COLD-FORMED STEEL FRAMING DESIGN GUIDE D110-16

Design Guide

American Iron and Steel Institute

AISI DESIGN GUIDE

Cold-Formed Steel Framing

Design Guide

2016 Edition

Approved by
AISI Committee on Framing Standards
AISI Committee on Specifications

Cold-Formed Steel Framing Design Guide

Third Edition

April 2016

Design Guide D110-16

American Iron and Steel Institute
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Washington, DC 20001

This Design Guide has been developed under the direction of the American Iron and Steel Institute (AISI) Education Subcommittee. The AISI Subcommittee wishes to acknowledge and express gratitude to Mr. Robert L. Madsen of Devco Engineering, Inc. for updating the third edition of this Design Guide.

With anticipated improvements in understanding of the behavior of cold-formed steel and the continuing development of new technology, this material might become dated. It is possible that AISI will attempt to produce updates of this guide, but it is not guaranteed.

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PREFACE

This publication is intended as a guide for designers of cold-formed steel framing systems for buildings. Cold-formed steel framing products include cold-formed studs, joists, rafters, trusses and miscellaneous bracing and connection components. They may be stick-built on site as individual members or panelized into preassembled systems for walls, floors or roofs.

The material presented in this publication has been prepared for the general information of the reader. While the material is believed to be technically correct and in accordance with recognized good practice at the time of publication, it should not be used without first securing competent advice with respect to its suitability for any given application. Neither the American Iron and Steel Institute, its members nor Devco Engineering, Inc. warrant or assume liability for the suitability of the material for any general or particular use.

Scope and Purpose of the Guide

This guide has been prepared to assist practicing structural engineers to design cold-formed steel framing systems. This is the third edition of the guide—the first and the second editions were developed and updated by Mr. Thomas Trestain, formerly of T. W. J. Trestain Structural Engineering, and was published in January 2002 and October 2007, respectively.

A general review of the basic structural principles is provided along with a number of detailed design examples covering wind bearing and axial load bearing stud walls and joists. The design examples are based on AISI S100-12, *North American Specification for the Design of Cold Formed Steel Structural Members, 2012 Edition (AISI 2012a)*, and AISI S240-15, *North American Standard for Cold-Formed Steel Structural Framing, 2015 Edition (AISI 2015)*. Reference is also made to ASCE/SEI 7-10 *(ASCE 2010*) and the *International Building Code, 2015 Edition (IBC 2015)*. The examples demonstrate how to translate the information available in load tables into complete structural systems. Both screwed and welded connection details are included, with an emphasis on screwed connections. Useful information on the strength of commonly used concrete anchors and self-drilling screws is also included.

A number of methodologies are proposed to handle design problems not covered in AISI S100 or AISI S240. These include an approximate method to check the bearing stresses under the bottom track of axial load bearing stud wall assemblies and a method to check the strength and stiffness of inner and outer top track assemblies for wind bearing applications.

A universal designator system for cold-formed steel framing members has been used throughout the guide. This product identification method is described in Appendix J.

Changes From the Second Edition of the Design Guide

The third edition of the Design Guide has been rewritten to reflect improvements in the design of cold-formed steel framing members and connections.

- This edition of AISI D110 references AISI S240-15. A cross-reference to the Framing Standards documents used in the previous edition of AISI D110, namely AISI S200, S210, S211, S213 and S214, is provided in Appendix K.
- The load combination factors as required by ASCE/SEI 7-10 have been used, including a 0.42 factor on wind for deflection calculations from IBC 2015 Table 1604.3, Footnote f.
- The design examples have been revised to conform to the latest design standards, including AISI S100-12 and AISI S240-15.
- Distortional buckling calculations have been added to Example #2.
- The slide clip detail for connecting wind bearing studs has been modified to illustrate a clip more typical of modern construction.
- Where warping torsion is checked, the provisions of Section C3.6 of AISI S100-12 are referenced.
- The alternative parapet design using cantilevering HSS posts has been modified to reflect an approach with more flexibility for field installation.
- A new Example #5 has been added to illustrate the design of load-bearing framing using ledger framing.
- Appendix L has been added to provide a complete listing of the Cold-Formed Steel Engineers Institute (CFSEI) Technical Notes.

Load Tables

In the second edition of the guide, industry standard software AISIWIN[1] *(Simpson Strong-Tie® 2014)* was used for determining section properties, allowable spans and allowable loads. That methodology is followed in this third edition as well, but with an up-to-date version of the software. Thus, in the design examples where reference is made to "load tables" or "manufacturer's tables", it is actually AISIWIN output that has been used. Note that the AISIWIN output conforms to AISI S100-12.

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[1] AISIWIN is an industry standard steel stud and joist software package by Simpson Strong-Tie®.

Other Sources of Information

There are a number of other valuable resource documents for the design of cold-formed steel structures. These are either referenced in the Design Guide or are available at the following websites:

- American Iron and Steel Institute (AISI): www.steel.org; www.aisistandards.org and www.buildingusingsteel.org
- Association of the Wall and Ceiling Industries: www.awci.org
- Center for Cold-Formed Steel Structures (CCFSS): www.CCFSSonline.org
- Cold-Formed Steel Engineers Institute (CFSEI): www.cfsei.org
- Steel Framing Alliance (SFA): www.steelframing.org

Acknowledgements

The American Iron and Steel Institute (AISI) would like to acknowledge the contribution of Mr. Robert L. Madsen, PE, Senior Project Engineer, Devco Engineering, Inc., Enterprise, Oregon, who was retained for the preparation of this third edition. Mr. Madsen is experienced in the design and installation of cold-formed steel framing products and is an active member of the AISI Committee on Specifications and the AISI Committee on Framing Standards. AISI would also like to acknowledge the contribution of Mr. Tom Trestain, P. Eng., formerly of T.W.J. Trestain Structural Engineering, Toronto, Canada, who was retained for the preparation of the first two editions of this publication. Special appreciation is also expressed to the members of the Project Monitoring Task Group, which consisted of Paul Dalia, Roger LaBoube, Sutton Stephens and Brandon Wahl, for adding their engineering knowledge and expertise to this project.

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AISI COLD-FORMED STEEL FRAMING DESIGN GUIDE

INTRODUCTION

1. AISI D110 Focus

AISI D110 was written with a focus on the fundamental principles of cold-formed steel design as they relate to cold-formed steel framing construction. It shows how to use product literature published by the cold-formed steel framing manufacturers when executing the design of building systems.

It was necessary to focus on fundamentals because the versatility of cold-formed steel framing construction makes an all-inclusive design guide virtually impossible. By following the examples provided, engineers should gain the confidence necessary to execute their own designs with the knowledge that there is nothing mysterious about cold-formed steel design. The same basic structural design principles that work with every other building material will also work with cold-formed steel framing.

An intimate knowledge of AISI S100-12, *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI 2012a) and AISI S240-15, *North American Standard for Cold-Formed Steel Structural Framing (AISI 2015)*, while desirable, is not essential. The examples focus on those areas of AISI S100 and AISI S240 that require the designer's attention. Much of the work has already been completed during the preparation of product literature by the cold-formed steel framing manufacturer with section properties and load tables calculated and ready to use in tabular form.

In AISI D110, the examples have been prepared with more detail than required for routine design. With experience, the designer will learn which secondary effects can be ignored to streamline the design process. In addition, the examples are not intended to preclude other design approaches and details. There are many satisfactory ways to design cold-formed steel framing systems.

Note that AISI D110 is almost entirely dedicated to hand calculation methods. Hand calculation is useful for illustrative purposes but may not be the most efficient approach for routine design. Experienced practitioners typically automate the design process as much as possible by writing their own spreadsheet type programs, purchasing commercial cold-formed steel software packages, or both.

2. LRFD Versus ASD

AISI S100 permits two different design approaches, Allowable Strength Design (ASD) or Load and Resistance Factor Design (LRFD). Either approach is acceptable, but the cold-formed steel framing industry continues to use the ASD approach almost exclusively. To reflect this practice, AISI D110 is based on ASD.

3. Loads

3.1 Wind, Earthquake and Gravity Nominal Loads

AISI D110 does not attempt to interpret the wind and gravity load provisions in the various building codes. Instead, the nominal design wind and gravity loads are assumed.

3.2 Load Combination Factors

The load combination factors for Allowable Strength Design have been taken from ASCE/SEI 7-10 Section 2.4 *(ASCE 2010)*. These load combination factors are consistent with the 2015 *International Building Code (IBC 2015).*

The deflection limit state for wall studs is checked for 0.42 times the nominal wind load *(for components & cladding)*. The 0.42 factor is taken from AISI S240.

4. Design Strengths for Cold-Formed Steel Framing Elements

4.1 Member Design Strengths

Member capacities in the form of yield moment, distortional buckling moment, shear, and web crippling design strengths and moments of inertia for checking deflection are generally available in published load tables. These tables also typically contain load data for wind and axial load bearing studs and roof and floor joists.

For this edition of AISI D110, the allowable spans, loads and section properties have been derived using AISIWIN (*Simpson Strong-Tie® 2014*), which is an industry standard software product. The output from AISIWIN conforms to AISI S100. The increase in strength due to cold work of forming has been included for flexure where applicable.

Unless noted otherwise, the following yield and tensile strength values for both stud and track have been used:

- For thicknesses less than or equal to 0.0451", F_y = 33 ksi and F_u = 45 ksi.
- For thicknesses greater than 0.0451", F_y = 50 ksi and F_u = 65 ksi.

These assumed yield and tensile strength values are common for studs whereas track is more typically available with yield strength of 33 ksi for all thicknesses. Check with the local cold-formed steel framing manufacturers before specifying track with yield strength of 50 ksi—a special order may be required.

Occasionally, a designer may wish to derive a member capacity when published values are to be confirmed or when confronted with a special case. Member design is covered by AISI S100, which includes provisions for local buckling, distortional buckling, members in tension, bending, compression and combined axial load and bending. A commentary is also available *(AISI 2012b)*. For axially loaded wall studs clad with imperfect sheathing (i.e., sheathing that does not completely restrain the studs), refer to AISI S240. Lastly, the American Iron and Steel Institute publishes AISI D100, *Cold-Formed Steel Design Manual (AISI 2013),* which contains additional helpful information such as computational aids; supplementary formulas; worked examples for beams columns and connections; properties of steels; and test procedures. The Cold-Formed Steel Engineers Institute (CFSEI) also publishes Technical Notes which are an excellent source of information on the analysis and design of cold-formed steel framing. See Appendix L for a list of relevant CFSEI Technical Notes.

Note that the design expressions in Section C3.6 of AISI S100 include members subject to torsional loading between bracing points. Determination of torsional stresses of open cross-sections is challenging and typically requires testing (Chapter F of AISI S100) or rational analysis.

4.2 Member Design Strength as a Function of Bracing

Cold-formed steel framing members, whether studs or joists, rely on supplementary bracing to resist lateral instability, weak axis buckling and the torsion resulting from loads not applied through the shear center.

4.2.1 Bracing for Wind Bearing Studs

Wind loads are transferred to studs by sheathing materials or by connectors such as brick ties. These loads are typically eccentric with respect to the shear center of the stud, and torsion therefore results. Figure I(a) illustrates the torsional eccentricity for the case of sheathing loading the top flange of a joist. Figure I(b) illustrates a larger torsional eccentricity more typical for wall studs with an old style wrap around brick tie or for sheathing attachment when the screw is in tension.

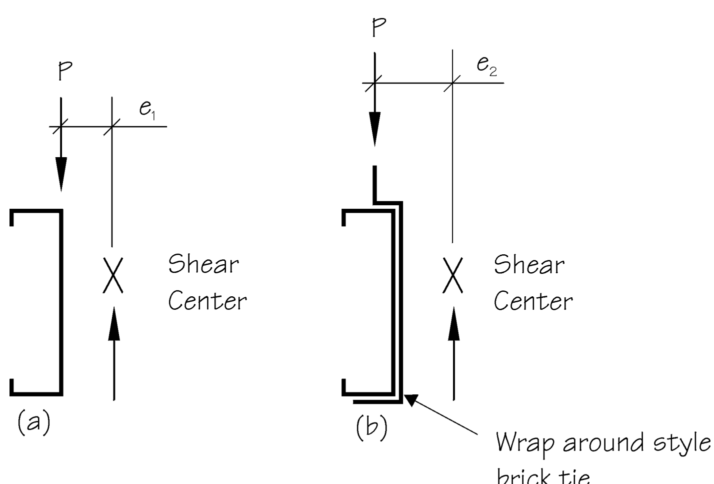
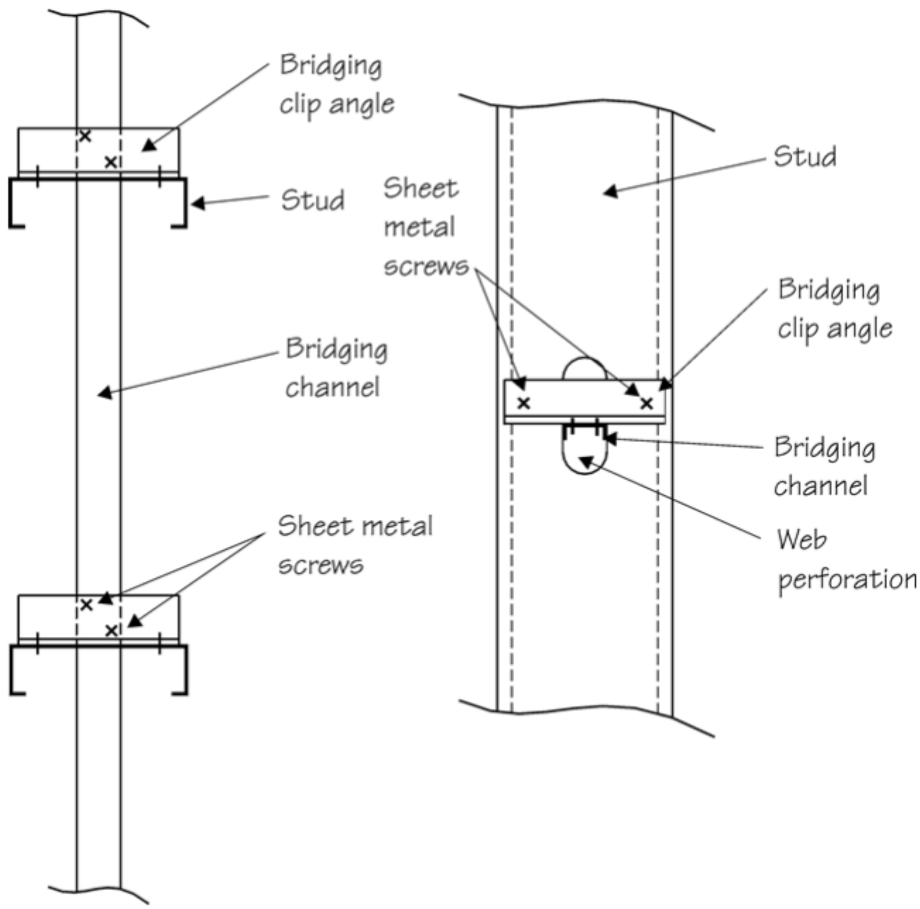


FIGURE I

Three types of bracing are commonly used to resist the torsional component of the load and the tendency of the studs to buckle laterally. These are illustrated in Figures II, III and IV.

The through-the-punchout bridging in Figure II is designed to form a rigid moment connection between the stud and the continuous bridging channel. The torsion in any individual stud is resisted by the major axis bending strength of the bridging channel and the neighboring studs.



THROUGH THE PUNCHOUT BRIDGING

FIGURE II

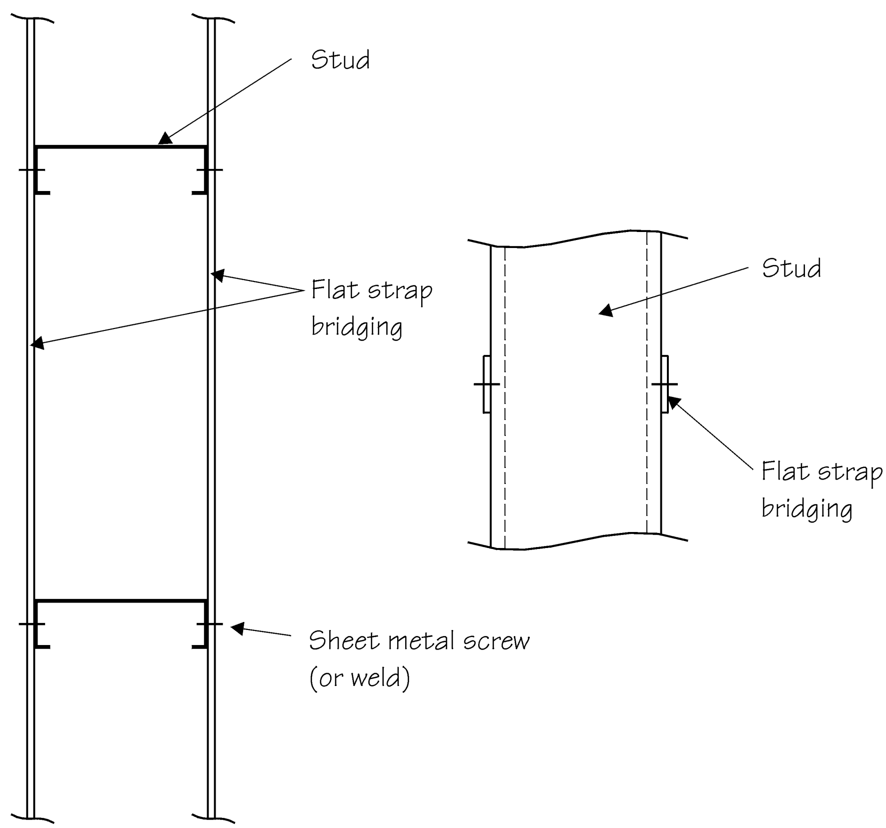
Advantages:

- Periodic anchorage of the bridging to the structure is not as critical as with face bridging (anchorage is only required to resist translation – not rotation).
- Bridging is easily installed from one side.
- Provides support for batt type insulations.

Disadvantages:

- Pre-punched web punchouts must align.
- Each connection requires a clip angle and a minimum of 4 screws or welding.
- Not as stiff as face bridging, particularly in thinner material.
- May not be effective for bridging studs with webs deeper than 8 inches.

The steel strap face bridging in Figure III is designed to act only in tension. Because the studs all have a tendency to twist in the same direction, the straps must be periodically anchored to the primary structure and/or blocking-in between the studs is required every few stud spaces as required structurally.



STEEL STRAP FACE BRIDGING

FIGURE III

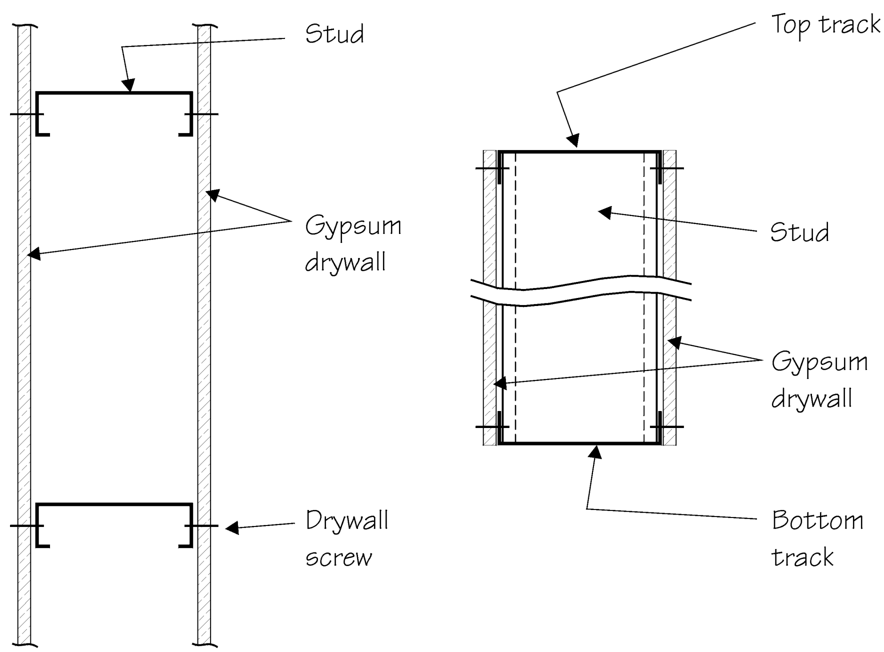
Advantages:

- Provides stiffest form of bridging, even if installed with some initial slackness *(Miller 1989, and Drysdale 1991)*.
- Requires only 2 screws per stud (i.e., 1 screw per flange) or welding.
- Can be installed independently of web punchouts.

Disadvantages:

- To install the bridging, access is required to both sides of the wall assembly (unless connections are welded).
- Bridging forces accumulate over a number of studs and periodic anchorage or blocking-in is required.
- Tension straps are prone to field abuse.

Sheathing as bracing is illustrated in Figure IV. The sheathing may be steel, plywood, cementitious or gypsum wallboard, stucco on lath, wafer board, etc., with the most common being gypsum wallboard. Note that industry practice is to supplement sheathings with a minimum amount of steel bridging in order to align members and to provide the necessary structural integrity during erection and in the completed structure.



SHEATHING AS BRACING

FIGURE IV

Advantages:

- Sheathings provide near continuous support to the studs.
- The diaphragm strength of sheathings transfers bracing forces to the top and bottom tracks.
- Sheathings are usually required to satisfy architectural or building science considerations and are available to act as bracing at little or no additional cost.

Disadvantages:

- Sheathings must be installed on both sides of the stud (or on one side supplemented by steel bridging on the other).
- Gypsum wallboard sheathings will restrain studs in thinner material (0.0346") but may require supplementary steel bridging to effectively restrain studs in thicker material *(Drysdale 1991)*.
- When subjected to wetting, the bracing performance of gypsum wallboard sheathings deteriorates significantly *(Drysdale 1991)*.
- Since the sheathings transfers bracing forces to the top and bottom tracks, the tracks must be designed to accept these forces. These forces are easily removed with typical bottom track detailing but require consideration where slip track type detailing is used at the top.

Allowable height tables for wind bearing studs typically assume that the studs are clad with perfect sheathings on both sides. These sheathings are assumed to completely restrain the studs laterally with no consideration given to lateral instability or secondary torsional stresses. When using such load tables, care is required to insure that this assumption can be achieved in practice. See Note I.

> *Note I*
>
> *If sheathings are absent or cannot be relied on to act as a structural brace, then the typical sheathed manufacturers' load tables can still be used provided there is sufficient steel bridging so that the effects of lateral-torsional buckling and the secondary stresses due to torsion can be neglected.*

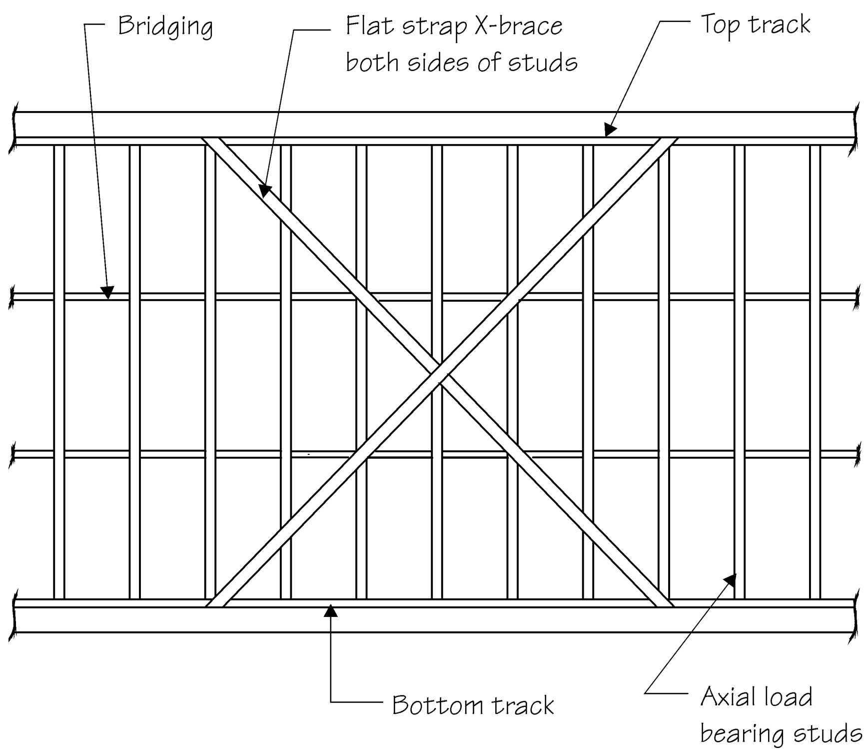
4.2.2 Bracing for Axial Load Bearing Studs

Axial load bearing studs resist both wind and axial loads.

With the superposition of wind and axial loads, bracing is required to provide lateral buckling and torsional restraint for wind *(discussed previously under 4.2.1)* as well as column buckling restraint about the weak axis. Typical bracing types are illustrated in Figures II, III and IV.

Both the through-the-punchout and face bridging in Figures II and III are designed to resist the torsional component of the load and the tendency of the studs to buckle laterally for wind. In addition, they must also prevent weak axis buckling of the studs due to axial loads. These weak axis bracing forces accumulate over a number of studs and the bridging, therefore, requires periodic anchorage to the primary structure. Figure V illustrates a common method for transferring bridging forces to the structure through the use of steel flat strap cross bracing.

An alternative design method is to rely on the shear diaphragm strength of the sheathings (Figure IV) to transfer accumulating bridging forces to the top and bottom tracks, while any individual stud is designed as an all -steel subsystem with no reliance on the sheathing. This design approach is based on the concept that the sheathings may be locally damaged or ineffective and therefore unable to support an individual stud but still structurally adequate over a length of wall to serve as a shear diaphragm.



BRACING FOR AXIAL LOAD BEARING STUDS

FIGURE V

AISI S240 proposes another sheathing braced design approach where the sheathings are adequate to act alone without the benefit of steel bridging (although bridging is required for short-term loading in the absence of sheathing). The capacity of the stud is limited by a number of strength limit states, with the local strength of the sheathing-to-stud connection frequently controlling. With the exception of residential construction, this design approach has not been widely used by the stud industry for a number of reasons:

- The design expressions do not give credit to the presence of supplementary steel bridging which is typically installed in order to align members and to provide the necessary structural integrity during erection and in the completed structure.

- Provided there is adequate steel bridging, this sheathing approach can produce a lower capacity than an all-steel approach.
- The most popular sheathing, gypsum wallboard, is seen by some as too sensitive to moisture and load cycle to act as a reliable structural brace for the service life of a structure.
- The design method does not recognize that the sheathing and the sheathing-to-stud fasteners may already be doing other structural work. This other structural work might include sheathings used as the diaphragm elements in shearwalls, sheathings used to resist torsion in studs, and sheathings used as air barriers.
- The design method does not provide a minimum length of sheathing for a wall segment. Very short lengths of wall may not perform as well as predicted.

Load tables for axial load bearing studs typically assume one of three possible bracing conditions:
(i) The studs are clad with perfect sheathings which completely restrain the studs laterally and only allow column buckling about the stud major axis. When using such load tables, care is required to ensure that this assumption can be achieved in practice.
(ii) The studs are designed with an all-steel approach with no reliance on sheathings. Overall major axis column buckling is checked along with minor axis flexural and torsional-flexural effects between the lines of bridging. The secondary stresses due to wind-induced torsion are usually considered to be small enough to be neglected. *(Very few load tables explicitly account for warping torsional stresses.)*
(iii) The studs are clad with imperfect sheathings and designed in accordance with AISI S240.

4.2.3 Bracing for Joists and Rafters

Joists and rafters are typically designed neglecting torsion and lateral instability effects because sheathings such as plywood subfloors in combination with finished ceilings provide the necessary diaphragm strength. Where sheathing is absent on one or both sides, bridging is usually required to prevent twisting.

In any case, it is standard practice in the industry to supply a minimum amount of bridging to align members and to provide the necessary structural integrity during construction as well as in the completed structure. Load tables for joists and rafters typically assume complete restraint by top and bottom sheathings.

4.3 Design Strengths for Connections

4.3.1 Welds

The unit strengths of fillet and flare groove welds are defined in AISI S100 Sections E2.5 and E2.6, respectively. The unit strength is a function of the weld type, the weld length and the direction of loading.

The design examples in this document use a simplified conservative approach outlined in Appendix A.

4.3.2 Screws

The design strength for sheet metal and self-drilling screw connections is defined in AISI S100. Analytical expressions are provided with the exception of the shear and tensile strength of the screw itself. These tensile and shear strengths are provided in Appendix A.

4.3.3 Concrete Anchors and Fasteners

Suggested design values for power-actuated fasteners (PAFs) are presented in Appendix B. Screw and expansion type concrete anchors are almost exclusively designed using manufacturers' software. This practice is reflected herein, but does not reference a particular manufacturer or software.

5. Serviceability of Cold-Formed Steel Framing Elements

5.1 Member Deflections

Design of cold-formed steel framing typically includes serviceability limits in addition to strength limits discussed above. These limits are intended to ensure that in addition to having adequate strength, members are sufficiently stiff to prevent wall or ceiling finishes from cracking and to limit noticeable deflections in floors.

Deflection limits are commonly specified as a ratio of the span of the member. For example, a wall with plaster finish may have a specified deflection limit of L/360 where L is the span length of the member. The examples in this document are presented on this basis. Deflection limits are typically found in the building codes or may be provided in project specifications.

DESIGN EXAMPLE #1
WIND BEARING INFILL WALL WITH SCREWED CONNECTIONS AND A SHEATHED DESIGN APPROACH

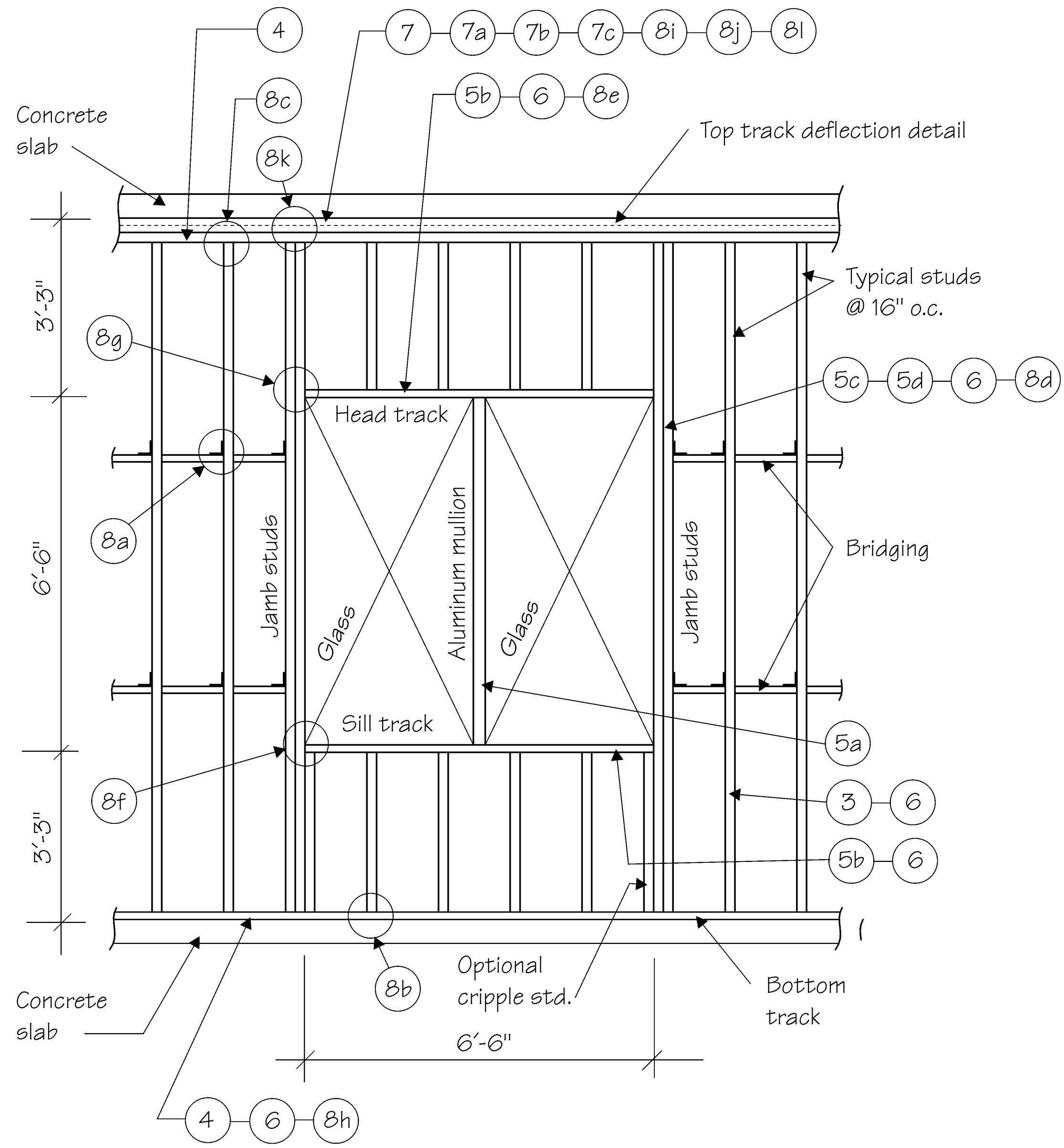


FIGURE 1-1

Introduction

This design example is based on the sheathed design approach which assumes that the sheathing is structurally adequate to resist the torsional component of loads not applied through the shear center and to resist the effects of lateral instability. Members are designed using simple beam theory. All connections are fastened with self-drilling screws.

