"	3'-1"	3'-10"	4'-7"
120	100	2'-3"	2'-8"	3'-1"	2'-10"	3'-7"	4'-3"
130	110	2'-0"	2'-5"	2'-10"	2'-8"	3'-3"	3'-11"
	120	1'-10"	2'-3"	2'-7"	2'-5"	3'-0"	3'-7"
	130	1'-9"	2'-1"	2'-4"	2'-3"	2'-9"	3'-3"

For SI: 1 inch = 25.4 mm, 1 psf = 0.0479 kN/m², 1 mph = 1.61 km/hr, 1 foot = 0.305 m

¹ Deflection criteria: $L/240$

² Stud height, h , measured from wall bottom track to wall top track or brace connection height at center of opening.

³ Head track spans are based on components and cladding wind speeds and tributary span $h/2$.

Table 4.1
Sill Plate Connections - Wall Bottom Track to Floor Joist or Track^{1,2,3,4}

Basic Wind Speed (mph)		Wall Bottom Track to Floor Joist or Track Connection				
Exposure		Stud Height, h (ft)				
A/B	C	10 < h ≤ 14	14 < h ≤ 18	18 < h ≤ 22	22 < h ≤ 26	26 < h ≤ 30
85		1-No.8 screw @ 12" o.c.	1-No.8 screw @ 12" o.c.	1-No.8 screw @ 12" o.c.	1-No.8 screw @ 12" o.c.	1-No.8 screw @ 12" o.c.
90		1-No.8 screw @ 12" o.c.	1-No.8 screw @ 12" o.c.	1-No.8 screw @ 12" o.c.	1-No.8 screw @ 12" o.c.	2-No.8 screws @ 12" o.c.
100	85	1-No.8 screw @ 12" o.c.	1-No.8 screw @ 12" o.c.	1-No.8 screw @ 12" o.c.	2-No.8 screws @ 12" o.c.	1-No.8 screw @ 8" o.c.
110	90	1-No.8 screw @ 12" o.c.	1-No.8 screw @ 12" o.c.	2-No.8 screws @ 12" o.c.	1-No.8 screws @ 8" o.c.	2-No.8 screws @ 8" o.c.
120	100	1-No.8 screw @ 12" o.c.	2-No.8 screws @ 12" o.c.	1-No.8 screw @ 8" o.c.	2-No.8 screws @ 8" o.c.	2-No.8 screws @ 8" o.c.
130	110	2-No.8 screws @ 12" o.c.	1-No.8 screw @ 8" o.c.	2-No.8 screws @ 8" o.c.	2-No.8 screws @ 8" o.c.	2-No.8 screws @ 8" o.c.
	120	2-No.8 screws @ 12" o.c.	2-No.8 screws @ 8" o.c.	2-No.8 screws @ 8" o.c.	2-No.8 screws @ 8" o.c.	2-No.8 screws @ 8" o.c.
	130	2-No.8 screws @ 8" o.c.	2-No.8 screws @ 8" o.c.	2-No.8 screws @ 8" o.c.	2-No.8 screws @ 8" o.c.	-

¹ Ref. Sections E11 - E13 for additional requirements for high wind and seismic areas (AISI, 2002).

² Ref. Table 4.2 for wall bottom track to foundation connections.

³ Where attachment is not given, special design is required.

⁴ Stud height, h, measured from wall bottom track to wall top track or brace connection height.

Table 4.2
Sill Plate Connections – Wall Bottom Track to Foundation^{1,2,3}

Basic Wind Speed (mph)		Minimum Spacing for 1/2" ϕ Sill Bolts				
		Stud Height, h (ft)				
A/B	C	10 < h $\leq$ 14	14 < h $\leq$ 18	18 < h $\leq$ 22	22 < h $\leq$ 26	26 < h $\leq$ 30
85		4'-0" o.c.	4'-0" o.c.	4'-0" o.c.	3'-0" o.c.	3'-0" o.c.
90		4'-0" o.c.	4'-0" o.c.	3'-0" o.c.	3'-0" o.c.	2'-0" o.c.
100	85	4'-0" o.c.	3'-0" o.c.	3'-0" o.c.	2'-0" o.c.	2'-0" o.c.
110	90	4'-0" o.c.	3'-0" o.c.	2'-0" o.c.	2'-0" o.c.	1'-4" o.c.
120	100	3'-0" o.c.	2'-0" o.c.	2'-0" o.c.	1'-4" o.c.	1'-4" o.c.
130	110	2'-0" o.c.	2'-0" o.c.	1'-4" o.c.	1'-4" o.c.	1'-4" o.c.
	120	2'-0" o.c.	2'-0" o.c.	1'-4" o.c.	1'-4" o.c.	-
	130	2'-0" o.c.	1'-4" o.c.	1'-4" o.c.	-	-

¹ Where attachment is not given, special design is required.

² Stud height, h, measured from wall bottom track to wall top track or brace connection height.

³ Refer to Figure 4.1 or 4.2 for connection.

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APPENDIX A – DESIGN CALCULTIONS

A1 Balloon Framing Method Calculations (Section 2.1)

A1.1 Gable End Overhang Design Calculation (Figure 2.3)

Design Objective:

Determine the required number of No. 8 screws to connect the outlooker to the wall top track in Figure 2.3 Option #1. For Option #2 in Figure 2.3, determine the required number of No. 8 screws to attach the outlooker track at each wall stud, and the required spacing of No. 8 screws to connect the sheathing to the wall top track. All references are to the *North American Specification for the Design of Cold-Formed Steel Structural Members*, unless otherwise noted (AISI, 2001)

A1.1.1 Overhang Option #1 (Figure A1.1):

Design Assumptions:

Stories = 2
Roof Overhang (Rake) = 1 ft
Stud Spacing = 24 in
Rafter Spacing = 24 in
Outlooker Thickness = 33 mil
Steel Yield Strength, F_y = 33 ksi
Roof Pitch = 3:12
Building Width = 32 ft
Wall height = 10 ft
Roof/Ceiling Dead Load = 12 psf
Wind Speed = 130 mph
Wind Exposure Category = C

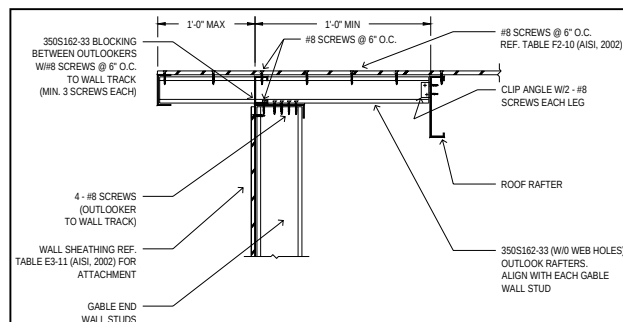


Figure A1.1 - Overhang Option #1 Detail

Design Approach:

Determine required number of screws from the outlooker to the wall top track to resist uplift forces, and the required spacing of No. 8 screws into the blocking to resist shear.

Solution:

Stud Height, $h = 10 \text{ ft} + 16 \text{ ft}(3/12) = 14 \text{ ft}$
Roof Height = $2(10 \text{ ft}) + 16 \text{ ft}(3/12) = 24 \text{ ft}$
Effective Area for Wind (uplift) = $(1 \text{ ft})(2 \text{ ft}) = 2 \text{ ft}^2$
Roof Slope = $\tan^{-1}(3/12) = 14.04^\circ$

Design Loads:

Dead Load:

Roof/Ceiling Dead Load = $(12 \text{ psf})(4 \text{ ft}^2) = 48 \text{ lbs}$

Wind Loads (Shear):

Wind loads are calculated in accordance with ASCE 7-2002 Method 2 for Components and Cladding (C&C) (ASCE, 2002).

Equation:

$$p = q_h \left[(GC_p) - (GC_{pi}) \right]$$

$$q_h = 0.00256 K_z K_{zt} K_d V^2 I \quad (\text{lb/ft}^2)$$

Comments:

Low-Rise Building

Reference:

6.5.12.4, Eq. 6-22

6.5.10, Eq 6-15

Where:

$$K_z = 0.932$$

$$K_{zt} = 1.0$$

$$K_d = 0.85$$

$$V = 130 \text{ mph}$$

$$I = 1.0$$

Exp. C, Case 1, $z = 24 \text{ ft}$

No Topographic Factor

Table 6-3

Table 6-4

3-Second Gust Wind Speed

Residential Building (Cat I)

Table 1-1, Table 6-1

$$q_h = 0.00256(0.932)(1.0)(0.85)(130)^2(1.0) = 34.27 \text{ lb/ft}^2$$

$$GC_p = -1.1, +1.0$$

$$GC_{pi} = \pm 0.18$$

Assume Eff. Area $< 10 \text{ ft}^2$

Enclosed Building

Figure 6-11A

$$p^+ = (34.27 \text{ lb/ft}^2) \left[(1.0) - (-0.18) \right] = 40.44 \text{ lb/ft}^2$$

$$p^- = (34.27 \text{ lb/ft}^2) \left[(-1.10) - (0.18) \right] = -43.87 \text{ lb/ft}^2$$

Controls

Wind Loads (Uplift):

Wind loads are calculated in accordance with ASCE 7-2002 Method 2 for Components and Cladding (C&C) (ASCE, 2002).

Equation:

$$p = q_h \left[(GC_p) - (GC_{pi}) \right]$$

$$q_h = 0.00256 K_z K_{zt} K_d V^2 I \quad (\text{lb/ft}^2)$$

Comments:

Low-Rise Building

Reference:

6.5.12.4, Eq. 6-22

6.5.10, Eq 6-15

Where:

$$K_z = 0.932$$

$$K_{zt} = 1.0$$

$$K_d = 0.85$$

$$V = 130 \text{ mph}$$

$$I = 1.0$$

Exp. C, Case 1, $z = 24 \text{ ft}$

No Topographic Factor

Table 6-3

Table 6-4

3-Second Gust Wind Speed

Residential Building (Cat I)

Table 1-1, Table 6-1

$$q_h = 0.00256(0.932)(1.0)(0.85)(130)^2(1.0) = 34.27 \text{ lb/ft}^2$$

$$GC_p = -3.7$$

$$GC_{pi} = 0$$

Overhang, Area = 2 ft^2

No Interior Surface

Figure 6-11C

$$p^- = (34.27 \text{ lb/ft}^2) \left[-3.7 \right] = -126.81 \text{ lb/ft}^2$$

Controls

Load Combinations:

1. $0.9D + 1.6W$

2. $1.6W$

Uplift Check

Shear Check

Design Loads:

1. Maximum Uplift, $P_u = 0.9(48 \text{ lbs}) - 1.6(126.81 \text{ psf})(2 \text{ ft}^2) = -362.6 \text{ lbs}$
2. Maximum Shear, $w_u = 1.6(43.87 \text{ psf})(14 \text{ ft}/2) = 491.3 \text{ plf}$

Material Properties:

$$\begin{aligned} F_y \text{ (track and outlooker)} &= 33,000 \text{ psi} \\ F_u \text{ (track and outlooker)} &= 45,000 \text{ psi} \end{aligned}$$

Tensile Capacity of No. 8 Screws:

Must check pull-out, pull-over, and tension capacity of screws per *Specification*, Section E4.4:

Pull-Out Capacity (Section E4.4.1):

$$P_{not} = 0.85t_c d F_{u2} \quad \text{Eq. E4.4.1-1}$$

where

$$\begin{aligned} t_c &= t_2 = 0.0346 \text{ in} && 33 \text{ mil material} \\ d &= 0.164 \text{ in} && \text{No. 8 screw} \\ F_{u2} &= 45,000 \text{ psi} && 33 \text{ ksi steel} \end{aligned}$$

$$P_{not} = 0.85(0.0346 \text{ in})(0.164 \text{ in})(45,000 \text{ psi}) = 217.05 \text{ lbs}$$

Pull-Over Capacity (Section E4.4.2):

$$P_{nov} = 1.5t_1 d_w F_{u1} \quad \text{Eq. E4.4.2-1}$$

where

$$\begin{aligned} t_1 &= 0.0346 \text{ in} && 33 \text{ mil material} \\ d_w &= 0.322 \text{ in} && \text{No. 8 screw} \\ F_{u1} &= 45,000 \text{ psi} && 33 \text{ ksi steel} \end{aligned}$$

$$P_{nov} = 1.5(0.0346 \text{ in})(0.322 \text{ in})(45,000 \text{ psi}) = 752.03 \text{ lbs}$$

Tension in Screw Capacity (Section E4.4.3):

Assume tensile capacity of screw is exceeds pullout capacity of 33 mil material

Therefore, $P_n = P_{not} = \mathbf{217.05 \text{ lbs}}$

Determine Required Number of Screws:

$$\begin{aligned} \phi P_n &\geq P_u \\ \phi &= 0.50 \end{aligned} \quad \text{Section E4}$$

$$N = \frac{P_u}{\phi P_n} = \frac{362.6 \text{ lbs}}{0.5(217.05 \text{ lbs})} = 3.34 \text{ Screws} - \text{Need 4 Screws}$$

Therefore, use minimum of 4 – No. 8 screws at each outlooker to wall top track connection as indicated in Figure 2.3.

No. 8 Screw Shear Capacity (Section E4.3):

Connection Shear Limited by Tilting and Bearing (Section E4.3.1):

$$t_1 = t_2 = 0.0346 \text{ in}$$

33 mil Material

$$d = 0.164 \text{ in}$$

No. 8 Screw per Table C-E4-1

$$F_{u1} = F_{u2} = 45,000 \text{ psi}$$

33 ksi Steel

$$\frac{t_2}{t_1} = 1.0$$

Therefore, nominal shear strength, P_{ns} , is the smallest of the following:

$$P_{ns} = 4.2\sqrt{t_2^3 d F_{u2}} = 4.2\sqrt{(0.0346)^3 (0.164) (45,000)} = 492.6 \text{ lbs} \quad \text{Eq. E4.3.1-1}$$

$$P_{ns} = 2.7t_1 d F_{u1} = 2.7(0.0346)(0.164)(45,000) = 689.4 \text{ lbs} \quad \text{Eq. E4.3.1-2}$$

$$P_{ns} = 2.7t_2 d F_{u2} = 2.7(0.0346)(0.164)(45,000) = 689.4 \text{ lbs} \quad \text{Eq. E4.3.1-3}$$

$$P_{ns} = 492.6 \text{ lbs}$$

Connection Shear Limited by End Distance (Section E4.3.2):

$$\text{Assume end distance} \geq 1.5d = 1.5(0.164 \text{ in}) = 0.25 \text{ in}$$

Therefore, shear capacity will not be limited by end distance

Shear in Screws (Section E4.3.3):

Assume shear capacity of screw exceeds tilting/bearing capacity of 33 mil material

Therefore, $P_{ns} = 492.6 \text{ lbs}$

Determine Required Spacing of Screws:

$$\phi P_{ns} \geq P_u$$

Section E4

$$\phi = 0.50$$

$$\text{Spacing, } s = \frac{\phi P_n}{w_u} = \frac{0.5(492.6 \text{ lbs})}{491.3 \text{ plf}} = (0.50 \text{ ft})(12 \text{ in / ft}) = 6 \text{ in}$$

Therefore, use No. 8 screws at 6" o.c. to connect blocking to wall track, as shown in Figure 2.3, Option #1.

A1.1.2 Overhang Option #2 (Figure A1.2):

Design Assumptions:

See Section A1.1.1

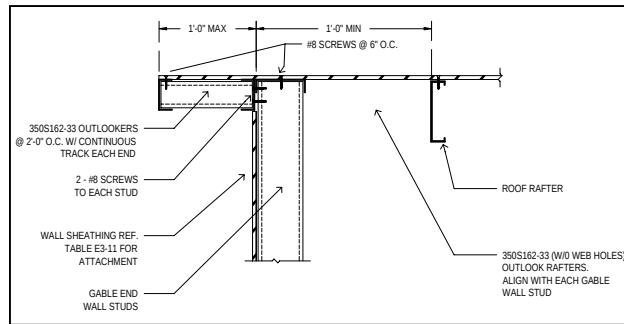


Figure A1.2 - Overhang Option #2 Detail

Design Approach:

Determine required spacing of No. 8 screws from the sheathing to wall top track to resist shear forces, and the required number of No. 8 screws from the outlooker track to the studs to resist the uplift forces.

Load Combinations:

1. $0.9D + 1.6W$
2. $1.6W$

Uplift Check
Shear Check

Design Loads:

1. Maximum Uplift, $P_u = 0.9(48 \text{ lbs}) - 1.6(126.81 \text{ psf})(2 \text{ ft}^2) = -362.6 \text{ lbs}$
2. Maximum Shear, $w_u = 1.6(43.87 \text{ psf})(14 \text{ ft}/2) = 491.3 \text{ plf}$

Spacing of Screws to Resist Shear:

Shear Capacity of No. 8 Screws:

From above calculation, $\phi P_n = 0.50(492.6 \text{ lbs}) = 246.3 \text{ lbs}$

Factored Load, $w_u = 491.3 \text{ plf}$

$$\text{Maximum Spacing, } s = \frac{\phi P_n}{w_u} = \frac{246.3 \text{ lbs}}{491.3 \text{ plf}} = (0.50 \text{ ft})(12 \text{ in} / \text{ft}) = 6 \text{ in} \geq 6 \text{ in} - \text{OK} !$$

Number of Screws per Stud to Resist Uplift:

Shear Capacity of No. 8 Screws:

From above calculation, $\phi P_n = 0.50(492.6 \text{ lbs}) = 246.3 \text{ lbs}$

$P_u = -362.6 \text{ lbs}$

$$N = \frac{P_u}{\phi P_n} = \frac{362.6 \text{ lbs}}{0.5(217.05 \text{ lbs})} = 3.34 \text{ Screws} - \text{Need 2 Screws}$$

Per Figure 2.3, use No. 8 Screws @ 6" o.c. from roof sheathing to wall top track to resist shear, and 2 - No. 8 Screws from outlooker track to each stud to resist uplift.

A1.2 Full Height Wall Stud Design Calculation (Table 2.1)

Design Objective:

Determine the minimum thickness for the gable end wall studs in the design described below. The length of the studs, h , is the full height of the gable end, with lateral bracing of the flanges at 48" on center. All references are to the *North American Specification for the Design of Cold-Formed Steel Structural Members* unless otherwise noted (AISI, 2001)

Design Assumptions:

Building Width =	36 ft
Roof Overhang (Rake) =	1 ft
Stud Spacing =	16 in
Rafter Spacing =	16 in
Stud Size =	350S162
Steel Yield Strength, F_y =	33 ksi
Roof Pitch =	4:12
Ground Snow Load =	70 psf
Snow Reduction Factor =	0.7
Multiplier for Unbal. Snow =	1.5
Minimum Roof Live Load =	16 psf
Roof/Ceiling Dead Load =	12 psf
Wind Speed =	100 mph
Wind Exposure Category =	B
Building Length =	50 ft
Ceiling Height =	8 ft
Deflection Limit =	$h/240$

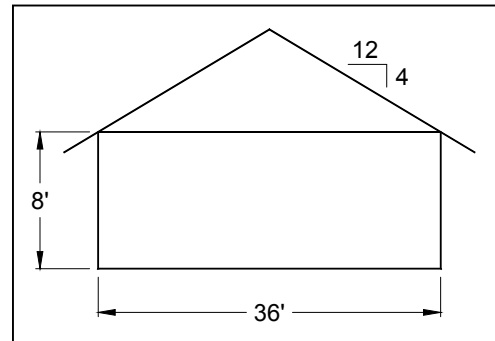


Figure A1.3 - Gable End Wall Elevation

Design Approach:

Design king stud (tallest) of gable end wall and use resulting thickness for all gable end studs.

Solution:

$$\text{Stud Height, } h = 8 \text{ ft} + (36 \text{ ft}/2)(4/12) = 14 \text{ ft}$$

Design Loads:

Dead Load:

$$\text{Roof/Ceiling Dead Load} = (12 \text{ psf})(16/12 \text{ ft}) = 16 \text{ plf}$$

Live Loads:

$$\text{Roof Live Load} = (16 \text{ psf})(16 \text{ in})(1 \text{ ft}/12 \text{ in}) = 21.3 \text{ plf}$$

$$\text{Roof Snow Load} = (0.7 \times 70 \text{ psf} \times 1.5)(16 \text{ in})(1/12) = 98 \text{ plf}$$

Therefore, roof snow loads control over roof live loads

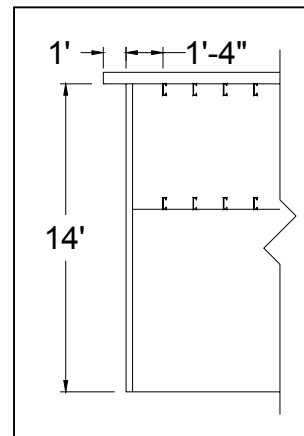


Figure A1.4 - Wall Section

Wind Loads:

Wind loads are calculated in accordance with ASCE 7-2002 Method 2 for Main Wind Force Resisting System (MWFRS) and Components and Cladding (C&C) (ASCE, 2002).

MWFRS:

$$p = q_h \left[(GC_{pf}) - (GC_{pi}) \right] \text{ (lb/ft}^2\text{)}$$

$$q_h = 0.00256 K_z K_{zt} K_d V^2 I \text{ (lb/ft}^2\text{)}$$

Comments:

Low-Rise Building

Reference:

6.5.12.2.2, Eq. 6-18

6.5.10, Eq 6-15

Where:

$$K_z = 0.70$$

$$K_{zt} = 1.0$$

$$K_d = 0.85$$

$$V = 100 \text{ mph}$$

$$I = 1.0$$

$$GC_{pf} = 0.40 \text{ or } -0.29$$

$$GC_{pi} = \pm 0.18$$

Exp. B, Case 1, $z = 14 \text{ ft}$

No Topographic Factor

Table 6-3

Table 6-4

3-Second Gust Wind Speed

Residential Building (Cat I)

Flat Roof, Bldg. Surface 1/4

Enclosed Building

Table 1-1, Table 6-1

Figure 6-10

Figure 6-5

$$q_h = 0.00256(0.70)(1.0)(0.85)(100)^2(1.0) = 15.23 \text{ lb/ft}^2$$

$$p^+ = (15.23 \text{ lb/ft}^2) [(0.40) - (-0.18)] = 8.8 \text{ lb/ft}^2$$

Controls

$$p^- = (15.23 \text{ lb/ft}^2) [(-0.29) - (0.18)] = -7.16 \text{ lb/ft}^2$$

ASCE Section 6.1.4.1 requires a minimum MWFRS wind load of 10 psf

Therefore, $p = 10 \text{ psf}$

C&C:

$$p = q_h \left[(GC_p) - (GC_{pi}) \right]$$

$$q_h = 15.23 \text{ lb/ft}^2$$

$$GC_p = 1.0 \text{ or } -1.05$$

$$GC_{pi} = \pm 0.18$$

$$p^+ = (15.23 \text{ lb/ft}^2) [(1.0) - (-0.18)] = 18.0 \text{ lb/ft}^2$$

$$p^- = (15.23 \text{ lb/ft}^2) [(-1.05) - (0.18)] = -18.7 \text{ lb/ft}^2$$

Comments:

Low-Rise Building

See Above Calculation

Eff. Wind Area = 18.7 ft^2

Enclosed Building

Reference:

6.5.12.4, Eq. 6-22

Figure 6-11A

Figure 6-5

Controls

Load Combinations:

1. $1.4D$

2. $1.2D + 1.6L + 0.5(L_r \text{ or } S)$

3. $1.2D + 0.5L + 1.6(L_r \text{ or } S)$

4. $1.2D + 0.8W + 1.6(L_r \text{ or } S)$

5. $1.2D + 1.6W + 0.5L + 0.5(L_r \text{ or } S)$

6. $1.6W$

7. $0.7W$

Using MWFRS for W

Using MWFRS for W

Using C&C for W – Bending Check

Using C&C for W – Deflection Check

Design Loads:

Reaction at Wall Stud = $(2.33 \text{ ft} \times w)(1.17 \text{ ft})/1.33 \text{ ft} = (2.04 \text{ ft})w$ See Figure A1.4

<u>Axial Loads:</u>		<u>Lateral Loads:</u>	
1.	$1.4(16 \text{ plf}) = 22.4 \text{ plf}$	None	
2.	$1.2(16 \text{ plf}) + 1.6(0) + 0.5(98 \text{ plf}) = 68.2 \text{ plf}$	None	
3.	$1.2(16 \text{ plf}) + 0.5(0) + 1.6(98 \text{ plf}) = 176 \text{ plf}$	None	
4.	$1.2(16 \text{ plf}) + 1.6(98 \text{ plf}) = 176 \text{ plf}$	$0.8(10 \text{ psf})(16/12) = 10.7 \text{ plf}$	
5.	$1.2(16 \text{ plf}) + 0.5(98 \text{ plf}) = 68.2 \text{ plf}$	$1.6(10 \text{ psf})(16/12) = 21.3 \text{ plf}$	
6.	None	$1.6(18.7 \text{ psf})(16/12) = 39.9 \text{ plf}$	
7.	None	$0.7(18.7 \text{ psf})(16/12) = 17.5 \text{ plf}$	

Controlling Load Combinations:

<u>Axial Loads:</u>		<u>Lateral Loads:</u>	
3.	$(2.04 \text{ ft})(176 \text{ plf}) = 359 \text{ lbs}$	None	
4.	$(2.04 \text{ ft})(176 \text{ plf}) = 359 \text{ lbs}$	10.7 plf	
5.	$(2.04 \text{ ft})(68.2 \text{ plf}) = 139.1 \text{ lbs}$	21.3 plf	
6.	None	39.9 plf	
7.	None	17.5 plf	

Material Properties:

$F_y = 33,000 \text{ psi}$
 $E = 29,500,000 \text{ psi}$
 $G = 11,300 \text{ psi}$
 $\mu = 0.30$ (Poisson's Ratio)

Full Section Properties:

Select a 350S162-54 section as indicated in Table 3.2.

Section property calculations based on *Design Guide for AISI Cold-Formed Steel Specification*, Section 3 (AISI, 1996).

Refer to Figure A1.4 for all section property calculations

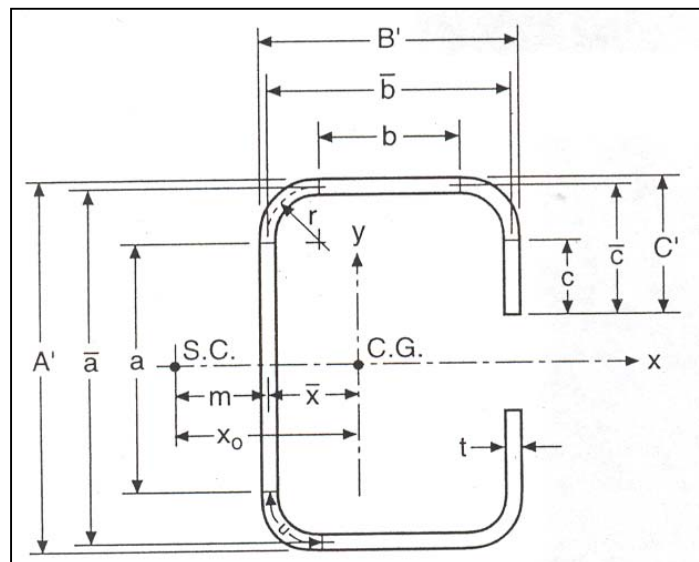


Figure A1.5 - C-Section Dimensions

$$\begin{aligned}
A' &= 3.5 \text{ in} \\
B' &= 1.625 \text{ in} \\
C' &= 0.5 \text{ in} \\
a &= A' - (2r + t) = 3.5 - (2(0.1363) + 0.0566 \text{ in}) = 3.1708 \text{ in} \\
\bar{a} &= A' - t = 3.5 - 0.0566 \text{ in} = 3.4434 \text{ in} \\
b &= B' - (2r + t) = 1.625 - (2(0.1363) + 0.0566 \text{ in}) = 1.2958 \text{ in} \\
\bar{b} &= B' - t = 1.625 - 0.0566 \text{ in} = 1.5684 \text{ in} \\
c &= C' - (r + t/2) = 0.5 - (0.1363 + 0.0566 \text{ in} / 2) = 0.3354 \text{ in} \\
\bar{c} &= C' - t/2 = 0.5 - 0.0566 \text{ in} / 2 = 0.4717 \text{ in} \\
R &= 0.108 \text{ in} \\
r &= R + t/2 = 0.108 + 0.0566 \text{ in} / 2 = 0.1363 \text{ in} \\
u &= \frac{\pi}{2} r = 1.57(0.1363 \text{ in}) = 0.2141 \text{ in} \\
t &= 0.0566 \text{ in}
\end{aligned}$$

Cross-sectional Area, A:

$$\begin{aligned}
A &= t(a + 2b + 2c + 4u) = 0.0566 \text{ in}(3.4434 + 2(1.2958) + 2(0.3354) + 4(0.2141 \text{ in})) = 0.4126 \text{ in}^2 \\
A \text{ from CFS} &= 0.4126 \text{ in}^2 - \text{OK !}
\end{aligned}$$

Moment of Inertia about x-axis, I_x :

$$\begin{aligned}
I_x &= 2t \left\{ \begin{aligned} &0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149r^3 + \\ &0.0833c^3 + \frac{c}{4}(a - c)^2 + u(a/2 + 0.637r)^2 + 0.149r^3 \end{aligned} \right\} \\
I_x &= 2(0.0566) \left\{ \begin{aligned} &0.0417(3.1708)^3 + 1.2958 \left(\frac{3.1708}{2} + 0.1363 \right)^2 + \\ &0.2141 \left(\frac{3.1708}{2} + 0.637(0.1363) \right)^2 + 0.149(0.1363)^3 + \\ &0.0833(0.3354)^3 + \frac{0.3354}{4}(3.1708 - 0.3354)^2 + \\ &0.2141 \left(\frac{3.1708}{2} + 0.637(0.1363) \right)^2 + 0.149(0.1363)^3 \end{aligned} \right\} = 0.7976 \text{ in}^4
\end{aligned}$$

$$I_x \text{ from CFS} = 0.7977 \text{ in}^4 - \text{OK !}$$

Location of Centroid, $\bar{x}$:

$$\begin{aligned}
\bar{x} &= \frac{2t}{A} \{ b(b/2 + r)^2 + u(0.363r) + u(b + 1.637r) + c(b + 2r) \} \\
\bar{x} &= \frac{2(0.0566)}{0.4126} \left\{ \begin{aligned} &1.2958 \left(\frac{1.2958}{2} + 0.1363 \right)^2 + 0.2141[0.363(0.1363)] + \\ &0.2141[1.2958 + 1.637(0.1363)] + 0.3354[1.2958 + 2(0.1363)] \end{aligned} \right\} = 0.5152 \text{ in}
\end{aligned}$$

$$\bar{x} = 0.5436 \text{ in from CFS} \approx 0.5152 \text{ in}$$

Distance between Centroid and Shear Center, m:

$$m = \bar{b} \left[\frac{3\bar{a}^2\bar{b} + \bar{c}(6\bar{a}^2 - 8\bar{c}^2)}{\bar{a}^3 + 6\bar{a}^2\bar{b} + \bar{c}(8\bar{c}^2 - 12\bar{a}\bar{c} + 6\bar{a}^2)} \right]$$

$$m = 1.5684 \left[\frac{3(3.4434)^2(1.5684) + 0.4717(6(3.4434)^2 - 8(0.4717)^2)}{3.4434^3 + 6(3.4434)^2(1.5684) + 0.4717(8(0.4717)^2 - 12(3.4434)(0.4717) + 6(3.4434)^2)} \right] = 0.7816 \text{ in}^2$$

Distance between Centroid and Shear Center, x_o :

$$x_o = -(\bar{x} + m) = -(0.5152 + 0.7816 \text{ in}) = -1.2968 \text{ in} \approx -1.2933 \text{ in from CFS} - \text{OK} !$$

Moment of Inertia about y-axis, I_y :

$$I_y = 2t \left\{ \begin{aligned} &b(b/2 + r)^2 + 0.0833b^3 + 0.356r^3 + \\ &c(b + 2r)^2 + u(b + 1.637r)^2 + 0.149r^3 \end{aligned} \right\} - A\bar{x}^2$$

$$I_y = 2(0.0566) \left\{ \begin{aligned} &1.2958\left(\frac{1.2958}{2} + 0.1363\right)^2 + 0.0833(1.2958)^3 + \\ &0.356(0.1363)^3 + 0.3354[1.2958 + 2(0.1363)^2] + \\ &0.2141[1.2958 + 1.637(0.1363)]^2 + 0.149(0.1363)^3 \end{aligned} \right\} - 0.4126(0.5152)^2 = 0.1506 \text{ in}^4$$

$$I_y \text{ from CFS} = 0.1508 \text{ in}^4 - \text{OK} !$$

Elastic Section Modulus, S:

$$S_x = \frac{I_x}{c}$$

$$c = 3.5 \text{ in}/2 = 1.75 \text{ in}$$

$$S_x = \frac{0.7976 \text{ in}^4}{1.75 \text{ in}} = 0.45577 \text{ in}^3 \approx 0.45583 \text{ in}^3 \text{ from CFS} - \text{OK} !$$

$$S_y = \frac{I_y}{c}$$

$$c(r) = 1.625 \text{ in} - \bar{x} = 1.625 \text{ in} - 0.5152 \text{ in} = 1.1098 \text{ in}$$

$$c(l) = \bar{x} = 0.5152 \text{ in}$$

$$S_y(r) = \frac{0.1506 \text{ in}^4}{1.1098 \text{ in}} = 0.13570 \text{ in}^3 \approx 0.13944 \text{ in}^3 \text{ from CFS} - \text{OK} !$$

$$S_y(l) = \frac{0.1506 \text{ in}^4}{0.5152 \text{ in}} = 0.29231 \text{ in}^3 \approx 0.27743 \text{ in}^3 \text{ from CFS} - \text{OK} !$$

Radius of Gyration, r:

$$r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{0.7976 \text{ in}^4}{0.4126 \text{ in}^2}} = 1.3904 \text{ in} \approx 1.3905 \text{ in from CFS} - \text{OK} !$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{0.1506 \text{ in}^4}{0.4126 \text{ in}^2}} = 0.6042 \text{ in} \approx 0.6046 \text{ in from CFS} - \text{OK} !$$

$$r_o = \sqrt{r_x^2 + r_y^2 + x_o^2}$$

$$r_o = \sqrt{(1.3904 \text{ in})^2 + (0.6042 \text{ in})^2 + (-1.2968 \text{ in})^2} = 1.9950 \text{ in} \approx 1.9928 \text{ in from CFS} - \text{OK} !$$

St. Venant Torsion Constant, J:

$$J = \frac{t^3}{3} (a + 2b + 2c + 4u)$$

$$J = \frac{(0.0566 \text{ in})^3}{3} (3.1708 + 2(1.2958) + 2(0.3354) + 4(0.2141 \text{ in})) = 0.0004406 \text{ in}^4$$

J from CFS = 0.0004406 in⁴ – OK !