For welded connections and an unsheathed design approach, see Design Example #2 where the secondary effects of torsion and lateral instability are included.

Figure 1-1 shows the components of a wind bearing infill wall assembly. The numbers shown in Figure 1-1 correspond to the applicable design step used in this example. The basic design steps are as follows:

Step 1: Given
Step 2: Design Wind Load
Step 3: Typical Stud Selection
Step 4: Bottom and Inner Top Track
Step 5: Window Framing Members
Step 6: Final Stud and Track Member Selection
Step 7: Top Track Deflection Detail
Step 8: Connection Design

Step 1 – Given

- EIFS (Exterior Insulation Finish System) exterior finish that applies a uniform load to the studs.
- Stud spacing = 16" o.c.
- Stud height = 13'-0"
- Interior and exterior sheathings provide adequate torsional restraint for loads not applied through the shear center and for lateral instability.
- No axial loads other than the self-weight of the assembly.
- L/360 deflection limit.
- Stud depth = 6" for architectural considerations.

Step 2 – Design Wind Load

From the governing building code, the nominal wind load = ±46.7 psf.

Load combination factors for Allowable Strength Design (ASD) are based on ASCE/SEI 7-10 *(ASCE 2010)* Section 2.4. For strength, the full ASD wind load is used. For deflection, 0.42 times the nominal wind load is used. For further discussion, refer to the Introduction Item 3.2.

Design wind load for strength = 0.6(46.7) = ±28 psf
Design wind load for deflection = 0.42(46.7) = ±19.6 psf

Step 3 – Typical Stud Selection

Refer to a manufacturer's wind bearing stud allowable height table with the following details:

- Height = 13'-0"
- Spacing = 16" o.c.
- ASD wind load = 28 psf
- Deflection limit = L/360

Note that typical wind bearing tables include checks on the following:

- *Deflection check at 0.42 times nominal wind load*
- *Mid-span moment check at nominal wind load*
- *End shear check at nominal wind load*
- *Web crippling may or may not be flagged in the allowable height tables*

> *Note 1-1*
>
> *AISI S240, B1.2.2.4 requires that when sheathing-braced design is used, wall studs be evaluated without sheathing for the following load combination: 1.2D + (0.5L or 0.2S) + 0.2W. This load combination is not commonly included in manufacturers' tables and for simplicity is not checked here. See Design Example #2 for methods used to check studs that do not use sheathing bracing.*

Try 600S162-43 (33) stud:

(Note: 600S162-43 (33) is a universal designator system adopted in AISI S201 for product identification. For a description of the system, see Appendix J.)

From manufacturers' tables – conservatively choose the next highest ASD wind load = 30 psf. Note that most manufacturers' tables are based on ASD wind pressures.

H_{MAX} = 15'-4" > 13'-0" ***OK***

Web crippling is flagged and therefore needs to be checked. *(See Note 1-2.)*

Note 1-2

For a typical stud-to-track connection (welded or screwed), the allowable web crippling strength of the stud is adequately predicted by AISI S100 Eq. C3.4.1-1 assuming a nominal bearing length of 1 inch (Drysdale 1991) provided there is at least that much bearing between the stud and the vertical leg of the track. For this web crippling calculation to be valid, web punchouts are not permitted near the end of the stud. Load tables typically set the distance from the center line of the last punchout to the end of the stud at 12" minimum as stipulated in AISI S201. Punchouts closer than 12" to the end of the stud may or may not result in a reduction to the allowable web crippling strength. Refer to AISI S100 web crippling provisions for guidance.

Research on the stud-to-track connection has resulted in an alternate fastened EOF web crippling expression available in AISI S240 Section B3.2.5.1. The AISI S240 web crippling equation predicts higher capacities than the AISI S100 equivalent used here.

Required P_{ext} = 0.5 wL (spacing/12)
= 0.5(28)(13)(16/12)
= 243 lb.

From tables for 1" bearing length and end one flange (EOF) fastened condition:

P_{all} = 259 lb. > 243 lb. ***OK***

Step 4 – Bottom and Inner Top Track

AISI S240 Section B3.2.5.1 provides a design procedure for checking local failure (tear through) of the track. AISI S240 does not require this failure mechanism to be checked when the thickness of the track is greater than or equal to the thickness of the stud.

Try a track that is thinner than the typical stud – 600T125-33 (33) track:

From the previous step:
Required P_{ext} = 243 lb.

Check track tear through using AISI S240 Section B3.2.5.1:

$P_{nst} = 0.6\, t_t\, w_{st}\, F_{ut}$

where:
P_{nst} = nominal strength for stud-to-track connection when subjected to transverse loads
t_t = track design thickness

w_{st} = 20 t_t + 0.56α (α = 1 when t_t is in inches and α = 25.4 when t_t is in mm)
F_{ut} = tensile strength of the track
Ω = 1.70

For 600T125-33 (33),

t_t = 0.0346"
F_{ut} = 45 ksi

P_{nst} = 0.6(0.0346)[20(0.0346) + 0.56(1)](45)(1000)
= 1170 lb.

P_{all} = P_{nst} /Ω = 1170/1.70
= 688 lb. > 243 lb. ***OK***

Therefore, 600T125-33 (33) track is acceptable for typical stud tear through. For a discussion of final track selection, see Step 6.

Step 5 – Window Framing Members

Distribution of wind loads on glass to supporting members.

The transfer of wind loads from the window assembly to the surrounding stud framing is a complicated issue depending on the structural behavior of the window itself and the connection of the window to the surrounding cold-formed steel framing members.

It is generally sufficient to assume either a 4-way or a 2-way wind load distribution as illustrated in Figure 1-2.

For this example:

$\frac{\text{Height}}{\text{Width}} = \frac{6.50}{3.25} = 2.00$ for glass and the 2-way distribution is appropriate.

See Note 1-3.

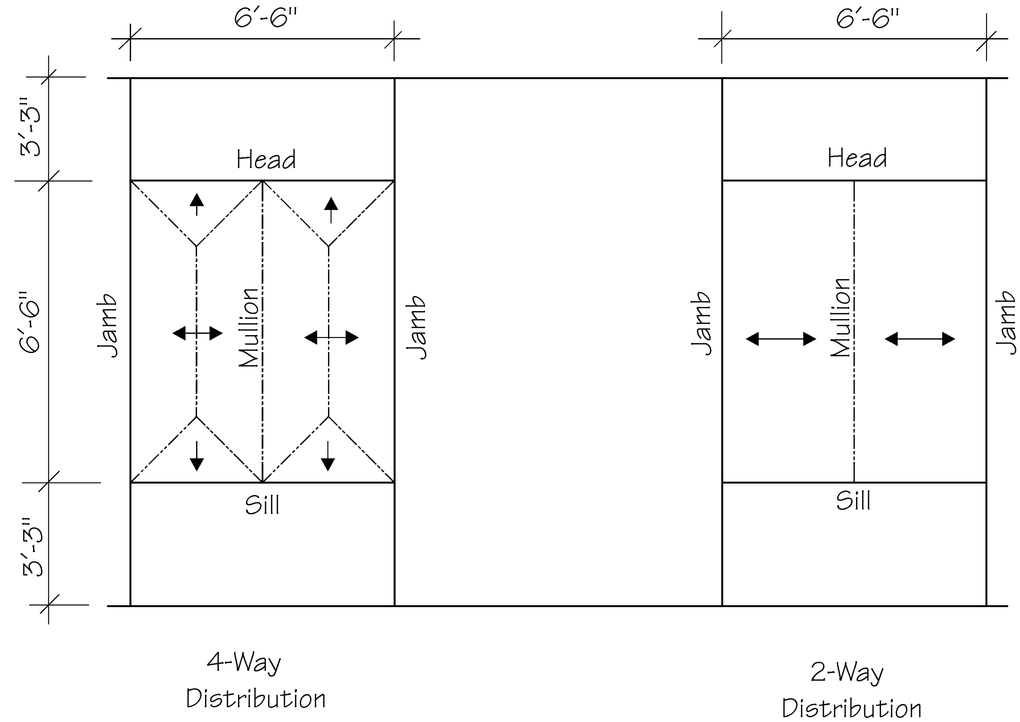


FIGURE 1-2

Note 1-3

A 2-way wind load distribution (Fig. 1-2) is usually adequate for windows with a height/width ratio greater than or equal to 2. This design example was also checked with a 4-way distribution (calculations not included here) indicating the following "errors" in the 2-way assumption:

Differences Between 4-Way and 2-Way Assumption

Sill and Head Track

Moment	*4-Way/2-Way*	*= 1.00*
Shear	*4-Way/2-Way*	*= 1.125*
Deflection	*4-Way/2-Way*	*= 1.05*

Jamb Stud

Moment	*4-Way/2-Way*	*= 0.99*
Shear	*4-Way/2-Way*	*= 1.00*
Deflection	*4-Way/2-Way*	*= 0.99*

Note that the as-built behavior of the window sill, head and jamb framing may vary from either the 2-way or 4-way assumption depending on both the structural behavior of the window itself and the as-built connection of the window to the surrounding cold-formed steel framing members. Designing for the actual load transfer details around windows is complicated, often not known at the time of stud design and usually not required.

Step 5(a) – Aluminum Mullion Loading

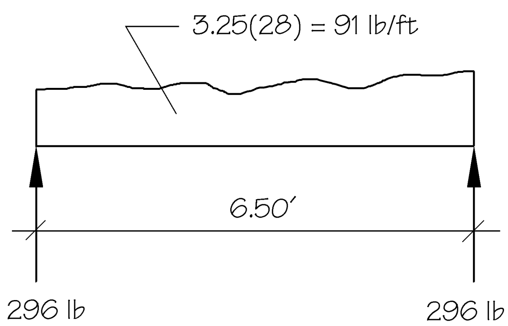


FIGURE 1-3

Step 5(b) – Sill and Head Track Design

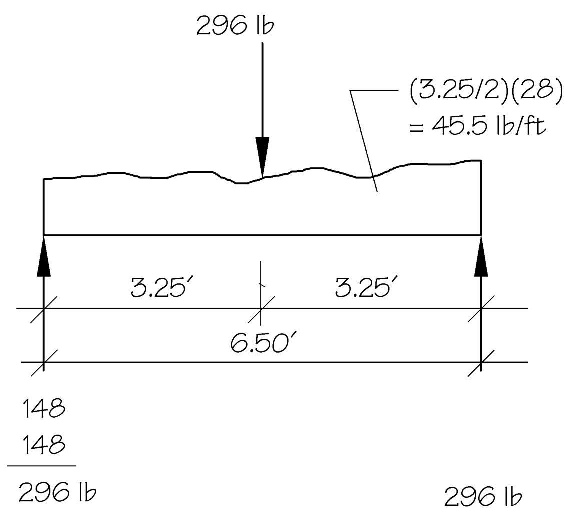


FIGURE 1-4

Required Moment:

$$M_{req} = \frac{PL}{4} + \frac{wL^2}{8}$$

$$= \left[\frac{296(6.5)}{4} + \frac{45.5(6.5)^2}{8}\right]\left[\frac{12}{1000}\right]$$

$$= 8.66 \text{ in.kips}$$

Required Shear:

$$V_{req} = 296 \text{ lb.}$$

Required Moment of Inertia:

$$\begin{aligned}\delta &= \frac{PL^3}{48EI} + \frac{5wL^4}{384EI} \\ &= \left[\frac{296\,(78)^3}{48\,(29.5)(10^6)\,I} + \frac{5(45.5/12)(78)^4}{384(29.5)(10^6)\,I}\right][0.70] \\ &= \frac{0.1128}{I}\end{aligned}$$

For deflection, limit = L/360

$$\delta = 78/360 = 0.217 \text{ in.}$$

$$I_{req} = \frac{0.1128}{0.217} = 0.520 \text{ in}^4$$

Try 600T125-43 (33) track:

M_{all}	= 9.11 in.kips	> 8.66 in.kips	***OK***
V_{all}	= 1377 lb.	> 296 lb.	***OK***
$I_{x(def)}$	= 1.77 in^4	> 0.520 in^4	***OK***

Reinforced window head for gravity loads – see Figures 1-6A and 1-6B.

The 600T125-43 (33) track has adequate major axis strength to resist wind loads. Some strengthening may be required, however, to resist the tendency for the window head to sag under the weight of the wall assembly above. The sagging will be further aggravated by the friction between the inner and outer top track when some relative slab movement occurs. The inner top track *(if the inner and outer top track deflection detail is used as shown in Figure 1-12)* cannot be relied on as a stiffening element because it may not be continuous over the window.

On narrow windows, the sagging is insignificant and can be ignored. On wider windows, a lintel may be required. Note that with the inherent stiffness of welded construction, sagging is less of a concern.

For windows of intermediate width, the window head reinforcement detail shown in Figure 1-6A may suffice. The detail in Figure 1-6B is appropriate for wide windows.

For this window, try creating a built-up section as in Figure 1-6A. Assume the additional stud and track sections resist gravity load and the remaining track section resists wind.

Weight of EIFS wall assembly above window head = 9 psf (assumed).

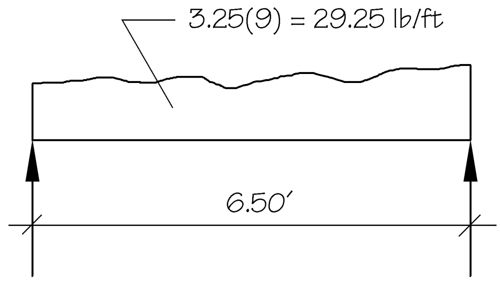


FIGURE 1-5

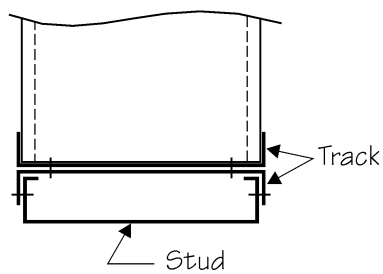


FIGURE 1-6A

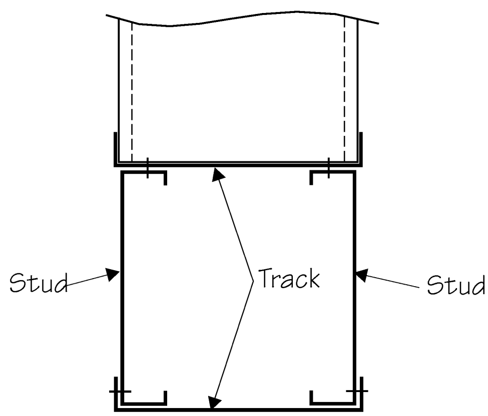


FIGURE 1-6B

Required moment (dead load only):

$$M_{req} = \frac{wL^2}{8}$$

$$= \left[\frac{3.25(9)(6.5)^2}{8}\right]\left[\frac{12}{1000}\right] = 1.85 \text{ in.kips}$$

Required moment of inertia (dead load only):

$$\delta = \frac{5wL^4}{384EI}$$
$$= \frac{5(3.25)(9/12)(78)^4}{384(29.5)(10^6)I}$$
$$= \frac{0.0398}{I}$$

For deflection, limit = L/360 say:

$$\delta = 78/360 = 0.217 \text{ in.}$$
$$I_{req} = \frac{0.0398}{0.217} = 0.183 \text{ in}^4$$

Try 600T125-43 (33) track plus 600S162-43 (33) stud:

As an approximation, assume the reinforcing stud acts alone for strength. For deflection, assume the moment of inertia is given by the simple addition of the weak axis properties of the reinforcing stud and track.

Using the lesser of web or lips in compression:
M_{all} = 2.13 in.kips > 1.85 in.kips ***OK***

$I_{y(def)} \approx$ fully effective weak axis moment of inertia for stud and track
= 0.148 + 0.044 = 0.192 in^4 > 0.183 in^4 ***OK***

Note that the fully effective (unreduced for local buckling) weak axis moments of inertia have been used for the deflection check since manufacturers' tables rarely show effective weak axis moments of inertia appropriate for deflection calculation. This is an unconservative assumption but adequate given the additional stiffening from attached sheathings and the inner top track (even if discontinuous over the window) not accounted for here.

In Figure 1-6B, sag is resisted by the major axis strength and stiffness of 2 - 600S162-43 (33) (the typical stud):

$I_{y(def)}$ = 2(2.32) = 4.64 in^4 >> 0.183 in^4 ***OK***

Step 5(c) – Jamb Stud Design

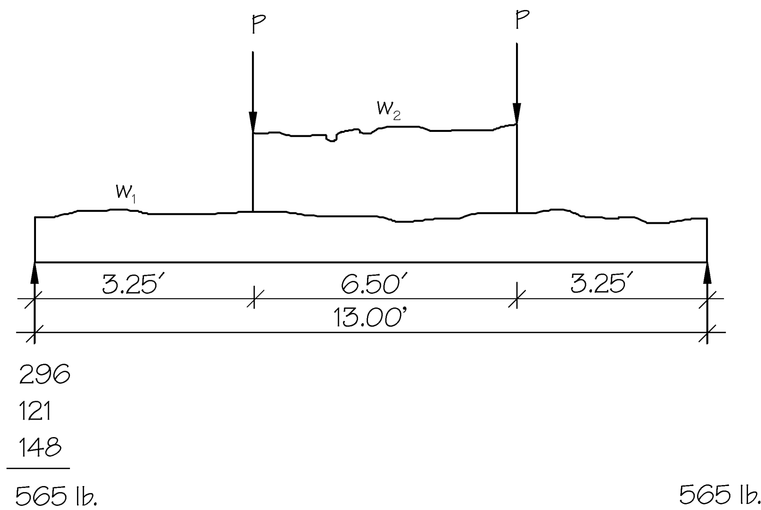


FIGURE 1-7

The loading on the jamb stud is shown in Figure 1-7.

w_1 = (1.33/2)(28) = 18.6 lb/ft
w_2 = (3.25/2)(28) = 45.5 lb/ft
P = 296 lb. (head/sill reaction)

Required Moment (*maximum at mid-span*):

M_{req} = [565(6.5)-296(3.25)-18.6(6.50)(3.25)-45.5(3.25)(1.625)] [12/1000]
= 24.9 in.kips

Required Shear and Web Crippling:

V_{req} = 565 lb.

Required Moment of Inertia:

Approximate deflections by replacing partial uniformly distributed load with a point load, P_1, as shown in Figure 1-8.

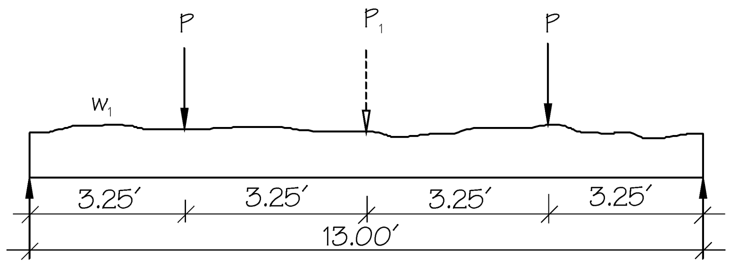


FIGURE 1-8

P = 296 lb.
P_1 = 6.50(45.5) = 296 lb.
w_1 = 18.6 lb/ft
L = 156 in. (span length)
a = 39 in. (distance from support to P)

$$\delta = \frac{5w_1L^4}{384EI} + \frac{P_1L^3}{48EI} + \frac{Pa}{24EI}\left(3L^2 - 4a^2\right)$$

$$= \frac{0.70}{29.5(10^6)I}\left[\frac{5(18.6/12)(156)^4}{384} + \frac{296(156)^3}{48} + \frac{296(39)\left[3(156)^2 - 4(39)^2\right]}{24}\right]$$

$$= \frac{1.603}{I}$$

Note that by computer

$$\delta_{exact} = \frac{1.542}{I}$$

Therefore, replacing the partial uniform distributed load (UDL) with a point load is conservative by 4.0%.

For a deflection limit of L/360, δ = 13(12)/360 = 0.433 in.

Using the exact solution:

$$I_{req} = \frac{1.542}{0.433} = 3.56\ in^4$$

Possible Built-Up Member Configurations:

See Figures 1-9, 1-10 and 1-11.

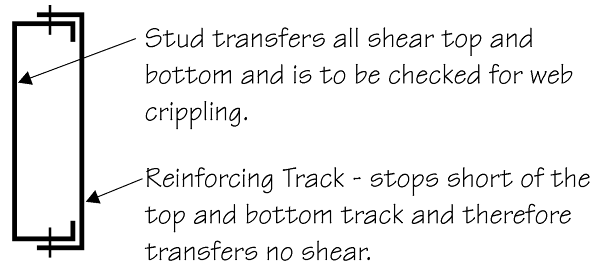


FIGURE 1-9

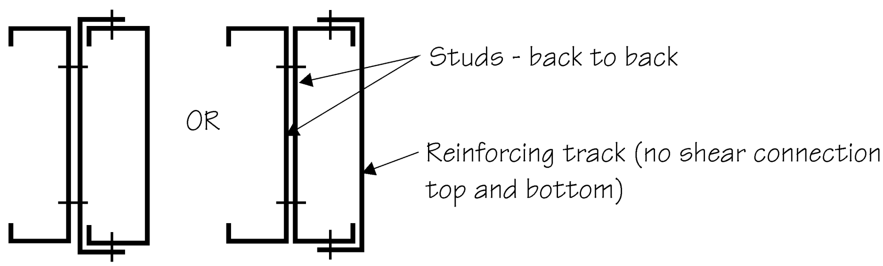


FIGURE 1-10

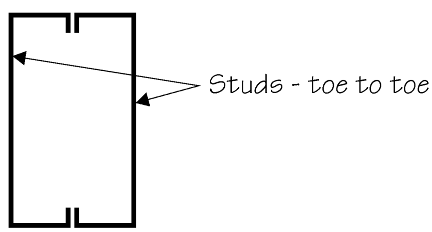


FIGURE 1-11

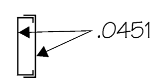
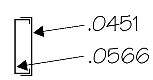
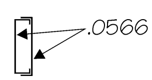
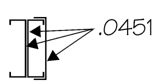
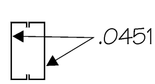
Step 5(d) – Jamb Selection

The calculations for the jamb selection are summarized in Table 1-1.

This table is based on the design approximation that the allowable moment and moment of inertia of the built-up sections are the simple addition of the component parts. See Note 1-3 for an alternative approach.

Note that the track section used as part of the built-up member will exceed 10'-0" in length, which may require a special order. Check with the local manufacturers. For studs, a punched section is assumed for allowable shear, allowable moment and moment of inertia. The track is not punched.

Table 1-1 Jamb Stud Selection Table

		Moment (in.kips)	Moment of Inertia (in^4)	Web Crippling (lb.)	Shear (lb.)
	Required ⟶	24.9	3.56	565	565
Built-Up Section	Component Sections				
A 	Track Stud Sum	9.11 16.68 25.79	1.77 2.32 4.09	0 259 259	0 1240 1240
B 	Track Stud Sum	9.11 30.33 39.44	1.77 2.86 4.63	0 599 599	0 947 947
C 	Track Stud Sum	17.73 30.33 48.06	2.24 2.86 5.10	0 599 599	0 1947 1947
D 	Track Stud Stud Sum	9.11 16.68 16.68 42.47	1.77 2.32 2.32 6.41	0 259 259 518	0 1240 1240 2480
E 	Stud Stud Sum	16.68 16.68 33.36	2.32 2.32 4.64	259 259 518	1240 1240 2480

Built-Up Section A: Unsatisfactory for web crippling.
Built-Up Section B: ***OK***
Built-Up Section C: ***OK***

Built-Up Section D: Unsatisfactory for web crippling.

Built-Up section D consists of two back-to-back studs reinforced with a track section that does not transfer any end shear *(see the back-to-back alternative, Figure 1-10)*. The allowable web crippling strength, P_{ext}, of two studs connected back-to-back is actually greater than two times the web crippling strength for a single stud. Refer to *AISI S100* Section C3.4.1 and the following calculations:

$$P_{all} = \frac{C\,t^2\,F_y \sin\theta}{\Omega}\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right)$$

Choose coefficients for built-up sections and end one flange loading. Coefficients and safety factors are the same for fastened and unfastened conditions.

R	= 0.0712"
t	= 0.0451"
Depth	= 6"
h	= Depth - 2t - 2R = 5.767"
N	= 1"
F_y	= 33 ksi
θ	= 90 degrees
C	= 10
C_R	= 0.14
C_N	= 0.28
C_h	= 0.001
Ω	= 2.00

Substituting:
P_{all} = 0.634 kips per web

For back-to-back:
P_{all} = 2(634) = 1268 lb. > 565 lb. ***OK***

Note that the track reinforcing for the built-up jamb stud may not be necessary in order to satisfy strength or stiffness requirements, but is required to facilitate connections at the window head and sill and for the connection of the window frame itself.

It was noted earlier under typical bottom and inner top track that if the track thickness is equal to or greater than the thickness of the stud, then track tear through need not be checked. This conclusion is based on single stud-to-track connections and may not apply to studs back-to-back. However, the design expression in Step 4 for a single stud can be applied to the back-to-back case to check the track thickness.

From Step 4, for 600T125-33:
$P_{all} = P_{nst}/\Omega$ = 688 lb. > 565 lb. ***OK***

Built-Up Section E: Unsatisfactory for web crippling.

Two studs toe-to-toe are not recommended as a built-up member in screwed construction because it is difficult to effectively connect the studs together. This toe-to-toe configuration is only recommended in welded construction.

Note 1-4

As an alternative approach to built-up jamb member selection, the load, W, carried by each of the component parts can be apportioned according to the relative stiffness of the members. By equating deflections, the following formulas can be obtained:

$$W_{STUD} = \frac{W_{TOTAL}}{1 + \frac{I_{X(TRACK)}}{I_{X(STUD)}}}$$

$$W_{TRACK} = W_{TOTAL} - W_{STUD}$$

This relative stiffness approach can produce more conservative results when moment controls the jamb selection. Usually, deflection or web crippling govern and the simple addition approach used here is adequate. When moment controls, the simple addition approach is likely still valid because the member that first reaches yield is assumed to shed any additional loading to the other parts of the built-up member that still have strength reserve. This assumption has not been confirmed by testing. Note that the relative stiffness approach does not apply to web crippling because the stud section(s) are assumed to carry the entire load for this case.

Step 6 – Final Stud and Track Member Selection

> *Note 1-5*
>
> *It can be impractical to mix different thicknesses of stud and different thicknesses of track on the same project - or at least on the same floor.*
> - *Mixed thicknesses can result in the wrong thickness in the wrong place on site.*
> - *Manufacturers do not stock stud and track, but rather they roll to order. By specifying one type of stud and track, the delivery time is reduced and the cost premium for small production runs is eliminated.*

There is little justification for mixing thicknesses of stud and track on this project. See Note 1-5. The following member selections are therefore appropriate:

- Typical Stud: 600S162-43 from Step 3
- Bottom Track: 600T125-33 is adequate for track tear through from Step 4. But 600T125-43 is required for the window head and sill. Use 600T125-43 for bottom track as well.
- Inner Top Track: Match bottom track thickness = 0.0451".
- Window Sill Track: 600T125-43 from Step 5(b)
- Built-Up Window Head: 2 - 600T125-43 track
1 - 600S162-43 stud
from Step 5(b)
- Built-Up Window Jamb: 2 - 600S162-43 stud
1 - 600T125-43 track
Built-Up Section D from Step 5(d)

Step 7 – Top Track Deflection Detail

Two different top track deflection details are proposed – an inner and outer top track (Figure 1-12) and a single top track (Figure 1-13).

In either case, the top track detail will be used to accommodate slab deflections (and the possible effect of column shortening) such that the studs are not loaded axially. This detail also accommodates construction tolerance in the slab-to-slab height such that the studs do not have to be custom cut to length on site. Allow for a construction tolerance of say ± 1/4" (The ±1/4" implies considerably better than average concrete tolerances on this project).

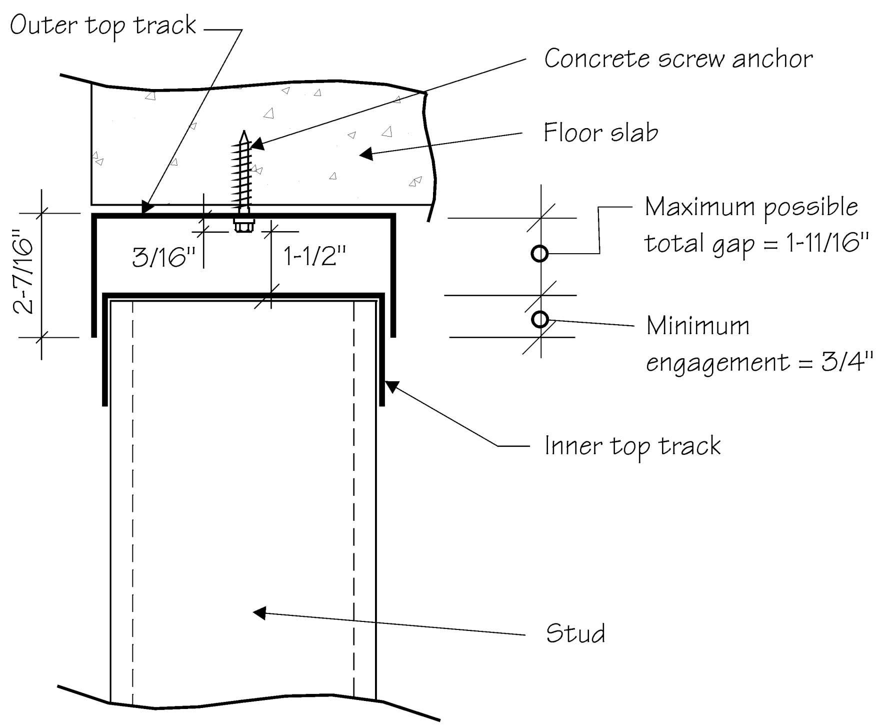


FIGURE 1-12

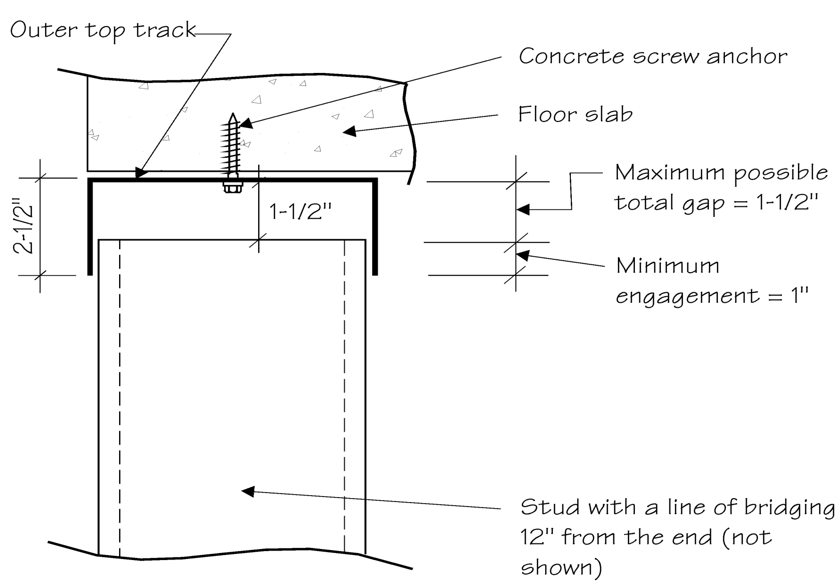


FIGURE 1-13

From the project structural engineer, the specified long-time slab deflection due to all sustained loads and the immediate deflection due to live load occurring after attachment of steel stud wall = 1/2" upper floor slab relative to lower floor slab and vice versa. The effect of column shortening is assumed to be negligible.