Torsional Warping Constant, C_w (Based on Square Corners - Refer to Figure A1.2.3):

$$C_w = \frac{\bar{a} \bar{b} \bar{t}}{12} \left\{ \frac{\begin{aligned} &2\bar{a}^3 \bar{b} + 3\bar{a}^2 \bar{b}^2 + 48\bar{c}^4 + 112\bar{b}\bar{c}^3 + 8\bar{a}\bar{c}^3 \\ &+ 48\bar{a}\bar{b}\bar{c}^2 + 12\bar{a}^2 \bar{c}^2 + 12\bar{a}^2 \bar{b}\bar{c} + 6\bar{a}^3 \bar{c} \end{aligned}}{6\bar{a}^2 \bar{b} + (\bar{a} + 2\bar{c})^3 - 24\bar{a}\bar{c}^2} \right\}$$

$$C_w = 0.54267 \text{ in}^6 > 0.39754 \text{ in}^6 \text{ from CFS}$$

Note: CFS 4.13 calculates C_w as the sum of the individual C_w for the section parts. For a section with many connecting parts like a stiffened channel, this may result in a much lower value for C_w as indicated here. The above calculated value will be used in all calculations. Refer to the CFS help file for more information.

Combined Axial and Bending Capacity:

Check following interaction equations from *Specification*, Section C5.2:

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

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

When $\frac{P_u}{\phi_c P_n} \leq 0.15$, the following equation may be used in place of the above two

equations:

$$\frac{P_u}{\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0 \quad \text{Eq. C5.2.2-3}$$

where

$P_u =$	Required compressive axial strength
$M_{ux}, M_{uy} =$	Required flexural strengths about the x and y-axis, respectively
$P_n =$	Nominal axial strength in accordance with Section C4
$P_{no} =$	Nominal axial strength in accordance with Section C4, with $F_n = F_y$
$M_{nx}, M_{ny} =$	Nominal flexural strengths about the x and y-axis, respectively, determined in accordance with Section C3.1

$$\alpha_x = 1 - \frac{P_u}{P_{Ex}} \quad \text{Eq. C5.2.2-4}$$

$$\alpha_y = 1 - \frac{P_u}{P_{Ey}} \quad \text{Eq. C5.2.2-5}$$

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

$$P_{Ey} = \frac{\pi^2 EI_y}{(K_y L_y)^2} \quad \text{Eq. C5.2.2-7}$$

$\phi_b =$	Factor of Safety = 0.95	C3.1.1
$\phi_c =$	Factor of Safety = 0.85	C4
$I_x =$	0.7976 in ⁴	
$I_y =$	0.1506 in ⁴	
$L_x =$	Unbraced length for bending about x-axis = (14 ft)(12in/ft) = 168 in	
$L_y =$	Unbraced length for bending about y-axis = 48 in	
$K_x =$	Effective length factor for buckling about x-axis = 1.0	
$K_y =$	Effective length factor for buckling about y-axis = 1.0	
$C_{mx}, C_{my} =$	Coefficients = 1.0	C5.2

$$P_{Ex} = \frac{\pi^2 (29,500,000 \text{ psi})(0.7976 \text{ in}^4)}{(1.0 \times 168 \text{ in})^2} = 8,228 \text{ lbs}$$

$$\alpha_x = 1 - \frac{P_u}{8,228 \text{ lbs}}$$

Calculation of P_n in accordance with Section C.4:

$$P_n = A_e F_n \quad \text{Eq. C4-1}$$

where

$A_e =$	Effective area calculated at stress F_n
$F_n =$	Function of F_e calculated as the minimum of the elastic flexural buckling, torsional, or torsional-flexural buckling stress (Sections C4.1 – C4.4).

Find F_e :

1. F_e for sections not subject to torsional or torsional-flexural buckling (Section C4.1):

$$F_e = \frac{\pi^2 E}{(K_y L_y / r_y)^2} \quad \text{Eq. C4.1-1}$$

$$\frac{K_y L_y}{r_y} = \frac{(1.0)(48 \text{ in})}{0.6042 \text{ in}} = 79.44 < 200 - \text{OK!} \quad \text{Section C4(c)}$$

$$F_e = \frac{\pi^2 (29,500,000 \text{ psi})}{(79.44)^2} = 46,132 \text{ psi}$$

2. F_e for sections subject to torsional or torsional-flexural buckling (Section C4.2):

Torsional Buckling:

$$F_e = \sigma_t = \frac{1}{A r_o^2} \left[GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2} \right]$$

where

$$\begin{aligned} A &= 0.4126 \text{ in}^2 \\ r_o &= 1.9950 \text{ in} \\ G &= 11,300,000 \text{ psi} \\ J &= 0.0004406 \text{ in}^4 \\ C_w &= 0.54267 \text{ in}^6 \\ K_t &= \text{Effective length factor for twisting} = 1.0 \\ L_t &= \text{Unbraced length of member for twisting} = 48 \text{ in} \end{aligned}$$

$$F_e = \sigma_t = \frac{1}{(0.4126 \text{ in}^2)(1.9950 \text{ in})^2} \left[(11,300,000 \text{ psi})(0.0004406 \text{ in}^4) + \frac{\pi^2 (29,500,000 \text{ psi})(0.54267 \text{ in}^6)}{(1.0 \times 48 \text{ in})^2} \right]$$

$$F_e = 89,360 \text{ psi}$$

Torsional-Flexural Buckling:

$$F_e = \frac{1}{2\beta} \left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t} \right] \quad \text{Eq. C4.2-1}$$

$$\sigma_{ex} = \frac{\pi^2 E}{(K_x L_x / r_x)^2}$$

$$\frac{K_x L_x}{r_x} = \frac{(1.0)(14 \text{ ft})(12 \text{ in / ft})}{1.3904 \text{ in}} = 120.83 < 200 - \text{OK!} \quad \text{Section C4(c)}$$

$$\sigma_{ex} = \frac{\pi^2 (29,500,000 \text{ psi})}{(120.83)^2} = 19,943 \text{ psi}$$

$$\sigma_t = \frac{1}{A r_o^2} \left[GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2} \right] = 89,360 \text{ psi} \quad \text{See Above}$$

$$\beta = 1 - \left(\frac{x_0}{r_0} \right)^2 \quad \text{Eq. C4.2-3}$$

$$\begin{aligned} x_0 &= -1.2968 \text{ in} \\ r_0 &= 1.9950 \text{ in} \end{aligned}$$

$$\beta = 1 - \left(\frac{-1.2968 \text{ in}}{1.9950 \text{ in}} \right)^2 = 0.5775$$

$$\begin{aligned} F_e &= \frac{1}{2(0.5775)} \left[(19,943 + 89,360 \text{ psi}) - \sqrt{(19,943 + 89,360 \text{ psi})^2 - 4(0.5775)(19,943 \text{ psi})(89,360 \text{ psi})} \right] \\ &= 18,020 \text{ psi} \end{aligned}$$

Therefore, $F_e = 18,020 \text{ psi}$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{33,000}{18,020}} = 1.35 \leq 1.50 \quad \text{Eq. C4-4}$$

$$F_n = (0.658^{\lambda_c^2}) F_y \quad \text{Eq. C4-2}$$

$$F_n = (0.658^{(1.35)^2}) (33,000 \text{ psi}) = 15,333 \text{ psi}$$

Determine P_n based on F_n and effective area, A_e :

Web: (standard 1.5 in punchout, unstiffened compression element)

$d_h =$	1.5 in	Maximum per <i>Method</i>
Hole spacing =	24 in	Minimum per <i>Method</i>
w (each side of hole) = $d/2 - d_h/2 - (R + t)$		Flat part of web
$d =$	3.5 in	
$R =$	0.108 in	Table 3.1
$t =$	0.0566 in	54 mil material

$$w = 3.5 \text{ in}/2 - 1.5 \text{ in}/2 - (0.108 + 0.0566 \text{ in}) = 0.8354 \text{ in}$$

$$W = \frac{w}{t} = \frac{0.8354 \text{ in}}{0.0566 \text{ in}} = 14.76$$

$$k = 0.43 \quad \text{Section B3.1(a)}$$

$$F_{cr} = k \frac{\pi^2 E}{12(1 - \mu^2)} \left(\frac{t}{w} \right)^2 \quad \text{Eq. B2.1-5}$$

$$F_{cr} = 0.43 \frac{\pi^2 (29,500,000 \text{ psi})}{12(1 - 0.30^2)} \left(\frac{0.0566}{0.8354} \right)^2 = 52,627 \text{ psi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad \text{Eq. B2.1-4}$$

$$f = F_n = 15,333 \text{ psi}$$

See Above

$$\lambda = \sqrt{\frac{15,333}{52,627}} = 0.540 \leq 0.673$$

$$b = w \text{ when } \lambda \leq 0.673$$

Eq. B2.1-1

$$b_w = w = 0.8354 \text{ in}$$

Flanges: (stiffened compression element)

$$f = F_n = 15,333 \text{ psi}$$

See Above

$$w = 1.625 \text{ in} - 2(R + t)$$

Flat width of flange

$$R = 0.108 \text{ in}$$

$$t = 0.0566 \text{ in}$$

$$w = 1.625 \text{ in} - 2(0.108 + 0.0566 \text{ in}) = 1.2958 \text{ in}$$

$$W = \frac{w}{t} = \frac{1.2958 \text{ in}}{0.0566 \text{ in}} = 22.89$$

$$S = 1.28 \sqrt{\frac{E}{f}} = 1.28 \sqrt{\frac{29,500,000}{15,333}} = 56.14$$

Eq. B4-1

$$0.328S = 0.328(56.14) = 18.42 < W = 22.89, \text{ therefore use Case II, B4.2(a)}$$

$$d = 0.5 \text{ in} - (R + t)$$

Flat width of stiffener

$$d = 0.5 \text{ in} - (0.108 + 0.0566 \text{ in}) = 0.3354 \text{ in}$$

$$I_s = \frac{td^3}{12} = \frac{(0.0566 \text{ in})(0.3354 \text{ in})^3}{12} = 0.00017796 \text{ in}^4$$

$$I_{a1} = 399t^4 \left[\frac{w/t}{S} - 0.328 \right]^3 = 399(0.0566 \text{ in})^4 \left[\frac{22.89}{56.14} - 0.328 \right]^3 = 0.00000208 \text{ in}^4$$

$$I_{a2} = t^4 \left[115 \frac{w/t}{S} + 5 \right] = (0.0566 \text{ in})^4 \left[115 \frac{22.89}{56.14} + 5 \right] = 0.000533 \text{ in}^4$$

$$I_a = \text{Min}(I_{a1}, I_{a2}) = I_{a1} = 2.08 \times 10^{-6} \text{ in}^4$$

$$(R_l) = I_s/I_a \leq 1$$

Eq. B4.2-9

$$\frac{I_s}{I_a} = \frac{0.000178}{0.00000208} = 85.6 > 1.0$$

$$\text{Therefore, } (R_l) = 1.0$$

$$D = 0.5 \text{ in}$$

$$D/w = 0.5 \text{ in}/1.2958 \text{ in} = 0.386$$

$$k = \left(4.82 - \frac{5D}{w} \right) (R_l)^n + 0.43 \leq 4$$

Table B4.2

$$n = \left[0.582 - \frac{w/t}{4S} \right] \geq \frac{1}{3} = \left[0.582 - \frac{22.89}{4(56.14)} \right] = 0.480 \geq \frac{1}{3} \quad \text{Eq. B4.2-11}$$

$$k = \left(4.82 - \frac{5(0.5 \text{ in})}{1.2958 \text{ in}} \right) (1.0)^{0.480} + 0.43 = 3.32$$

$$F_{cr} = k \frac{\pi^2 E}{12(1-\mu^2)} \left(\frac{t}{w} \right)^2 \quad \text{Eq. B2.1-5}$$

$$F_{cr} = 3.32 \frac{\pi^2 (29,500,000 \text{ psi})}{12(1-0.30^2)} \left(\frac{0.0566}{1.2958} \right)^2 = 168,886 \text{ psi}$$

$$\lambda = \sqrt{\frac{15,333}{168,886}} = 0.301 \leq 0.673$$

Therefore, $b_f = w = 1.2958 \text{ in}$
Edge Stiffeners: (unstiffend compression element)

$$k = 0.43$$

$$w = d = 0.3354 \text{ in}$$

Section B3.1(a)
 See Above

$$F_{cr} = k \frac{\pi^2 E}{12(1-\mu^2)} \left(\frac{t}{w} \right)^2 \quad \text{Eq. B2.1-5}$$

$$F_{cr} = 0.43 \frac{\pi^2 (29,500,000 \text{ psi})}{12(1-0.30^2)} \left(\frac{0.0566}{0.3354} \right)^2 = 326,493 \text{ psi}$$

$$\lambda = \sqrt{\frac{15,333}{326,493}} = 0.217 \leq 0.673 - \text{Edge stiffener is fully effective}$$

$$d'_s = d = 0.3354 \text{ in}$$

$$d_s = d'_s(R_l) = 0.3354 \text{ in}$$

Length of Corners:

$$R = 0.108 \text{ in}$$

$$t = 0.0566 \text{ in}$$

$$r = R + t/2 = 0.108 + 0.0566 \text{ in}/2 = 0.1363 \text{ in}$$

$$u = \frac{\pi}{2} r = 1.57(0.1363 \text{ in}) = 0.2141 \text{ in}$$

Total Effective Area, A_e :

$$A_e = t \left[2b_w + 2(d_s + b_f) + 4u \right]$$

$$A_e = 0.0566 \text{ in} \left[2(0.8354 \text{ in}) + 2(0.3354 + 1.2958 \text{ in}) + 4(0.2141 \text{ in}) \right] = 0.3277 \text{ in}^2$$

Calculate P_n :

$$P_n = A_e F_n = (0.3277 \text{ in}^2)(15,333 \text{ psi}) = 5,025 \text{ lbs} \quad \text{Eq. C4-1}$$

$$P_{no} = A_e F_y = (0.3277 \text{ in}^2)(33,000 \text{ psi}) = 10,814 \text{ lbs}$$

Calculation of M_{nx} in accordance with Section C.3:

1. Nominal Section Strength (Section C3.1.1):

Use Procedure I:

$$M_n = S_e F_y \quad \text{Eq. C3.1.1-1}$$

Find Effective Section Modulus about x-axis, S_{ex} :

$$S_{ex} = S_x - S_{Hole}$$

$$S_x = 0.45577 \text{ in}^3$$

See Above

$$S_{Hole} = \frac{I_x}{c} = \frac{0.0566 \text{ in}(1.5 \text{ in})^3}{12(0.75 \text{ in})} = 0.02123 \text{ in}^3$$

$$S_{ex} = 0.45577 - 0.02123 \text{ in}^3 = 0.4345 \text{ in}^3$$

$$F_y = 33,000 \text{ psi}$$

$$M_n = (0.4345 \text{ in}^3)(33,000 \text{ psi})(1 \text{ ft}/12 \text{ in}) = 1,195 \text{ lb-ft}$$

2. Lateral-Torsional Buckling Strength (Section C3.1.2.1):

$$M_n = S_c F_c \quad \text{Eq. C3.1.2.1-1}$$

S_c = Elastic section modulus of effective section calculated relative to extreme compression fiber at F_c .

Find F_c :

For $F_e > 2.78F_y$

$$F_c = F_y \quad \text{Eq. C3.1.2.1-2}$$

For $2.78F_y > F_e > 0.56F_y$

$$F_c = \frac{10}{9} F_y \left(1 - \frac{10F_y}{36F_e} \right) \quad \text{Eq. C3.1.2.1-3}$$

For $F_e < 0.56F_y$

$$F_c = F_e \quad \text{Eq. C3.1.2.1-4}$$

F_e = Elastic critical lateral-torsional buckling stress calculated according to the following:

a) For singly-, doubly-, and point-symmetric sections:

$$F_e = \frac{C_b r_o A}{S_f} \sqrt{\sigma_{ey} \sigma_t} \quad \text{Eq. C3.1.2.1-5}$$

where

$C_b =$	1.0 (Conservative)
$r_o =$	1.9950 in
$A =$	0.4126 in ²
$S_f =$	0.45577 in ³

$$\sigma_{ey} = \frac{\pi^2 E}{(K_y L_y / r_y)^2} \quad \text{Eq. C3.1.2.1-8}$$

$$(K_y L_y / r_y) = 79.44 \quad \text{See } P_n \text{ Calc. Above}$$

$$\sigma_{ey} = \frac{\pi^2 (29,500,000 \text{ psi})}{(79.44)^2} = 46,136 \text{ psi}$$

$$\sigma_t = \frac{1}{A r_0^2} \left[GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2} \right] \quad \text{Eq. C3.1.2.1-9}$$

$$\sigma_t = 89,360 \text{ psi} \quad \text{See } P_n \text{ Calc. Above}$$

$$F_e = \frac{(1.0)(1.9950 \text{ in})(0.4126 \text{ in}^2)}{0.45577 \text{ in}^2} \sqrt{(46,136 \text{ psi})(89,360 \text{ psi})} = 115,963 \text{ psi}$$

$$2.78 F_y = 2.78(33,000 \text{ psi}) = 91,740 \text{ psi} < F_e = 115,963 \text{ psi}$$

$$\text{Therefore, } F_c = F_y = 33,000 \text{ psi} \quad \text{Eq. C3.1.2.1-2}$$

$$S_c = S_{ex} = 0.4345 \text{ in}^3$$

$$M_n = S_c F_c = (0.4345 \text{ in}^3)(33,000 \text{ psi})(1 \text{ ft}/12 \text{ in}) = 1,195 \text{ lb-ft}$$

Check Interaction Equations for Load Combinations:

Load Combination 3:

Axial Load, $P_u = 359 \text{ lbs}$

Lateral Load, $M_{ux} = 0$

$$\frac{P_u}{\phi_c P_n} = \frac{359 \text{ lbs}}{0.85(5,025 \text{ lbs})} = 0.084 \leq 0.15, \text{ Therefore use Equation C5.2.2-3:}$$

$$\frac{359 \text{ lbs}}{0.85(5,025 \text{ lbs})} = 0.084 \leq 1.0 - \text{OK!} \quad \text{Eq. C5.2.2-3}$$

From CFS, Equation Result = 0.098 $\approx$ 0.084 – OK !

Load Combination 4:

Axial Load, $P_u = 359 \text{ lbs}$

$$\text{Lateral Load, } M_{ux} = \frac{(10.7 \text{ plf})(14 \text{ ft})^2}{8} = 262.2 \text{ lb-ft}$$

$$\frac{P_u}{\phi_c P_n} = \frac{359 \text{ lbs}}{0.85(5,025 \text{ lbs})} = 0.084 \leq 0.15, \text{ Therefore use Equation C5.2.2-3:}$$

$$\frac{359 \text{ lbs}}{0.85(5,025 \text{ lbs})} + \frac{(262.2 \text{ lb} - \text{ft})}{0.95(1,195 \text{ lb} - \text{ft})} = 0.315 \leq 1.0 - \text{OK !} \quad \text{Eq. C5.2.2-3}$$

From CFS, Equation Result = 0.338 $\approx$ 0.315 – OK !

Load Combination 5:

Axial Load, $P_u = 159.1 \text{ lbs}$

$$\text{Lateral Load, } M_{ux} = \frac{(21.3 \text{ plf})(14 \text{ ft})^2}{8} = 521.9 \text{ lb} - \text{ft}$$

$$\frac{P_u}{\phi_c P_n} = \frac{159.1 \text{ lbs}}{0.85(5,025 \text{ lbs})} = 0.037 \leq 0.15, \text{ Therefore use Equation C5.2.2-3:}$$

$$\frac{159.1 \text{ lbs}}{0.85(5,025 \text{ lbs})} + \frac{(521.9 \text{ lb} - \text{ft})}{0.95(1,195 \text{ lb} - \text{ft})} = 0.497 \leq 1.0 - \text{OK !} \quad \text{Eq. C5.2.2-3}$$

From CFS, Equation Result = 0.521 $\approx$ 0.497 – OK !

Load Combination 6:

Axial Load, $P_u = 0 \text{ lbs}$

$$\text{Lateral Load, } M_{ux} = \frac{(39.9 \text{ plf})(14 \text{ ft})^2}{8} = 977.6 \text{ lb} - \text{ft}$$

$$\frac{P_u}{\phi_c P_n} = 0 \leq 0.15, \text{ Therefore use Equation C5.2.2-3:}$$

$$\frac{(977.6 \text{ lb} - \text{ft})}{0.95(1,195 \text{ lb} - \text{ft})} = 0.86 \leq 1.0 - \text{OK !} \quad \text{Eq. C5.2.2-3}$$

From CFS, Equation Result = 0.895 $\approx$ 0.86 – OK !

Check Mid-height Deflection:

Load Combination 7:

Axial Load, $P_u = 0 \text{ lbs}$

Lateral Load, $w = 70\%$ of C&C = 17.5 plf

$$\delta = \frac{5wL^4}{384EI_{xe}}$$

where

$L = \text{Span} = 14 \text{ ft}$

$I_x = 0.7976 \text{ in}^4$

$$\delta = \frac{5(17.5 \text{ plf})(14 \text{ ft})^4}{384(29,500,000 \text{ psi})(0.7976 \text{ in}^4)} \times \frac{1728 \text{ in}^3}{\text{ft}^3} = 0.64 \text{ in} \leq \frac{h}{240} = \frac{(14 \text{ ft})(12)}{240} = 0.70 \text{ in} - \text{OK !}$$

Check Web Crippling:

Verify capacity stud web against web crippling at end of stud using Equation C4.2-1 from the AISI Wall Stud Standard (2004) – Public Review Copy (AISI, 2004b):

$$\phi P_{nst} = Ct^2 F_y \left(1 - C_R \sqrt{\frac{R}{t}} \right) \left(1 + C_N \sqrt{\frac{N}{t}} \right) \left(1 - C_h \sqrt{\frac{h}{t}} \right) \quad (\text{Eq. C4.2-1})$$

$$\phi P_{nst} \geq P_u$$

where

P_{nst} = Nominal Crippling Strength

P_u = Reaction at end of tallest wall stud due to lateral load in combination 6

C = Web Crippling Coefficient = 3.7

C_R = Inside Bend Radius Coefficient = 0.19

C_N = Bearing Length Coefficient = 0.74

C_h = Web Slenderness Coefficient = 0.019

R = Stud Inside Bend Radius

N = Stud Bearing Length

h = Depth of Flat Portion of Stud Web Measured along Plane of Web

t = Stud Design Thickness

ϕ = 0.90

For a 350S162-54 section:

R = 0.108 in

N = 1.125 in

t = 0.0566 in

h = 3.5 in – 2(0.108 + 0.0566 in) = 3.1708 in

Table 3.1 - 54 mil

Assume 1.25" track section

54 mil stud

$$\phi P_{nst} = 0.90 (3.7) (0.0566)^2 (33,000) \left(1 - 0.19 \sqrt{\frac{0.108}{0.0566}} \right) \left(1 + 0.74 \sqrt{\frac{1.125}{0.0566}} \right) \left(1 - 0.019 \sqrt{\frac{3.1708}{0.0566}} \right)$$

$$\phi P_{nst} = 957.5 \text{ lbs}$$

$$P_u = (39.9 \text{ plf})(14 \text{ ft}/2) = 279.3 \text{ lbs}$$

$$\phi P_{nst} = 957.5 \text{ lbs} \geq P_u = 279.3 \text{ lbs} - \text{OK !}$$

Therefore, a 350S162-54 section is adequate, and CFS results have been verified
Check next thinner size, 350S162-43, using CFS 4.13:

Load Combination 3:

Axial Load, P_u = 359 lbs

Lateral Load, M_{ux} = 0

From CFS, Equation Eq. C5.2.2-1 Result = $0.122 \leq 1.0$ – OK !

From CFS, Equation Eq. C5.2.2-2 Result = $0.052 \leq 1.0$ – OK !

Load Combination 4:

Axial Load, $P_u = 359$ lbs

Lateral Load, $M_{ux} = 262.2$ lb-ft

From CFS, Equation Eq. C5.2.2-1 Result = $0.434 \leq 1.0$ – OK !

From CFS, Equation Eq. C5.2.2-2 Result = $0.347 \leq 1.0$ – OK !

Load Combination 5:

Axial Load, $P_u = 159.1$ lbs

Lateral Load, $M_{ux} = 521.9$ lb-ft

From CFS, Equation Eq. C5.2.2-1 Result = $0.656 \leq 1.0$ – OK !

From CFS, Equation Eq. C5.2.2-2 Result = $0.610 \leq 1.0$ – OK !

Load Combination 6:

Axial Load, $P_u = 0$ lbs

Lateral Load, $M_{ux} = 977.6$ lb-ft

From CFS, Equation Eq. C5.2.2-3 Result = $1.100 > 1.0$ – NG !

Therefore, a 350S162-43 section is not adequate for this gable end wall, and a 350S162-54 section should be selected, as indicated in Table 2.1.

A1.3 Required Lengths for Ceiling Diaphragms (Tables 2.3 – 2.6)

A1.3.1 Gypsum Board Ceiling Diaphragms (Tables 2.3 – 2.4)

Design Objective:

Determine the minimum required length for a gypsum board ceiling diaphragm as illustrated in Figure A1.6. All references are to the *North American Specification for the Design of Cold-Formed Steel Structural Members*, unless otherwise noted (AISI, 2001)

Design Assumptions:

Building Width = 28 ft
Steel Yield Strength, F_y = 33 ksi
Roof Pitch = 4:12
Wind Speed = 90 mph
Wind Exposure Category = B
Ceiling Height = 8 ft
Max. Aspect Ratio = 2:1

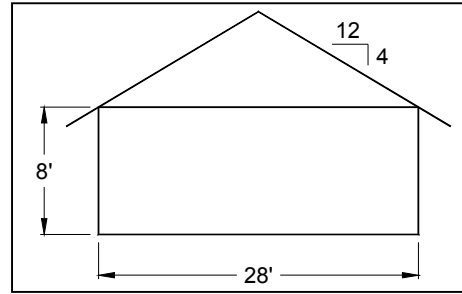


Figure A1.6 - Gable End Wall Elevation

Design Approach:

Find the required gypsum board ceiling diaphragm length to provide lateral support for the gable end wall studs.

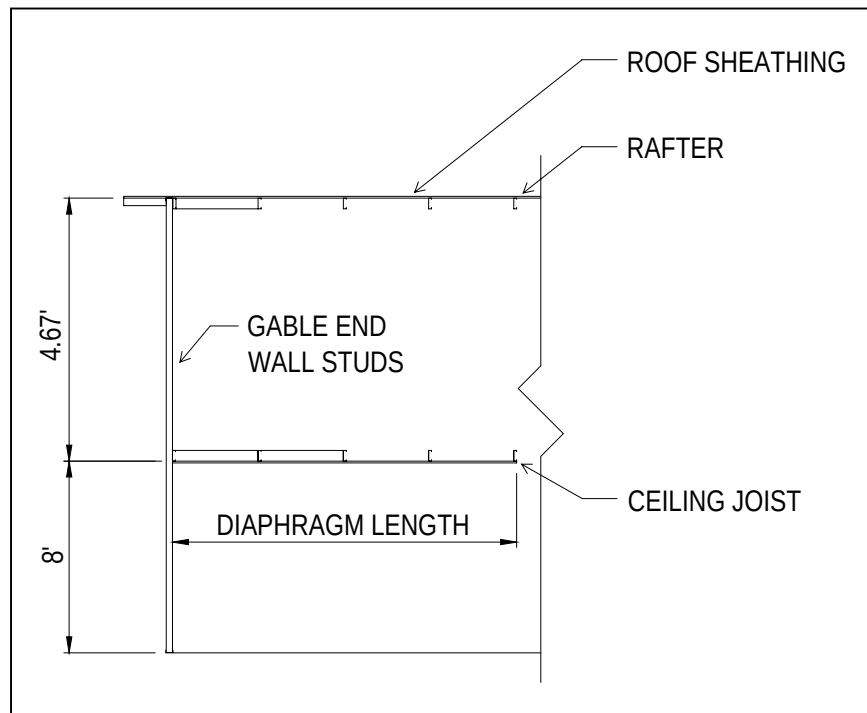


Figure A1.7 - Diaphragm Section

Solution:

Wall Height, $H = 8 + (14 \text{ ft})(4/12) = 12.67 \text{ ft}$
 Stud Height, $h = 8 \text{ ft}$

Wind Loads:

Wind loads are calculated in accordance with ASCE 7-2002 Method 2 for windward Components and Cladding loads (ASCE, 2002).

$$\text{Wind Area} = (8 \text{ ft}/2)(28 \text{ ft}) + [0.5(28 \text{ ft})(12.67 - 8)/2] = 144.67 \text{ ft}^2$$

Equation:

$$p = qGC_p - q_i(GC_{pi})$$

$$q = q_i = q_h = 0.00256K_zK_{zt}K_dV^2I \text{ (lb/ft}^2\text{)}$$

Comments:

Building of all Heights

Reference:

6.5.12.2, Eq. 6-17

6.5.10, Eq 6-15

Where:

$$K_z = 0.70$$

Exp. B, Case 1, $z = 15 \text{ ft}$

Table 6-3

$$K_{zt} = 1.0$$

No Topographic Factor

$$K_d = 0.85$$

MWFRS

Table 6-4

$$V = 90 \text{ mph}$$

3-Second Gust Wind Speed

$$I = 1.0$$

Residential Building (Cat I)

Table 1-1, Table 6-1

$$G = 0.85$$

Gust Effect Factor

6.5.8.1

$$C_p = 0.8$$

Windward

Figure 6-6

$$C_p = -0.5$$

Leeward

Figure 6-6

$$GC_{pi} = \pm 0.18$$

Enclosed Building

Figure 6-5

$$q_h = 0.00256(0.70)(1.0)(0.85)(90)^2(1.0) = 12.34 \text{ lb/ft}^2$$

$$p_{ww} = (12.34 \text{ lb/ft}^2)(0.85)(0.8) - 12.34(-0.18) = 10.61 \text{ lb/ft}^2$$

$$p_{LW} = (12.34 \text{ lb/ft}^2)(0.85)(-0.5) - 12.34(-0.18) = -3.02 \text{ lb/ft}^2$$

Total Horizontal Shear:

$$p = 10.61 + |-3.02| = 13.6 \text{ lb/ft}^2$$

Load Combinations:

1. 1.6W

Using MWFRS for W

Design Loads:

$$\text{Horizontal Diaphragm Force} = 13.6 \text{ psf}(144.67 \text{ ft}^2) = 1,968 \text{ lbs}$$

$$\text{Force to each side wall} = 1,968 \text{ lbs}/2 = 984 \text{ lbs}$$

$$P_u = 1.6(984 \text{ lbs}) = 1,574 \text{ lbs}$$

Determine Required Diaphragm Length:

Diaphragm capacity based on AISI Lateral Design Standard (AISI, 2004b):

For ½" gypsum board shear wall with screws @ 6" o.c. in field, 12" @ edges (blocked):
R_n = 262.5 plf by interpolation Table C2.1-2

Reduce for unblocked diaphragm per Table C2.1-2 footnote 3:

$$R_n = 0.35(262.5 \text{ plf}) = 91.88 \text{ plf}$$

$$\phi = 0.65$$

$$\phi R_n = 0.65(91.88 \text{ plf}) = 59.7 \text{ plf}$$

$$\text{Total Required Length} = \frac{P_u}{\phi R_n} = \frac{1574 \text{ lbs}}{59.7 \text{ plf}} = 26.4 \text{ ft}$$

$$\text{Required length at each end} = 26.4 \text{ ft}/2 = 13.2 \text{ ft}$$

$$\text{Aspect Ratio} = \text{Width/Length} = 28 \text{ ft}/13.2 \text{ ft} = 2.12 > 2 \text{ (Maximum)} - \text{N.G.}!$$

$$\text{Round up to nearest 2 ft length} = \mathbf{14 \text{ ft}}$$

$$\text{Aspect Ratio} = \text{Width/Length} = 28 \text{ ft}/14 \text{ ft} = 2.0 = 2 \text{ (Maximum)} - \text{O.K.}!$$

Therefore, diaphragm must be constructed over a minimum length of 14 ft at each end of the structure as indicated in Table 2.3.

A1.3.2 Wood Structural Panel Sheathed Ceiling Diaphragms (Tables 2.5 – 2.6)

Design Objective:

Determine the minimum required length for a wood sheathed ceiling diaphragm as illustrated in Figure A1.8. All references are to the *North American Specification for the Design of Cold-Formed Steel Structural Members*, unless otherwise noted (AISI, 2001)

Design Assumptions:

Building Width = 28 ft
Steel Yield Strength, F_y = 33 ksi
Roof Pitch = 4:12
Wind Speed = 90 mph
Wind Exposure Category = B
Ceiling Height = 10 ft
Max. Aspect Ratio = 3:1

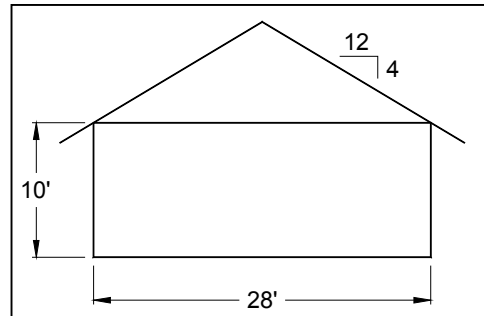


Figure A1.8 - Gable End Wall Elevation

Design Approach:

Find the required wood sheathed ceiling diaphragm length to provide lateral support for the gable end wall studs.