At the time of installation, the deflection gap should be 3/4" plus or minus the construction tolerance of 1/4". This results in a minimum possible gap at the time of installation of 1/2" which is adequate to accommodate slab deflections above, assuming the slab below does not deflect. Conversely, if the slab below deflects 1/2" and the slab above does not deflect, then the maximum possible gap is 3/4" + 1/4" + 1/2" = 1-1/2". See Figures 1-12 and 1-13.

Step 7(a) – Inner and Outer Top Track Deflection Detail With Concrete Screw Anchors

The deflection gap is taken as the clear distance between the head of the concrete screw anchor and the inner top track. The maximum total gap is given by 1-1/2" + 3/16" = 1-11/16". *(Note that wedge-type expansion anchors are not practical in this application because the exposed portion of the fastener interferes too much with the deflection gap.)*

Assuming a minimum engagement of 3/4", then the leg of the outer top track must be 1-11/16" + 3/4" = 2-7/16".

The 1-11/16" maximum total gap is used in the calculations that follow to determine the thickness of the outer top track.

Summary:

Tolerance	= ±1/4"
Deflection	= 1/2"
Concrete screw anchor head	= 3/16"
Minimum installation gap	= Deflection + Screw Head
	= 1/2" + 3/16" = 11/16"
Maximum installation gap	= Deflection + Screw Head + 2 x Tolerance
	= 1/2" + 3/16" + 1/2" = 1-3/16"
Maximum possible gap	= Maximum installation gap + Deflection
	= 1-3/16" + 1/2" = 1-11/16"

One leg of the outer top track is assumed to be loaded uniformly by the inner top track which spreads the concentrated reactions from the studs.

This assumption is reviewed in Appendix E where the inner top track is analyzed as a beam on an elastic foundation (i.e. the outer top track).

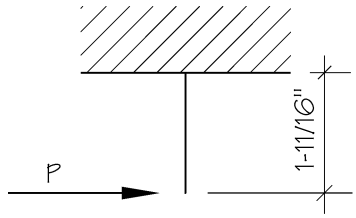


FIGURE 1-14

Figure 1-14 illustrates the cantilever design assumption for the outstanding leg of the outer top track. Check the required track thickness:

$$\begin{aligned} P &= \left(\frac{\text{Stud Height}}{2}\right)w \\ &= (13/2)(28) \\ &= 182 \text{ lb/ft of length for strength} \end{aligned}$$

$$\begin{aligned} M_{req} &= 1.6875P = 1.6875(182) \\ &= 307 \text{ in.lb/ft. of length} \end{aligned}$$

Assuming an elastic section modulus, F_y = 50 ksi and a 1-foot length of track:

$$\begin{aligned} M_{all} &= (1/6)bt^2F_y/\Omega \\ &= (1/6)(12)t^2(50{,}000)/1.67 \\ &= 59900t^2 \end{aligned}$$

For $M_{all} = M_{req}$

$$t \geq \sqrt{\frac{307}{59900}} = 0.0716 \text{ in.}$$

Use standard t = 0.0713 in. (≈ 0.0716 in.) with F_y = 50 ksi.

Note 1-6

1. *The outer top track has been sized using an elastic section modulus. If a plastic section modulus, $Z=(1/4)bt^2$, had been used, the allowable moment, M_{all}, would have been 50% higher.*

2. *This 50% reserve strength is required to offset the errors in the assumption that the inner top track loads the outer top track uniformly. See Appendix E.*

3. *Some manufacturers sell outer top track in F_y = 33 ksi material which will increase the required thickness accordingly.*

Check the outer top track horizontal movement using the formula developed in Appendix D:

$$\delta \geq \frac{P}{EI}\left[\frac{L_2^2 L_1}{8} + \frac{L_2^3}{3}\right]$$

where:

$$P = \left(\frac{Stud\ \ Height}{2}\right)w$$

$$= (13/2)(46.7)(0.42) \quad = 127\ lb.\ for\ deflection\ (per\ foot\ of\ \ length)$$

L_1 = 6" (track width)
L_2 = 1-11/16" (maximum gap)
E = 29,500,000 psi
I = $bt^3/12 = 12(0.0713)^3/12 = 0.362(10^{-3})$ in^4/ft of length

Substituting into the above equation and solving gives:

$\delta \geq 0.044$" *(see Note 1-7)*

Note 1-7

1. *Deflections in the outer top track may be locally higher. See Appendices D and E.*

2. *No limit on δ is proposed; however, horizontal movement in the top track detail should be expected and accounted for in the architectural detailing.*

Use 0.0713" thick outer top track with 2-7/16" leg length and F_y = 50 ksi.

Note 1-8

Sheathing is used in this design example to brace the studs to resist the torsional loads not applied through the shear center and to resist the effects of lateral instability. These sheathing forces are transferred to the top and bottom tracks where they accumulate until the track is connected to the primary structure.

Note in Figure 1-12 that the inner top track is not connected to the outer top track. This detail is commonly used where significant lateral drift is expected; for example, during an inelastic seismic event. The inner track is allowed to slide relative to the outer track in the plane of the wall so in-plane drift is accommodated without causing the framing and finishes in the plane of movement to rack. Where drift accommodation is a concern, considerable detailing effort is required to ensure compatibility of movements of adjacent structural and non-structural elements. A complete discussion of seismic drift accommodation is beyond the scope of this design guide.

Step 7(b) – Inner and Outer Top Track Deflection Detail With Power-Actuated Fasteners

Power-actuated fasteners have a negligible head dimension when installed, and no allowance for the head is required when detailing the deflection gap.

Deflection gap summary with power-actuated fasteners:

Tolerance	= ± 1/4"
Deflection	= 1/2"
Concrete anchor head	= 0"
Minimum installation gap	= Deflection + Anchor Head
	= 1/2"
Maximum installation gap	= Deflection + Anchor Head + 2 x Tolerance
	= 1/2" + 0" + 1/2" = 1"
Maximum possible gap	= Maximum installation gap + Deflection
	= 1" + 1/2" = 1-1/2"

Assuming a minimum engagement of 3/4", the leg of the outer top track must be 1-1/2" + 3/4" = 2-1/4".

Reworking the track thickness calculations from Step 7(a) with a cantilever leg length of 1-1/2" gives the following:

t = 0.0675"
Use next standard design thickness = 0.0713" with F_y = 50 ksi
and reworking the outer top track horizontal movement calculations with t = 0.0713" and L_2 = 1-1/2" gives:

$\delta \geq 0.033"$

See Note 1-7.

Step 7(c) – Single Top Track Deflection Detail

See Figure 1-13. Compared with the inner and outer top track detail, the single outer top track has the following advantages and disadvantages:

Advantages

- Easier to install.
- Fasteners that connect the top track to the primary structure can be inspected.
- Fasteners such as wedge-type expansion anchors can be used since the fastener head does not interfere with the deflection gap.
- Reduces material build-up. Build-up can be problematic for finishes.

Disadvantages

- The single top track deflection detail provides no torsional restraint to the top of the studs and a line of bridging is typically required close to the end of the stud. *(If through-the-punchout style bridging is used, the bridging is typically located 12 inches from the end of the stud. Flat strap bridging can be closer to the end because there is no punchout to compromise web crippling capacity of the stud.)*

- Only a local portion of the top track is mobilized to resist a stud reaction. This is particularly an issue for larger jamb reactions where a supplementary slide clip might be required.
- The web crippling capacity of the stud may be lower.

The deflection gap summary will be the same as Step 7(b) except that the minimum engagement is increased to maintain web crippling capacity (calculations to follow).

Assuming a minimum engagement of 1 in., the leg of the outer top track must be 1-1/2" + 1" = 2-1/2".

The thickness of the single top track can be checked using the provisions of AISI S240 Section B3.2.5.2.

From AISI S240 Equation B3.2.5.2-1:

$$P_{ndt} = \frac{w_{dt} t^2 F_y}{4e}$$

and

$$w_{dt} = 0.11(\alpha^2)(e^{0.5}/t^{1.5}) + 5.5\alpha \le S$$

where:

P_{ndt} = nominal strength of the deflection track when subjected to transverse loads
w_{dt} = effective track length
S = center-to-center spacing of studs
t = track design thickness
F_y = design yield strength of track material
e = design deflection gap
α = 1 when e, t and S are in inches
= 25.4 when e, t and S are in mm
Ω = 2.80

The above equations are valid within the following range of parameters:

<u>Stud Section</u>

Design Thickness:	0.0451 in. to 0.0713 in.
Design Yield Strength:	33 ksi to 50 ksi
Nominal Depth:	3.50 in. to 6.0 in.
Nominal Flange Width:	1.625 in. to 2.5 in.
Stud Spacing:	12 in. to 24 in.
Stud bearing Length:	3/4 in. minimum

<u>Track Section</u>

Design Thickness:	0.0451 in. to 0.0713 in.

Design Yield Strength: 33 ksi to 50 ksi
Nominal Depth: 3.50 in. to 6.0 in.
Nominal Flange Width: 2.0 in. to 3.0 in.

In addition, the clear distance from the stud to the end of the track must be greater than or equal to $w_{dt}/2$ *(AISI S240 B3.2.5.2)*.

Note that the thickness of the track cannot be solved for directly - use trial and error.

For a typical stud with S = 16":

P_{req} = (1/2)(stud height)(16/12)(28)
= (13/2)(16/12)(28)
= 243 lb. per stud

Try t = 0.0872" with e = 1.5" (no fastener head clearance required) and F_y = 50 ksi:

$$w_{dt} = 0.11(1)^2(1.5^{0.5}/0.0872^{1.5}) + 5.5(1)$$
$$= 10.73\,\text{in.} \le 16\,\text{in.}$$

$$P_{ndt} = \frac{10.73(0.0872)^2(50)}{4(1.5)}$$
$$= 0.680\ \text{kips}$$

$P_{all} = P_{ndt}/\Omega$ = 680/2.80
= 243 lb. = P_{req} ***OK***

Use the next standard design thickness = 0.1017 in.

Note that this design thickness exceeds the 0.0713 in. limit in AISI S240. This extrapolation of the design equations is deemed acceptable under the rational analysis provisions of *AISI S100* Section A1.2, and the Ω = 2.80 is more conservative than the safety factor requirements of that section.

Note also that for the single top track deflection detail, no analytical method for checking serviceability is currently available.

Recheck stud web crippling in the top track.

The typical stud web crippling check (Step 3) was based on 1" of bearing length and the end one flange (EOF) fastened condition. The fastened approach is not permitted for single top track deflection details because both stud flanges are not connected to the track flanges. Web crippling, therefore, reverts to the expression in A*ISI S100* for unfastened end one flange loading. See *AISI S100* Table C3.4.1-2.

$$P_{all} = \frac{C\,t^2\,F_y \sin\theta}{\Omega}\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right)$$

where:

R	= 0.0712"
t	= 0.0451"
Depth	= 6"
h	= Depth - 2t - 2R = 5.767"
N	= 3/4" or 1"
F_y	= 33 ksi
θ	= 90 degrees
C	= 4
C_R	= 0.14
C_N	= 0.35
C_h	= 0.02
Ω	= 1.85 (unfastened)

Substituting:

P_{all} = 225 lb. with N = 3/4"
= 245 lb. with N = 1"

and from the previous calculations:
P_{req} = 243 lb. per stud > 225 lb. with N = 3/4" ***UNSATISFACTORY***
< 245 lb. with N = 1" ***OK***

Therefore, use the minimum engagement of 1" for the single top track deflection detail.

For a jamb stud, assume the single top track design provisions apply *(from AISI S240 Section B3.2.5.2),*

From Step 5(c), the jamb reaction is:
P_{req} = 565 lb.

t = 0.1017 in.
e = 1.5 in.
F_y = 50 ksi

Substituting gives:
w_{dt} = 9.65 in.
P_{ndt} = 0.832 kips

$P_r = P_{ndt} / \Omega$ = 832/2.80
= 297 lb. < 565 lb. ***UNSATISFACTORY***

The jamb stud overstresses the single top track. Provide a proprietary slide clip to connect the top of the jamb stud to the primary structure.

Step 8 – Connection Design

Member selection has been based on the assumption that the inner and outer wall sheathings provide adequate torsional restraint for loads not applied through the shear center and for lateral instability.

Provided the sheathing acts as a brace, a number of connection details have no required forces to resist and the detailing of these connections is therefore based on industry practice rather than structural design. These details include bridging and stud to top and bottom track connections. Other connection details require engineering.

Step 8(a) – Bridging

Space bridging in accordance with manufacturers' recommendations. A maximum spacing of 5'-0" o.c. is common and is used here. Therefore, for a 13'-0" span, two rows of bridging are required at third points.

Use 150U050-54 continuous through-the-punchout bridging channel with 1-1/2" × 1-1/2" × 0.0566" × 5-1/2" long clip angles at each stud. Connect bridging channel to clip angles and clip angles to studs with 2 - #10 self-drilling screws. The screw locations in the clip angles may be defined by pre-drilled pilot holes provided by the manufacturer. See Figure 2-12.

Step 8(b) – Stud to Bottom Track Connection

Use #10 self-drilling screws with low profile heads to connect stud to track (flange to flange). See Figure 2-16.

Step 8(c) – Stud to Inner Top Track Connection

Figure 1-15 illustrates the details for the stud to top track connection using self-drilling screws. See Note 1-9.

Note 1-9

1. *With welded construction, the long-legged inner top track can be replaced with conventional track. The welds do not interfere with the sliding connection.*

2. *Do not install drywall screws above the line of the #10-16 self-drilling screw shown; otherwise, the performance of the sliding connection will be impaired.*

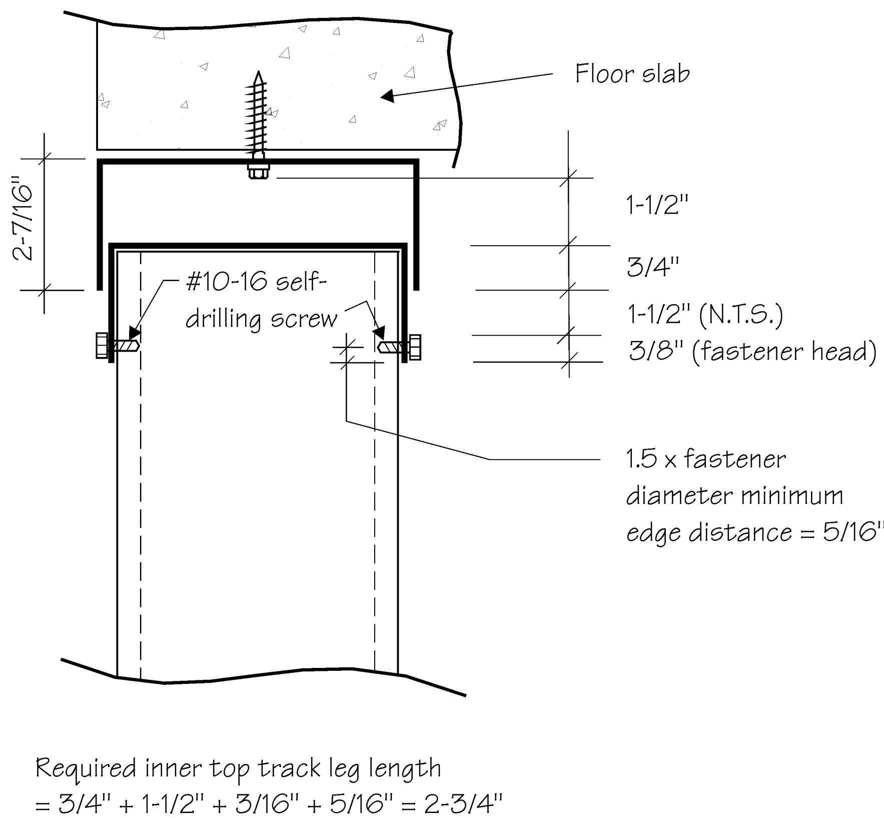


FIGURE 1-15

Step 8(d) – Built-Up Jamb Stud Interconnection

The connection requirements for a track and stud jamb member are not defined in *AISI S100.* Experience in the field indicates that a connection spacing of 24" o.c. is adequate. The details are shown in Figure 1-16

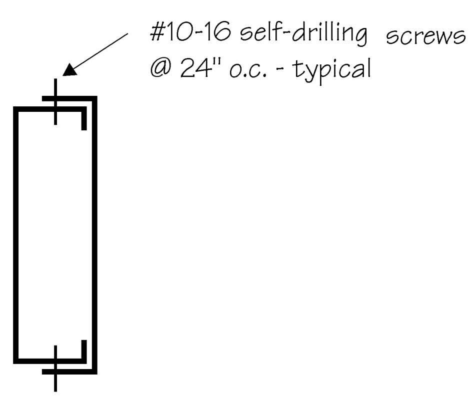


FIGURE 1-16

Step 8(e) – Built-Up Window Head Interconnection

The connection requirements shown in Figure 1-17 are similar to the built-up jamb. These fastener requirements would also apply to the alternative built-up window head in Figure 1-6B.

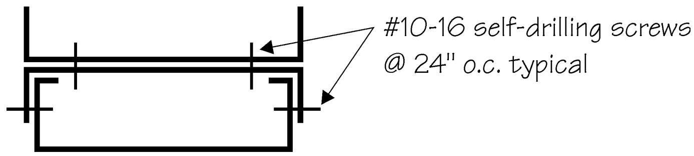


FIGURE 1-17

Step 8(f) – Sill Track to Jamb Stud Connection

Figure 1-18 illustrates the details of the screwed sill track to jamb stud connection.

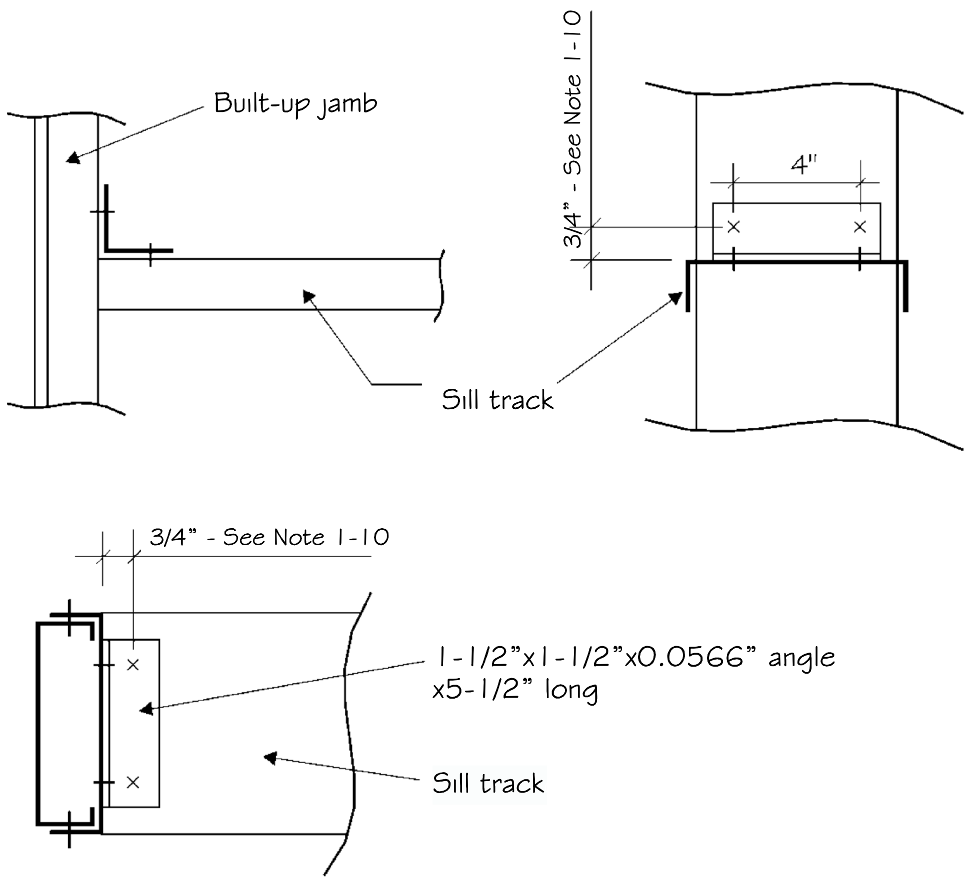


FIGURE 1-18

Note 1-10

1. *Minimum dimension for screw gun clearance varies depending on the manufacturer of the screw gun. A minimum of 5/8" is generally adequate.*

2. *Choose angle one thickness heavier than the connected members but not less than 0.0566". This rule of thumb is intended to control deformation in the angle connector. A more rigorous analysis of this connection is also acceptable and may result in a lighter angle.*

3. *It is generally good practice to install self-drilling screws through the thinner material into the thicker. This connection detail is an exception to the rule.*

The eccentricity of the connection is assumed to be resisted as illustrated in Figures 1-19 and 1-20. This is the most efficient distribution of the eccentric forces since the fasteners are only subjected to shear. Because of the symmetry of the connection, the required load and the allowable strength for all four screws is the same.

Sill track end shear = V = 296 lb. See Step 5(b).

See Figures 1-19 and 1-20 for connection force distributions.

$V = 296$ lb.
$V_1 = 296/2 = 148$ lb.
$V_2 = Ve/4 = 296(0.75)/4 = 55.5$ lb.
V_{req} = Shear resultant
$= \sqrt{V_1^2 + V_2^2} = 158$ lb.

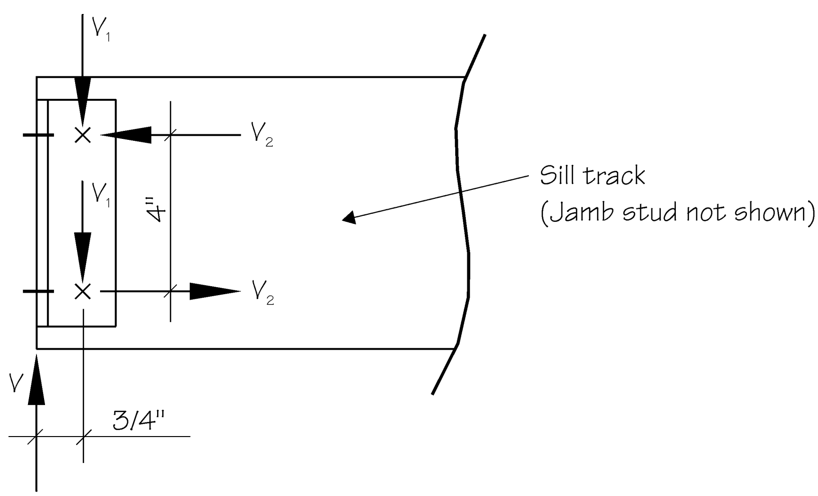


FIGURE 1-19

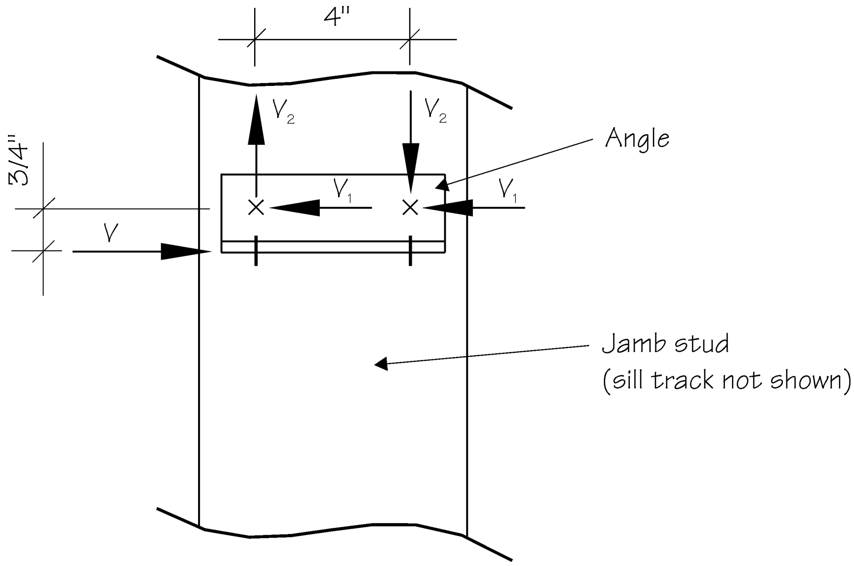


FIGURE 1-20

Determine screw shear capacities by AISI S100 E4.3 assuming #10-16 self-drilling screws.

> *Note 1-11*
>
> *The AISI provisions in E4 are based on a statistical review of a large number of screw tests including a variety of screw types and connection details. AISI S100 allows the use of test values in lieu of the design expressions in E4.*

Screw design input values:

Angle	t_1 = 0.0566"	F_{u1} = 65 ksi
Track	t_2 = 0.0451"	F_{u2} = 45 ksi
Jamb	t_2 = 0.0451"	F_{u2} = 45 ksi
Screw	Size = #10-16	d = 0.190" *(Appendix A Table A-2)*

Allowable shear:

Screw allowable shear is limited by E4.3.1 tilting and bearing.

t_2/t_1 = 0.797 < 1.0; therefore, choose the governing P_{ns} from AISI S100 Equations E4.3.1-1, E4.3.1-2 and E4.3.1-3.

$$P_{ns} = 4.2\,(t_2^3 d)^{1/2}\,F_{u2} = 789\,\text{lb.} - \text{governs}$$
$$P_{ns} = 2.7 t_1 d F_{u1} = 1887\ \text{lb.}$$
$$P_{ns} = 2.7 t_2 d F_{u2} = 1041\ \text{lb.}$$

Gives:

$V_{all} = P_{ns}/\Omega = 789/3 = 263$ lb.

Check end distance per AISI S100 Section E4.2:

Minimum specified end distance = 1.5d = 0.285". End distance for both the clip and angle is roughly 3/4" for angle and track with the thinner track governing, easily meeting this requirement.

Screw allowable shear limited by AISI S100 Section E4.3.3 shear in the screws themselves.

$P_{ns} = P_{ss}$

Where P_{ss} = nominal shear capacity of screw. See Appendix A, Table A-1.

$P_{ns} = P_{ss} = 1400$ lb.

Gives:

$V_{all} = P_{ns} / \Omega = 1400/3$
$= 467$ lb.

The governing V_{all} is from AISI S100 Section E4.3.1 and is given by:

$V_{all} = 263$ lb. > 158 lb. ***OK***

Note 1-12

Add a stud under the connection to resist dead load and construction abuse at the time of window installation. See optional cripple stud in Figure 1-1. As a design alternative, this additional stud could also be designed to pick up the end reaction due to wind from the sill track and thereby eliminate the need for a clip angle connection. The connection between the sill track and the additional stud could be analyzed using the provisions of AISI S240, B3.2.5.1 (e).

Step 8(g) – Built-Up Head to Jamb Stud Connection

Provide angles top and bottom at window head to resist wind load plus dead load and construction abuse, particularly at the time of window installation.

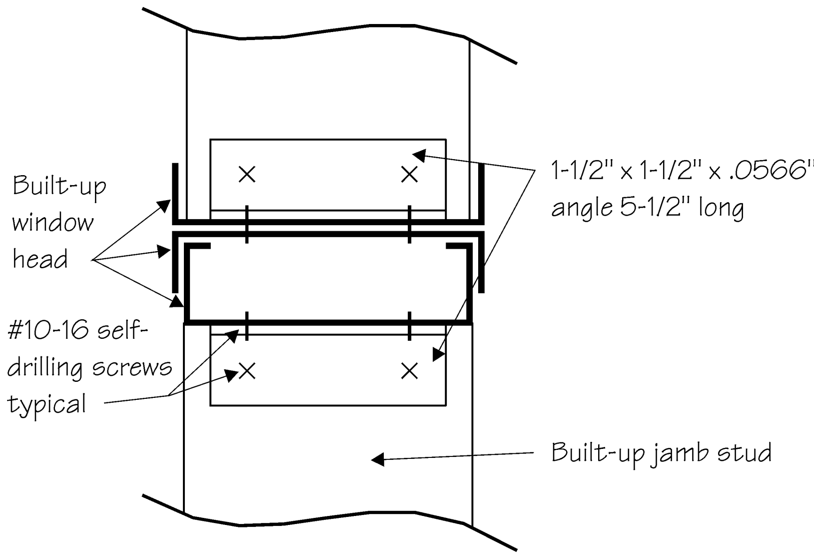


FIGURE 1-21

Step 8(h) – Bottom Track to Concrete Connection

The bottom track to concrete connection *(see Figure 1-22)* is designed for both wedge type and screw type concrete anchors.

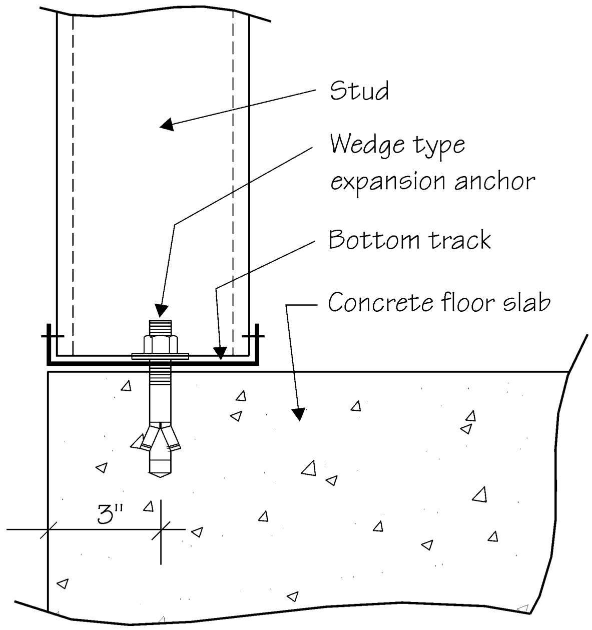


FIGURE 1-22

Note 1-13

1. *Reference Drysdale, 1991 recommends an anchor spacing less than or equal to 2'-8" o.c. regardless of the type of anchor used. This spacing is necessary to control local and overall track deformations. Similarly, an anchor should be located not more than 8″ from track ends or splices unless splice details are provided that are capable of creating flexural continuity in the track.*

2. *The bottom track anchor is assumed to be loaded in shear only with negligible pull-out due to prying.*

Applied load to bottom track:

V_{req} = (1/2)(Stud Height)(28)
= (13/2)(28) = 182 lb./ft. for strength

i) Alternative (a) – 3/8" diameter wedge type expansion anchor:

See Appendix B.1 for a discussion of wedge anchor design and selection. For this example, manufacturers' software was used to determine that 3/8″ × 2-3/8″ nominal embedment wedge anchors spaced 32″ o.c. and with 3″ edge distance are adequate.

Check bearing by *AISI S100* Section E3.3.1 (without consideration of bolt hole deformation):

$P_n = Cm_f dtF_u$

$d/t = 0.375/0.0451 = 8.31 < 10$
Therefore, $C = 3$

$m_f = 0.75$ (without washer)

$P_n = 3(0.75)(0.375)(0.0451)(45)(1000)$
$= 1712$ lb.

$V_{all} = P_n / \Omega = 1712/2.50$
$= 685$ lb. $> 182(32/12) = 485$ lb **_OK_**

Use 32" o.c. *(See Note 1-13)* **_OK_**

Fastener requirements at jamb studs:

V_{req} = jamb bottom reaction = 565 lb. *(from Step 5c)* **_OK_**

From manufacturers' software, the same anchor used at 32″ o.c. along the wall bottom track is adequate at the jambs. Therefore, use one additional anchor at the jamb. Space this anchor at least 6″ from the closest wall anchor to avoid significant reductions in anchor capacity.