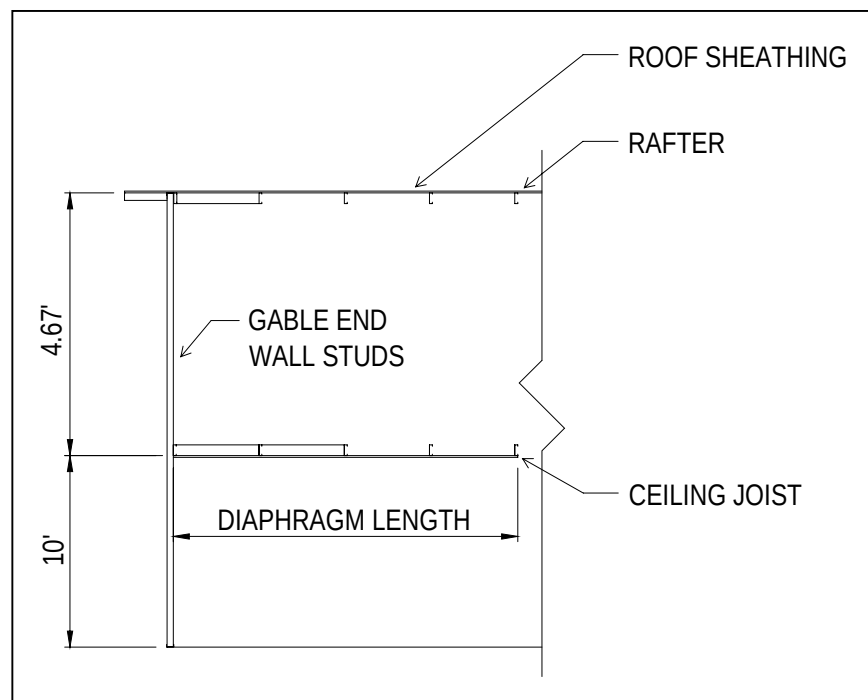


Figure A1.9 - Diaphragm Section

Solution:

Per Table 3.8, must assume 6:12 roof pitch for calculations:

$$\text{Wall Height, } H = 10 + (14 \text{ ft})(4/12) = 14.67 \text{ ft}$$

$$\text{Stud Height, } h = 10 \text{ ft}$$

Wind Loads:

Wind loads are calculated in accordance with ASCE 7-2002 Method 2 for windward Components and Cladding loads (ASCE, 2002).

$$\text{Wind Area} = (10 \text{ ft}/2)(28 \text{ ft}) + [0.5(28 \text{ ft})(14.67 - 10)/2] = 172.67 \text{ ft}^2$$

Equation:

$$p = qGC_p - q_i(GC_{pi})$$

$$q = q_i = q_h = 0.00256K_zK_{zt}K_dV^2I \text{ (lb/ft}^2\text{)}$$

Comments:

Buildings of All Heights

Reference:

6.5.12.2, Eq. 6-17

6.5.10, Eq 6-15

Where:

$$K_z = 0.70$$

$$K_{zt} = 1.0$$

$$K_d = 0.85$$

$$V = 90 \text{ mph}$$

$$I = 1.0$$

$$G = 0.85$$

$$C_p = 0.8$$

$$C_p = -0.5$$

$$GC_{pi} = \pm 0.18$$

Exp. B, Case 1, $z = 14.67 \text{ ft}$

No Topographic Factor

MWFRS

3-Second Gust Wind Speed

Residential Building (Cat I)

Gust Effect Factor

Windward

Leeward

Enclosed Building

Table 6-3

Table 6-4

Table 1-1, Table 6-1

6.5.8.1

Figure 6-6

Figure 6-6

Figure 6-5

$$q_h = 0.00256(0.70)(1.0)(0.85)(90)^2(1.0) = 12.34 \text{ lb/ft}^2$$

$$p_{ww} = (12.34 \text{ lb/ft}^2)(0.85)(0.8) - 12.34(-0.18) = 10.61 \text{ lb/ft}^2$$

$$p_{Lw} = (12.34 \text{ lb/ft}^2)(0.85)(-0.5) - 12.34(-0.18) = -3.02 \text{ lb/ft}^2$$

Total Horizontal Shear:

$$p = 10.61 + |-3.02| = 13.6 \text{ lb/ft}^2$$

Load Combinations:

1. 1.6W

Using MWFRS for W

Design Loads:

$$\text{Horizontal Diaphragm Force} = 13.6 \text{ psf}(172.67 \text{ ft}^2) = 2,348.3 \text{ lbs}$$

$$\text{Force to each side wall} = 2,348.2 \text{ lbs}/2 = 1,174 \text{ lbs}$$

$$P_u = 1.6(1,174 \text{ lbs}) = 1,879 \text{ lbs}$$

Determine Required Diaphragm Length:

Diaphragm capacity based on AISI Lateral Design Standard (AISI, 2004b):

For unblocked diaphragm with 3/8" sheathing and load perp. to unblocked edges:

$R_n = 615 \text{ plf}$

Table D2-1

$\phi = 0.65$

$$\phi R_n = 0.65(615 \text{ plf}) = 399.8 \text{ plf}$$

$$\text{Required Length} = \frac{P_u}{\phi R_n} = \frac{1879 \text{ lbs}}{399.8 \text{ plf}} = 4.70 \text{ ft}$$

Aspect Ratio = Width/Length = 28 ft/4.70 ft = 6.0 > 3 (Maximum) – O.K.!

Maximum Length = 28 ft/3 = 9.33 ft – Round up to nearest 2 ft length = **10 ft**

Therefore, diaphragm must be constructed over a minimum length of 10 ft as indicated in Table 2.6.

A2 Openings in Gable End Walls (Section 3.0)

A2.1 Maximum Head Track Span Calculation (Tables 3.2 - 3.6)

Design Objective:

Determine the maximum span, L , for a 550T125-43 head track in the gable end wall shown below. All references are to the *North American Specification for the Design of Cold-Formed Steel Structural Members* unless otherwise noted (AISI, 2001).

Design Assumptions:

Building Width =	30 ft
Stories =	2
Roof Overhang (Rake) =	1 ft
Stud Spacing =	24 in
Stud Size =	550S162
Steel Yield Strength, F_y =	33 ksi
Roof Pitch =	8:12
Wind Speed =	120 mph
Wind Exposure Category =	B
Ceiling Height =	10 ft
Deflection Limit =	$L/240$

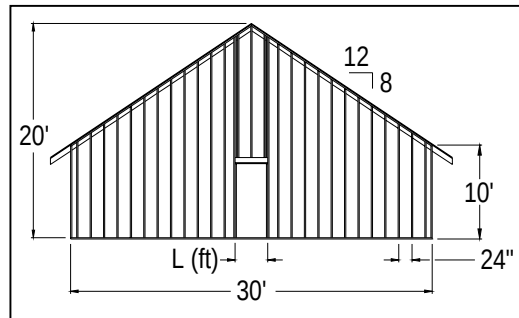


Figure A2.1 - Gable End Wall Elevation

Design Approach:

Determine the maximum head track span for the above conditions. Track resists wind forces in bending about its strong axis, and is assumed to be simply supported at ends. The wall studs are assumed to support the track against twisting and weak axis bending.

Solution:

Stud Height, $h = 10 + (15 \text{ ft})(8/12) =$	20 ft
Must use 22 ft Table, so $h =$	22 ft
Track Flange Support Spacing = Stud Spacing =	24 in
Effective Width = $(22 \text{ ft}/2) =$	11 ft
Roof Height, $z =$	20 ft

Design Loads:

Wind Loads:

Wind loads are calculated in accordance with ASCE 7-2002 Method 2 for Components and Cladding (C&C) (ASCE, 2002).

Equation:

$$p = q_h \left[(GC_p) - (GC_{pi}) \right]$$
$$q_h = 0.00256 K_z K_{zt} K_d V^2 I \quad (\text{lb/ft}^2)$$

Comments:

Low-Rise Building

Reference:

6.5.12.4, Eq. 6-22

6.5.10, Eq 6-15

Where:

$K_z = 0.70$	Exp. B, Case 1, $z = 20$ ft	Table 6-3
$K_{zt} = 1.0$	No Topographic Factor	
$K_d = 0.85$		Table 6-4
$V = 120$ mph	3-Second Gust Wind Speed	
$I = 1.0$	Residential Building (Cat I)	Table 1-1, Table 6-1
$GC_p = 0.95$ or -1.05	Assume Wind Area ≥ 20 ft ²	Figure 6-11A
$GC_{pi} = \pm 0.18$	Enclosed Building	Figure 6-5

$$q_h = 0.00256(0.70)(1.0)(0.85)(120)^2(1.0) = 21.93 \text{ (lb/ft}^2\text{)}$$

$$p^+ = (21.93 \text{ lb/ft}^2) [(0.95) - (-0.18)] = 24.78 \text{ lb/ft}^2$$

$$p^- = (21.93 \text{ lb/ft}^2) [(-1.05) - (0.18)] = -26.97 \text{ lb/ft}^2$$

Controls

Load Combinations:

- | | |
|---------|------------------------------------|
| 1. 1.6W | Using C&C for W – Bending Check |
| 2. 0.7W | Using C&C for W – Deflection Check |

Design Loads:

Lateral Loads:

- | | |
|----|---|
| 1. | $1.6(26.97 \text{ psf})(11 \text{ ft}) = 474.7 \text{ plf}$ |
| 2. | $0.7(26.97 \text{ psf})(11 \text{ ft}) = 207.7 \text{ plf}$ |

Material Properties:

$F_y = 33,000$ psi
 $E = 29,500,000$ psi
 $G = 11,300$ psi
 $\mu = 0.30$ (Poisson's Ratio)

Full Section Properties:

Select a 550T125-43 track section from Table 3.4 (round height up to 22 ft)
 Section property calculations based on CFS 4.13
 Refer to full height stud calculation in Section A1.2 for verification of CFS 4.13 for section property calculations.

$A = 0.3588 \text{ in}^2$
 $I_x = 1.4929 \text{ in}^4$
 $\bar{x} = 0.5445 \text{ in}$
 $x_o = -0.5458 \text{ in}$
 $I_y = 0.0428 \text{ in}^4$
 $S_x = 0.52885 \text{ in}^3$
 $S_y(r) = 0.04119 \text{ in}^3$
 $S_y(l) = 0.20155 \text{ in}^3$

$$\begin{aligned}
r_x &= 2.0398 \text{ in} \\
r_y &= 0.3452 \text{ in} \\
r_o &= 2.1396 \text{ in} \\
J &= 0.0002433 \text{ in}^4 \\
C_w &= 0.24446 \text{ in}^6
\end{aligned}$$

Bending Capacity:

Calculate nominal flexural strength, M_n , per *Specification* Section C3.1:

Calculation of M_{nx} in accordance with Section C.3:

Need Effective Area, A_e :

Web: (no punchouts, stiffened element under stress gradient– Section B2.3)

Assuming web in both tension and compression, use section B2.3(a)(i):

For Compression Portion:

$$k = 4 + 2(1 + \psi)^3 + 2(1 + \psi) \quad \text{Eq. B2.3-2}$$

$$\psi = \left| \frac{f_2}{f_1} \right| = 1.0 \quad f_1 = f_2 \text{ (uniform load on symm. Section)} \quad \text{Eq. B2.3-1}$$

$$k = 4 + 2(1 + 1)^3 + 2(1 + 1) = 24 \approx 23.9 \text{ from Table C-B2-1}$$

Use $k = 23.9$

$$h_o = 5.5 \text{ in}$$

$$b_o = 1.25 \text{ in}$$

$$h_o / b_o = 5.50 / 1.25 = 4.4 > 4$$

Figure B2.3-2

Figure B2.3-2

$$b_1 = \frac{b_e}{(3 + \psi)} \quad \text{Eq. B2.3-6}$$

$$b_2 = \frac{b_e}{(1 + \psi)} - b_1 \quad \text{Eq. B2.3-7}$$

b_e = Effective width per Section B2.1 with $f = f_1$ and $k = 24$:

$$b_e = w \quad \text{for } \lambda \leq 0.673 \quad \text{Eq. B2.1-1}$$

$$b_e = \rho w \quad \text{for } \lambda > 0.673 \quad \text{Eq. B2.1-2}$$

$$w = d - 2(R + t)$$

$$d = 5.50 \text{ in}$$

$$R = 0.09375 \text{ in}$$

$$t = 0.0451 \text{ in}$$

Flat portion

Table 3.1

43 mil material

$$w = 5.50 \text{ in} - 2(0.09375 + 0.0451 \text{ in}) = 5.2223 \text{ in}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad \text{Eq. B2.1-4}$$

$$F_{cr} = k \frac{\pi^2 E}{12(1-\mu^2)} \left(\frac{t}{w} \right)^2 \quad \text{Eq. B2.1-5}$$

$$F_{cr} = 23.9 \frac{\pi^2 (29,500,000 \text{ psi})}{12(1-0.30^2)} \left(\frac{0.0451}{5.2223} \right)^2 = 47,525 \text{ psi}$$

$$f = f_1 = \frac{Mc}{I_x} \quad \text{Flexural stress}$$

Assume a 5 ft head track span (verified later):

$$M = \frac{w_u l^2}{8} = \frac{(474.7 \text{ plf})(5 \text{ ft})^2}{8} = (1483.44 \text{ lb-ft}) \left(\frac{12 \text{ in}}{\text{ft}} \right) = 17,801 \text{ lb-in}$$

$$c = 2.6115 \text{ in}$$

$$I_x = 1.4929 \text{ in}^4$$

$$f_1 = \frac{(17,801 \text{ lb-in})(2.6115 \text{ in})}{1.4929 \text{ in}^4} = 31,139 \text{ psi}$$

$$\lambda = \sqrt{\frac{31,139}{47,525}} = 0.809 > 0.673$$

$$b = \rho w \quad \text{Eq. B2.1-2}$$

$$\rho = \frac{(1 - 0.22/\lambda)}{\lambda} = \frac{(1 - 0.22/0.809)}{0.809} = 0.900$$

$$\text{Effective width, } b_e = 0.9(5.2223 \text{ in}) = 4.6982 \text{ in}$$

$$b_1 = \frac{4.6982 \text{ in}}{(3+1)} = 1.1745 \text{ in}$$

$$b_2 = \frac{4.6982 \text{ in}}{(1+1)} - 1.1745 \text{ in} = 1.1745 \text{ in}$$

$$\text{Total effective compressive width, } w_c = b_1 + b_2 = 2(1.1745 \text{ in}) = 2.3491 \text{ in}$$

$$\text{Effective tensile width, } w_t = w/2 = 2.6115 \text{ in}$$

Top Flange: (unstiffend compression element)

$$k = 0.43 \quad \text{Section B3.1(a)}$$

$$w = b_0 - (R + t)$$

$$w = 1.25 \text{ in} - (0.09375 + 0.0451 \text{ in}) = 1.1112 \text{ in}$$

$$F_{cr} = k \frac{\pi^2 E}{12(1-\mu^2)} \left(\frac{t}{w} \right)^2 \quad \text{Eq. B2.1-5}$$

$$F_{cr} = 0.43 \frac{\pi^2 (29,500,000 \text{ psi}) \left(\frac{0.0451}{1.1112} \right)^2}{12(1-0.30^2)} = 18,888 \text{ psi}$$

For Section C3.1.1, $f = F_y = 33,000 \text{ psi}$

$$\lambda = \sqrt{\frac{33,000}{18,888}} = 1.747 > 0.673$$

$$b = \rho w$$

Eq. B2.1-2

$$\rho = \frac{\left(1 - \frac{0.22}{\lambda}\right)}{\lambda} = \frac{\left(1 - \frac{0.22}{1.747}\right)}{1.747} = 0.500$$

Effective compression flange width, $b_{fc} = 0.5(1.1112 \text{ in}) = 0.5556 \text{ in}$

For Section C3.1.2, $f = F_c$

Find F_c :

For $F_e > 2.78F_y$

$$F_c = F_y$$

Eq. C3.1.2.1-2

For $2.78F_y > F_e > 0.56F_y$

$$F_c = \frac{10}{9} F_y \left(1 - \frac{10F_y}{36F_e} \right)$$

Eq. C3.1.2.1-3

For $F_e < 0.56F_y$

$$F_c = F_e$$

Eq. C3.1.2.1-4

F_e = Elastic critical lateral-torsional buckling stress calculated according to the following:

a) For singly-, doubly, and point-symmetric sections:

$$F_e = \frac{C_b r_o A}{S_f} \sqrt{\sigma_{ey} \sigma_t}$$

Eq. C3.1.2.1-5

where

$$C_b = 1.0 \text{ (Conservative)}$$

$$r_o = 2.1396 \text{ in}$$

$$A = 0.3588 \text{ in}^2$$

$$S_f = 0.52885 \text{ in}^3$$

$$\sigma_{ey} = \frac{\pi^2 E}{\left(K_y L_y / r_y \right)^2}$$

Eq. C3.1.2.1-8

$$K_y = 1.0 \text{ (Pinned)}$$

$$L_y = \text{Stud Spacing} = 24 \text{ in}$$

$$r_y = 0.3452 \text{ in}$$

$$\frac{K_y L_y}{r_y} = \frac{1.0(24)}{0.3452} = 69.52 \leq 200 - \text{OK !}$$

$$\sigma_{ey} = \frac{\pi^2(29,500,000 \text{ psi})}{(69.52)^2} = 60,234 \text{ psi}$$

$$\sigma_t = \frac{1}{Ar_0^2} \left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2} \right] \quad \text{Eq. C3.1.2.1-9}$$

$$\sigma_t = \frac{1}{(0.3588)(2.1396)^2} \left[(11,300,000)(0.0002433) + \frac{\pi^2(29,500,000)(0.24446)}{(1.0 \times 24)^2} \right] = 76,903 \text{ psi}$$

$$F_e = \frac{(1.0)(2.1396 \text{ in})(0.3588 \text{ in}^2)}{0.52885 \text{ in}^3} \sqrt{(60,234 \text{ psi})(76,903 \text{ psi})} = 98,800 \text{ psi}$$

$$2.78F_y = 2.78(33,000 \text{ psi}) = 91,740 \text{ psi} < F_e = 98,800 \text{ psi}$$

$$\text{Therefore, } F_c = F_y = 33,000 \text{ psi}$$

$$\text{Eq. C3.1.2.1-2}$$

$$\text{Effective compression flange width, } b_{fc} = 0.5(1.1112 \text{ in}) = 0.5556 \text{ in}$$

See Above

$$\text{Effective tensile flange width, } b_{ft} = w = 1.1112 \text{ in}$$

Therefore, A_e is same for both Section C3.1.1 and C3.1.2.

Length of Corners:

$$R = 0.09375 \text{ in}$$

$$t = 0.0451 \text{ in}$$

$$r = R + t/2 = 0.09375 + 0.0451 \text{ in}/2 = 0.1163 \text{ in}$$

$$u = \frac{\pi}{2} r = 1.57(0.1163 \text{ in}) = 0.1827 \text{ in}$$

Total Effective Area, A_e :

$$A_e = t [b_{fc} + w_c + w_t + b_{ft} + 2u]$$

$$A_e = 0.0451 \text{ in} [0.5556 + 2.3491 + 2.6115 + 1.1112 + 2(0.1827 \text{ in})] = 0.3154 \text{ in}^2$$

1. Nominal Section Strength (Procedure 1 - Section C3.1.1):

$$M_n = S_e F_y$$

$$\text{Eq. C3.1.1-1}$$

Find S_e :

Need I_{xe} of effective section (refer to Figure A3.1.2):

For simplicity, consider all elements as lines and neglect corner bend radius'.

Find Location of N.A.:

Element:	A (in ²)	y (in)	Ay (in ³)
1	0.03030	5.45490	0.16528
2	0.05822	4.80950	0.28000
3	0.17599	1.95115	0.34338
4	0.05536	0	0
ΣA =	0.31987 in ²		ΣAy = 0.78866 in ³

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{0.78866 \text{ in}^3}{0.31987 \text{ in}^2} = 2.46557 \text{ in}$$

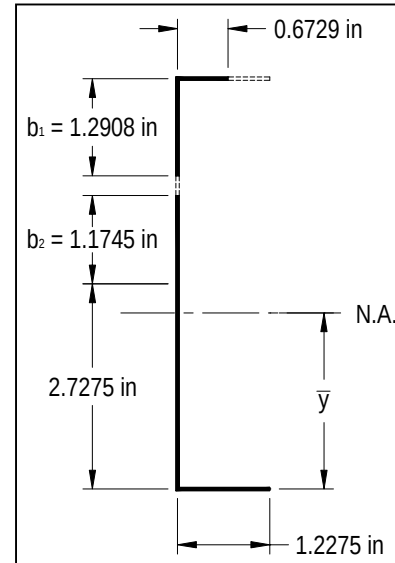


Figure A3.2 – Effective Track Section

Find I_{xe} :

Element:	I_{xx} (in ⁴)	A (in ²)	d (in)	Ad^2 (in ⁴)
1	0	0.03030	2.98933	0.27076
2	0.00808	0.05822	2.34393	0.31986
3	0.22334	0.17599	0.51442	0.04657
4	0	0.05536	2.46557	0.33654
Total I_{xx} =	0.23142 in ⁴		ΣAd ² =	0.97373 in ⁴

$$I_{xe} = I_{xx} + \Sigma Ad^2 = 0.23142 + 0.97373 \text{ in}^4 = 1.2052 \text{ in}^4$$

$$c_{top} = 5.50 - 2.46557 + 0.0451 \text{ in}/2 = 3.05698 \text{ in}$$

$$S_e = \frac{I_{xe}}{c} = \frac{1.2052 \text{ in}^4}{3.05698 \text{ in}} = 0.3942 \text{ in}^3$$

$$F_y = 33,000 \text{ psi}$$

$$M_n = (0.3942 \text{ in}^3)(33,000 \text{ psi})(1 \text{ ft}/12 \text{ in}) = 1084.2 \text{ lb-ft}$$

2. Lateral-Torsional Buckling Strength (Section C3.1.2.1):

$$M_n = S_c F_c \quad \text{Eq. C3.1.2.1-1}$$

S_c = Elastic section modulus of effective section calculated relative to extreme compression fiber at F_c .

$$F_c = F_y = 33,000 \text{ psi} \quad \text{See Above}$$

$$S_c = \frac{I_{xe}}{c}$$

$$c_{top} = 5.50 - 2.46557 + 0.0451 \text{ in}/2 = 3.05698 \text{ in}$$

$$S_e = \frac{I_{xe}}{c} = \frac{1.2052 \text{ in}^4}{3.05698 \text{ in}} = 0.3942 \text{ in}^3$$

$$M_n = S_e F_c = (0.3942 \text{ in}^3)(33,000 \text{ psi})(1 \text{ ft}/12 \text{ in}) = 1084.2 \text{ lb-ft}$$

Therefore, $M_n = 1084.2 \text{ lb-ft}$

Find Maximum Span, L, for Load Combinations:

Load Combination 1:

$$M_{ux} \leq \phi_b M_n$$

$$\phi_b = 0.90$$

$$M_n = 1084.2 \text{ lb-ft}$$

$$M_{ux} = \frac{w_u L^2}{8}$$

$$w_u = 474.7 \text{ plf}$$

$$L = \sqrt{\frac{8\phi_b M_n}{w_u}} = \sqrt{\frac{8(0.90)(1084.2 \text{ lb-ft})}{474.7 \text{ plf}}} = 4.055 \text{ ft}$$

Therefore, estimate of 5 ft span for calculation of $f_1 = \text{OK} !$

C3.1.1 - Unstiff. Flgs.

See Above

Simply Supported

Uniform Load

Load Combination 2:

Axial Load, $P_u = 744 \text{ lbs}$

Load Combination 7:

Axial Load, $P_u = 0 \text{ lbs}$

Lateral Load, $w = 70\%$ of C&C = 30.9 plf

$$\delta = \frac{5wL^4}{384EI_x} \leq \frac{L}{240}$$

$$L = \sqrt[3]{\frac{384EI_x}{5w(240)}}$$

where

$L = \text{Track Span}$

$$I_x = 1.4929 \text{ in}^4$$

$$w = (207.7 \text{ plf})(1 \text{ ft}/12 \text{ in}) = 17.308 \text{ lb/in}$$

$$L = \sqrt[3]{\frac{384(29,500,000)(1.4929)}{5(17.308)(240)}} = (93.38 \text{ in})(1/12) = 7.78 \text{ ft} > 4.055 \text{ ft}$$

Therefore, Bending Controls: $L = 4.055 \text{ ft} = 4'-1" \approx 4'-2"$ from Table 3.4 – OK !

A2.2 Box Beam/Back-to-Back Header Design Calculations

A2.2.1 Header Design for Span, $L \leq 6'-0''$

Design Objective:

Verify the adequacy of (2) 600S162-43 C-sections in a back-to-back or box beam header arrangement for header spans up to 6'-0". All references are to the *North American Specification for the Design of Cold-Formed Steel Structural Members* unless otherwise noted (AISI, 2001)

Design Assumptions:

Building Width =	40 ft
Stories =	2
Roof Overhang (Rake) =	1 ft
Roof Pitch =	12:12
Stud Spacing =	24 in
Truss Spacing =	24 in
Floor Joist Spacing =	24 in
Header Span =	6 ft
Header Size =	(2) 600S162-43
Steel Yield Strength, F_y =	33 ksi
Ground Snow Load =	70 psf
Snow Reduction Factor =	0.7
Multiplier for Unbal. Snow =	1.5
Minimum Roof Live Load =	16 psf
Roof/Ceiling Dead Load =	12 psf
Wall Dead Load =	10 psf
Floor Dead Load =	10 psf
Floor Live Load =	40 psf
Ceiling Height =	10 ft
Deflection Limit =	$L/240$

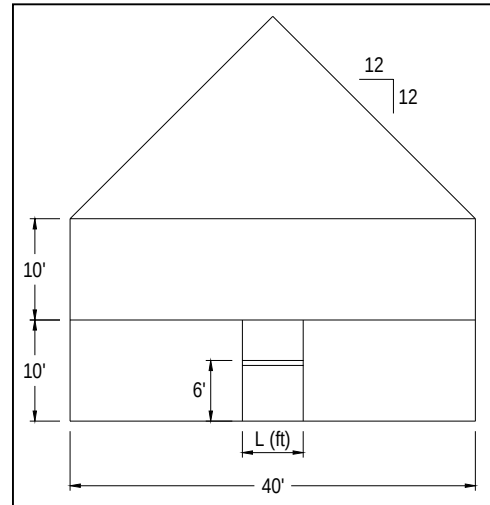


Figure A2.2 – Gable End Wall Elevation

Design Approach:

Assume header is located 6 ft above ground level in center of lower wall. Loads transferred to header are considered uniformly distributed along its length.

Solution:

Total Wall Height, $z = 2(10 \text{ ft}) + 1 \text{ ft} + (20 \text{ ft})(12/12) = 41 \text{ ft}$

Wall Height Above Header = $41 \text{ ft} - 6 \text{ ft} = 35 \text{ ft}$

Design Loads:

Dead Load:

Roof/Ceiling Dead Load = $(12 \text{ psf})(1 \text{ ft} + 2 \text{ ft}/2) =$	24 plf
Floor Dead Load = $(10 \text{ psf})(2 \text{ ft}/2) =$	10 plf
Wall Dead Load = $(10 \text{ psf})(35 \text{ ft}) =$	350 plf

Live Loads:

Roof Live Load = (16 psf)(1 ft + 2 ft/2) = 32 plf
 Roof Snow Load = (0.7x70 psf x 1.5)(1 ft + 2 ft/2) = 147 plf
 Therefore, roof snow loads control over roof live loads.
 Floor Live Load = (40 psf)(2 ft/2) = 40 plf

Load Combinations:

1. 1.4D
2. 1.2D + 1.6L + 0.5(L_r or S)
3. 1.2D + 0.5 L + 1.6(L_r or S)

Design Loads:

1. 1.4(24 + 10 + 350 plf) = 537.6 plf
2. 1.2(24 + 10 + 350 plf) + 1.6(40 plf) + 0.5(147 plf) = 598.3 plf
3. 1.2(24 + 10 + 350 plf) + 0.5(40 plf) + 1.6(147 plf) = 716 plf

Controlling Load Combination:

3. 716 plf

Maximum Moment, $M_{ux} =$

$$\frac{(716 \text{ plf})(6 \text{ ft})^2}{8} = (3222 \text{ lb} - \text{ft}) \left(\frac{12 \text{ in}}{\text{ft}} \right) \left(\frac{1 \text{ kip}}{1000 \text{ lbs}} \right) = 38.66 \text{ k} - \text{in}$$

Check Adequacy of (2) 600S162-43 with CFS 4.13:

Consider compression flange continuously supported by track section: $L_b = 0$

$$\phi M_{nx} \geq M_u$$

From CFS, $\phi M_{nx} = 52.77 \text{ k} - \text{in} \geq 38.66 \text{ k} - \text{in}$ - OK !

Check Web Crippling @ Stud Location:**Design Loads:**

From above, $w_u = 716 \text{ plf}$
 Stud Spacing = 2 ft
 Stud Axial Load $P_u = (716 \text{ plf})(2 \text{ ft}) = 1432 \text{ lbs}$

Determine Web Crippling Strength per Section C3.4:

$$P_n = C_t^2 F_y \sin \theta \left(1 - C_R \sqrt{\frac{R}{t}} \right) \left(1 + C_N \sqrt{\frac{N}{t}} \right) \left(1 - C_h \sqrt{\frac{h}{t}} \right) \quad \text{Eq. C3.4.1-1}$$

For Back-to-Back Headers Fastened to Support with Interior Load (Table C3.4.1-1):

t =	0.0451 in	43 mil web
R =	0.09375 in	Table 3.1
N =	1.125 in	1.25 in track
h =	6 in – 2(0.09375 + 0.0451 in) = 5.7223 in	Flat part of web
F _y =	33,000 psi	33 ksi steel

$$\begin{aligned}\theta &= 90^\circ \\ C &= 20 \\ C_R &= 0.15 \\ C_N &= 0.05 \\ C_h &= 0.003 \\ \phi_w &= 0.90\end{aligned}$$

$$R/t = \frac{0.09375}{0.0451} = 2.08 \leq 5 - \text{OK !}$$

$$\phi_w P_n = 0.90(20)(0.0451)^2(33,000)\sin 90^\circ \left(1 - 0.15\sqrt{\frac{0.09375}{0.0451}}\right) \left(1 + 0.05\sqrt{\frac{1.125}{0.0451}}\right) \left(1 - 0.003\sqrt{\frac{5.7223}{0.0451}}\right)$$

$$\phi_w P_n = 1143 \text{ lbs}$$

$$\phi_w P_n = 2(1143 \text{ lbs}) = 2286 \text{ lbs} \geq P_u = 1432 \text{ lbs} - \text{OK !}$$

2 Sections

For Box Beam Headers Fastened to Support with Interior Load (Table C3.4.1-2):

t = 0.0451 in	43 mil web
R = 0.09375 in	Table 3.1
N = 1.125 in	1.25 in track
h = 6 in – 2(0.09375 + 0.0451 in) = 5.7223 in	Flat part of web
F _y = 33,000 psi	33 ksi steel

$$\theta = 90^\circ$$

$$C = 13$$

$$C_R = 0.23$$

$$C_N = 0.14$$

$$C_h = 0.01$$

$$\phi_w = 0.90$$

$$R/t = \frac{0.09375}{0.0451} = 2.08 \leq 5 - \text{OK !}$$

$$\phi_w P_n = 0.90(13)(0.0451)^2(33,000)\sin 90^\circ \left(1 - 0.23\sqrt{\frac{0.09375}{0.0451}}\right) \left(1 + 0.14\sqrt{\frac{1.125}{0.0451}}\right) \left(1 - 0.01\sqrt{\frac{5.7223}{0.0451}}\right)$$

$$\phi_w P_n = 791 \text{ lbs}$$

For box beam headers, increase above by factor $\alpha = 2.3 \frac{t_t}{t_c} \geq 1.0$

$$t_t = 0.0346$$

$$t_c = 0.0451$$

Assume 33 mil track
43 mil web

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

$$\phi_w P_n = 1.765(791 \text{ lbs})(2) = 2791 \text{ lbs} \geq P_u = 1432 \text{ lbs} - \text{OK !}$$

Check Deflections (Total Load):

$$\delta = \frac{5wL^4}{384EI_x}$$

where

L = Span = 6 ft

w = 24 + 10 + 350 + 40 + 147 plf = 571 plf

I_x = 4.6004 in⁴ (from CFS 4.13)

$$\delta = \frac{5(571 \text{ plf})(6 \text{ ft})^4}{384(29,500,000 \text{ psi})(4.6004 \text{ in}^4)} \times \frac{1728 \text{ in}^3}{\text{ft}^3} = 0.12 \text{ in} \leq \frac{L}{240} = \frac{(6 \text{ ft})(12)}{240} = 0.30 \text{ in} - \text{OK !}$$

Check Adequacy of (2) 600S162-33 with CFS 4.13:

$$\phi M_{nx} \geq M_u$$

From CFS, $\phi M_{nx} = 35.80 \text{ k-in} < 38.66 \text{ k-in} - \text{NG !}$

Therefore, use (2) 600S162-43 in back-to-back or box beam header configuration for header spans, $L \leq 6 \text{ ft}$.

A2.2.2 Header Design for Span, $L > 6'-0"$

Design Objective:

Verify the adequacy of (2) 800S162-54 C-sections in a back-to-back or box beam header arrangement for header spans up to maximum head track span in Tables 3.2 – 3.6. All references are to the *North American Specification for the Design of Cold-Formed Steel Structural Members* unless otherwise noted (AISI, 2001)

Design Assumptions:

See Section A2.2.1

Design Approach:

Assume header is located 6 ft above ground level in center of lower wall. Loads transferred to header are considered uniformly distributed along its length.

Solution:

From Section A2.2.1, $w_u = 716$ plf

Maximum Moment, $M_{ux} =$

$$\frac{(716 \text{ plf})(8.75 \text{ ft})^2}{8} = (6852.3 \text{ lb-ft}) \left(\frac{12 \text{ in}}{\text{ft}} \right) \left(\frac{1 \text{ kip}}{1000 \text{ lbs}} \right) = 82.23 \text{ k-in}$$

Check Adequacy of (2) 800S162-54 with CFS 4.13:

Consider compression flange continuously supported by track section: $L_b = 0$

$$\phi M_{nx} \geq M_u$$

From CFS, $\phi M_{nx} = 83.35 \text{ k-in} \geq 82.23 \text{ k-in}$ - OK !