Check bottom track bending strength between fasteners.

Assume simple span with worst-case location of stud end reactions.

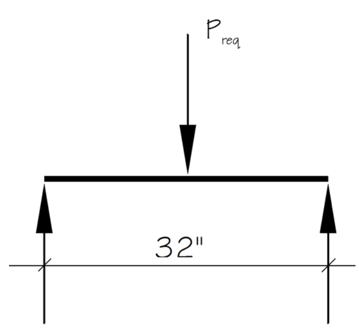


FIGURE 1-23

Stud reaction $= P_{req} = (13/2)(16/12)(28)$
$= 243$ lb. (strength)

From standard beam diagrams – for 2 equal concentrated loads, the moment is maximized with one of the loads located at mid-span as in Figure 1-23.

$M_{req} = P_{req} (L/4)$
$= 243(32/4)/1000$
$= 1.94$ in.kips

For 600T125-43 (33) track
$M_{all} = 9.11$ in.kips >> 1.94 in.kips ***OK***

ii) Alternative (b) – 1/4"-diameter concrete screw anchor:

See Appendix B.1 for a discussion of screw anchor design and selection. For this example, manufacturers' software was used to determine that 1/4″ x 1-5/8″ nominal embedment screw anchors spaced 24″ o.c. and with 3″ edge distance are adequate.

Check bearing by *AISI S100* Section E3.3.1 (without consideration of bolt hole deformation):

$P_n = Cm_f dtF_u$

$d/t = 0.25/0.0451 = 5.54 < 10$
Therefore, C = 3

$m_f = 0.75$ (without washer)

$P_n = 3(0.75)(0.25)(0.0451)(45)(1000)$
$= 1142$ lb.

$V_{all} = P_n / \Omega = 1142/2.50$
$= 457$ lb. > 182(24/12) = 364 lb ***OK***

Fastener requirements at jamb studs:

V_{req} = jamb bottom reaction = 565 lb. *(from Step 5c)*

A single screw anchor as used for the typical base track anchorage is not adequate. Therefore, provide 2 fasteners adjacent to jamb spaced 4″ o.c. as designed using manufacturers' software.

iii) Alternative (c) – 0.157" diameter power-actuated fastener:

Allowable shear - for load data see Appendix B.2 – Table B.2-1.

f_c' = 3000 psi
Edge distance = 3"

Choose 1-1/4" embedment.

From Table B.2-1, interpolate between concrete strengths for allowable fastener shear. *(Interpolation is assumed to apply - not explicitly allowed by ICC Evaluation Report No. ESR-2269.)*

f_c' (psi)	V_{all} (lb.)
2000	310
4000	310
3000	310 (by inspection)

Then V_{all} = 310 lb.

Check bearing by *AISI S100* Section E3.3.1 (without consideration of bolt hole deformation). *(Assume the provisions for bolt bearing apply to PAFs.)*

$P_n = Cm_f dtF_u$

d/t = 0.157/0.0451 = 3.48 < 10
Therefore, C = 3

m_f = 0.75 (without washer)

P_n = 3(0.75)(0.157)(0.0451)(45)(1000)
= 717 lb.

$V_{all} = P_n / \Omega$ = 717/2.50
= 287 lb.

V_{all} = 287 lb. per fastener governs

Required fastener spacing
= $(V_{all} / V_{req})(12)$ = (287/182)(12)
= 18.9" o.c.

Use 16" o.c. < 2'-8" o.c. *(See Note 1-13)* ***OK***

Fastener requirements at jamb studs:

V_{req} = jamb bottom reaction = 565 lb. *(from Step 5c)*

Provide 2 fasteners adjacent to jamb

V_{all} = 2(287) = 574 lb. > 565 lb. ***OK***

Space fasteners at S_{cr} = 4" o.c.

Step 8(i) – Inner and Outer Top Track to Concrete Connection

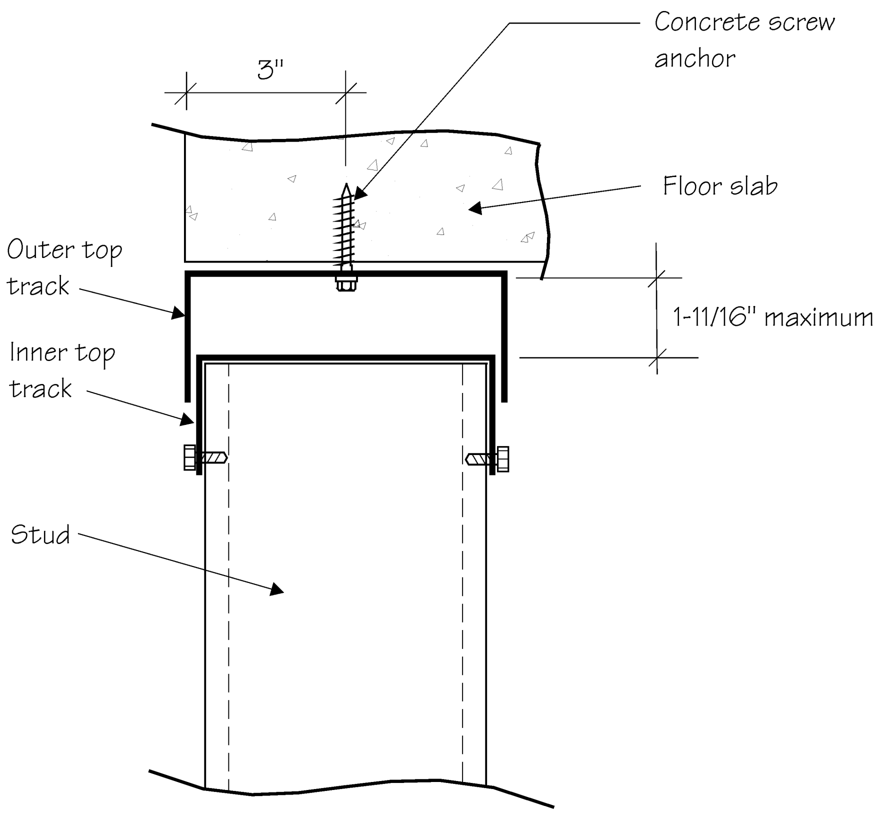


FIGURE 1-24

Note 1-14

1. *The maximum anchor spacing of 2'-8" o.c. recommended in Reference 1 applies to bottom track. For top track, this recommended maximum spacing is reduced to 2'-0" to account for the absence of torsional restraint from the studs. This 2'-0" recommendation is based on engineering judgement and has not been confirmed by testing.*

2. *The anchor is loaded in shear and pull-out due to prying.*

3. *Do not place bottom reinforcing steel along the line of the concrete anchors.*

4. *Wedge type expansion anchors are not practical in this application because the exposed portion of the fastener interferes too much with the deflection gap.*

5. *Unlike the bottom track condition, extra fasteners are not required at jamb studs because of load spreading by the inner top track.*

Applied load to top track:

V_{req} = (1/2)(Stud Height)(28)
= (13/2)(28) = 182 lb./ft. for strength

M_{req} = 182(1.6875) = 307.1 in-lb/ft.

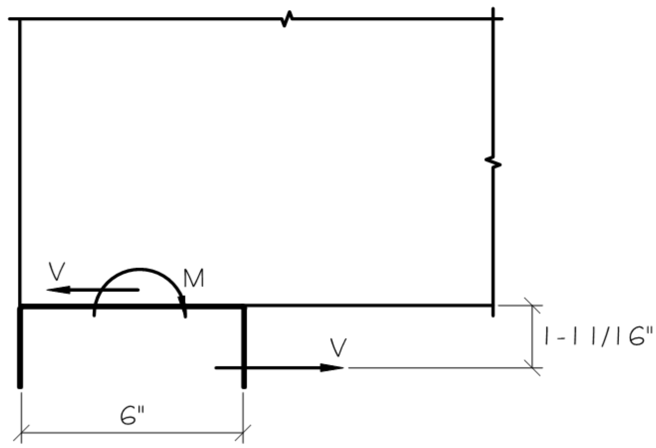


FIGURE 1-25

i) Alternative (a) – 1/4" diameter concrete screw anchor

See Appendix B.1 for a discussion of screw anchor design and selection. For this example, manufacturers' software was used to determine that a 1/4″ × 1 5/8″ nominal embedment screw anchors spaced 16″ o.c. and with 3″ edge distance are adequate.

Use ¼″ × 1 5/8″ nominal embed screw anchors 16" o.c. with 3″ edge distance

ii) Alternative (b) – 0.157" diameter powder actuated fastener

For this case the deflection gap is reduced to 1-1/2" because there is no fastener head interference. See Figure 1-26 and the caution in Note 1-15.

V_{req} = 182 lb/ft
T_{req} = 182(1.50/3.00) = 91 lb/ft *(moments about "a")*

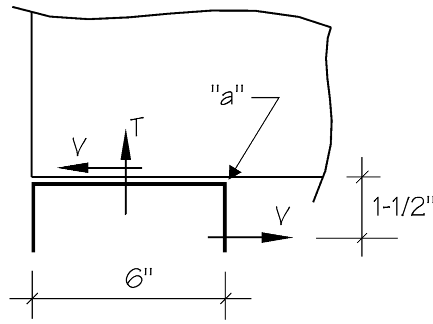


FIGURE 1-26

Allowable shear and tension – for load data see Appendix B.2 – Table B.2-1.

f_c' = 3000 psi
Edge distance = 3"

Choose 1-1/4" embedment.

Allowable shear

From Step 8(h)iii:
V_{all} = 310 lb. will control *(bearing does not control in this material thickness by inspection.)*

Note 1-15

ASCE 7-10 does not explicitly prohibit the use of PAF in this application in Seismic Design Categories D, E and F. However, Evaluation Reports for PAF often provide strict limitations on their use. Refer to the Evaluation Reports to ensure their applicability for the application.

Allowable tension:

From Table B.2-1, interpolate between concrete strengths for allowable fastener tension. *(Interpolation is assumed to apply - not explicitly allowed by ICC ESR 2269.)*

f_c' (psi)	V_{all} (lb.)
2000	240
4000	280
3000	260 (by interpolation)

Then T_{all} = 260 lb.

Check interaction equation from Table B.2-1 Note 3 with s = fastener spacing in feet:

$$\left(\frac{sP}{P_a}\right)+\left(\frac{sV}{V_a}\right)\le 1.0$$

Substituting and solving for "s":

$$\left(\frac{s91}{260}\right)+\left(\frac{s182}{310}\right)\le 1.0$$

s = 1.06′

Use 12" o.c.

Note that power-actuated fasteners have relatively small head diameters and pull-over and the interaction of shear and pull-over should be checked as possible limit states. Check for the 12" o.c. fastener spacing.

V_{req} = (12/12)(182) = 182 lb. per fastener
T_{req} = (12/12)(91) = 91 lb. per fastener

Use AISI S100 Section E5.2.3 for Pull-Over Strength of PAF.

$P_{nov} = \alpha_w t_1 d_w F_{u1}$

where:
α_w = 1.5 for simple PAF
t_1 = 0.0713" (outer top track)
d_w = 0.32" (fastener head diameter Table B.3-1)
F_{u1} = 65 ksi

P_{nov} = 1.5(0.0713)(0.32)(65)(1000)
= 2225 lb.

$P_{all} = P_{nov} / \Omega$ = 2225/3
= 742 lb. > 91 lb. ***OK***

AISI S100 Section E5 does not contain explicit provisions for combined shear and pullover. Therefore, assume that the similar provisions for screws are a reasonable approximation. Check interaction for combined shear and pull-over using *AISI S100 Section E4.5.1.*

$$\frac{Q}{P_{ns}}+0.71\frac{T}{P_{nov}}\le\frac{1.10}{\Omega}$$

$Q = V_{req} = 182$ lb.
$T = T_{req} = 91$ lb.
$P_{ns} = 2.7 t_1 d F_{u1} = 2.7(0.0713)(0.157)(65)(1000)$
$= 1965$ lb.
and from above
$P_{nov} = 2225$ lb.

$$\frac{182}{1965} + \frac{(0.71)(91)}{2225} \leq \frac{1.10}{2.35}$$

Gives $0.12 < 0.47$ ***OK***

Step 8(j) – Single Outer Top Track to Concrete Connection

For this case, the deflection gap is reduced to 1-1/2" because there is no fastener head interference. See Figure 1-26.

$V_{req} = (1/2)(\text{Stud Height})(28)$
$= (13/2)(28) = 182$ lb./ft. for strength

$T_{req} = 182(1.50/3.00) = 91$ lb/ft. *(moments about "a")*

i) Alternative (a) – 1/4" diameter concrete screw anchor:

While the deflection gap is slightly reduced, thus reducing the prying moment on the anchor, analysis shows that the same anchor spacing is required for this condition as was used for the inner and outer top track case (16").

> *Note 1-16*
>
> *The single top deflection track design provisions in AISI S240 do not include any limit on fastener spacing. However, the background research (as reported in Gerloff 2004) was based on a maximum fastener spacing equal to the stud spacing and this is proposed here as an upper limit.*

Use 16" o.c. *(See Note 1-16)* ***OK***

ii) Alternative (b) – 0.157" diameter power-actuated fastener:

The analysis here will be the same as for the inner and outer top track for Step 8(i)ii. See Note 1-15.

Use 12" o.c.

iii) Alternative (c) – 3/8" diameter wedge type expansion anchor:

Wedge type expansion anchors generally do not come in sizes as small as the screw type anchor specified. The larger wedge type anchor would, by inspection, work at the same spacing as the screw anchor. However, based on the maximum recommended anchor spacing of 16" o.c. (See Note 1-16 above), there is no advantage to using them in this condition.

Table 1-2
Top and Bottom Track to Concrete Connection Summary

Top or Bottom Track	Anchor Size & Type	Embedment Depth (inches)	Spacing (inches)	Number of Fasteners at Jambs
Bottom Track	3/8" diameter wedge anchor	2	32	1
	1/4" diameter screw anchor	1-1/2	24	2
	0.145" diameter power-actuated anchor	1-1/4	16	2
Inner/Outer Top Track	1/4" diameter screw anchor	1-1/2	16	*Note 1*
	0.145" diameter power-actuated anchor	1-1/4	12	*Note 1 & 3*
Single Outer Top Track	1/4" diameter screw anchor	1-1/2	16	*Note 2*
	0.145" diameter power-actuated anchor	1-1/4	12	*Note 2 & 3*

Notes:

1. *For the inner and outer top track, no additional top track fasteners are required at jamb locations since concentrated loads are spread by the inner/outer top track detail.*

2. *For the single outer top track used in this design example, no additional top track fasteners are required at jamb locations because the jamb is connected at the top with a proprietary slide clip. See Section 8(k) for the slide clip connection details.*

3. *For top track deflection details where earthquake design is a consideration, there may be restrictions on the use of power-actuated fasteners. See Note 1-15.*

Step 8(k) – Slide Clip to Concrete Connection

From Step 7(c), a proprietary slide clip is required to transfer the top jamb reaction to the underside of the concrete floor slab. The engineering for the connection between the stud and the clip is assumed to be provided by the slide clip manufacturer. The connection between the slide clip and the concrete remains the responsibility of the cold-formed steel framing designer.

It is assumed that two fasteners are required between the side clip and the concrete in order to provide some torsional restraint. Based on the sheathed design assumption in this example, there is no direct torsion applied to the connection from the jamb stud but there will be some inherent eccentricities in the connection itself. Assume all the connection eccentricity is resisted at the slide clip to concrete connection. See Figure 1-27.

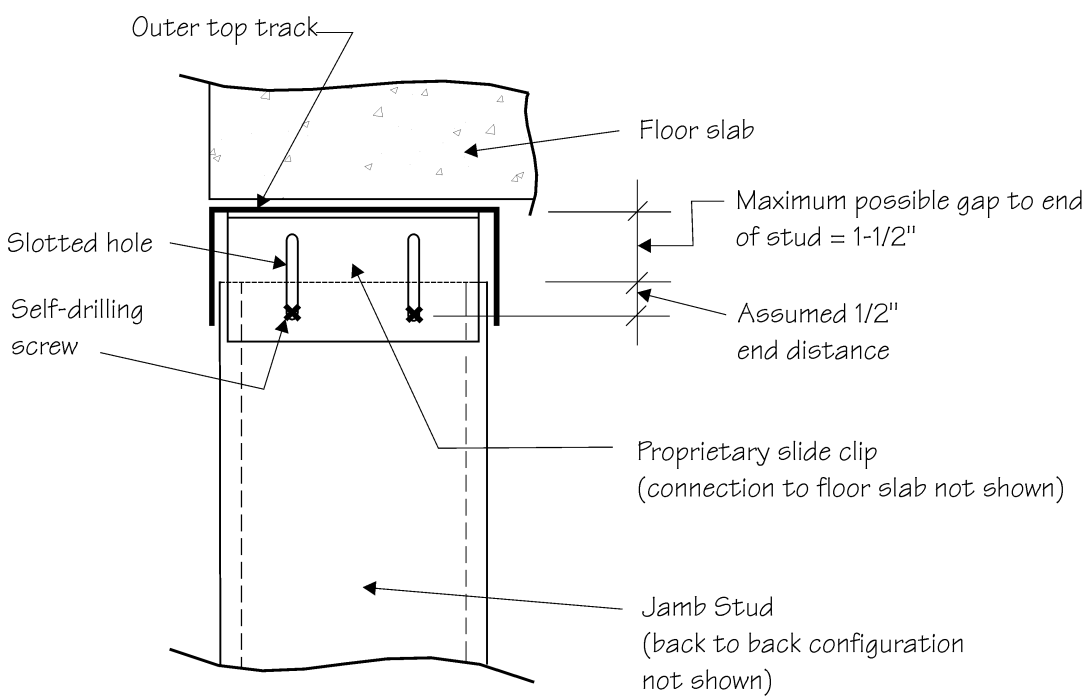


FIGURE 1-27

For this connection, the 3/8" diameter screw type anchor has been selected with 2-1/2" nominal embedment.

The design of the fasteners will be sensitive to their specific location based on prying forces, edge distance and minimum spacing requirements. Use 2.5" minimum edge distance to allow for some variation in the location of the slab edge along with the 2.5" minimum spacing. Using these distances, the concrete anchors will be asymmetrically placed as shown in Figures 1-28 and 1-29.

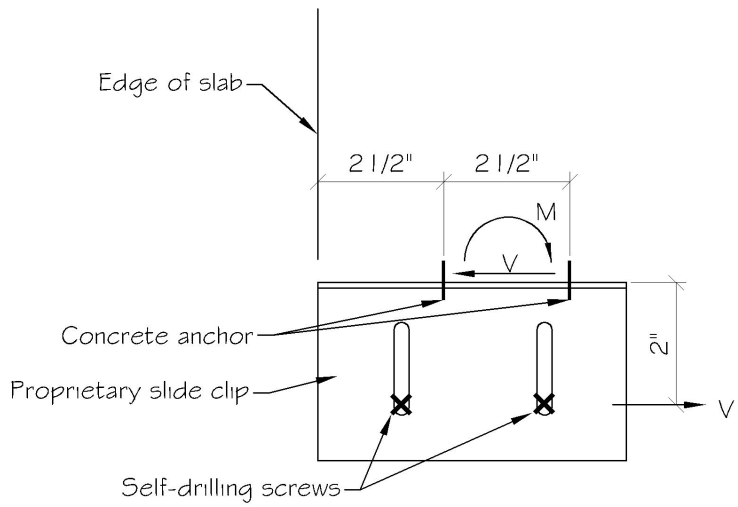


FIGURE 1-28

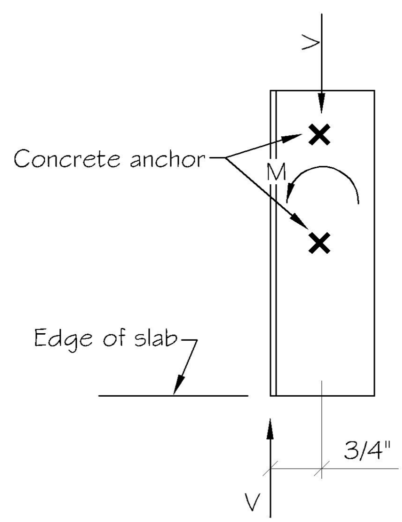


FIGURE 1-29

Required shear and tension:

V = jamb reaction = 565 lb. from Step 5(c).

$M_1 = V(2) = 565(2) = 1130$ in-lb

Torsional Moment = $V(0.75) = 565(0.75) = 424$ in-lb

Using manufacturers' software to analyze for the geometry, shear and moment results in a satisfactory anchor design.

Step 8(l) – Outer Top Track to Embedded Plate Connection

On some projects, the top track is connected to the underside of a spandrel beam rather than the underside of the slab. The quantity of bottom reinforcing steel in the spandrel beam may make the installation of drilled anchors difficult. For projects such as this, an embedded plate may be more practical. Suggested details are shown in Figures 1-30 and 1-32.

Assume embedded plates at maximum recommended spacing for anchoring outer top track = 24" o.c. *(Note 1-14).*

i) Alternative (a) – field welding:

Check weld as shown in Figures 1-30 and 1-31.

Applied load to track:

$V = (13/2)(28)$
$= 182$ lb./ft

For welds at 24" o.c.:
$V_1 = 2(182)/2 = 182$ lb. per weld

By taking moments,

$T_1 = 1.50(2)(182)/6$
$= 91$ lb. per weld

Resultant shear per weld:

$$V_{req} = \sqrt{V_1^2 + T_1^2} = \sqrt{182^2 + 91^2}$$
$= 203$ lb. per weld

Allowable strength for a weld of length L inches *(See Note 1-17)*:

$V_{all} = 0.75tLF_u/\Omega$
$= 0.75(0.0713)L(65)(1000)/3.05$
$= 1140L$ lb.

$L_{req} = 203/1140 = 0.18$" of weld length

Use L = 1" as a minimum practical weld length

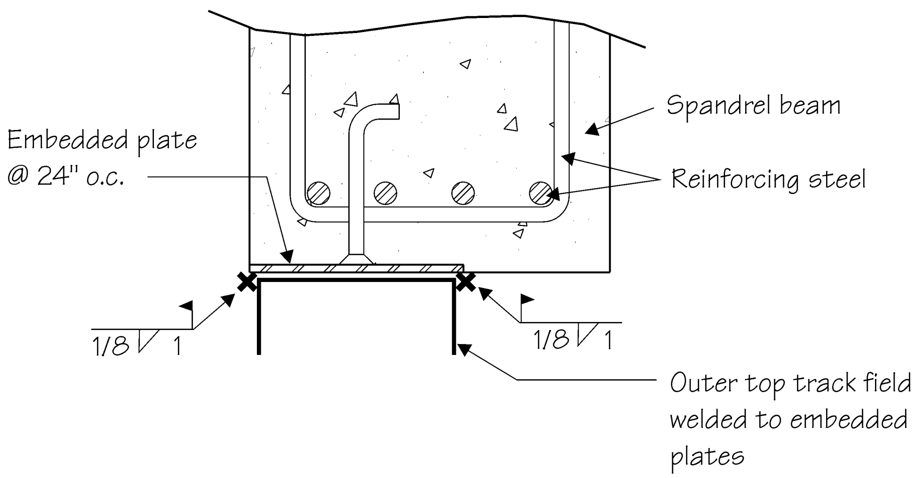


FIGURE 1-30

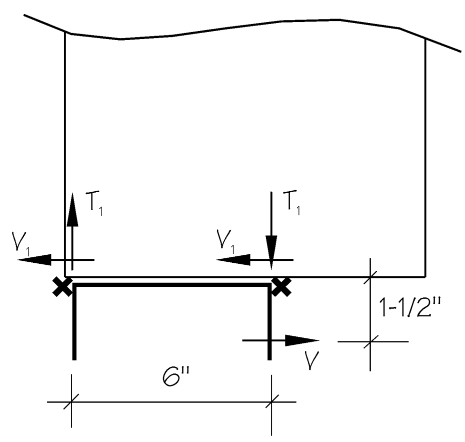


FIGURE 1-31

Note 1-17

1. *A simplified approach to weld strengths is used in this design guide. Refer to Appendix A, Section A.1, for the origin of the general formula for the nominal unit strength of fillet and flare-bevel groove welds, $0.75tLF_u/\Omega$ with $\Omega = 3.05$.*

2. *For this simplified method, the strength of fillet and flare groove welds in cold-formed steel thicknesses less than or equal to 0.10" is a function of the tensile strength of the sheet and the length of the weld. It is assumed that the necessary weld leg size is available to develop the strength of the parent material.*

3. *Show a nominal weld size on drawings of say 1/8" accompanied by a note: "For material less than or equal to 0.10" thick, drawings show nominal weld leg sizes. For such material, the effective throat of welds shall not be less than the thickness of the thinnest connected part."*

4. *The F_u values for various ASTM steels can be found in AISI D100 (AISI 2013), Part I.*

ii) Alternative (b) – power-actuated fasteners:

Applied loads on fasteners. See Figures 1-32 and 1-33.

From previous step:
V = 182 lb/ft

For PAFs at 24" o.c.:
$V_{req} = V_1 = 2(182)/2$
= 182 lb. per fastener

For moments about "a":

$T_{req} = 2(182)(1.5)/(4+1)$
= 109 lb. per fastener

Allowable shear and tension are taken from Appendix B.3, Table B.3-1 assuming a 1/4" thick embedded plate.

Minimum edge distance = 1/2" < 1" ***OK***

V_{all} = 775 lb.
T_{all} = 720 lb.

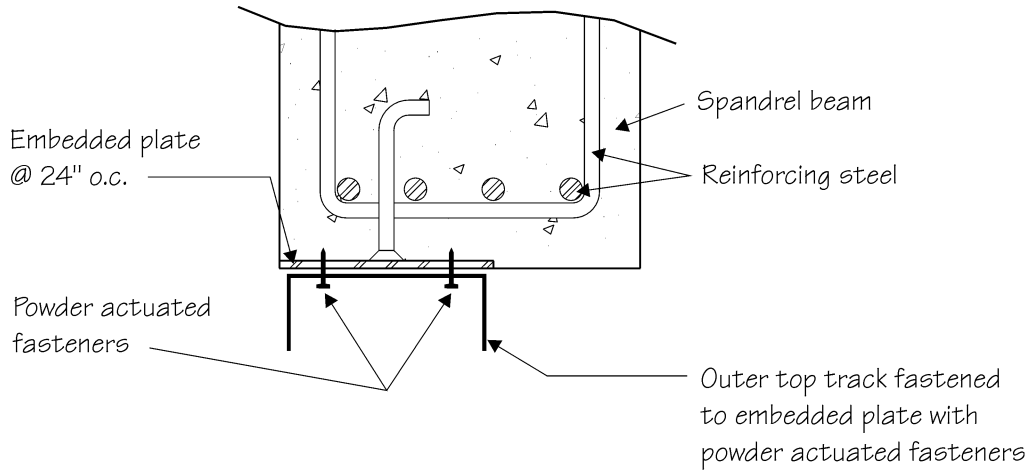


FIGURE 1-32

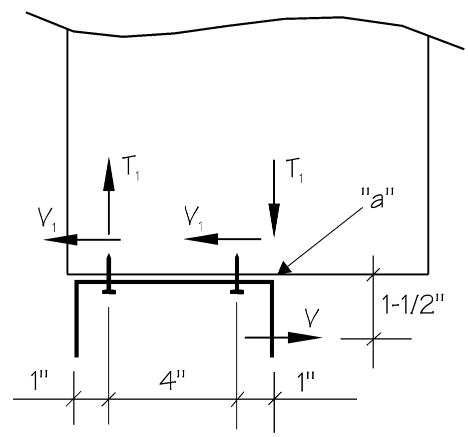


FIGURE 1-33

Interaction check:

$$\left(\frac{p}{P_a}\right)+\left(\frac{V}{V_a}\right)\leq 1.00$$

$$\left(\frac{109}{775}\right)+\left(\frac{182}{720}\right)=0.39<1.00$$

OK

Pullover ***OK*** by inspection – see Step 8(i)ii.

Note that where earthquake design is a consideration, there may be restrictions on the use of power-actuated fasteners–see Note 1-15.

DESIGN EXAMPLE #2
WIND BEARING INFILL WALL WITH AN UNSHEATHED DESIGN APPROACH AND WELDED OR SCREWED CONNECTIONS

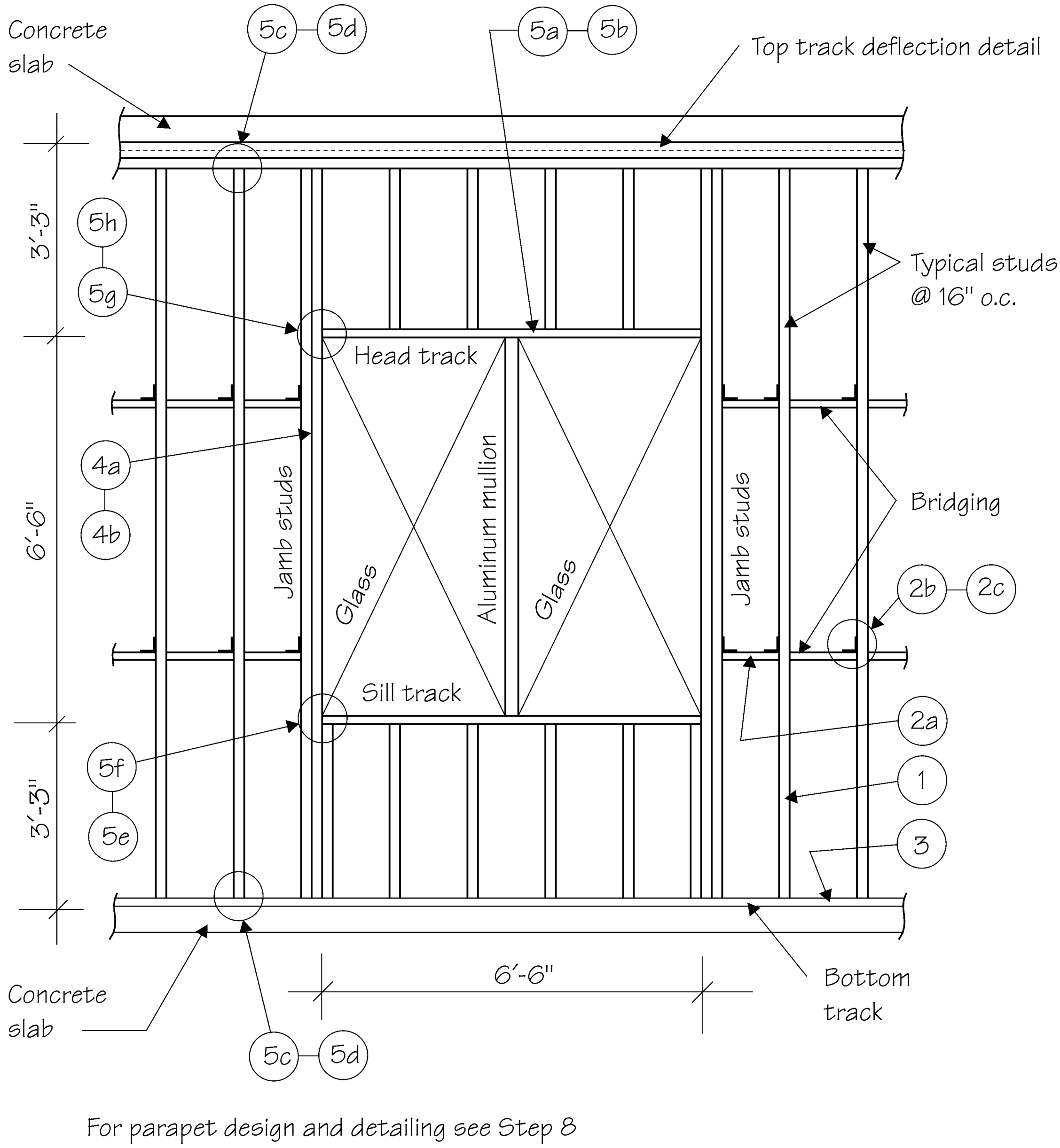


FIGURE 2-1

Introduction

This design example assumes an all-steel system where the restraint of the sheathings is ignored. All connections are designed as welded or screwed. An inner and outer top track deflection detail is assumed.

Members are checked for torsional effects of loads not applied through the shear center, lateral instability, and distortional buckling. Bridging is checked for the accumulated torsion between bridging lines.

The layout of the members and the design wind loads are identical to Design Example 1. See Figure 2-1. The numbers shown in Figure 2-1 correspond to the applicable design step used in this example. Refer also to the following:

Step 1 – Typical Stud Design
Step 2 – Through-the-Punchout Bridging Design
Step 3 – Check Bottom Track and Sill Track for Lateral Instability
Step 4 – Jamb Stud
Step 5 – Miscellaneous Connection Design
Step 6 – General Comments on Welded Connections
Step 7 – Details at Shearwalls
Step 8 – Parapets

Step 1 – Typical Stud Design

Step 1(a) – Check Trial Stud for Warping Torsion

Check the typical stud selected in Design Example 1 for warping torsion using the approximate method outlined in Appendix C and Section C3.6 of AISI S100.

This design example uses a torsional eccentricity from the shear center to the center line of the web as illustrated in Figure 2-3A. This eccentricity would be typical for positive wind pressures loading the exterior sheathing which in turn load the compression flange of the stud. Abbreviated calculations are also provided for a torsional eccentricity from the shear center to the center line of the flange (Figure 2-3B). This eccentricity would be typical for negative wind pressures loading the exterior sheathing with the attaching screws in tension. Note that the torsional analyses assume the sheathings load the studs but do not provide any meaningful or reliable torsional restraint.

The stud spans from A to D as shown in Figure 2-2 with bridging at points B and C. This gives an unsupported length L_u = 4'-4". The applied wind load = 37.3 lb/ft is assumed to act through the web center line. A component of this load, F, will act on the half-beam which is analyzed as a continuous beam with the top and bottom track and the bridging lines acting as supports.

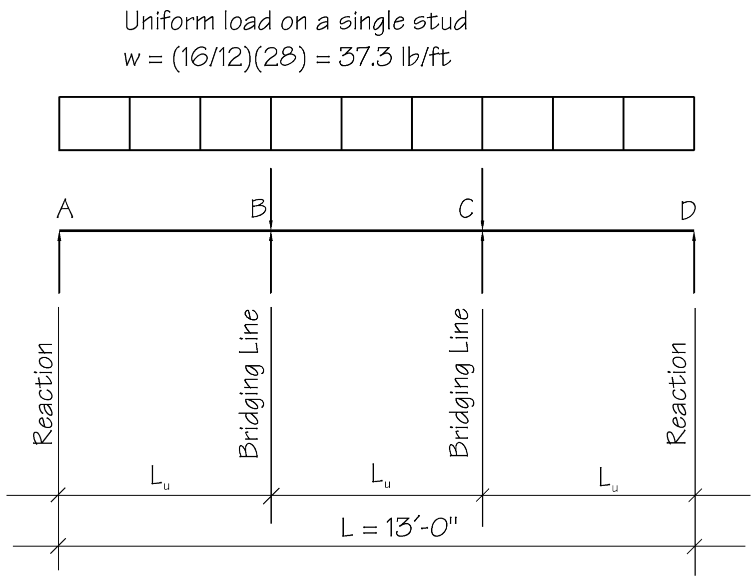


FIGURE 2-2

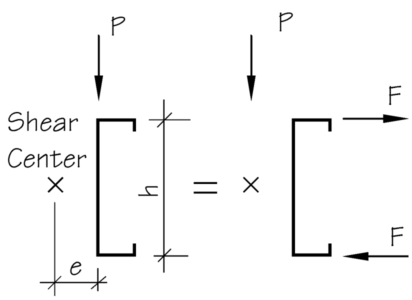


FIGURE 2-3A

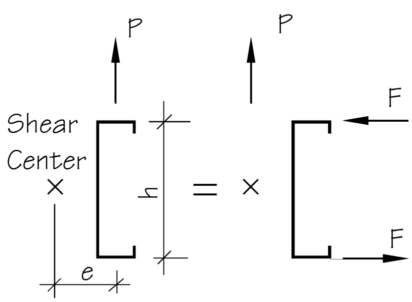


FIGURE 2-3B

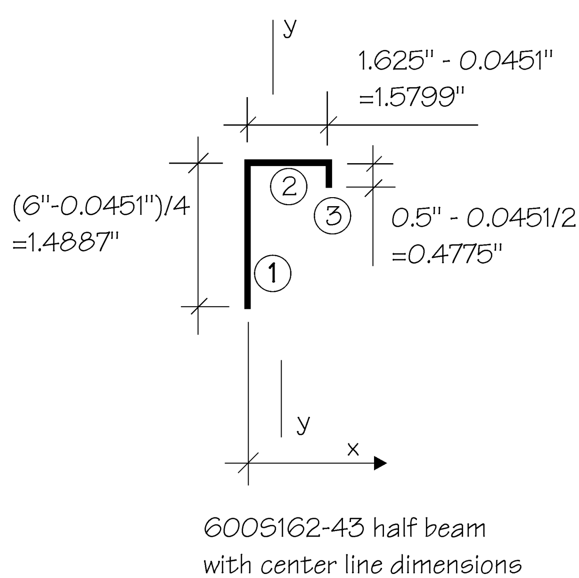


FIGURE 2-4

Derive the properties for the torsional "half-beam" using the linear method. For further examples of the linear method and properties of line elements, see AISI D100 *(AISI 2013b),* Part I, Section 3. For this example, neglect the corner radii.

Element	L	X	LX	LX^2	I_0/t
1	1.4887	0.0000	0.0000	0.0000	0.0000
2	1.5799	0.7900	1.2481	0.9860	0.3286
3	0.4775	1.5799	0.7544	1.1919	0.0000
Σ	3.5461		2.0025	2.1779	0.3286

$X_{cg} = \Sigma LX/\Sigma L$
$= 2.0025/3.5461 = 0.5647$ in. (from web centerline)

$I_{cg} = [\ \Sigma LX^2 + \Sigma I_0/t - \Sigma L(X_{cg})^2\ ]\ t$
$= [\ 2.1779 + 0.3286 - (3.5461)(0.5647)^2\ ][\ 0.0451\ ]$
$= 0.06204\ in^4$

$$S_{y(web)} = \frac{I}{X_{cg} + t/2} = \frac{0.06204}{0.5647 + 0.0451/2}$$
$$= 0.1056\ in^3$$

$$S_{y(lip)} = \frac{I}{Flg - X_{cg} - t/2} = \frac{0.06204}{1.625 - 0.5647 - 0.0451/2}$$
$$= 0.0598\ in^3$$

Load on "half-beam"

The torsional loads, F, illustrated in Figure 2-3A are derived as follows:

$F = P(e/h)$

$P = 37.3$ lb/ft
$e = m = 0.670$ in. (distance from shear center to web center line)
$h = \text{depth} - t = 6 - 0.0451 = 5.95$ in.

$F = 37.3(0.670/5.95)$
$= 4.20$ lb/ft

Required maximum moment on the half-beam is given by the 3-span continuous beam analogy where the moment over the first support (bridging line) is given by the following:

$M_{t(req)} = FL_u^2/10$
$= 4.20(4.33)^2/10$
$= 7.87$ ft.lb

(If the moment on the half-beam is to be checked at the mid-height of the stud—i.e., mid-distance between bridging lines—then $M_{req} = FL_u^2/40$. See Note 2-1, Item 1.)

Warping torsional stress:

$$f_{torsion} = M_{t(req)}/S_{y(lip)}$$
$$= 7.87(12)/0.0598 = 1580 \text{ psi}$$

For combined bending and warping torsion, calculate the moment reduction factor, R, per AISI S100 Section C3.6, defined as follows:

> "…the reduction factor, R, shall be equal to the ratio of the maximum normal stresses due to bending alone divided by the combined stresses due to both bending and torsional warping at the point of maximum combined stress on the cross-section."

$$R = \frac{f_{bending\_max}}{f_{bending} + f_{torsion}}$$

Major axis moment:

$$M_{x(req)} = wL^2/8 = (16/12)(28)(13)^2(12)/8$$
$$= 9460 \text{ in.lb}$$

$$f_{bending\_max} = M_{x(req)}/S_{x(eff)}$$
$$= 9460/0.767$$
$$= 12300 \text{ psi}$$

$$R = \frac{12300}{12300 + 1580} = 0.886$$

Reduce allowable moment:
$= RM_{all}$
$= 0.886(16.68)$
$= 14.78$ in.kips

$M_{x(req)} = 9.46$ in.kips < 14.78 in.kips ***OK***

Note 2-1

1. *For this design example, the maximum warping torsional effects occur at a line of bridging, whereas the maximum primary bending moment occurs at mid-span. A more rigorous design procedure would check M_{all}(reduced) both at the line of bridging and at mid-span against $M_{x(req)}$ calculated for each of the two locations. As a design expediency, this example assumes the maximum warping torsion and the maximum primary bending moment both occur at the same location even though this is not the case.*

2. *There is some interaction between warping torsion and lateral instability not accounted for in this procedure. Refer to the discussion at the end of Appendix C.*

3. *AISI S100, Section C3.6 requires that the reduction factor, R, be applied to "the available flexural strength [factored resistance] calculated in accordance with Section C3.1.1(a)." The reduction factor need not be applied to the allowable distortional buckling moment calculated per Section C3.1.4.*

4. *Where the restraint of sheathings is to be ignored, the effect of warping torsional stresses can generally be neglected for routine design provided there is adequate bridging.*

5. *For torsional eccentricity to the flange center line:*

   $e = m + (\text{center line flange width})/2$
   $= 0.670 + (1.625 - 0.0451)/2$
   $= 1.46$ *in.*

   $F = 9.15$ *lb/ft*

   $M_{t(req)} = 17.16$ *ft.lb*

   $f_{torsion} = 3440$ *psi*

   $R = 0.781$

   Reduced allowable moment $= RM_{all} = 13.03$ *in.kips*

   $M_{x(req)} = 9.46$ *in.kips* < 13.03 *in.kips* ***OK***

Step 1(b) – Check Trial Stud for Lateral Instability

From Figure 2-2:

L_u = 4'-4" = 52"

Assume:

$K_t = K_y = 1$

From load tables:

J = 0.000303 in^4
C_w = 1.10 in^6
r_0 = 2.58 in.
A = 0.447 in^2
r_y = 0.576 in.

S_f = full unreduced section modulus
= 0.7719 in^3
M_{all} = 16.68 in.kips *(fully braced allowable moment for subsequent calculation of F_{ya})*

and
E = 29,500 ksi
G = 11,300 ksi

From AISI S100 Section C3.1.2:
C_b = 1 (Conservative – only a small benefit for *AISI S100* Equation C3.1.2.1-5 for the middle L_u)

$$\sigma_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L_y}{r_y}\right)^2} = \frac{\pi^2 (29500)}{\left[\frac{(1)(52)}{0.576}\right]^2}$$
$$= 35.7 \text{ ksi}$$

$$\sigma_t = \frac{1}{A r_0^2}\left[GJ + \frac{\pi^2 E C w}{(K_t L_t)^2}\right]$$
$$= \frac{1}{(0.447)(2.58)^2}\left[11300\,(0.000303) + \frac{\pi^2 (29500)(1.10)}{[(1)(52)]^2}\right]$$
$$= 41.0 \text{ ksi}$$

$$F_e = \frac{C_b r_0 A\sqrt{\sigma_{ey}\sigma_t}}{S_f}$$
$$= \frac{(1)(2.58)(0.447)\sqrt{(35.7)(41.0)}}{0.772}$$
$$= 57.2 \text{ ksi}$$

Assuming F_{ya} is not available in load tables *(both M_{all} and $S_{x(eff)}$ are based on the fully braced condition).*
$F_{ya} = M_{all}\Omega/S_{x(eff)} = 16.68(1.67)/0.767$
$= 36.3$ ksi

$2.78\ F_{ya} = 2.78(36.3) = 100.9$ ksi
$0.56\ F_{ya} = 0.56(36.3) = 20.3$ ksi

For $2.78F_{ya} > F_e > 0.56F_{ya}$

$$F_c = \frac{10}{9}F_{ya}\left(1 - \frac{10F_{ya}}{36F_e}\right)$$
$$= \frac{10}{9}(36.3)\left[1 - \frac{10(36.3)}{36(57.2)}\right]$$
$$= 33.2 \text{ ksi}$$

See Note 2-2.

Note 2-2

1. *The next step in AISI S100 is to calculate a revised effective section modulus, S_c, with F_{ya} replaced by F_c. The moment capacity reflecting lateral buckling is then given by $M_{all} = M_n/\Omega = S_cF_c/\Omega$. For typical lightweight steel framing profiles, calculating S_c requires considerable work with little benefit. Use the following procedure instead:*

   $M_{all\ (lateral\ buckling)} = M_{x\ (fully\ braced)} \times (F_c/F_{ya})$

   This expression is always conservative.

2. *Where section capacity does not include the effect of cold work of forming, F_{ya} is replaced by F_y throughout Step 1(b).*

$M_{all\ (lateral\ buckling)} = M_{all\ (fully\ braced)}$
$= 16.68(33.2/36.3)$
$= 15.3$ in.kips

From Step 1(a):
$M_{x(req)}$ = 9.46 in.kips < 15.3 in.kips ***OK***

Therefore, from Steps 1(a) and 1(b), the allowable moment of the typical stud is reduced by warping torsion and by lateral instability. However, there is sufficient bridging and strength reserve such that the basic stud selection is unaffected.

Step 1(c) – Check Trial Stud for Distortional Buckling

Distortional buckling must be checked in accordance with AISI S100 Section C3.1.4. The distortional buckling allowable moment is a function of the elastic distortional buckling stress, F_d, calculated in accordance with AISI S100 Eq. C3.1.4-6:

$$F_d = \beta \frac{k_{\phi fe} + k_{\phi we} + k\phi}{k_{\phi fg} + k_{\phi wg}}$$

$$\beta = 1.0$$

Note that β accounts for moment gradient within distortional buckling half wavelength. The distortional buckling half wavelength for typical stud members is relatively short, so there is little value in including the moment gradient factor. β =1.0 is a conservative simplification.

To determine the various k_ϕ values, properties of the compression flange must be calculated. *CFSEI 2008* can be used to determine these properties using the line method, ignoring corners as follows:

h_0 = 6.0 (in) - Out-to-out web depth
b = 1.625 – 0.0451 = 1.5799 (in) – Center line Flange Width (inches)
d = 0.5 – 0.0451/2 = 0.4775 (in) – Center line Lip Length (inches)

$$\begin{aligned} A_f &= (b + d)t = 0.886 \\ &= (1.5799 + 0.4775)0.0451 \\ &= 0.09279(\text{in}^2) \end{aligned}$$

$$\begin{aligned} J_f &= \frac{1}{3}bt^3 + \frac{1}{3}dt^3 \\ &= \frac{1}{3}(1.5799)(0.0451)^3 + \frac{1}{3}(0.4775)(0.0451)^3 \\ &= 0.00006291(\text{in}^4) \end{aligned}$$

$$I_{xf} = \frac{t[t^2b^2 + 4bd^3 - 4bd^3(\cos\theta)^2 + t^2bd + d^4 - d^4(\cos\theta)^2]}{12(b+d)}$$

$$= \frac{0.0451[(0.0451)^2(1.5799)^2 + 4(1.5799)(0.4775)^3 + 0.0451^2(1.5799)(0.4775) + 0.4775^4]}{12(1.5799 + 0.4775)}$$

$$= 0.001363(\text{in}^4)$$

Note that cos(θ) = 0 for θ = 90° in these equations. Those terms are not included in the numerical expressions.

$$I_{yf} = \frac{t[b^4 + 4db^3 + 6d^2b^2(\cos\theta) + 4d^3b(\cos\theta)^2 + d^4(\cos\theta)^2]}{12(b+d)}$$

$$= \frac{0.0451[1.5799^4 + 4(0.4775)(1.5799)^3]}{12(1.5799 + 0.4775)}$$

$$= 0.02514(\text{in}^4)$$

$$I_{xyf} = \frac{tbd^2\sin(\theta)[b + d\cos(\theta)]}{4(b+d)}$$

$$= \frac{0.0451(1.5799)(0.4775)^2(1.5799)}{4(1.5799 + 0.4775)}$$

$$= 0.003118(\text{in}^4)$$

$$I_{0f} = \frac{tb^3}{3} + \frac{bt^3}{12} + \frac{td^3}{3}$$

$$= \frac{0.0451(1.5799)^3}{3} + \frac{1.5799(0.0451)^3}{12} + \frac{0.0451(0.4775)^3}{3}$$

$$= 0.06093(\text{in}^4)$$

$$x_{0f} = \frac{b^2 - d^2\cos(\theta)}{2(b+d)}$$

$$= \frac{1.5799^2}{2(1.5799 + 0.4775)}$$

$$= 0.6066(\text{in})$$

$$y_{0f} = \frac{-d^2\sin(\theta)}{2(b+d)}$$

$$= \frac{-(0.4775^2)}{2(1.5799 + 0.4775)}$$

$$= -0.05540(\text{in})$$

$$h_{xf} = \frac{-[b^2 + 2bd + d^2\cos(\theta)]}{2(b+d)}$$

$$= \frac{-[1.5799^2 + 2(1.5799)(0.4775)]}{2(1.5799 + 0.4775)}$$

$$= -0.9733(\text{in})$$

$$h_{yf} = \frac{-d^2\sin(\theta)}{2(b+d)}$$

$$= \frac{-(0.4775^2)}{2(1.5799 + 0.4775)}$$

$$= -0.05540(\text{in})$$

$$C_{wf} = 0 \quad (\text{in}^6)$$

Per AISI S100 Eq. C3.1.4-8:

$$L_{cr} = \left(\frac{4\pi^4 h_o(1-\mu^2)}{t^3}\left(I_{xf}(x_{of}-h_{xf})^2 + C_{wf} - \frac{I^2_{xyf}}{I_{yf}}(x_{of}-h_{xf})^2\right) + \frac{\pi^4 h_o^4}{720}\right)$$

$$= \left(\frac{4\pi^4 6(1-0.3^2)}{0.0451^3}\left(0.001363[0.6066-(-0.9733)]^2 + 0 - \frac{0.003118^2}{0.02514}[0.6066-(-0.9733)]^2\right) + \frac{\pi^4 6^4}{720}\right)^{1/4}$$

$$= 15.43(\text{in})$$

In the equations that follow, L is taken as the minimum of L_m and L_{cr}. Since no discrete bracing is used, $L_m > L_{cr}$, so $L = L_{cr}$. E = 29,500 ksi and G = 11,300 ksi:

$$k_{\phi fe} = \left(\frac{\pi}{L}\right)^4\left(EI_{xf}(x_{of}-h_{xf})^2 + EC_{wf} - E\frac{I^2_{xyf}}{I_{yf}}(x_{of}-h_{xf})^2\right) + \left(\frac{\pi}{L}\right)^2 GJ_f$$

$$= 0.1530(\text{kips})$$

$$k_{\phi we} = \frac{Et^3}{12(1-\mu^2)}\left(\frac{3}{h_o} + \left(\frac{\pi}{L}\right)^2\frac{19h_o}{60} - \left(\frac{\pi}{L}\right)^4\frac{h_o^3}{240}\right)$$

$$= 0.1438(\text{kips})$$

$$k_\phi = 0$$

For an all-steel design, there is no distortional buckling restraint provided by sheathing. As such, $k_\phi = 0$. Per CFSEI 2008, for walls sheathed with gypsum board, $k_\phi = 0$ as well, since gypsum sheathing has been shown to have inadequate strength to restrain distortional buckling.

$$k_{\phi fg} = \left(\frac{\pi}{L}\right)^2 \left[Af\left((xof - hxf)^2 \left(\frac{Ixyf}{Iyf}\right)^2 - 2yof(xof - hxf)\left(\frac{Ixyf}{Iyf}\right) + h^2{}_{xf} + y^2{}_{of} \right) + Ixf + Iyf \right]$$

$$= 0.004984(in^2)$$

$$k_{\phi wg} = \frac{hot\pi^2}{13440}\left(\frac{[45360(1-\xi web) + 62160]\left(\frac{L}{h^2{}_o}\right)^2 + 448\pi^2 + \left(\frac{ho}{L}\right)^2 [53 + 3(1-\xi web)]\pi^4}{\pi^4 + 28\pi^2\left(\frac{L}{ho}\right)^2 + 420\left(\frac{L}{ho}\right)^4} \right)$$

$$= 0.001138(in^2)$$

Note that $\xi_{web} = (f_1\text{-}f_2)/f_1$ where f_1 and f_2 are the stresses at the opposite ends of the web. For bending of symmetric sections, $f_1 = -f_2$ and $\xi_{web} = 2$.

$$F_d = \beta \frac{k\phi fe + k\phi we + k\phi}{k\phi fg + k\phi wg}$$

$$= 1.0\frac{0.1530 + 0.1438 + 0}{0.004984 + 0.001138}$$

$$= 48.48(ksi)$$

$$M_y = S_{fy}F_y$$

$$= 0.7719(33)$$

$$= 25.47(in-kips)$$

$$M_{crd} = S_f F_d$$

$$= 0.7719(48.48)$$

$$= 37.42(in-kips)$$

$$\lambda_d = \sqrt{My / Mcrd}$$

$$= \sqrt{25.47 / 37.42}$$

$$= 0.8250$$

For $\lambda_d > 0.673$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y$$

$$= \left(1 - 0.22\left(\frac{37.42}{25.47}\right)^{0.5}\right)\left(\frac{37.42}{25.47}\right)^{0.5} 25.47$$

$$= 22.64(\text{in} - \text{kips})$$

$$M_a = M_n / \Omega_b$$

$$= 22.67 / 1.67$$

$$= 13.56(\text{in} - \text{kips})$$

From Step 1(a)

$M_{x(req)}$ = 9.46 in.kips < 13.56 in.kips

Therefore, the allowable flexural strength is limited by distortional buckling, but the initial stud selection is adequate.

Step 2 – Through-the-Punchout Bridging Design

Step 2(a) – Bridging Channel Design

The bridging channel is designed as a continuous beam supported by the major axis bending strength of each stud and loaded by the twisting moment from each stud. This is illustrated in Figure 2-5. Assume a 5-span (i.e., 5 stud spaces) condition as shown in Figure 2-6.

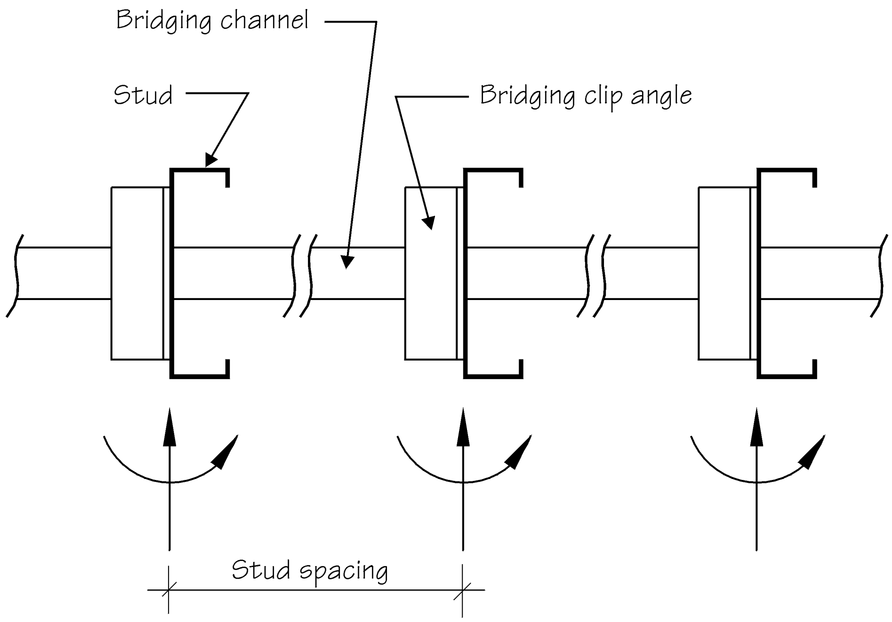


FIGURE 2-5

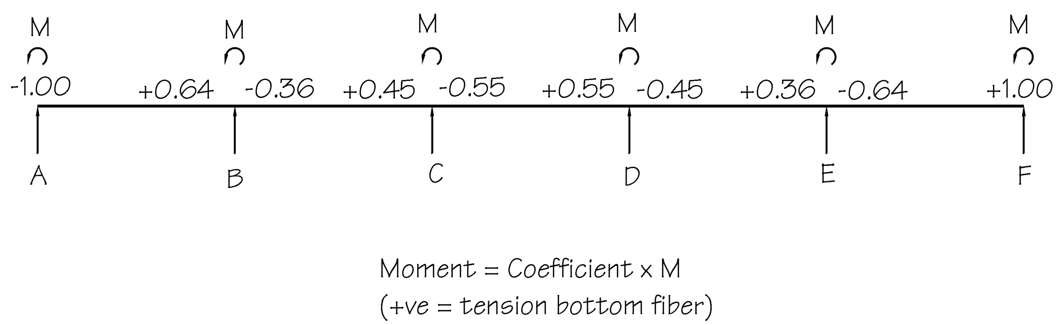


FIGURE 2-6

FIGURE 2-7

The outside span is critical and is shown with the moment coefficients in Figure 2-7. The moment, M, is derived from the top and bottom flange brace requirements given in *AISI S100* Section D3.2.1.

$P_L = 1.5(m/d)W$

where:

W = wa
w = load/ft on the stud for strength = (16/12)(28) = 37.3 lb/ft
a = bridging spacing = 4.33 ft.
m = stud web center line to shear center = 0.670"
d = 6"

$P_L = 1.5(0.670/6)(37.3)(4.33)$
$= 27.1$ lb.

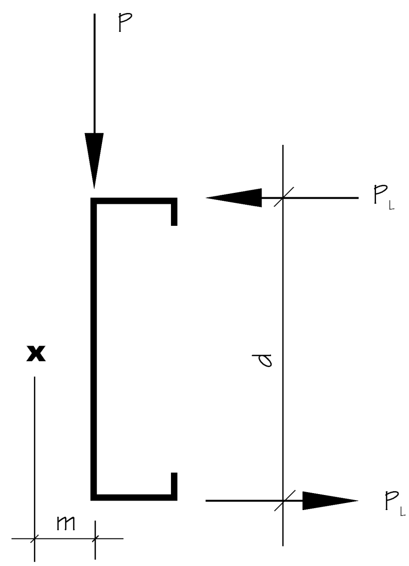


FIGURE 2-8

Then the moment resisted by the bridging channel is given by the flange brace couple with a lever arm equal to the depth of the stud. See Figure 2-8.

M = 27.1(6) = 162.6 in.lb and the resulting moment values in the outside span are illustrated in Figure 2-9.

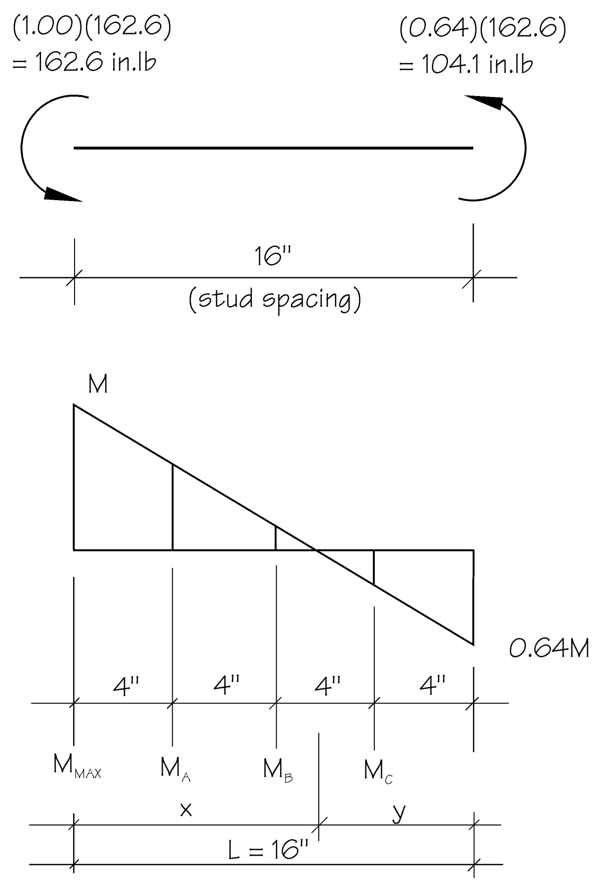


FIGURE 2-9

Try 150U50-54 (33) bridging channel.

Properties from product literature *or from AISI D100 formulas.*

Section will be fully effective at F_y = 33 ksi *(i.e., $\lambda \leq 0.673$ for all elements at $f = F_y$ AISI S100 B2.1 – calculations not shown here).*

S_x = 0.0520 in^3
r_y = 0.145 in.
A = 0.130 in^2
J = 0.000138 in^4
C_w = 0.00104 in^6
x_0 = 0.254 in.
r_0 = 0.622 in.

Check strength:

$M_{all} = F_y S_x / \Omega$

= (33)(0.0520)/1.67
= 1.03 in.kips

M_{req} = 0.163 in.kips << 1.03 in.kips ***OK***

Check lateral instability from AISI S100 Section C3.1.2:

E = 29,500 ksi
G = 11,300 ksi
$K_t = K_y = 1$

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$

where M_{max}, M_A, M_B and M_C are illustrated in Figure 2-9. Using similar triangles and absolute values:

x = 0.610L
y = 0.390L

M_{max} = M
M_A = 0.590M
M_B = 0.180M
M_C = 0.230M

Substituting in the expression for C_b gives:

C_b = 2.20

$$\sigma_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L_y}{r_y}\right)^2} = \frac{\pi^2 (29500)}{\left[\frac{(1)(16)}{0.145}\right]^2}$$

$$= 23.91 \text{ ksi}$$

$$\sigma_t = \frac{1}{Ar_0^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right]$$

$$= \frac{1}{(0.130)(0.622)^2}\left[11300(0.000138) + \frac{\pi^2 (29500)(0.00104)}{[(1)(16)]^2}\right]$$

$$= 54.52 \text{ ksi}$$

$$F_e = \frac{C_b r_0 A \sqrt{\sigma_{ey}\sigma_t}}{S_f}$$
$$= \frac{(2.20)(0.622)(0.130)\sqrt{(23.91)(54.52)}}{0.0520}$$
$$= 123.5 \text{ ksi}$$

$2.78F_y = 2.78(33) = 91.7$ ksi

For $F_e \geq 2.78F_y$

$F_c = F_y$ and there is no reduction in allowable moment for lateral instability.

$M_{all} = 1.03$ in.kips

$M_{req} = 0.163$ in.kips << 1.03 in.kips ***OK***

Use 150U50-54 (33) bridging channel.

Note 2-3

For torsional eccentricity to the flange center line:

e = m + (flange width)/2 = 0.670 + (1.625 - 0.451)/2
= 1.460"

P_L = 1.50(e/d)W = 1.50(1.460/6)(37.3)(4.33)
= 59.0 lb.

M_{req} = 59.0(6) = 354 in.lb

M_{all} = 1.03 in.kips > 0.354 in.kips ***OK***

Step 2(b) – Bridging Welded Connection

Refer also to Design Example #2, Step 6 for general comments on welded construction.

From Step 2(a), the twisting moment transferred from stud to bridging channel:

M_{req} = 162.6 in.lb

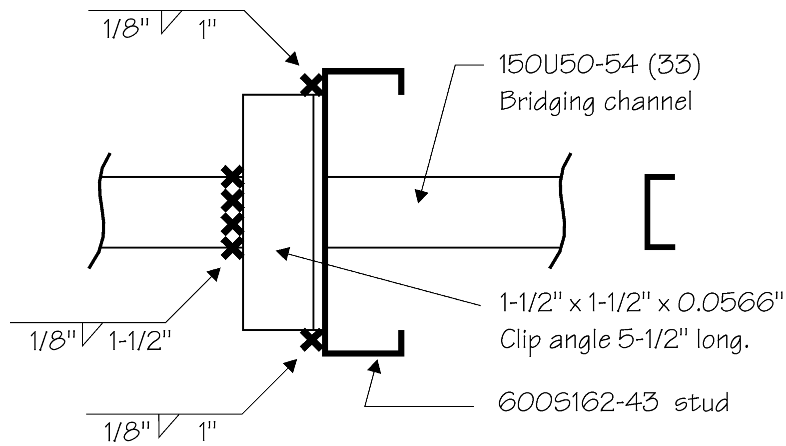


FIGURE 2-10

Clip angle size:

See Note 2-4.