Check Web Crippling @ Stud Location:

Design Loads:

From above, $w_u = 716$ plf

Stud Spacing = 2 ft

Stud Axial Load $P_u = (716 \text{ plf})(2 \text{ ft}) = 1432 \text{ lbs}$

Determine Web Crippling Strength per Section C3.4:

$$P_n = Ct^2 F_y \sin \theta \left(1 - C_R \sqrt{\frac{R}{t}} \right) \left(1 + C_N \sqrt{\frac{N}{t}} \right) \left(1 - C_h \sqrt{\frac{h}{t}} \right) \quad \text{Eq. C3.4.1-1}$$

For Back-to-Back Headers Fastened to Support with Interior Load (Table C3.4.1-1):

$t =$	0.0566 in	54 mil web
$R =$	0.108 in	Table 3.1
$N =$	1.125 in	1.25 in track
$h =$	8 in – 2(0.108 + 0.0566 in) = 7.6708 in	Flat part of web
$F_y =$	33,000 psi	33 ksi steel
$\theta =$	90°	

$$\begin{aligned}
C &= 20 \\
C_R &= 0.15 \\
C_N &= 0.05 \\
C_h &= 0.003 \\
\phi_w &= 0.90
\end{aligned}$$

$$R/t = \frac{0.108}{0.0451} = 1.91 \leq 5 - \text{OK !}$$

$$\phi_w P_n = 0.90(20)(0.0566)^2(33,000) \sin 90^\circ \left(1 - 0.15 \sqrt{\frac{0.108}{0.0566}} \right) \left(1 + 0.05 \sqrt{\frac{1.125}{0.0566}} \right) \left(1 - 0.003 \sqrt{\frac{7.6708}{0.0566}} \right)$$

$$\phi_w P_n = 1780 \text{ lbs}$$

$$\phi_w P_n = 2(1780 \text{ lbs}) = 3561 \text{ lbs} \geq P_u = 1432 \text{ lbs} - \text{OK !}$$

2 Sections

For Box Beam Headers Fastened to Support with Interior Load (Table C3.4.1-2):

t = 0.0566 in	43 mil web
R = 0.108 in	Table 3.1
N = 1.125 in	1.25 in track
h = 8 in – 2(0.108 + 0.0566 in) = 7.6708 in	Flat part of web
F _y = 33,000 psi	33 ksi steel

$$\theta = 90^\circ$$

$$C = 13$$

$$C_R = 0.23$$

$$C_N = 0.14$$

$$C_h = 0.01$$

$$\phi_w = 0.90$$

$$R/t = \frac{0.108}{0.0451} = 1.91 \leq 5 - \text{OK !}$$

$$= 0.90(13)(0.0566)^2(33,000) \sin 90^\circ \left(1 - 0.23 \sqrt{\frac{0.108}{0.0566}} \right) \left(1 + 0.14 \sqrt{\frac{1.125}{0.0566}} \right) \left(1 - 0.01 \sqrt{\frac{7.6708}{0.0566}} \right)$$

$$\phi_w P_n = 1211 \text{ lbs}$$

For box beam headers, increase above by factor $\alpha = 2.3 \frac{t_t}{t_c} \geq 1.0$

$$t_t = 0.0346$$

$$t_c = 0.0566$$

Assume 33 mil track
43 mil web

$$\alpha = 2.3 \left(\frac{0.0346}{0.0566} \right) = 1.406$$

$$\phi_w P_n = 1.406(1211 \text{ lbs})(2) = 3406 \text{ lbs} \geq P_u = 1432 \text{ lbs} - \text{OK !}$$

Check Deflections (Total Load):

$$\delta = \frac{5wL^4}{384EI_x}$$

where

L = Span = 8.75 ft

w = 24 + 10 + 350 + 40 + 147 plf = 571 plf

I_x = 11.400 in⁴ (from CFS 4.13)

$$\delta = \frac{5(571 \text{ plf})(8.75 \text{ ft})^4}{384(29,500,000 \text{ psi})(11.400 \text{ in}^4)} \times \frac{1728 \text{ in}^3}{\text{ft}^3} = 0.22 \text{ in} \leq \frac{L}{240} = \frac{(8.75 \text{ ft})(12)}{240} = 0.43 \text{ in} - \text{OK !}$$

Check Adequacy of (2) 800S162-43 with CFS 4.13:

$$\phi M_{nx} \geq M_u$$

From CFS, $\phi M_{nx} = 63.74 \text{ k-in} < 82.23 \text{ k-in} - \text{NG !}$

Therefore, use (2) 800S162-54 in back-to-back or box beam header configuration for header spans, 6 ft < L ≤ 8.75 ft (maximum in Tables 3.2 – 3.6).

A2.3 Wall Bottom Track Connections (Section 4.0)

A2.3.1 Wall Bottom Track to Floor Joist or Track Calculation (Table 4.1)

Design Objective:

Determine the required number and spacing of No. 8 screws to connect the wall bottom track of the upper wall to the floor joists/rim boards or floor sheathing based on the design shown below. All references are to the *North American Specification for the Design of Cold-Formed Steel Structural Members*, unless otherwise noted (AISI, 2001)

Design Assumptions:

Stories =	2
Roof Overhang (Rake) =	1 ft
Stud Spacing =	24 in
Rafter Spacing =	24 in
Bottom Track Thickness =	33 mil
Floor Joist Thickness =	33 mil
Steel Yield Strength, F_y =	33 ksi
Floor Sheathing =	$\frac{3}{4}$ " plywood
Roof Pitch =	3:12
Building Width =	32 ft
Wall height =	10 ft
Roof/Ceiling Dead Load =	12 psf
Wall Dead Load =	10 psf
Wind Speed =	130 mph
Wind Exposure Category =	C

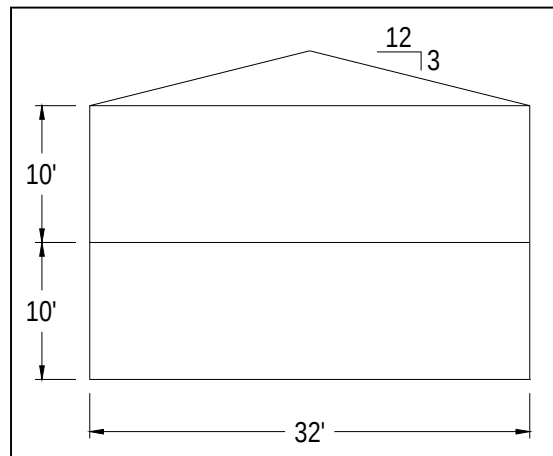


Figure A2.3 - Gable End Wall Elevation

Design Approach:

Determine required number of screws and spacing at critical location for shear and wind uplift. Check combined effect of shear and tension on screws using interaction equation. Critical location for shear and uplift is at center of gable end wall.

Solution:

$$\begin{aligned}\text{Stud Height, } h &= 10 \text{ ft} + (16 \text{ ft})(3/12) = 14 \text{ ft} \\ \text{Roof Height, } z &= 10 + 14 \text{ ft} = 24 \text{ ft}\end{aligned}$$

Design Loads:

Dead Load (per stud):

$$\text{Roof/Ceiling Dead Load per Stud} = [(12 \text{ psf})(2 \text{ ft})(3 \text{ ft})(1.5 \text{ ft})]/2 \text{ ft} = 54 \text{ lbs}$$

$$\text{Uniform Dead Load} = 54 \text{ lbs}/2 \text{ ft} = 27 \text{ plf}$$

$$\text{Wall Dead Load} = (10 \text{ psf})(14 \text{ ft}) = 140 \text{ plf}$$

Wind Loads (Shear):

Wind loads are calculated in accordance with ASCE 7-2002 Method 2 for Main Wind Force Resisting Systems (MWFRS) (ASCE, 2002).

Equation:

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

$$q_h = 0.00256 K_z K_{zt} K_d V^2 I \quad (\text{lb/ft}^2)$$

Where:

$$K_z = 0.932$$

$$K_{zt} = 1.0$$

$$K_d = 0.85$$

$$V = 130 \text{ mph}$$

$$I = 1.0$$

$$q_h = 0.00256(0.932)(1.0)(0.85)(130)^2(1.0) = 34.27 \text{ lb/ft}^2$$

$$GC_{pf} = 0.48, -0.37, -0.45, 0.72, -0.56$$

$$GC_{pi} = \pm 0.18$$

Comments:

Low-Rise Building

Exp. C, Case 1, $z = 24 \text{ ft}$

No Topographic Factor

3-Second Gust Wind Speed

Residential Building (Cat I)

Enclosed Building

Reference:

6.5.12.2.2, Eq. 6-18

6.5.10, Eq 6-15

Table 6-3

Table 6-4

Table 1-1, Table 6-1

Figure 6-10 (cont'd)

Figure 6-5

$$p^+ = (34.27 \text{ lb/ft}^2)[(0.48) - (-0.18)] = 22.62 \text{ lb/ft}^2$$

$$p^- = (34.27 \text{ lb/ft}^2)[(-0.37) - (0.18)] = -18.85 \text{ lb/ft}^2$$

$$p^- = (34.27 \text{ lb/ft}^2)[(-0.45) - (0.18)] = -21.59 \text{ lb/ft}^2$$

$$p^+ = (34.27 \text{ lb/ft}^2)[(0.72) - (-0.18)] = 30.84 \text{ lb/ft}^2$$

$$p^- = (34.27 \text{ lb/ft}^2)[(-0.56) - (0.18)] = -25.36 \text{ lb/ft}^2$$

Controls

Wind Loads (Uplift):

Wind loads are calculated in accordance with ASCE 7-2002 Method 2 for Main Wind Force Resisting Systems (MWFRS) (ASCE, 2002).

Equation:

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

$$q_h = 34.27 \text{ (lb/ft}^2)$$

Comments:

Low-Rise Building

See Above

Reference:

6.5.12.2.2, Eq. 6-18

$$GC_{pf} = -0.69, -0.44, -1.07, -0.63$$

$$GC_{pi} = 0$$

$$G = 0.85$$

$$C_p = 0.80$$

Top of roof overhang

No Interior Surface

Bottom of roof overhang

Figure 6-10 (cont'd)

6.5.8.1

6.5.11.4.1

$$p^- = (34.27 \text{ lb/ft}^2)[-0.69 - (0.80)(0.85)] = -46.95 \text{ lb/ft}^2$$

$$p^- = (34.27 \text{ lb/ft}^2)[-0.44 - (0.80)(0.85)] = -38.38 \text{ lb/ft}^2$$

$$p^- = (34.27 \text{ lb/ft}^2)[-1.07 - (0.80)(0.85)] = -59.974 \text{ lb/ft}^2$$

$$p^- = (34.27 \text{ lb/ft}^2)[-0.63 - (0.80)(0.85)] = -44.89 \text{ lb/ft}^2$$

Controls

Load Combinations:

1. 0.9D + 1.6W
2. 1.6W

Uplift Check
Shear Check

Design Loads:

1. Maximum Uplift, $w_u = 0.9(27 + 140 \text{ plf}) - 1.6(59.97 \text{ psf})(2 \text{ ft}) = -41.60 \text{ plf}$
2. Maximum Shear, $w_u = 1.6(30.84 \text{ psf})\left(\frac{14 \text{ ft}}{2}\right) = 345.41 \text{ plf}$

Shear Due to Wind Forces:**Material Properties:**

F_y (track and outlooker) = 33,000 psi
 F_u (track and outlooker) = 45,000 psi

No. 8 Screw Shear Capacity in Metal Joists or Rim (Section E2):

Reference for shear is from *2004 Supplement to the Commentary on the Standard for Cold-Formed Steel Framing*

$$\text{Shear} = \frac{P_n}{\Omega} = 164 \text{ lbs}$$

where

$$\phi = 0.50$$

$$\Omega = 3$$

Section E4

$$P_n = 164\Omega = 164(3) = 492 \text{ lbs}$$

$$\phi P_n = \phi 492 = 0.5(492) = 246 \text{ lbs}$$

Determine Required Number of Screws:

Assume Spacing = 8 in o.c.

$$\phi P_{ns} \geq P_u$$

$$P_u = (345.41 \text{ plf})(8 \text{ in})(1 \text{ ft}/12 \text{ in}) = 230.27 \text{ lbs}$$

$$N = \frac{P_u}{\phi P_n} = \frac{230.27 \text{ lbs}}{246 \text{ lbs}} = 0.936 \text{ Screws} = 1 \text{ Screw}$$

Therefore, track requires 1-No. 8 Screw at 8" o.c. to resist pure shear forces.

Tension Due to Uplift Forces:**No. 8 Screw Tension Capacity in Metal Joists or Rim (Section E2):**

Reference for pullout is from *2004 Supplement to the Commentary on the Standard for Cold-Formed Steel Framing*

$$Pullout = \frac{P_n}{\Omega} = 72 \text{ lbs}$$

where

$$\phi = 0.50$$

$$\Omega = 3$$

Section E4

$$P_n = 72\Omega = 72(3) = 216 \text{ lbs}$$

$$\phi P_n = \phi 216 = 0.5(216) = 108 \text{ lbs}$$

No. 8 Screw Tension Capacity in 3/4" Plywood Flooring:

References in this section are to *Structural Design in Wood* (Stalnaker & Harris, 1999) unless otherwise noted.

Withdrawal Resistance, Z_w (kips):

$$Z_w = 9.47DG^2p_n f$$

$$D = 0.164 \text{ in}$$

$$G = 0.42$$

$$p = 0.75 \text{ in}$$

$$n_f = 1$$

$$Z_w = 9.47(0.164)(0.42)^2(0.75)(1.0) = 0.205 \text{ kips} = 205 \text{ lbs}$$

$$\phi = 0.65$$

$$\phi Z_w = 0.65(205 \text{ lbs}) = 133.5 \text{ lbs} > \phi P_n = 108 \text{ lbs} \text{ – Tension in Metal Controls}$$

$$\phi P_n = 108 \text{ lbs per screw}$$

Eq. 5-8
No. 8 Screw
(APA, 2004)
3/4" Plywood
Per Screw

Check No.8 Screws @ 8" on Center:

$$\phi P_n \geq P_u$$

$$P_u = (41.60 \text{ plf})(8 \text{ in})(1 \text{ ft}/12 \text{ in}) = 27.73 \text{ lbs per screw}$$

$$N = \frac{P_u}{\phi P_n} = \frac{27.73 \text{ lbs}}{108 \text{ lbs}} = 0.26 \text{ Screws} = 1 \text{ Screw}$$

Therefore, track requires 1-No. 8 Screw at 8" o.c. to resist uplift forces.

Combined Shear and Tension Capacity:

$$\text{Interaction Equation: } \left(\frac{T_u}{\phi T_n} \right)^2 + \left(\frac{V_u}{\phi V_n} \right)^2 \leq 1.0$$

For 1 #8 Screw @ 8" o.c.:

$$T_u = 41.60 \text{ plf}$$

$$\phi T_n = (108 \text{ lbs}) \frac{(1 \text{ Screw})}{0.667 \text{ ft}} = 161.92 \text{ plf}$$

$$V_u = 345.41 \text{ plf}$$

$$\phi V_n = (246 \text{ lbs}) \frac{(1 \text{ Screw})}{0.667 \text{ ft}} = 368.82 \text{ plf}$$

$$\left(\frac{41.60}{161.92} \right)^2 + \left(\frac{345.41}{368.82} \right)^2 = 0.94 \leq 1.0 - \text{OK !}$$

Therefore, use 1 No. 8 Screw @ 8" o.c.

Note that for this example, the number of screws required using Table 4.1 is 2 – No. 8 Screws at 8" o.c. This is because the example used the actual mean roof height while the tables were developed using the maximum mean roof height of 30-feet to cover all conditions.

A2.3.2 Wall Bottom Track to Foundation Calculation (Table 4.2)

Design Objective:

Determine the required spacing of ½" diameter A307 Grade A headed anchor bolts to connect the wall bottom track to the foundation in the design shown below. All references are to the *North American Specification for the Design of Cold-Formed Steel Structural Members*, unless otherwise noted (AISI, 2001)

Design Assumptions:

Roof Overhang (Rake) = 1 ft
Stud Spacing = 24 in
Roof Joist Spacing = 24 in
Bottom Track Thickness = 33 mil
Steel Yield Strength, F_y = 33 ksi
Roof Pitch = 3:12
Building Width = 32 ft
Wall height = 10 ft
Roof/Ceiling Dead Load = 12 psf
Wall Dead Load = 10 psf
Wind Speed = 130 mph
Wind Exposure Category = C

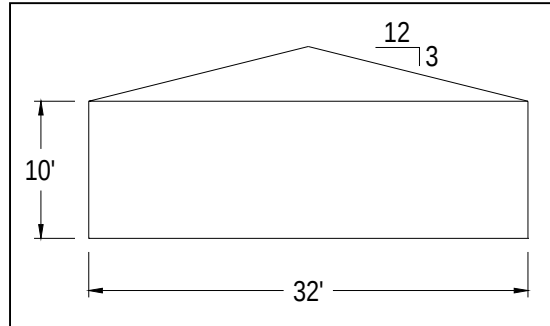


Figure A2.4 - Gable End Wall Elevation

Design Approach:

Determine required bolt spacing at critical location for shear, tension due to uplift on overhang, and combined effect of shear and tension. Critical location for shear and uplift is at center of gable end wall. Assume track rests on a 2x Douglas Fir Larch sill plate when checking shear.

Solution:

Stud Height, $h = 10 \text{ ft} + (16 \text{ ft})(3/12) = 14 \text{ ft}$
Roof Height, $z = 14 \text{ ft}$

Design Loads:

Dead Load:

Roof/Ceiling Dead Load = $(12 \text{ psf})(2 \text{ ft}) = 24 \text{ plf}$
Wall Dead Load = $(10 \text{ psf})(14 \text{ ft}) = 140 \text{ plf}$

Wind Loads (Shear):

Wind loads are calculated in accordance with ASCE 7-2002 Method 2 for Main Wind Force Resisting System (MWFRS) (ASCE, 2002).