Use t = 0.0566"

Note 2-4

The following design rules for wind bearing stud bridging clip angles are suggested:

- *Based on field experience and a limited amount of research (Drysdale 1991, Green 2004a), it is recommended that the thickness of the bridging clip angle be the greater of the thickness of the stud or 0.0566".*
- *Leg lengths to be 1-1/2".*
- *Length to be ≥ stud depth minus 1/2".*
- *Screws (or welds) placed in outer portion of the web of the stud.*

Bridging channel to clip angle weld (see Figure 2-10):

Clip angle:	t = 0.0566"	F_u = 65 ksi
Bridging channel:	t = 0.0566"	F_u = 45 ksi

Using the linear method, the maximum required load per inch of weld length is given by:

$$q_{req} = \frac{M_{req}}{S_{weld}} = \frac{162.6}{(1.5)^2 / 6} = 434 \text{ lb / in.}$$

With the lower strength bridging channel governing, the allowable weld capacity per inch of weld length is given by (See Appendix A.1):

$q_{all} = P_n / (\Omega L) = 0.75tF_u / \Omega$
$= 0.75(0.0566)(45)(1000)/3.05$
$= 626$ lb/in > 434 lb/in ***OK***

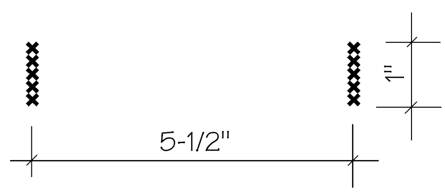


FIGURE 2-11

Clip angle to stud weld (see Figure 2-11):

Clip angle	t = 0.0566"	F_u = 65 ksi
Stud	t = 0.0451"	F_u = 45 ksi

Weld group allowable moment (stud material governs):

$M_{all} = 5.50 \times (0.75)tLF_u/\Omega$
$= 5.50(0.75)(0.0451)(1)(45)(1000)/3.05$
$= 2740$ in.lb >> 162.6 in.lb ***OK***

Note 2-5

For torsional eccentricity to the center line of the flange:

M_{req} = 354 in.lb from Note 2-3.

Bridging channel to clip angle weld:

$$q_{req} = \frac{M_{req}}{S_{weld}} = \frac{354}{(1.5)^2/6} = 944 \text{ lb/in.}$$

And from the Appendix A.1 conservative approach:
q_{all} = 626 lb/in < 944 lb/in **UNSATISFACTORY**

A more detailed approach to weld strength is provided in AISI S100 and is appropriate here. For fillet welds loaded transversely (Section E2.5(2)):

$q_{all} = P_n/(\Omega L) = tF_u/\Omega$
$= 0.0566(45)(1000)/2.35$
$= 1080$ lb/in > 944 lb/in **OK**

Clip angle to stud weld:

M_{all} = 2740 in.lb >> 354 in.lb **OK**

Step 2(c) – Bridging Screwed Connection

See Figure 2-12. For member sizes not shown, see Figure 2-10.

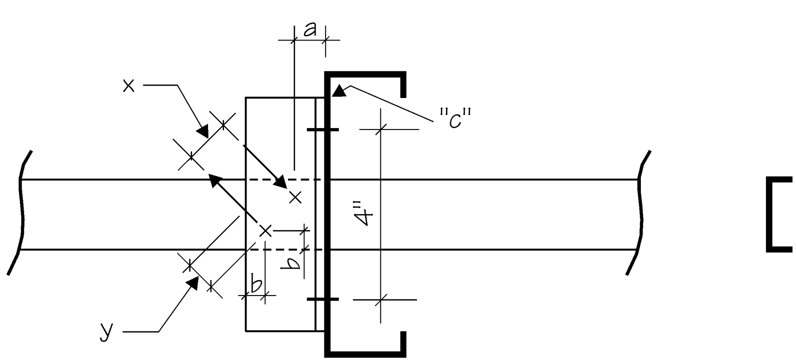


FIGURE 2-12

Clip angle size:

Use t = 0.0566" *(See Note 2-4)*

Bridging channel to clip angle screws:

Suggested dimensioning of the self-drilling screw locations is illustrated in Figure 2-12. Dimensions a and b are set and x and y are calculated. *(Note that bridging clip angles are frequently supplied with pre-drilled holes with dimensioning that varies from manufacturer to manufacturer. The dimensioning of pre-drilled holes should be used in design):*

a = 5/8" for screw gun clearance
b = minimum edge distance by *AISI S100* Section E4.2.
= 1.5d = 1.5(0.190) = 0.285"
x = lever arm for force couple = 1.101"
y = edge distance parallel to force = 0.337"

Required shear per screw:

$V_{req} = M_{req}/1.101$ *(M_{req} from Step 2(a))*
= 162.6/1.101
= 148 lb/screw

Determine screw shear capacities per AISI S100 E4.3 assuming #10-16 self-drilling screws.

Screw design input values:

Clip angle	t_1 = 0.0566"	F_{u1} = 65 ksi
Bridging channel	t_2 = 0.0566"	F_{u2} = 45 ksi
Screw	Size = #10-16	d = 0.190" *(App. A Table A-2)*

Allowable shear per screw:

Screw allowable shear limited by E4.3.1 tilting and bearing:

t_2/t_1 = 1.0; therefore, choose the governing P_{ns} from AISI S100 Equations E4.3.1-1, E4.3.1-2 and E4.3.1-3.

$$P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2} = 1109 \text{ lb.} - \text{governs}$$
$$P_{ns} = 2.7 t_1 d F_{u1} = 1887 \text{ lb.}$$
$$P_{ns} = 2.7 t_2 d F_{u2} = 1307 \text{ lb.}$$

Gives:

$V_{all} = P_{ns}/\Omega$ = 1109/3 = 370 lb.

Screw allowable shear is limited by E4.3.2 shear in the screws themselves. Refer to *AISI S100.*

$P_{ns} = P_{ss}$
where P_{ss} = nominal shear capacity of screw. See Appendix A, Table A-1:

$P_{ns} = P_{ss} = 1400$ lb.

Gives:

$V_{all} = P_{ns} / \Omega = 1400/3$
$= 467$ lb.

The governing V_{all} is from E4.3.1 and is given by:

$V_{all} = 370$ lb. > 148 lb. ***OK***

Clip angle to stud screws:

Required tension per screw *(from moments about "c" in Figure 2-12):*

$T_{req} = M_{req} / (4 + 0.75)$
$= 162.6/4.75$
$= 34.2$ lb.

Allowable tension per screw:

Screw nominal tension limited by E4.4.1 pull-out:

$P_{not} = 0.85 t_c d F_{u2}$ where $t_c = t_2 = 0.0451"$
$= 0.85(0.0451)(0.190)(45)(1000)$
$= 328$ lb.

Screw nominal tension limited by E4.4.2 pull-over:

$P_{nov} = 1.5 t_1 d_w F_{u1}$

Does not govern by inspection.

Screw nominal tension is limited by E4.4.3 tension in the screws themselves. Refer to *AISI S100:*

$P_{nt} = P_{ts} = 1936$ lb.
where P_{ts} = nominal tensile capacity of the screw. See Appendix A, Table A-1.

The governing nominal tension is given by P_{not} = 328 lb. and

$T_{all} = P_{not}/\Omega = 328/3$
$= 109$ lb. > 34.2 lb. **OK**

Note 2-6

For torsional eccentricity to the center line of the flange:

M_{req} = 354 in.lb from Note 2-3.

Bridging channel to clip angle screws:

V_{req} = 354/1.101 = 322 lb.

V_{all} = 370 lb. > 322 lb. ***OK***

Clip angle to stud screws:

T_{req} = 354/4.75 = 74.5 lb.

T_{all} = 109 lb > 74.5 lb. ***OK***

Step 3 – Check Bottom Track and Sill Track for Lateral Instability

L_u = stud spacing = 16"

Assume:

$K_t = K_y = 1$

Try 600T125-43 (33) track.

From load tables:

M_{all} = 9.11 in.kips
F_y = 33 ksi
J = 0.000260 in^4
C_w = 0.307 in^6
r_0 = 2.28 in.
A = 0.383 in^2
r_y = 0.337 in.

S_f = full unreduced section modulus
= 0.604 in^3

From AISI S100 Section C3.1.2.1:

E = 29,500 ksi
G = 11,300 ksi
C_b = 1 (*Conservative - see AISI S100 Equation C3.1.2.1-6 if less conservative value required.*)

$$\sigma_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L_y}{r_y}\right)^2} = \frac{\pi^2 (29500)}{\left[\frac{(1)(16)}{0.337}\right]^2}$$
$$= 129.2 \text{ ksi}$$

$$\sigma_t = \frac{1}{A r_0^2}\left[GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2}\right]$$
$$= \frac{1}{(0.383)(2.28)^2}\left[11300\,(.000260) + \frac{\pi^2 (29500)(0.307)}{[(1)(16)]^2}\right]$$
$$= 176.8 \text{ ksi}$$

$$F_e = \frac{C_b r_0 A\sqrt{\sigma_{ey}\sigma_t}}{S_f}$$
$$= \frac{(1)(2.28)(0.383)\sqrt{(129.2)(176.8)}}{0.604}$$
$$= 219 \text{ ksi}$$

$2.78F_y = 2.78(33) = 91.7$ ksi

For $F_e > 2.78M_y$

$F_c = F_y$ and no moment reduction for lateral instability. *(Where $F_c < F_y$, see Step 1(b) for a simplified approach.)*

Therefore, based on the fully braced strength checks from Design Example #1, the bottom track and sill track are ***OK***.

Step 4 – Jamb Stud

Step 4(a) – Welded Jamb Stud Interconnection

Refer to the Jamb Stud Selection Table 1-1 in Design Example #1.

Try built-up section. *AISI S100* does not define interconnection requirements for this type of built-up member. Experience indicates that a connection spacing of 16" o.c. is adequate. The welds are required to transfer shear between the two stud sections and to generate at least partially closed section torsional behavior. See Figure 2-13.

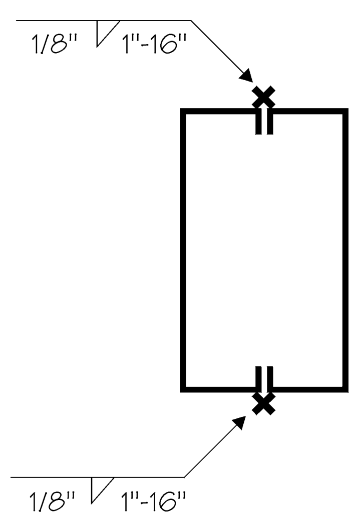


FIGURE 2-13

From Design Example #1, Table 1-1, this jamb stud is overstressed in web crippling:

P_{all} = 518 lb.
P_{req} = 565 lb. > 518 lb. ***UNSATISFACTORY***

Recalculate the web crippling, P_{all}, using instead the expression and coefficients provided in AISI S240 Section B3.2.5.1. See Notes 1-1 and 2-7. See also additional limitations on the use of this web crippling expression in AISI S240 Section B3.2.5.1.

$$P_{all} = \frac{P_{nst}}{\Omega} = \frac{C t^2 F_y}{\Omega}\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right)$$

R = 0.0712"
t = 0.0451"
Depth = 6"
h = Depth - 2t - 2R = 5.767"
N = 1"
F_y = 33 ksi
C = 3.7
C_R = 0.19
C_N = 0.74
C_h = 0.019
Ω = 1.70

Substituting:
P_{all} = 0.392 kips per web

For 2 studs toe-to-toe:
P_{all} = 2(392) = 784 lb. > 565 lb. ***OK***

> *Note 2-7*
>
> *The stud-to-track web crippling expression in AISI S240 assumes that both flanges of the stud are connected to both flanges of the track. For the inner and outer top track detail used here, this condition is met. However, for the single top track deflection detail, this condition is not met (there is no connection between the stud flanges and the single top track), and web crippling reverts to the expression provided in AISI S100 for the unfastened, end-one flange condition.*

Step 4(b) – Screwed Jamb Stud Interconnection

The toe-to-toe stud configuration from Step 4(a) is not recommended for screwed construction because it is difficult to effectively connect the studs together with screws. Instead use Built-Up Section D from Table 1-1 Design Example #1. See Design Example #1 Step 8(d) for suggested interconnection requirements.

Step 5 – Miscellaneous Connection Design

Step 5(a) – Welded Window Head Interconnection

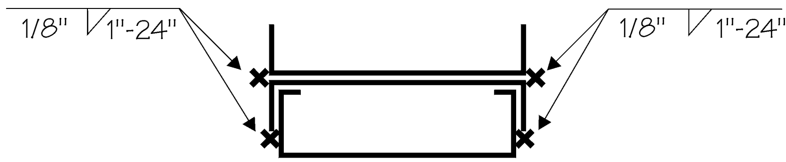


FIGURE 2-14

Note that the weld requirements for the alternative window head built-up section in Figure 1-6B would be similar.

Step 5(b) – Screwed Window Head Interconnection

See Design Example #1, Step 8(e).

Step 5(c) – Welded Stud to Top or Bottom Track Connection

See Note 2-8.

Note 2-8

Connection design assumptions:

- *All of the stud end shear is transferred in bearing against one upstanding leg of the track. Therefore, the welds (or screws) are not required to transfer any stud end shear.*
- *Welds (or screws) are required to provide torsional restraint to the end of the stud.*
- *Welds (or screws) are not required to provide torsional restraint to the track. (Depending on the fastening of the track to the primary structure, some torsional restraint might be required but it has been neglected here.)*
- *The general case of torsion at the end of the stud is provided in Appendix I. For the design of this connection, the torsional term, Kawm, described in Appendix I, is conservatively neglected.*

Required end reaction for typical stud:
= 0.5wL(spacing/12)
= 0.5(28)(13)(16/12)
= 243 lb.

From *AISI S100* Section D3.2.1:

See Figure 2-15.

$P_{req} = P_{L1} = - P_{L2}$ = load/weld to restrain end of stud torsionally
= (m/d) x (stud end reaction)
= (m/d)P

P_{req} = (0.670/6)(243)
= 27.1 lb.

and for 1" weld length
$P_{all} = 0.75tLF_u/\Omega$
= 0.75(0.0451)(1)(45)(1000)/3.05
= 499 lb. >> 27.1 lb. ***OK***

Note that the weld configuration shown in Figure 2-15 allows welding from one side.

Step 5(d) – Screwed Stud to Top or Bottom Track Connection

See Step 5(c) for design assumptions and required load per screw.

P_{req} = 27.1 lb/screw

Screw input values:

Track	t = 0.0451"	F_u = 45 ksi
Stud	t = 0.0451"	F_u = 45 ksi
Screw	Size = #10-16	d = 0.19" *(Table A-2)*

Allowable Shear per Screw

Screw allowable shear limited by AISI S100 Section E4.3.1, tilting and bearing:

t_2/t_1 = 1.0; therefore, choose the governing P_{ns} from *AISI S100* Equations E4.3.1-1, E4.3.1-2 and E4.3.1-3.

$$P_{ns} = 4.2\,(t_2^3 d)^{1/2}\, F_{u2} = 789 \text{ lb.} - \text{governs}$$
$$P_{ns} = 2.7\, t_1 d F_{u1} = 1041 \text{ lb.}$$
$$P_{ns} = 2.7\, t_2 d F_{u2} = 1041 \text{ lb.}$$

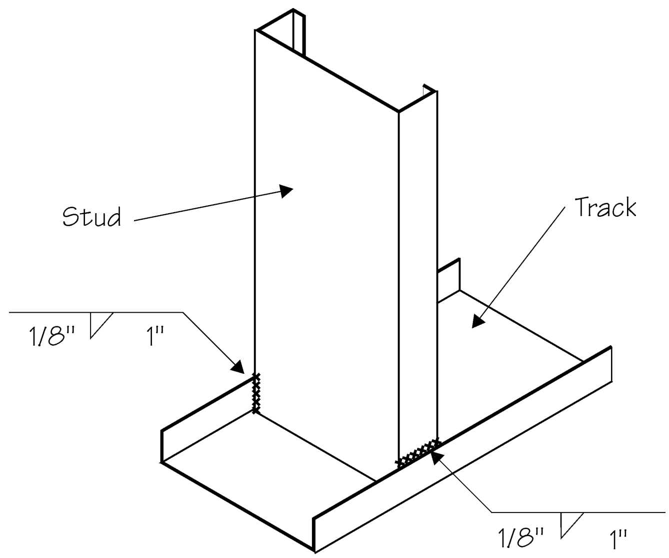


FIGURE 2-15

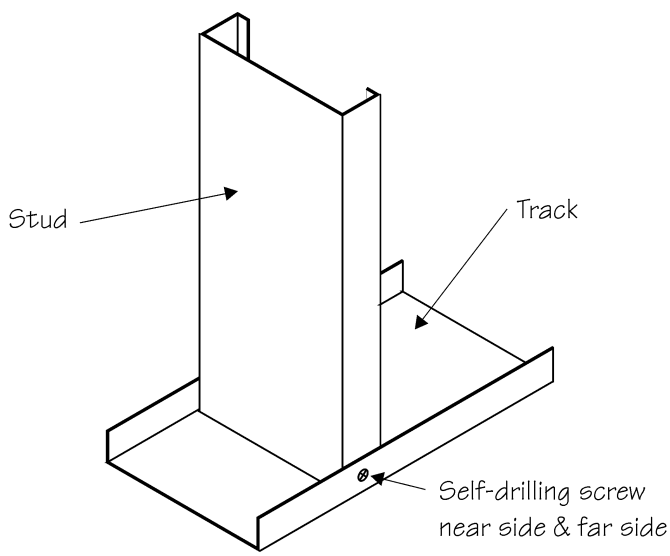


FIGURE 2-16

Gives:

$V_{all} = P_{ns}/\Omega = 789/3$
$= 263$ lb.

Screw allowable shear limited by AISI S100 Section E4.3.2, shear in the screws themselves. Refer to *AISI S100*.

$P_{ns} = P_{ss}$

where P_{ss} = nominal shear capacity of screw. See Appendix A, Table A-1.

$V_{all} = P_{ns}/\Omega = \phi P_{ss}/\Omega = 1400/3$
$= 467$ lb.

The governing V_{all} is from E4.3.1 and is given by

$V_{all} = 263$ lb. > 27.1 lb. ***OK***

Step 5(e) – Welded Sill to Jamb Connection

The end connection transfers both shear and torsion from the sill member. The torsional component is described as a general case in Appendix I. For the purposes of this design example, the term *Kawm (as described in Appendix I)* is neglected. See Figure 2-17.

Stud and sill track:
t = 0.0451" $F_u = 45$ ksi

Required end reaction (Design Example #1, Step 5(b)):
$V_{req} = 296$ lb.

and from Appendix I, the required torsional moment on the weld group *(neglecting Kawm)* is given by:

$M_{req} = Rm = V_{req}\, m$

From load tables for 600T125-43 track
m = 0.336"

$M_{req} = 296(0.336)$
$= 99.5$ in.lb

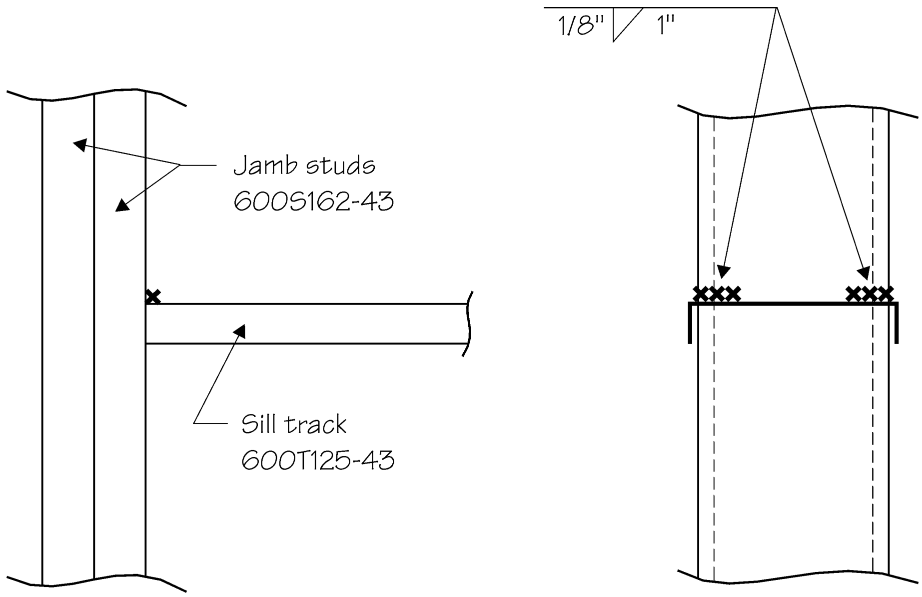


FIGURE 2-17

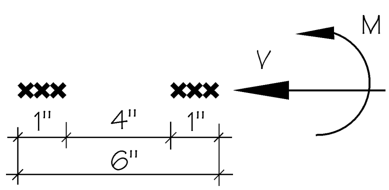


FIGURE 2-18

See Figure 2-18. Using the linear method, the maximum required load per inch of weld length is given by the vector addition of the two stress components:

$$q_{req} = \sqrt{\left(\frac{M_{req}}{S_{weld}}\right)^2 + \left(\frac{V_{req}}{A_{weld}}\right)^2}$$

$S_{weld} = I_{weld}/c$
$= 2\,[1/12(1)^3 + 1(2.5)^2\,]\;/\;3$
$= 4.22\ in^2$

$A_{weld} = L = 2(1) = 2$ in.

$$q_{req} = \sqrt{\left(\frac{99.5}{4.22}\right)^2 + \left(\frac{296}{2}\right)^2}$$

$$= 150 \text{ lb/in}$$

$q_{all} = P_n/(L\Omega) = 0.75tF_u/\Omega$
$= 0.75(0.0451)(45)(1000)/3.05$
$= 499 \text{ lb/in} > 150 \text{ lb/in}$ ***OK***

Step 5(f) – Screwed Sill to Jamb Connection

For detailing of this connection, refer to Figure 1-18. The forces on the fasteners are the same as Design Example #1, Step 8(f) except that here the torsional moment, M, is superimposed.

From Step 5(e), the moment and shear applied to the connection are given by the following specified values:

V_{req} = 296 lb.
M_{req} = 99.5 in.lb

For screws connecting the angle to the sill track, see Figures 2-19 and 2-20.

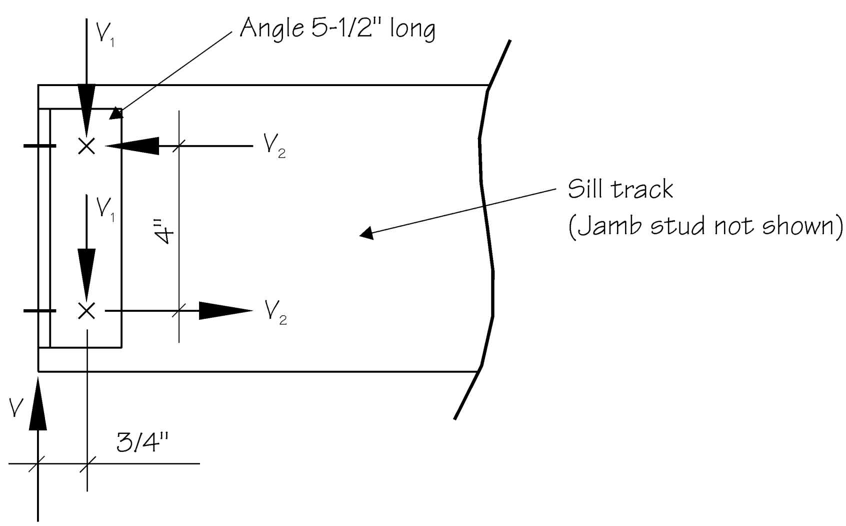


FIGURE 2-19

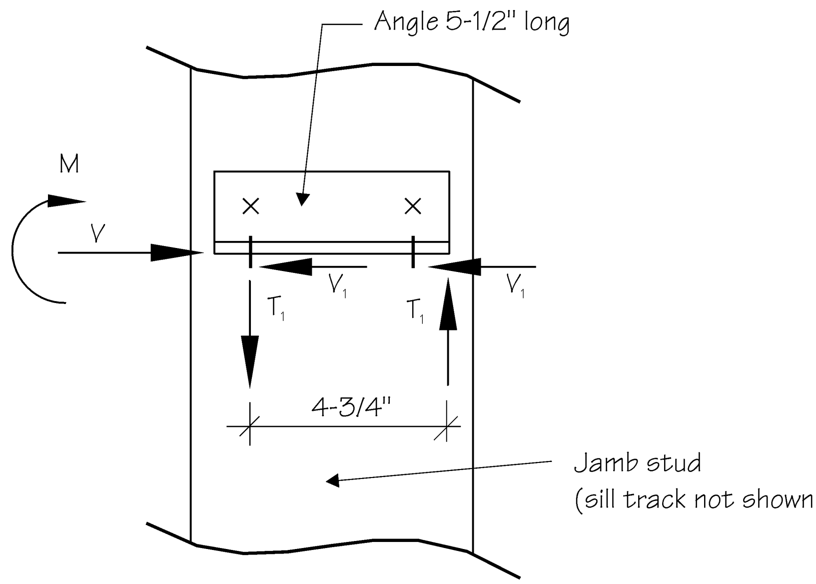


FIGURE 2-20

$M = M_{req} = 99.5$ in.lb
$V = V_{req} = 296$ lb.

$V_1 = V/2 = 296/2 = 148$ lb.
$V_2 = Ve/4 = 296(0.75)/4 = 55.5$ lb.

Shear resultant

$$V_{req} = \sqrt{V_1^2 + V_2^2} = \sqrt{148^2 + 55.5^2}$$
$$= 158 \text{ lb.}$$

$T_{req} = T_1 = M/4.75$
$= 99.5/4.75$
$= 20.9$ lb.

Screw allowable shear from Design Example #1, Step 8(f):

$V_{all} = 263$ lb. per screw

Screw allowable tension:

Screw nominal tensile strength limited by AISI S100 E4.4.1 pull-out:

$P_{not} = 0.85 t_c d F_{u2}$ where $t_c = t_2 = 0.0451"$
$= 0.85(0.0451)(0.190)(45)(1000)$
$= 328$ lb.

Screw nominal tensile strength limited by AISI S100 E4.4.2 pull-over:

$P_{nov} = 1.5t_1d_wF_{u1}$

Does not govern by inspection.

Screw nominal tensile strength limited by AISI S100 E4.4.3 tension in the screws themselves:

$P_{nt} = P_{ts} = 1936$ lb.

Where P_{ts} = nominal tensile strength of screw. See Appendix A, Table A-1.

The governing nominal tensile strength is given by P_{not} = 328 lb. and

$T_{all} = P_{not}/\Omega = 328/3$
$= 109$ lb.

Per AISI S100 Section E4.5.2, Combined Shear and Pull-Out:

$$\frac{Q}{P_{ns}} + \frac{T}{P_{not}} \le \frac{1.15}{\Omega}$$

$$\frac{158}{789} + \frac{20.9}{328} \le \frac{1.15}{2.55}$$

$0.264 \le 0.451$ ***OK***

For screws connecting the angle to the jamb stud see Figure 2-21.

The vertical component of shear, V_3, is relieved by the applied torsional moment, M, and the resulting net force on these screws will by less than in Design Example #1, Step 8(f).

Therefore, by inspection ***OK.***

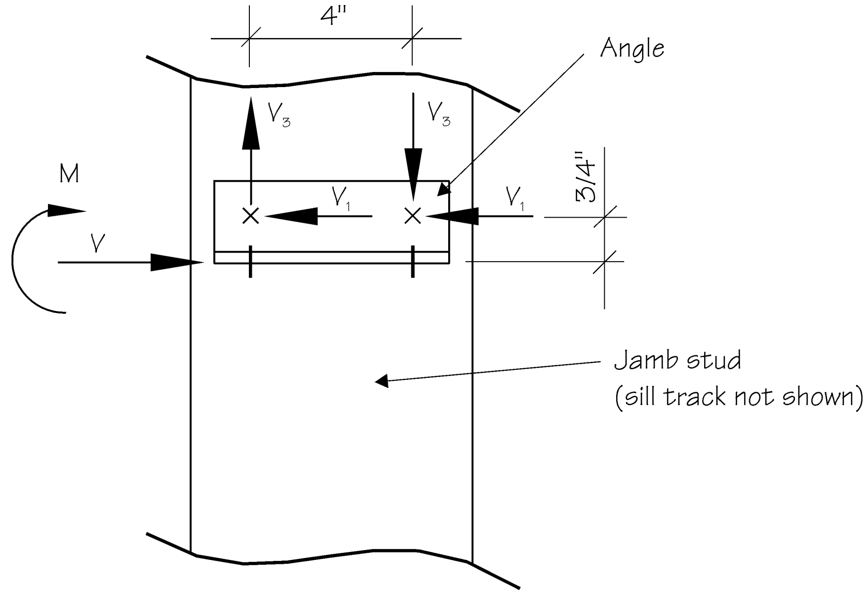


FIGURE 2-21

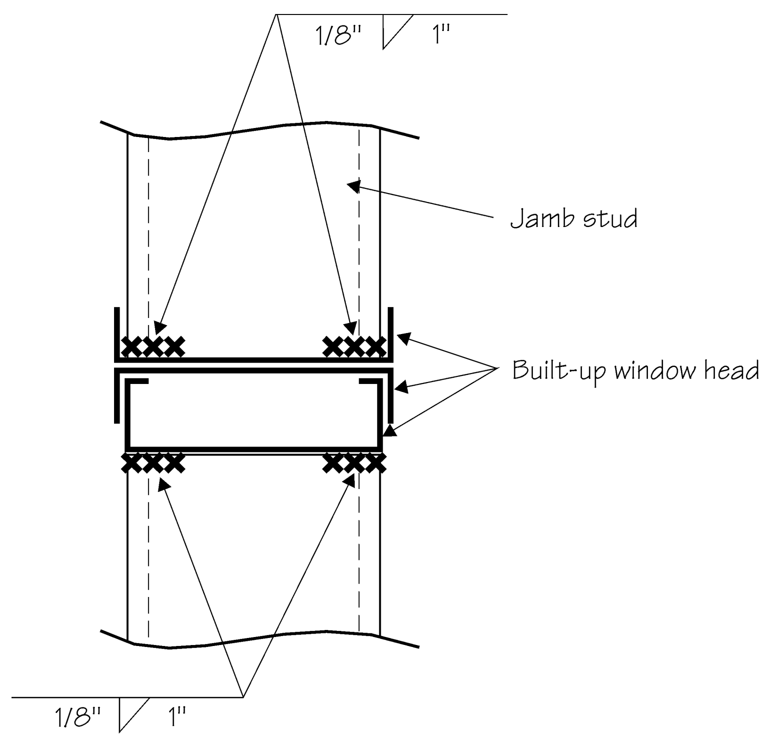


FIGURE 2-22

Step 5(g) – Welded Head to Jamb Connection

See Figure 2-22.

Step 5(h) – Screwed Head to Jamb Connection

See Figure 1-21.

Step 6 – General Comments on Welded Connections

The strength of welds in cold-formed steel in thicknesses less than or equal to 0.10 inches is a function of the tensile strength of the sheet and the length of the weld. It is assumed that the necessary weld leg size is available to develop the strength of the parent material.

Show a nominal weld size on drawings of, say, 1/8" accompanied by a note: "For material less than 0.10" thick, drawings show nominal weld leg sizes. For such material, the effective throat of welds shall not be less than the thickness of the thinnest connected part."

The minimum practical parent material thickness for welding varies with the skill of the welders and the welding procedure used. A common recommended minimum thickness is 0.0451". There is no minimum weld length requirement in *AISI S100*—a 1" minimum is used here.

Refer to Appendix A.1 for the origin of the general formula for the nominal strength of fillet and flare-bevel groove welds, $0.75tLF_u$.

The F_u values for various steels can be found in AISI D100, Part I.

Advantages of Welded Connections

1. Many connections can be made without supplementary clip angles, such as the window head and sill to jamb connections (Figures 2-17 and 2-22).

2. Special long-legged inner top tracks are not required for the inner and outer top track deflection detail.

3. Welded connections (with experienced welders) have generally less labor content than screwed connections. This is particularly the case in thicker material (say, ≥ 0.0566") and in shop conditions.

Disadvantages of Welded Connections

1. Experienced and certified welders capable of working with light gauge material may not be available. Damaged members from burn through are the usual consequence of inexperience.

2. In shop conditions, the fumes from the galvanizing vapors are toxic and require special air handling or special masks.

3. The galvanized coatings are locally damaged by the heat of welding, and touch-up with a zinc-rich paint is required.

4. Where a strength increase from cold work of forming is used (*as in this guide*), there is the possibility that this strength increase can be lost due to the heat of welding. AISI S100 requires testing to evaluate this effect. See Section A7.2(c). Alternatively, the cold work of forming effect can be conservatively ignored.

Step 7 – Details at Shearwalls

See Figure 2-23 and Note 2-9.

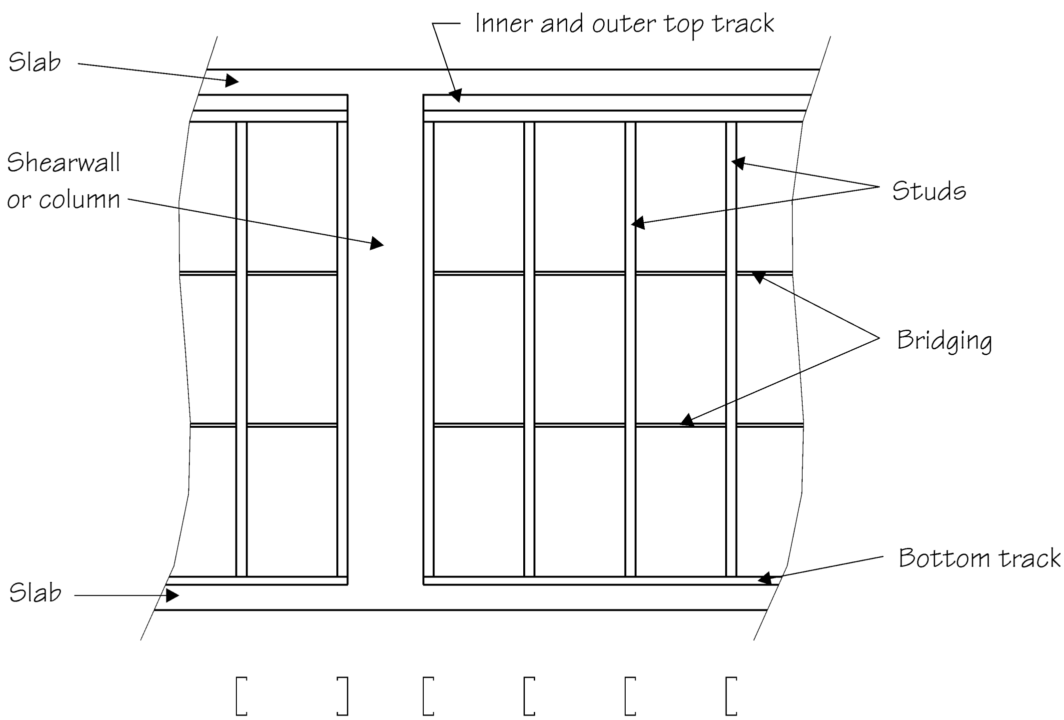


FIGURE 2-23

Note 2-9

In areas of low seismicity, or where framed walls are designed to rack with structure drift, anchor the last stud to the shearwall or column with wedge type expansion anchors at, say, 2'-8" o.c. This anchorage eliminates any differential wind load deflections between the stud and the shearwall or column, effectively anchors the bridging, and provides racking resistance in the plane of the wall.

Where building drift accommodation is based on a sliding system (i.e., where the top of the wall is disengaged from the structure in the plane of the wall), adequate separation between wall framing and finishes and the shearwall or column must be provided.

Step 8 – Parapets

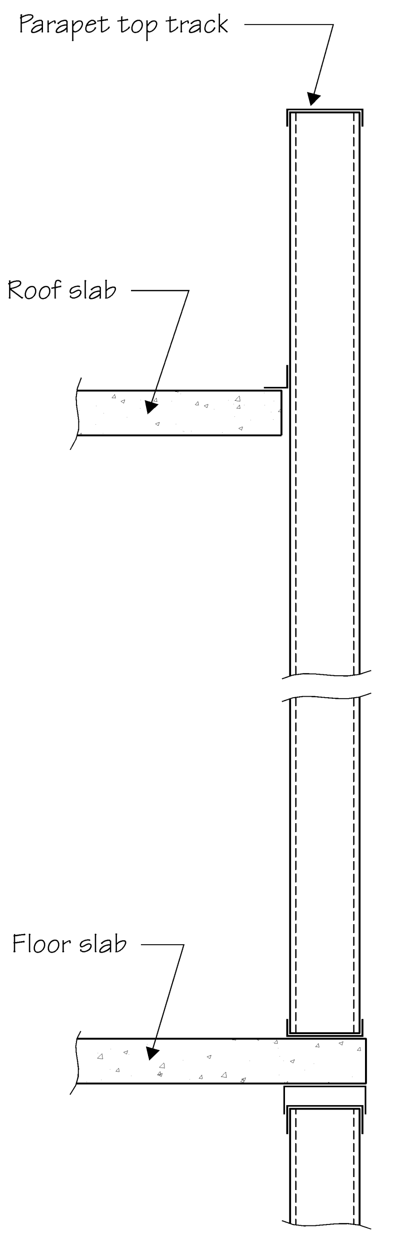


FIGURE 2-24

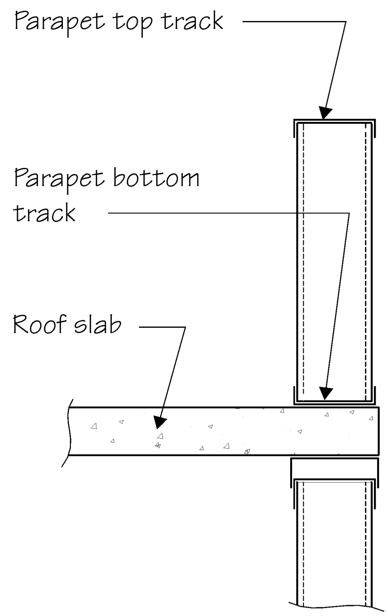


FIGURE 2-25

Note 2-10

1. *Some parapets are designed to accept substantial vertical and lateral loads from window washing equipment, swing stages, etc. and also may be required to function as a guard. It is assumed that these loads do not apply to this project. Nevertheless, parapet design should anticipate considerable abuse during construction (gravel buggies) and in the completed structure (re-roofing operations).*

2. *Figure 2-25, as illustrated, is generally not acceptable. A fixed base moment resisting condition is required which cannot be achieved by anchoring the bottom track to the top of the roof slab.*

   This detail will work if each stud is directly connected to the roof slab with a full moment connection. There are proprietary connection devices available for this purpose. Alternatively, provide anchor plates cast into the roof slab at regular intervals with hot-rolled angles, channels or hollow structural section members welded to the anchor plates and cantilevering over the height of the parapet to support the top track. The studs are then designed for the relatively trivial simply supported condition between the tracks. The top track is designed to span between the cantilevering posts and the bottom track is designed to span between fasteners into the roof slab, which will be controlled by the 2'-8" o.c. maximum recommended spacing in Drysdale 1991. This cantilevering post design approach is detailed in Step 8(a) that follows.

3. *The design calculations for the alternative configuration in Figure 2-24 are provided in Step 8(b).*

   In order to accommodate roof slab deflections, a proprietary slide clip connection detail has been shown – see Figure 2-32.

   If the specified slide clip is not capable of providing torsional restraint, additional bridging or blocking may be required within 12″ of the clip. Design of the bridging would be as illustrated in Steps 8(e) below.

Step 8(a) – Cantilevering Hallow Structural Section (HSS) Post Approach

See Figure 2-26.

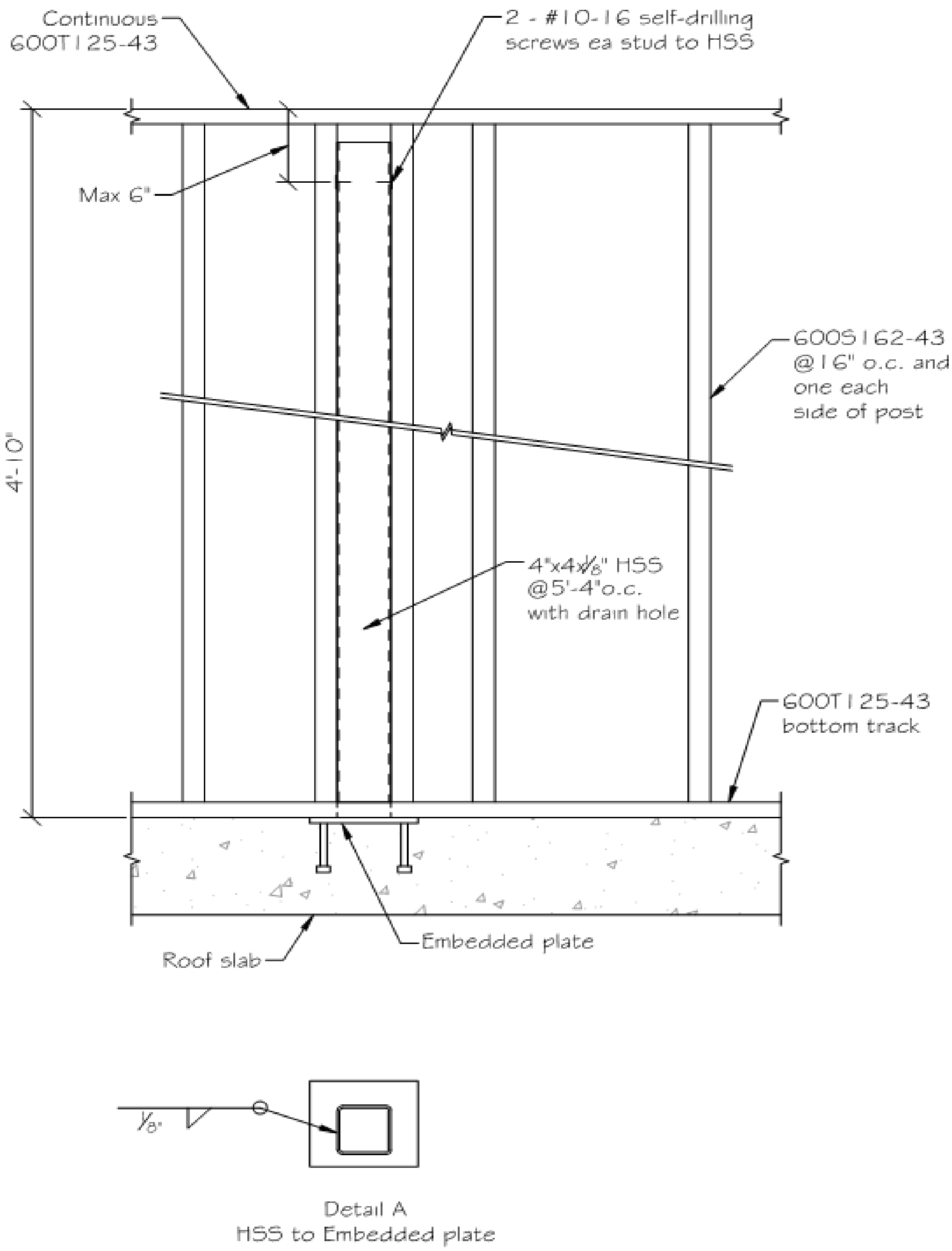


FIGURE 2-26

i) Top Track

Assume continuous top tack supported by posts at 5'-4" o.c. For strength check, assume a two-span condition and for deflection, assume one span.

Wind load on top track:
w = (parapet height/2)(28) = (4.83/2)(28)
= 67.6 lb/ft

Note that wind pressures on parapets are often much higher than typical wall pressures. For this example, 28 psf is used for convenience.

For the middle support of the two-span case:

$$M_{req} = \frac{wL^2}{8} = \frac{67.6(5.33)^2}{8}\left(\frac{12}{1000}\right)$$
$$= 2.88 \text{ in.kips}$$
$$V_{req} = \frac{1.25wL}{2} = \frac{1.25(67.6)(5.33)}{2(1000)}$$
$$= 0.225 \text{ kips}$$

From load tables for 600T125-43 (33):

M_{all} = 9.11 in.kips > 2.88 in.kips ***OK***

V_{all} = 1.377 kips > 0.225 kips ***OK***

Combined bending and shear - *AISI S100* Section C3.3.1

$$\sqrt{\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2}$$
$$= \sqrt{\left(\frac{M}{M_{all}}\right)^2 + \left(\frac{V}{V_{all}}\right)^2} = \sqrt{\left(\frac{2.88}{9.11}\right)^2 + \left(\frac{0.225}{1.377}\right)^2}$$

= 0.36 < 1.00 ***OK***

ii) Cantilevering Post

Try 4" × 4" × 1/8" HSS (F_y = 33 ksi) at 5'-4" o.c. For HSS, use *AISI S100.*

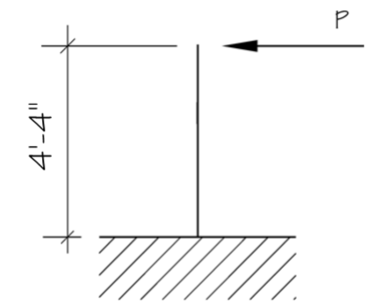


FIGURE 2-27

Required moment and shear - see Figure 2-27:

P = track reaction(4.83/4.33) – For connection max 6″ below top track
= 1.25wL(4.83/4.33) (for the two-span track condition - worst case)
= 1.25(67.6)(5.33)(4.83/4.33)
= 502 lb.

M_{req} = PL = 502(4.33)(12/1000)
= 26.1 in.kips

V_{req} = P = 0.502 kips

Properties from product literature *(STI 2005):*

w/t = 31.5
t = 0.116"
$S_x = 2.20 \text{ in}^3$
$I = 4.40 \text{ in}^4$
$A = 1.77 \text{ in}^2$
$J = 6.91 \text{ in}^4$

Check if the HSS is subject to local buckling by *AISI S100* Section B2.1.

A flat under uniform stress is the critical case:

$$F_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 = \frac{4\pi^2 29500}{12\left(1-0.3^2\right)}\left(\frac{1}{31.5}\right)^2$$
$$= 107.5 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{33}{107.5}}$$
= 0.554 < 0.673, therefore, no local buckling.

Allowable moment:

$M_{all} = F_y S_x / \Omega$
$= 33(2.20)/1.67$
$= 43.5$ in.kips > 26.1 in.kips ***OK***

Allowable shear:

By *AISI S100* Section C3.2.1

$$\sqrt{\frac{Ek_v}{F_y}} = \sqrt{\frac{29500(5.34)}{33}} = 69.1$$

h/t = 31.5 < 69.1 gives:

$F_v = 0.60F_y = 0.60(33)$
$= 19.8$ ksi

$V_n = A_w F_y = (h/t)t^2 F_y = 31.5(0.116)^2(33)$
$= 8.38$ kips/web

For 2 webs

$V_{all} = 2V_n / \Omega = 2(8.38)/1.60$
$= 10.5$ kips > 0.450 kips ***OK***

Combined bending and shear:

By *AISI S100* Section C3.3.1:

$$\sqrt{\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2}$$
$$= \sqrt{\left(\frac{M}{M_{all}}\right)^2 + \left(\frac{V}{V_{all}}\right)^2} = \sqrt{\left(\frac{26.1}{43.5}\right)^2 + \left(\frac{0.450}{10.5}\right)^2}$$

$= 0.60 < 1.00$ ***OK***

Deflection *(with 0.7 factor on ASD wind load = 0.42 factor on LRFD wind load) – approximated for full-height tube.*

$P_{top} = 1.25WL = 1.25(67.6)(5.33) = 450$ lb

$$\delta = \frac{0.7(P_{top})L^3}{3EI} = \frac{0.7(450)(4.83 \times 12)^3}{3(29500)(1000)I}$$
$$= \frac{0.693}{I}$$

The maximum permissible deflection is calculated using twice the length of the cantilever - see Note 2-12.

$$\delta_{max} = \frac{2L_{cant}}{360} = \frac{2(4.83)(12)}{360}$$
$$= 0.322 \text{ in.}$$

$$I_{req} = \frac{0.693}{0.322}$$

$= 2.15 \text{ in}^4 < 4.40 \text{ in}^4$ ***OK***

Lateral-torsional buckling for HSS:

From *AISI S100* Section C3.1.2.2

$$L_u = \frac{0.36 C_b \pi}{F_y S_f}\sqrt{EGJI_y}$$

C_b = 1 (conservative)
F_y = 33 ksi
S_f = 2.20 in^3
E = 29500 ksi
G = 11300 ksi
J = 6.91 in^4
I_y = 4.40 in^4

Substituting:

L_u = 1568 in. >> 58" ***OK***

Therefore, no moment reduction for lateral buckling.

iii) Cantilevering Post Welding

See Figure 2-26, Detail A.

The strength of welded joints with t > 0.10" is limited by the approximate method in Appendix A or shear through the effective throat of the weld itself.

Weld design data:

t_{min} = 0.116"
F_y = 33 ksi
Electrode F_{xx} = 60 ksi

Allowable strength for 1/8" fillet weld:

From Appendix A

$q_{all} = P_n/(L\Omega) = 0.75tF_u/\Omega$
$= 0.75(0.116)(45)/3.05$
$= 1.28$ kips/in

From *AISI S100* Section E2.5

$q_{all} = P_n/(L\Omega) = 0.75t_wF_{xx}/\Omega$
$= 0.75(0.707)(0.125)(60)/2.55$
$= 1.56$ kip/in

Use q_{all} = 1.28 kips/in

Embedded plate to HSS post weld *(Figure 2-26 Detail A):*

A_{weld} = 8 in. *(include only the welds to the webs here)*

$S_{weld} = I_{weld}/c$
$= 2[(1/12)(4)^3 + (4)(2)^2]/2$
$= 21.3$ in^2

M_{req} = 26.1 in.kips
V_{req} = 0.502 kips

The maximum required load per inch of weld length is given by the vector addition of the two stress components:

$$q_{req} = \sqrt{\left(\frac{M_{req}}{S_{weld}}\right)^2 + \left(\frac{V_{req}}{A_{weld}}\right)^2}$$

Substituting:

q_{req} = 1.23 kips/in < 1.28 kips/in ***OK***

iv) Stud to Post Connection

At connection 6″ below the top track, from ii) above, P = 502 lb. Allowable shear for #10 screws in 43-mil studs = 263 lb. Therefore, by inspection, use (2) #10-16 screws each stud to post.

Check stud web crippling at top track:
From the Jamb Stud design of Step 4, P_{all} = 0.392 kips/web. Therefore, for (2) studs at the post, P_{all} = 2(392) = 784 lb > 502 lb. No stiffeners are required.

Step 8(b) – Cantilevering Typical Stud Approach

The parapet dimensions and loading are illustrated in Figure 2-28. See also Figure 2-32.

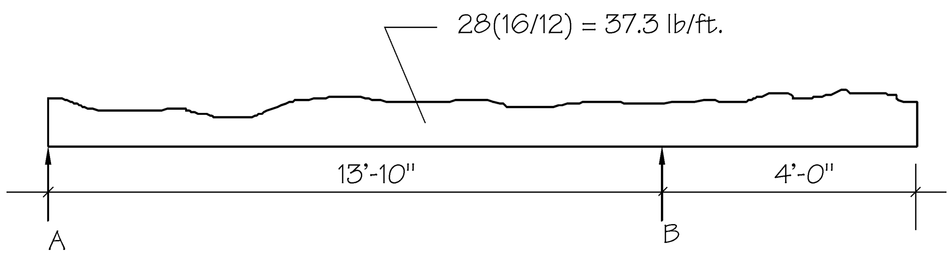


FIGURE 2-28

Reactions:

Moments about A
$13.83R_B - 37.3(17.83)^2/2 = 0$

R_B = 429 lb.

Required moment and shear at top reaction:

$M_{req} = M_B = [37.3(4)^2/2][12/1000] = 3.58$ in.kips

V_{req} (to the right of B) = 4(37.3) = 149 lb.

V_{req} (to the left of B) = R_B - 149 = 429 - 149 = 280 lb.

Try 600S162-43 (33) stud.

For Span A-B, recheck allowable span lengths from Design Example #1 *(a conservative approach that ignores the reduction in mid-span moment and deflection due to the cantilever loading):*

The extra 10" in span length is ***OK*** by inspection.

For cantilever,

M_{all} = 16.68 in.kips > 3.58 in.kips ***OK***
V_{all} (based on a punched web) = 1.24 kips > 0.280 kips ***OK***

Check web crippling at B. See Note 2-11.

Note 2-11

1. *For the web crippling check to be valid, web punchouts are not permitted in the vicinity of the reaction at B. The required distance can be checked from AISI S100 Section C3.4.2 Eq. C3.4.2-2 by setting R_c = 1 and solving which gives:*

   $x = 1.89h + 0.887d_h$
   $= 1.89(5.767) + 0.887(1.50)$
   $= 12.2"$

   Therefore, the required distance between the edge of bearing and the edge of the punchout must be ≥ 12.2" for no reduction in web crippling.

2. *Depending on the type of slide clip chosen, web crippling may not be a valid limit state. For example, slide clips that attach through the web of the stud do not result in bearing forces and web crippling need not be checked.*

For illustrative purposes, in cases where web crippling is a realistic limit state, derive the web crippling allowable strength at B for interior one-flange condition, unfastened and 3" bearing length. From *AISI S100* Section C3.4 and Table C3.4.1-2:

$$P_{all} = \frac{C t^2 F_y \sin\theta}{\Omega}\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right)$$

where:

R = 0.0712"
t = 0.0451"
Depth = 6"
h = Depth - 2t - 2R = 5.767"
N = 3"
F_y = 33 ksi
θ = 90 degrees
C = 13
C_R = 0.23
C_N = 0.14
C_h = 0.01
Ω = 1.65

Substituting:

P_{all} = 0.714 kips > 0.429 kips ***OK***

Check combined bending and web crippling at B *(AISI S100 Section C3.5.1):*

$$0.91\left(\frac{P}{P_n}\right)+\left(\frac{M}{M_{nxo}}\right)\le\frac{1.33}{\Omega}$$

where:

$P = R_B = 0.429$ kips
$M = M_B = 3.58$ in.kips
$P_n = \Omega_w P_{all} = 1.65(0.714) = 1.178$ kips
$M_{nxo} = \Omega_b M_{all} = 1.67(16.68) = 27.9$ in.kips
$\Omega = 1.70$

Substituting:

$$0.91\left(\frac{0.429}{1.178}\right)+\left(\frac{3.58}{27.9}\right)\le\frac{1.33}{1.70}$$

$0.46 \le 0.782$ ***OK***

Combined bending and shear:

By *AISI S100* Section C3.3.1,

$$\sqrt{\left(\frac{\Omega_b M}{M_{nxo}}\right)^2+\left(\frac{\Omega_v V}{V_n}\right)^2}=\sqrt{\left(\frac{M}{M_{all}}\right)^2+\left(\frac{V}{V_{all}}\right)^2}$$

where:

$M = M_B = 3.58$ in.kips
$V = V_B = 0.280$ kips
$M_{all} = 16.68$ in.kips
$V_{all} = 1.24$ kips (use punched shear)

Substituting:

$$\sqrt{\left(\frac{3.58}{16.68}\right)^2+\left(\frac{0.280}{1.24}\right)^2}\le 1.00$$

$0.31 \le 1.00$ ***OK***

Check cantilever deflection. *(See Note 2-12.)*

Note 2 - 12

In lightweight steel framing, it is common practice to check cantilever deflections assuming a deflection limit of L'/XXX (XXX = 360 for this example) where L' and the associated deflections are described in Figures 2-29 and 2-30. This approach is based on the assumption that a cantilever is analogous to a portion of a simply supported beam.

The simplified approach in Figure 2-29 is used here even though the slope is not zero at the base of the cantilever. A more rigorous model is shown in Figure 2-30 with L_s and δ_c taken to the middle of the back span. Where deflected shapes are different from that shown in Figure 2-30, L_s and δ_c can be taken instead to the point of zero slope.

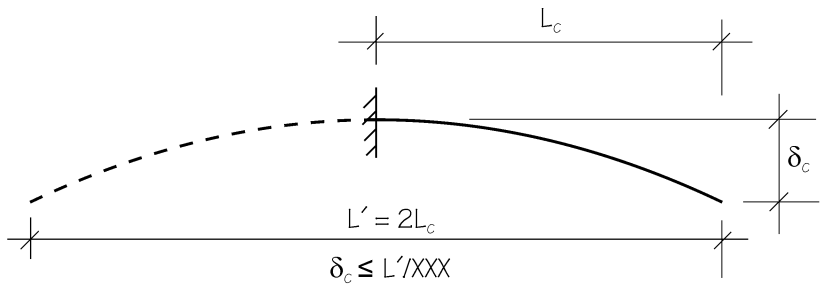


FIGURE 2-29
(from LGSEA 2001a)

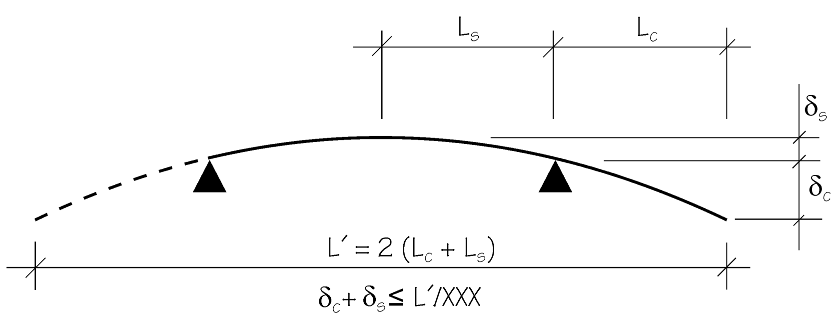


FIGURE 2-30
(from LGSEA 2001a)

By computer analysis with all spans loaded as in Figure 2-28 except:
w = 0.7(37.3) = 26.1 lb/ft

δ_c = 0.172"

L' = $2L_c$ = 2(48") = 96"

L'/360 = 96/360 = 0.267" > 0.172" ***OK***

Step 8(c) – Warping Torsional Stresses for Cantilevering Typical Stud

As shown in Figure 2-31, there is a distance of 12" between the roof reaction point and the line of bridging. The torsional eccentricity of the reaction point (with respect to the shear center of the section) will induce warping torsional stresses in the studs which can be checked using the model proposed in Appendix C. See also Step 1(a) from this design example for additional background to the procedure.

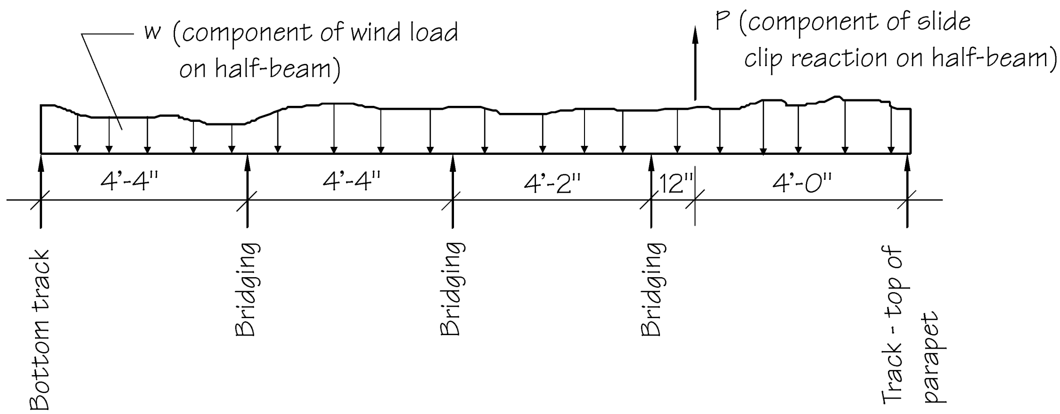


FIGURE 2-31

The torsional loads on the "half-beam", w and P, are illustrated in Figure 2-31 and are derived as follows:

e = m = 0.670 in. (distance from shear center to web center line)
h = depth – t = 6 – 0.0451 = 5.95 in.

w = 28(16/12)(e/h) = (28)(16/12)(0.670/5.95)
= 4.20 lb/ft

P = (Reaction at B)(e/h) = (0.670/5.95)(429)
= 48.3 lb.

The maximum required moment on the half-beam is found by computer analysis of the continuous beam illustrated in Figure 2-31. The top and bottom track and the bridging lines are supports for the half-beam. The maximum moment occurs at the location of the concentrated load, P *(the cantilever reaction point)*.

$M_{t(req)}$ = 267 in.lb

Warping torsional stress:

$\sigma_w = M_{t(req)}/S_y$ (S_y from Step 1(a))
= 267/0.0598
= 4460 psi

Major axis bending stress:

$S_{x(eff)} = 0.767 \text{ in}^3$

$M_{x(req)} = 3580$ in.lb (at the cantilever reaction point)

Then

$\sigma_{bend} = M_{x(req)} / S_{x(eff)} = 3580/0.767$
$= 4670$ psi

R factor from AISI S100 Section C3.6:

$$R = \frac{f_{bending\_max}}{f_{bending} + f_{torsion}} = \frac{4670}{4670 + 4460}$$
$$= 0.512$$

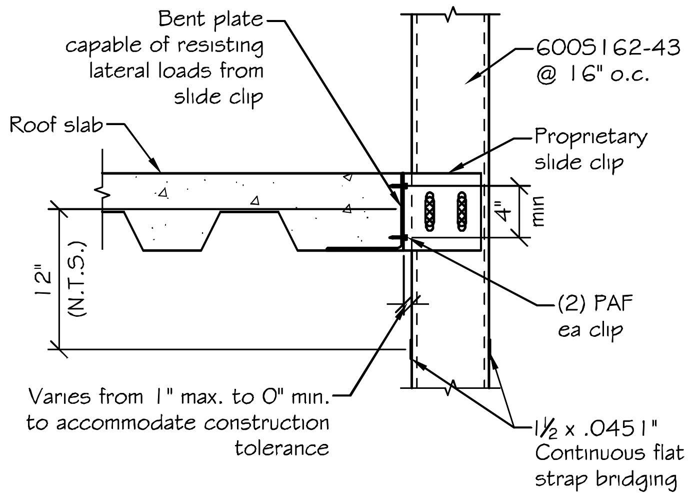
Reduced allowable moment
$= RM_{all} = 0.512(16.68)$
$= 8.54$ in.kips > 3.58 in.kips **OK**

Note 2-13

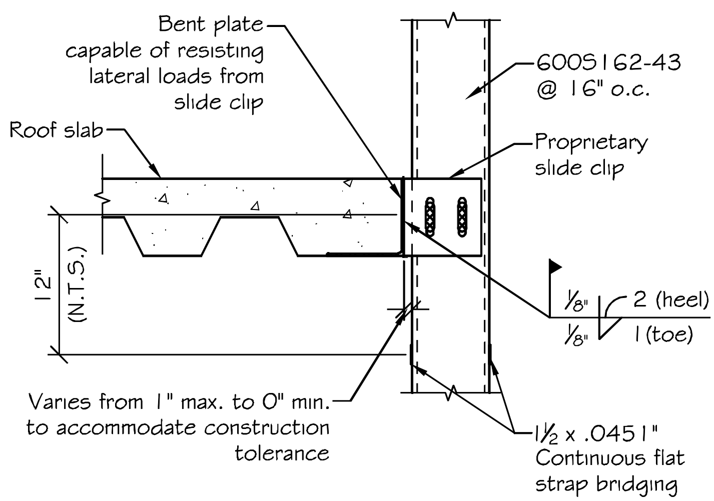
For torsional eccentricity to the center line of the flange:
$e = m$ + (flange width)/2 = 1.46"

For the half-beam
$w = 9.16$ lb/ft
$P = 105$ lb
$M_{t(req)} = 582$ in.lb
$\sigma_w = 9730$ psi
$RM_{all} = (0.324)(16.68) = 5.40$ in.kips > 3.58 in.kips ***OK***

Note that the RM_{all} should not be used in the interaction equations for moment and shear and moment and web crippling in Step 8(b).