Equation:

$$p = q_h [(GC_{pf}) - (GC_{pi})]$$
$$q_h = 0.00256 K_z K_{zt} K_d V^2 I \text{ (lb/ft}^2\text{)}$$

Comments:

Low-Rise Building

Reference:

6.5.12.2.2, Eq. 6-18

6.5.10, Eq 6-15

Where:

$K_z = 0.85$	Exp. C, Case 1, $z = 14$ ft	Table 6-3
$K_{zt} = 1.0$	No Topographic Factor	
$K_d = 0.85$		Table 6-4
$V = 130$ mph	3-Second Gust Wind Speed	
$I = 1.0$	Residential Building (Cat I)	Table 1-1, Table 6-1
$q_h = 0.00256(0.85)(1.0)(0.85)(130)^2(1.0) = 31.26$ lb/ft ²		
$GC_p = 0.48, -0.37, -0.45, 0.72, -0.56$		Figure 6-10 (cont'd)
$GC_{pi} = \pm 0.18$	Enclosed Building	Figure 6-5

$$p^+ = (31.26 \text{ lb/ft}^2)[(0.48) - (-0.18)] = 20.63 \text{ lb/ft}^2$$

$$p^- = (31.26 \text{ lb/ft}^2)[(-0.37) - (0.18)] = -17.19 \text{ lb/ft}^2$$

$$p^- = (31.26 \text{ lb/ft}^2)[(-0.45) - (0.18)] = -19.69 \text{ lb/ft}^2$$

$$p^+ = (31.26 \text{ lb/ft}^2)[(0.72) - (-0.18)] = 28.13 \text{ lb/ft}^2$$

Controls

$$p^- = (31.26 \text{ lb/ft}^2)[(-0.56) - (0.18)] = -23.13 \text{ lb/ft}^2$$

Wind Loads (Uplift):

Wind loads are calculated in accordance with ASCE 7-2002 Method 2 for Main Wind Force Resisting System (MWFRS) (ASCE, 2002).

Equation:

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

$$q_h = 31.26 \text{ (lb/ft}^2\text{)}$$

Comments:

Low-Rise Building

See Above

Reference:

6.5.12.2.2, Eq. 6-18

$$GC_{pf} = -0.69, -0.44, -1.07, -0.63$$

$$GC_{pi} = 0$$

$$G = 0.85$$

$$C_p = 0.80$$

Top of roof overhang

No Interior Surface

Bottom of roof overhang

Figure 6-10 (cont'd)

6.5.8.1

6.5.11.4.1

$$p^- = (31.26 \text{ lb/ft}^2)[-0.69 - (0.80)(0.85)] = -42.83 \text{ lb/ft}^2$$

$$p^- = (31.26 \text{ lb/ft}^2)[-0.44 - (0.80)(0.85)] = -35.01 \text{ lb/ft}^2$$

$$p^- = (31.26 \text{ lb/ft}^2)[-1.07 - (0.80)(0.85)] = -54.70 \text{ lb/ft}^2$$

$$p^- = (34.27 \text{ lb/ft}^2)[-0.63 - (0.80)(0.85)] = -40.9 \text{ lb/ft}^2$$

Controls

Load Combinations:

$$1. 0.9D + 1.6W$$

$$2. 1.6W$$

Uplift Check

Shear Check

Design Loads:

$$1. \text{ Maximum Uplift, } w_u = 0.9(24 + 140 \text{ plf}) - 1.6(54.70 \text{ psf})(2 \text{ ft}) = -27.44 \text{ plf}$$

$$2. \text{ Maximum Shear, } w_u = 1.6(28.13 \text{ psf})\left(\frac{14 \text{ ft}}{2}\right) = 315.06 \text{ plf}$$

Material Properties:Track:

$$F_y = 33,000 \text{ psi}$$

$$F_u = 45,000 \text{ psi}$$

Bolt:

$$F_{nt} = 45,000 \text{ psi}$$

$$F_{nv} = 27,000 \text{ psi}$$

Wood Sill (Douglas Fir-Larch):

$$G = 0.50 \text{ (Specific Gravity)}$$

Concrete:

$$f'_c = 3,000 \text{ psi}$$

Determine Shear Capacity of Connection:**Track/Bolt Capacity (Section E3):**Spacing and Edge Distance (E3.1):

$$P_n = teF_u$$

$$t = 0.0346 \text{ in}$$

$$e = 1.75 \text{ in}$$

$$F_u = 45,000 \text{ psi}$$

$$P_n = (0.0346 \text{ in})(1.75 \text{ in})(45,000 \text{ psi}) = 2,724 \text{ lbs}$$

$$\frac{F_u}{F_{sy}} = \frac{45}{33} = 1.36 \geq 1.08$$

$$\phi = 0.70$$

$$\phi P_n = 0.70(2724 \text{ lbs}) = 1907 \text{ lbs}$$

33 mil Track
350T125 Track
33 ksi Steel

E3.1(a)

Bearing (E3.3):

Assume deformation around bolt hole is a design consideration (Section E3.3.2):

$$P_n = (4.64\alpha t + 1.53)dtF_u$$

Eq. E3.3.2-1

$$\alpha = 1.0$$

$$t = 0.0346 \text{ in}$$

$$d = 0.5 \text{ in}$$

$$F_u = 45,000 \text{ psi}$$

US Customary Units
33 mil Track
Bolt Diameter
33 ksi Steel

$$P_n = (4.64(1.0)(0.0346) + 1.53)(0.5 \text{ in})(0.0346 \text{ in})(45,000 \text{ psi}) = 1316 \text{ lbs}$$

$$\phi = 0.65$$

$$\phi P_n = 0.65(1,316 \text{ lbs}) = 855 \text{ lbs}$$

Bolt Shear (Section E3.4):

$$P_n = A_b F_{nv}$$

Eq. E3.4-1

$$A_b = \frac{\pi(0.5 \text{ in})^2}{4} = 0.1963 \text{ in}^2$$

$$F_{nv} = 27 \text{ ksi}$$

Table E3.4-1

$$P_n = (0.1963 \text{ in}^2)(27,000 \text{ psi}) = 5,301 \text{ lbs}$$

$$\phi = 0.65$$

Table E3.4-1

$$\phi P_n = 0.65(5,301 \text{ lbs}) = 3445.6 \text{ lbs}$$

Therefore, bearing controls and track/bolt capacity $\phi P_n = 855 \text{ lbs/Bolt}$

Concrete Capacity:

References in this section are to ACI 318-02 – *Building Code Requirements for Structural Concrete* (ACI, 2002).

Breakout Strength (Section D.6.2):

For single anchor with shear force perpendicular to the edge:

$$V_{cb} = \frac{A_v}{A_{vo}} \psi_6 \psi_7 V_b \quad \text{Eq. D-20}$$

$$\frac{A_v}{A_{vo}} = 1.0$$

For Single Anchor

$$c_2 \gg c_1$$

D.6.2.6 - Interior Bolt

$$\psi_6 = 1.0$$

$$\psi_7 = 1.0$$

D.6.2.7

$$V_b = 7 \left(\frac{1}{d_o} \right)^{0.2} \sqrt{d_o} \sqrt{f'_c} (c_1)^{1.5} \quad \text{Eq. D-23}$$

$$l = h_{ef} = 7 \text{ in}$$

Section D.0

$$d_o = 0.5 \text{ in}$$

Bolt Diameter

$$f'_c = 3,000 \text{ psi}$$

$$c_1 = 1.75 \text{ in}$$

Edge Distance

$$V_b = 7 \left(\frac{7}{0.5} \right)^{0.2} \sqrt{0.5} \sqrt{3,000} (1.75)^{1.5} = 1,064 \text{ lbs}$$

$$V_{cb} = 1.0(1.0)(1.0)(1,064 \text{ lbs}) = 1,064 \text{ lbs}$$

$$\phi = 0.70$$

No Edge Reinf.

$$\phi V_{cb} = 0.70(1,064 \text{ lbs}) = 745 \text{ lbs}$$

Pryout Strength (Section D.6.3):

$$V_{cp} = k_{cp} N_{cb} \quad \text{Eq. D-28}$$

$$k_{cp} = 2.0$$

$h_{ef} \geq 2.5 \text{ in}$

$$N_{cb} = \frac{A_N}{A_{No}} \psi_2 \psi_3 N_b \quad \text{Eq. D-4}$$

$$A_N = (c_1 + 1.5h_{ef})(2 \times 1.5h_{ef})$$

$c_1 < 1.5h_{ef}$

$$A_N = (1.75 + 1.5(7 \text{ in}))(2 \times 1.5(7 \text{ in})) = 257.25 \text{ in}^2$$

Fig. RD.5.2.1(a)

$$A_{No} = 9h_{ef}^2 = 9(7 \text{ in})^2 = 441 \text{ in}^2$$

Fig. RD.5.2.1(a)

$$c_{\min} = c_1 = 1.75 \text{ in} < 1.5h_{ef} = 1.5(7 \text{ in}) = 10.5 \text{ in}$$

$$\psi_2 = 0.7 + 0.3 \frac{c_{\min}}{1.5h_{ef}} = 0.7 + 0.3 \frac{1.75}{1.5(7)} = 0.75 \quad \text{Eq. D-11}$$

$$\psi_3 = 1.25$$

Cast-in Anchor

$$N_b = k \sqrt{f'_c} h_{ef}^{1.5}$$

Eq. D-7

$$k = 24$$

Cast-in Anchor

$$N_b = 24 \sqrt{3,000} (7)^{1.5} = 24,345 \text{ lbs}$$

$$N_{cb} = \frac{257.25}{441} (0.75)(1.25)(24,345 \text{ lbs}) = 13,314 \text{ lbs}$$

$$V_{cp} = k_{cp} N_{cb} = 2.0(13,314 \text{ lbs}) = 26,628 \text{ lbs}$$

$$\phi = 0.70$$

No Edge Reinf.

$$\phi V_{cp} = 0.70(26,628 \text{ lbs}) = 18,639 \text{ lbs}$$

Therefore, breakout controls and concrete capacity $\phi V_c = 745 \text{ lbs/Bolt}$

Wood Sill Capacity:

References in this section are to the *National Design Specification for Wood Construction (2001 Edition) and Supplement: Design Values* (AF&PA, 2001).

$$G = 0.50$$

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$$\text{Main Member, } t_m = 3 \text{ in}$$

Max = $2t_s$

$$\text{Side Member, } t_s = 1.5 \text{ in}$$

2x Sill

$$\text{Bolt Diameter, } D = 0.5 \text{ in}$$

$$Z_{s\perp} = 3.3(370 \text{ lbs}) = 1,221 \text{ lbs}$$

Table 8.2A - LRFD

$$\phi = 0.65$$

$$\phi Z_{s\perp} = 0.65(1,221 \text{ lbs}) = 793 \text{ lbs/Bolt}$$

$$\text{Shear Capacity} = \text{Min}(855, 745, 793 \text{ lbs}) = 745 \text{ lbs/Bolt}$$

Determine Minimum Spacing for Shear

$$\text{From above calculation, } \phi P_n = 745 \text{ lbs}$$

$$\text{Factored Load, } w_u = 315.06 \text{ plf}$$

$$\text{Maximum spacing, } s = \frac{745 \text{ lbs}}{315.06 \text{ plf}} \left(\frac{12 \text{ in}}{\text{ft}} \right) = 28.38 \text{ in} > 24 \text{ in from Table 4.2— OK!}$$

Determine Tensile Capacity of Connection:

Bolt Capacity (Section E3):

Tension in Bolts (E3.4):

$$P_n = A_b F_{nv}$$

Eq. E3.4-1

$$A_b = \frac{\pi(0.5 \text{ in})^2}{4} = 0.1963 \text{ in}^2$$

$$F_{nv} = 45,000 \text{ psi}$$

Table E3.4-1

$$P_n = (0.1963 \text{ in}^2)(45,000 \text{ psi}) = 8,833 \text{ lbs}$$

$$\phi = 0.65$$

Table E3.4-1

$$\phi P_n = 0.65(8,833 \text{ lbs}) = 5,741.8 \text{ lbs}$$

Concrete Capacity:

References in this section are to ACI 318-02 – *Building Code Requirements for Structural Concrete* (ACI, 2002).

Breakout Strength (Section D.5.2):

For single anchor with shear force perpendicular to the edge:

$$N_{cb} = \frac{A_N}{A_{No}} \psi_2 \psi_3 N_b \quad \text{Eq. D-4}$$

$$N_{cb} = 13,314 \text{ lbs}$$

From Above

$$\phi = 0.70$$

$$\phi N_{cb} = 0.70(13,314 \text{ lbs}) = 9,319.8 \text{ lbs}$$

Pullout Strength (Section D.5.3):

$$N_{pn} = \psi_4 N_p \quad \text{Eq. D-12}$$

$$\psi_4 = 1.0$$

$$N_p = A_{brg} 8 f'_c \quad \text{Eq. D-13}$$

$$A_{brg} = \frac{(.875 \text{ in})^2 \pi}{4} = 0.6013 \text{ in}^2 \quad \text{Headed Area (AISC, 2001)}$$

$$f'_c = 3,000 \text{ psi}$$

$$N_p = 0.6013 \text{ in}^2 (8)(3,000 \text{ psi}) = 14,431 \text{ lbs}$$

$$\phi = 0.70$$

$$\phi N_p = 0.70(14,431 \text{ lbs}) = 10,102.2 \text{ lbs}$$

Side Face Blowout Strength (Section D.5.4):

$$c = 1.75 \text{ in} < 0.4 h_{ef} = 0.4(7 \text{ in}) = 2.8 \text{ in}$$

$$N_{sb} = 160c \sqrt{A_{brg}} \sqrt{f'_c} \quad \text{Eq. D-15}$$

$$A_{brg} = 0.6013 \text{ in}^2 \quad \text{See Above}$$

$$f'_c = 3,000 \text{ psi}$$

$$N_{sb} = 160(1.75) \sqrt{0.6013} \sqrt{3,000} = 11,892 \text{ lbs} \quad \text{Eq. D-15}$$

$$\phi = 0.70$$

$$\phi N_p = 0.70(11,892 \text{ lbs}) = 8,324.6 \text{ lbs}$$

Therefore, side face blowout controls and concrete capacity $\phi P_n = \phi N_p = \mathbf{8,324 \text{ lbs/Bolt}}$

Tensile Capacity = Min(5741, 8324 lbs) = **5,741 lbs/Bolt**

Determine Minimum Spacing for Tension

From above calculation, $\phi P_n = 5,741 \text{ lbs}$

Factored Load, $w_u = 27.44 \text{ plf}$

Maximum spacing, $s = \frac{5741 \text{ lbs}}{27.44 \text{ plf}} \left(\frac{12 \text{ in}}{\text{ft}} \right) = 251 \text{ in} > 24 \text{ in}$ from Table 4.2 – OK !

Determine Combined Shear and Tensile Capacity of Connection:**Bolt Capacity (Section E3):**

Assume bolt spacing = 24 in per Table 4.2

$$P_n = A_b F'_nt \quad \text{Eq. E3.4-1}$$

$$A_b = \frac{\pi(0.5 \text{ in})^2}{4} = 0.1963 \text{ in}^2$$

$$F'_nt = 52 - 2.4 f_v \leq 45,000 \quad \text{Table E3.4-3}$$

$$f_v = \frac{V_u}{A_b}$$

$$V_u = 315.06 \text{ plf} (24 \text{ in}) \left(\frac{1 \text{ ft}}{12 \text{ in}} \right) = 630.12 \text{ lbs per bolt}$$

$$f_v = \frac{630.12 \text{ lbs}}{0.1963 \text{ in}^2} \left(\frac{1 \text{ kip}}{1000 \text{ lb}} \right) = 3.21 \text{ ksi}$$

$$F'_nt = 52 - 2.4(3.21 \text{ ksi}) = 44.29 \text{ ksi} < 45 \text{ ksi}$$

$$P_n = (0.1963 \text{ in}^2)(44.29 \text{ ksi}) = 8.695 \text{ ksi} = 8695 \text{ lbs}$$

$$\phi = 0.75 \quad \text{Table E3.4-3}$$

$$\phi P_n = 0.75(8695 \text{ lbs}) = 6521 \text{ lbs}$$

$$\phi P_n \geq P_u \text{ (Tension)}$$

$$P_u = 27.44 \text{ plf} (24 \text{ in}) \left(\frac{1 \text{ ft}}{12 \text{ in}} \right) = 54.88 \text{ lbs per bolt}$$

$$\phi P_n = 6521 \text{ lbs} \geq 54.88 \text{ lbs} - \text{OK !}$$

Concrete Capacity:

References in this section are to ACI 318-02 – *Building Code Requirements for Structural Concrete* (ACI, 2002).

$$0.2\phi V_n = 0.2(745) = 149 \text{ lbs}$$

$$V_u = 630.12 \text{ lbs} > 149 \text{ lbs} \quad \text{Section D.7.1}$$

$$0.2\phi N_n = 0.2(5,741 \text{ lbs}) = 1,148 \text{ lbs}$$

$$N_u = 54.88 \text{ lbs} \leq 1,148 \text{ lbs} \quad \text{Section D.7.2}$$

Therefore, full strength in shear is permitted: $\phi V_n \geq V_u$

Maximum spacing = 28.38 in (from shear) > 24 in from Table 4.2

Therefore, use $\frac{1}{2}$ " ϕ headed anchor bolts @ 2'-0" o.c. as indicated in Table 4.2



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