PAF CONNECTION



WELDED CONNECTION

FIGURE 2-32

Step 8(d) – Stud to Roof Slab Connection Design

i) Proprietary Slide Clip:

Check capacity with manufacturer's published literature.

ii) Slide Clip to Bent Plate – Welded Connection:

V_{req} = 429 lb from Step 8(b) above.
Assume 68-mil slide clip, F_y = 50 ksi.

Allowable strength for 1/8" flare-groove weld:

From Appendix A

$q_{all} = P_n/(L\Omega) = 0.75tF_u/\Omega$
$= 0.75(0.0713)(65)/3.05$
$= 1.14$ kips/in

$L_{req'd} = V_{req}/q_{all}$
$L_{req'd} = 0.429/1.14 = 0.38$ inches

Note that the above is based on the force being concentric with the center line of the weld. For slide clips, the force may be applied either above or below the weld center line by the amount of the expected structural deflections. It is common to specify a weld length equal to 2x the expected deflection + $L_{req'd}$, as calculated above. For the case of ¾" upward or downward structural deflection, the total weld length along the heel of the clip would be 0.38+2(0.75) = 1.88 inches.

Therefore, use 2" weld at the heel of the clip and 1" stabilizing weld at the toe of the clip.

iii) Slide Clip to Bent Plate – PAF Connection:

Assuming PAF centered in the clip leg at the bent plate, the tension in the PAF will be $2xV_{req}$ = 858 lb to account for prying.

For a ¼" bent plate closure, per Appendix B, the allowable tension in the PAF is T_a = 775 lb/PAF.

Check pullover per AISI S100 Section E5.2.3:

$P_{nov} = \alpha_w t_1 d'_w F_{u1}$

α_w = 1.25 (conservatively taken for tapered standoff head PAF)
t_1 = 0.0713 inches for 68-mil clips
d'_w = 0.32 inches per manufacturer's data
F_{u1} = 65 ksi

Substituting:

P_{nov} = 1.85 kips

P_{all} = P_{nov}/Ω

= 1.85/3.0 = 0.618 kips/PAF **<u>*Controls*</u>**

For (2) PAF min 4" o.c. and assuming the reaction to be a maximum of ¾" above or below the center line of the PAF pair, the maximum load at any fastener, including 2x prying, is given by:

T_{max} = 2 x [V_{req}/2 + V_{req}(0.75)/4] = 590 lb/PAF

590 lb < 618 lb ***OK***

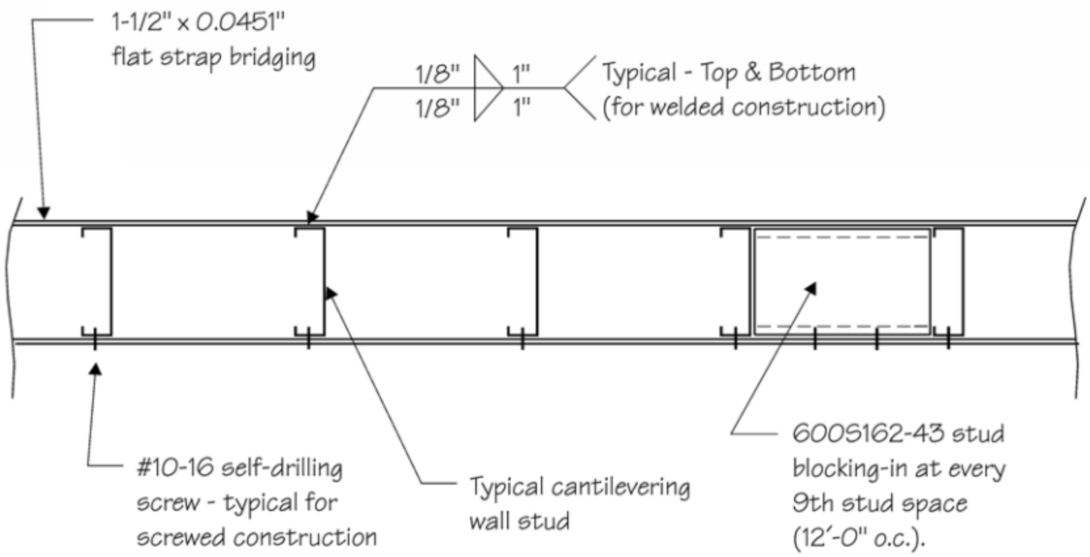
Therefore, use (2) PAF min 4" o.c.

Note 2-14

Where power-actuated fasteners act in tension, it is preferred to provide two or more fasteners in the interests of structural redundancy. Also, where earthquake design is a consideration, there may be restrictions on the use of power-actuated fasteners in tension - see Note 1-14.

Step 8(e) – Welded Flat Strap Bridging Design

As discussed in Note 2-10 Item 3, if the proprietary slide clip is assumed to provide no torsional restraint, a line of bridging is required. The line of bridging is shown in Figure 2-32, a distance of 12" from the roof reaction point (R_B). For the bridging flat strap details, see Figure 2-33. For blocking-in details, see Figure 2-34.



Note: For blocking-in connection details see Figures 2-34 and 2-35.

FIGURE 2-33

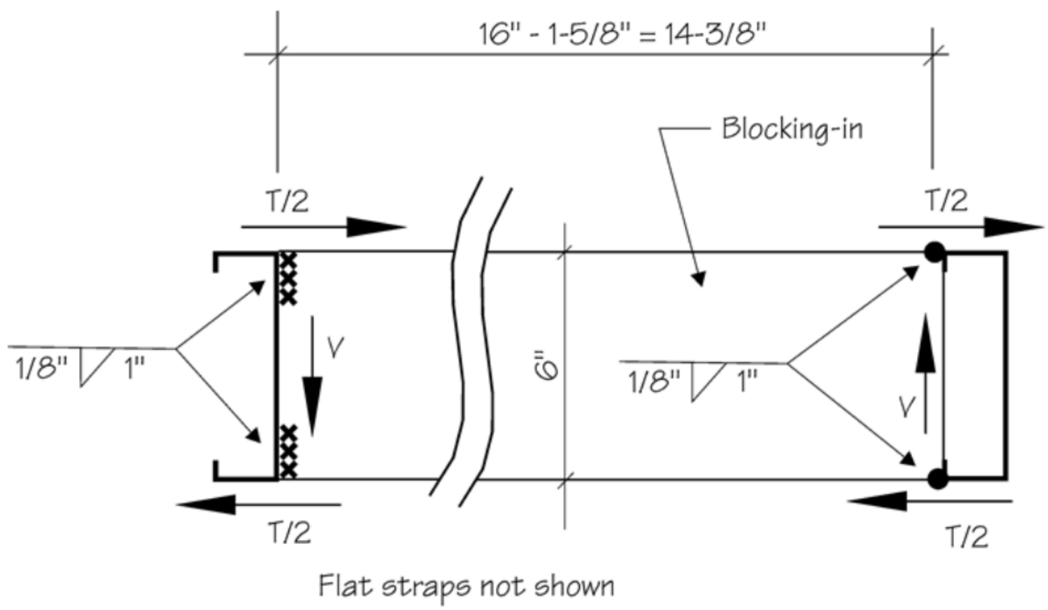


FIGURE 2-34

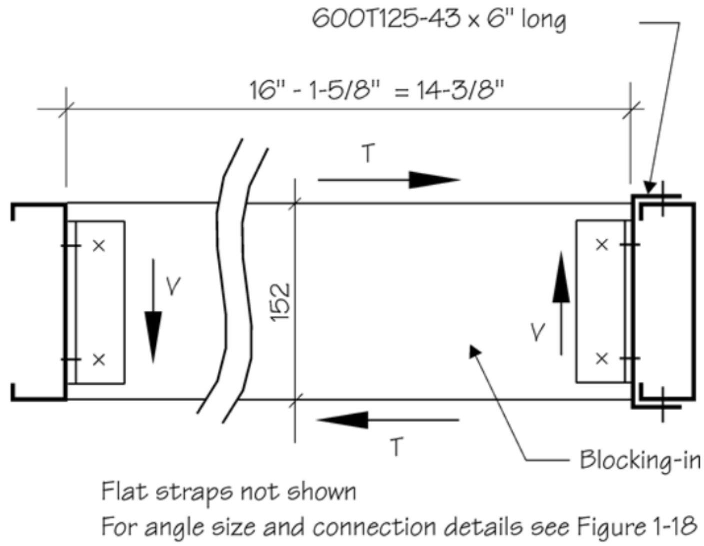


FIGURE 2-35

Check the bridging force required to restrain the stud according to *AISI S100* Section D3.2.1. The reaction at R_B will induce twisting in the stud, which in turn is partially relieved by the applied wind load.

From Step 8(b)
R_B = 429 lb.

and
m = 0.670"
d = 5.95" (center line stud depth)

The force, R_B, is within 0.3a of the bridging line, therefore:

$P_L = (m/d)R_B$
$= (0.670/5.95)(429)$
$= 48.3$ lb.

(The applied wind load 0.5a at either side of the bridging line offsets P_L and has been conservatively neglected here.)

Experience shows that a blocking spacing of 12′ typically results in workable force levels for both straps and connections. Therefore, assume blocking-in every 9 stud spaces = 12'-0" o.c.

The accumulated required bridging force
$T_{req} = T = 9(48.3) = 435$ lb.

Try 1-1/2" x 0.0451" flat strap bridging *(Figure 2-33).*

Check tensile capacity of flat strap – gross-cross section check only – *AISI S100* Section C2.

$T_{all} = T_n / \Omega = A_g F_y / \Omega$
$= (1.50)(0.0451)(33)/1.67$
$= 1.34 \text{ kips} > 0.435 \text{ kips}$ ***OK***

Blocking-in connection - see Figure 2-34:

The blocking-in is essentially a shear panel with equal and opposite applied loads, T, and with the internal forces, V, required for rotational equilibrium. The forces, V, are in turn resisted by the major axis bending strength of the connecting studs. Also see Note 2-15.

Note 2-15

The design of blocking-in is based on the assumption that the load in the interior flat strap is resisted by an equal and opposite load in the exterior flat strap. But equal and opposite flat strap loads are not always possible – the end of a run of wall is one such example. In this case, some additional flat strap anchorage is required such as connection to a built-up jamb with sufficient weak axis strength and stiffness or connection to a primary structural element such as a column or shearwall. Also, it may be advantageous to space the blocking-in more closely than shown here such that any unbalanced loads when they do occur are relatively small.

The details in Figure 2-34 assume that there is no direct weld between the flat strap and the blocking-in. *(It is difficult to weld a 1-1/2" wide flat strap to a 1-5/8" flange.)*

First, the strap loads are transferred to the flanges of two studs adjacent to the blocking-in. The weld is the typical strap to stud weld called up in Figure 2-33 *(not shown in Figure 2-34).*

Check weld capacity - strap load is transferred through 2 studs adjacent to the blocking-in with 2 welds per stud for a total weld length of 4 x 1" = 4".

$V_{all} = 0.75tLF_u / \Omega = (0.75)(0.0451)(4)(45)/3.05$
$= 2.00 \text{ kips} > 0.435 \text{ kips}$ ***OK***

Then the loads are transferred to the blocking-in via the welds shown in Figure 2-34. But these welds are also simultaneously resisting the internal shear force, V. Each of the four welds in Figure 2-34 is therefore subject to T/2 and V/2.

$V_{req} = V = 6T/14.375 = 6(435)/14.375$
$= 182$ lb.

For each 1" weld:

Required resultant load = $\sqrt{(V_{req}/2)^2 + (T_{req}/2)^2} = \sqrt{(182/2)^2 + (435/2)^2}$
$= 236$ lb. per weld

$V_{all} = 0.75tLF_u/\Omega = (0.75)(0.0451)(1)(45)(1000)/3.05$
$= 499$ lb. > 235 lb. ***OK***

For torsional eccentricity to the center line of the flange, see Note 2-16.

Step 8(f) – Screwed Flat Strap Bridging Design

Refer to Step 8(e) for the derivation of the required loads T_{req} and V_{req}. Refer also to Note 2-15. For the bridging flat strap details, see Figure 2-33. For blocking-in details, see Figure 2-35.

From Step 8(e), assuming again blocking-in every 9 stud spaces:
$T_{req} = T = 435$ lb.
$V_{req} = V = 182$ lb.

Try 1-1/2" × 0.0451" flat strap bridging *(Figure 2-33)*.

Check tensile capacity of flat strap – net cross-section check because of the screw hole. *AISI S100 E6.2 addresses tension rupture for members with holes.*

#10-16 self-drilling screw diameter = 0.190" *(Table A-2):*

$T_{all} = T_n/\Omega$ where $T_n = F_uA_e$

$A_e = U_{sl}A_{nt}$

Per AISI S100 Table E6.2-1, for multiple connectors in the line parallel to the force, $U_{sl} = 1.0$

For a single row of fasteners parallel to the line of force, g = 0 in AISI S100 Eq. E6.2-3. Therefore,

$A_{nt} = A_g - d_ht = 1.5(0.0451) – 0.190(0.0451) = 0.0591$ in^2

$T_{all} = 0.0591(45)/3.00$ ($\Omega = 3.00$ *per AISI S100 Table E6-1*)
$= 0.886$ kips > 0.435 kips ***OK***

Gross area check – see Step 8(e). ***OK***

From Step 5(d), the allowable shear strength of the #10-16 self-drilling screw is given by:
V_{all} = 263 lb/screw *($t_1 = t_2 = 0.0451"$)*

Unlike the welded detail, the flat strap is screw-connected directly to the blocking-in. This connection is shown in Figure 2-33.

The number of screws required for the flat strap to blocking-in connection is given by:

No. of screws = T_{req} /263 = 435/263 = 1.7 screws

Use 2 screws ***OK***

Blocking-in connection - see Figure 2-35:

The angle connection in shear was previously investigated in Design Example #1 Step 8(f). The same clip angle detailing as illustrated in Figure 1-18 is assumed here.

From that example:

V_{req} = 296 lb. *(required shear applied to the connection)*

resulting in:
V_{req} /screw = 158 lb.

But:
V_{all} /screw = 263 lb.

Therefore, the maximum allowable strength of the connection is given by:

V_{all} = (263/158)(296)
= 493 lb. > 182 lb. ***OK***

For torsional eccentricity to the center line of the flange, see Note 2-16.

> *Note 2-16*
>
> *For torsional eccentricity to the center line of the flange:*
> *$e = m + (\text{flange width})/2 = 1.46"$*
>
> *Gives:*
> *$T_{req} = 948$ lb.*
> *$V_{req} = 397$ lb.*
>
> *For welded blocking-in, use same details but reduce spacing to 8 stud spaces (calculations not shown here).*
>
> *For screwed blocking-in, use same spacing and details except increase the number of screws connecting the strap to blocking to 4 (calculations not shown here).*

Step 8(g) – Parapets and Cantilevering Studs at Window Locations

The calculations for window locations have not been done here. The following should be considered:

1. The built-up jamb stud should be carried through to the top of the parapet. Check for the same limit states as the typical stud. For built-up jamb studs, slide clips are typically used on both sides of the jamb.

2. The window built-up head detail should account for the extra weight, sag and accidental vertical loads applied to the parapet.

3. The cantilevering studs that extend upwards from the window head will alter the window head lateral loads compared with the typical case.

DESIGN EXAMPLE #3
WIND BEARING WALL WITH STRIP WINDOWS

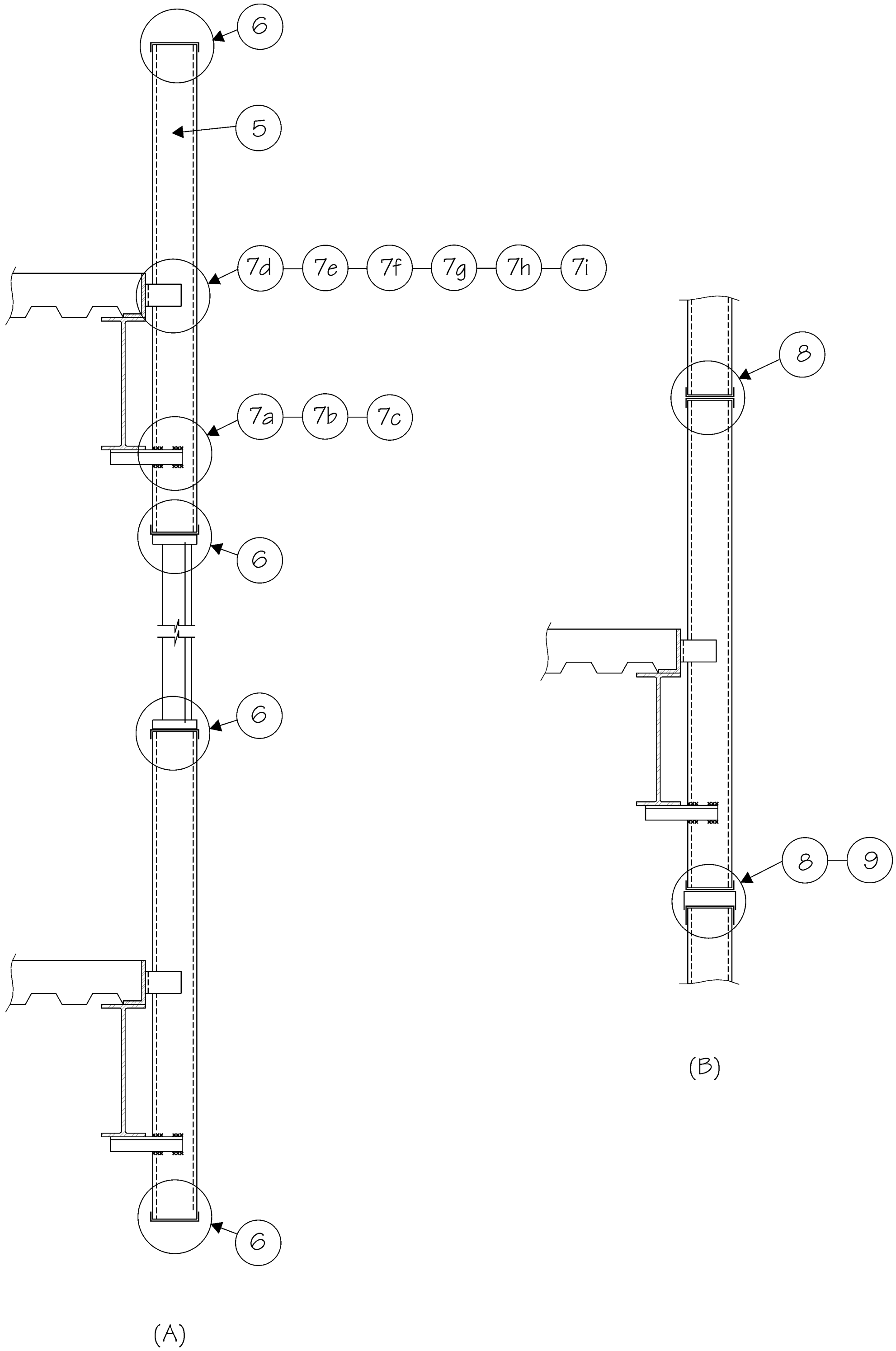


FIGURE 3-1

Introduction

This design example reviews three alternative methods for framing strip windows with cold-formed steel framing. Detailed design calculations are presented for the third alternative, studs outside the face of the structure.

The calculations assume welded connections and an all-steel system where the restraint of the sheathings is ignored. Where accounted for, torsional eccentricities are taken to the center line of the web.

The section numbers for the design of individual components are identified in Figure 3-1.

Refer also to the following:
Step 2 - Design Wind Loads
Step 3 - Design Earthquake Loads
Step 4 - Alternative Design Approaches
Step 9 - Alternative Detail for Shop-Applied Finishes

Step 1 – Given

- Stud spacing = 24" o.c.
- Stud depth = 6"
- Deflection limit = L/360
- Welded connections
- Sheathings are assumed not to provide torsional or weak axis restraint for the studs.
- Finishes are field-applied
- Weight of wall and window = 16 psf
- Geometry *(in elevation)* illustrated in Figure 3-2.

Step 2 – Design Wind Loads

Load combination factors for allowable strength design (ASD) are based on ASCE/SEI 7-10 *(ASCE 2010)* Section 2.4. For strength, the full ASD wind load is used. For deflection, 0.42 times the nominal wind load is used. For further discussion, refer to Introduction Item 3.2.

From the governing building code, the nominal wind load = 50 psf

Design wind load for strength = (0.6)50 = 30 psf
Design wind load for deflection = 0.42(50) = 21 psf

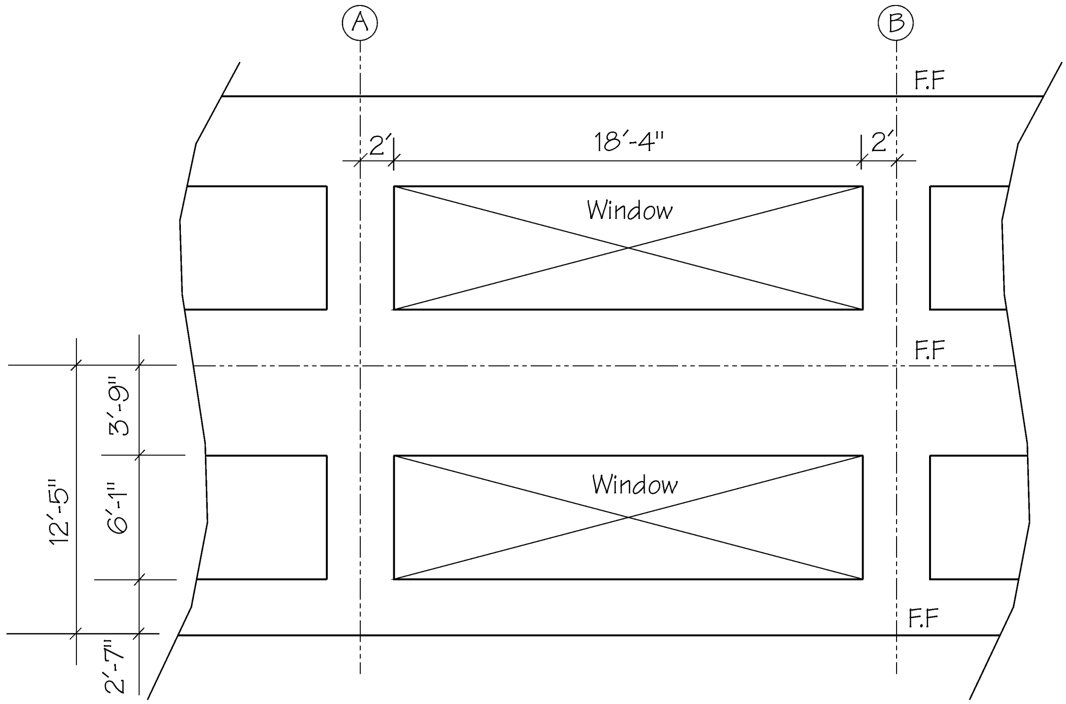


FIGURE 3-2

Step 3 – Design Earthquake Loads

From the governing building code for lateral force on elements of structures:

Nominal seismic force acting in any horizontal direction:

For exterior walls: $F_p = 0.288W_p$

For the body of connections: $F_p = 0.288W_p$

For fasteners in the connections: $F_p = 0.9W_p$

where W_p = the weight of an element or component

Step 4 – Alternative Design Approaches

Step 4(a) – Punched Window Approach (Design Alternative #1)

If each strip window is to be treated as a punched window, then the window head and sill member must be able to span horizontally between the window jambs spaced at 18'-4".

Check the ability of the window sill to span 18'-4".

Wind load tributary width:

= (6.08 + 2.58)/2 = 4.33'

Required moment:

M_{req} = [4.33(30)(18.33)^2/8][12/1000]
= 65.5 in.kips

Required inertia:

$$\delta = \frac{5wL^4}{384EI} = \frac{5(4.33)(50)(0.42)(220)^4}{12(384)(29.5)(10^6)I}$$

$$= \frac{7.83}{I} \text{ in.}$$

Substituting δ = L/360 = 220/360 in. and solving for I gives:

I_{req} = 7.83(360)/220 = 12.8 in^4

For the built-up window sill illustrated in Figure 3-3, try 600S162-68 (50) stud and 600T125-68 (50) track. *(Track with F_y = 50 ksi may require a special order.)*

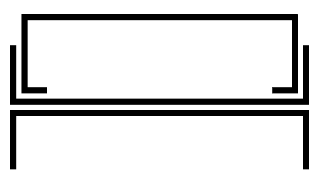


FIGURE 3-3

From load tables *(see Note 1-4 for an alternative approach to built-up member analysis)*:

M_{all} = 35.7 + 2(25.7)
= 87.1 in.kips> 65.5 in.kips ***OK***

$I_{x(def)}$ = 3.52 + 2(2.93)
= 9.38 in^4 < 12.8 in^4 ***UNSATISFACTORY***

Sill deflections are excessive even for a built-up section made with thick members. Therefore, the punched window approach with cold-formed steel framing is not practical.

Step 4(b) – Cold-Formed Steel Framing Mullions (Design Alternative #2)

The span length of window head and sill members can be reduced with the introduction of cold-formed steel framing mullions as shown in Figure 3-4.

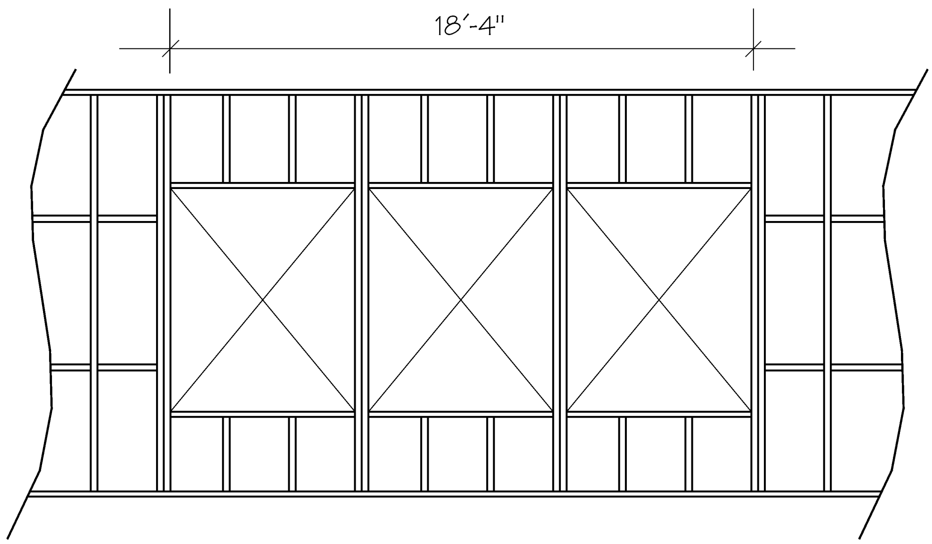


FIGURE 3-4

This alternative is not usually acceptable architecturally since the visual effect of the strip window is compromised.

For design purposes, the strip window has been reduced to a series of punched windows which are similar to the infill Design Examples #1 and #2.

Step 4(c) – Cantilevering Head and Sill (Design Alternative #3)

This detail as illustrated in Figure 3-5 is generally not acceptable. As was discussed under cantilevering parapets *(see Note 2-10, Item 2)*, anchoring conventional track to the structure will create neither a strong nor a stiff moment connection.

This detail will work if anchor plates are cast into the floor slabs at regular intervals and cantilevering hot-rolled angles, channels or hollow structural sections are welded in place. The cold-formed steel framing members are designed as infill around the hot-rolled cantilevers. This is similar to the approach illustrated in Design Example #2, Step 8(a).

Note that with this type of detail, the differential slab deflections will be accommodated within the aluminum extrusions for the window. This requirement should be specified on the project contract documents.

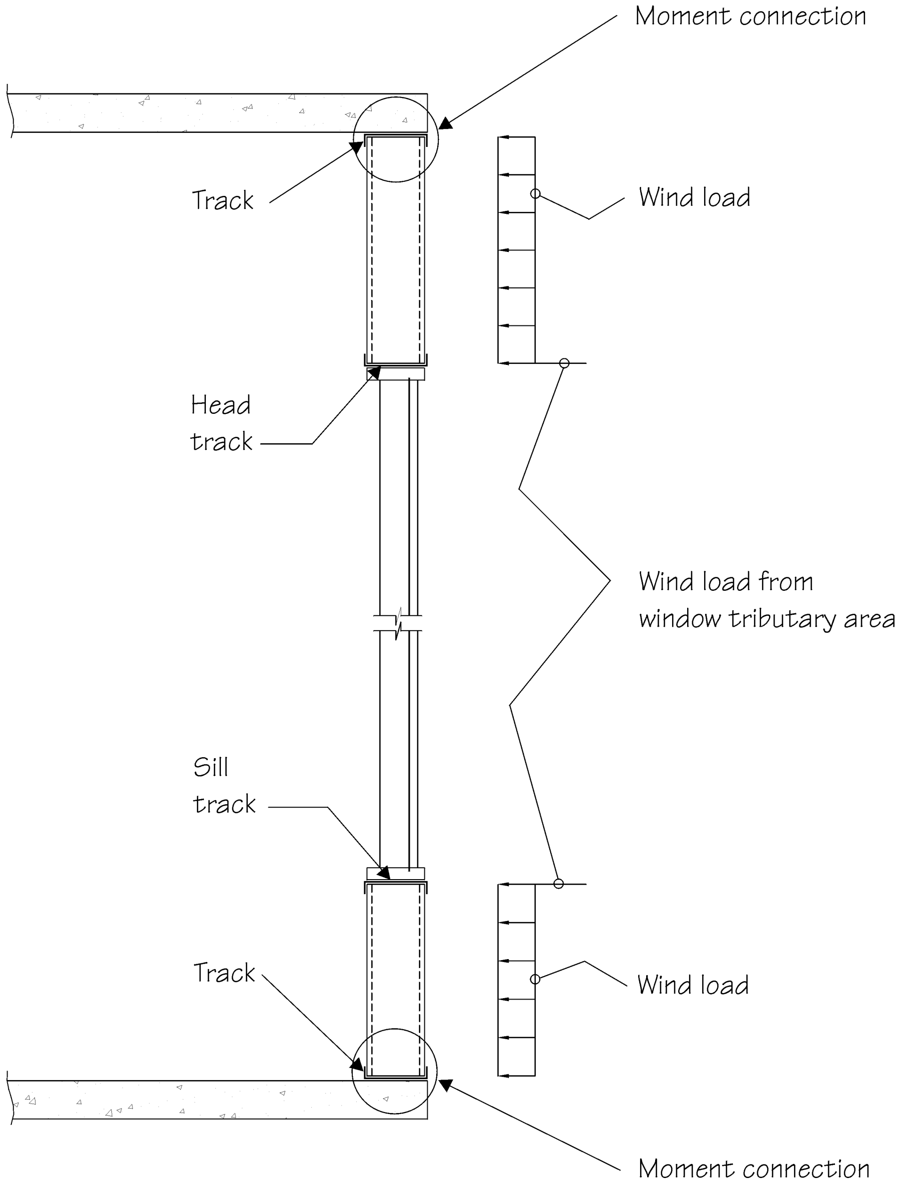


FIGURE 3-5

Step 4(d) – Studs Outside the Face of the Structure (Design Alternative #4)

See Figure 3-6. This approach is generally the most satisfactory way of framing strip windows and it will be used in the following design example.

As for Step 4(c), Design Alternative #3, the differential slab deflections will be accommodated within the aluminum extrusions for the window. This requirement should be specified on the project contract documents. For purposes of the design example, assume a structural steel building with deck reinforced slab floors.

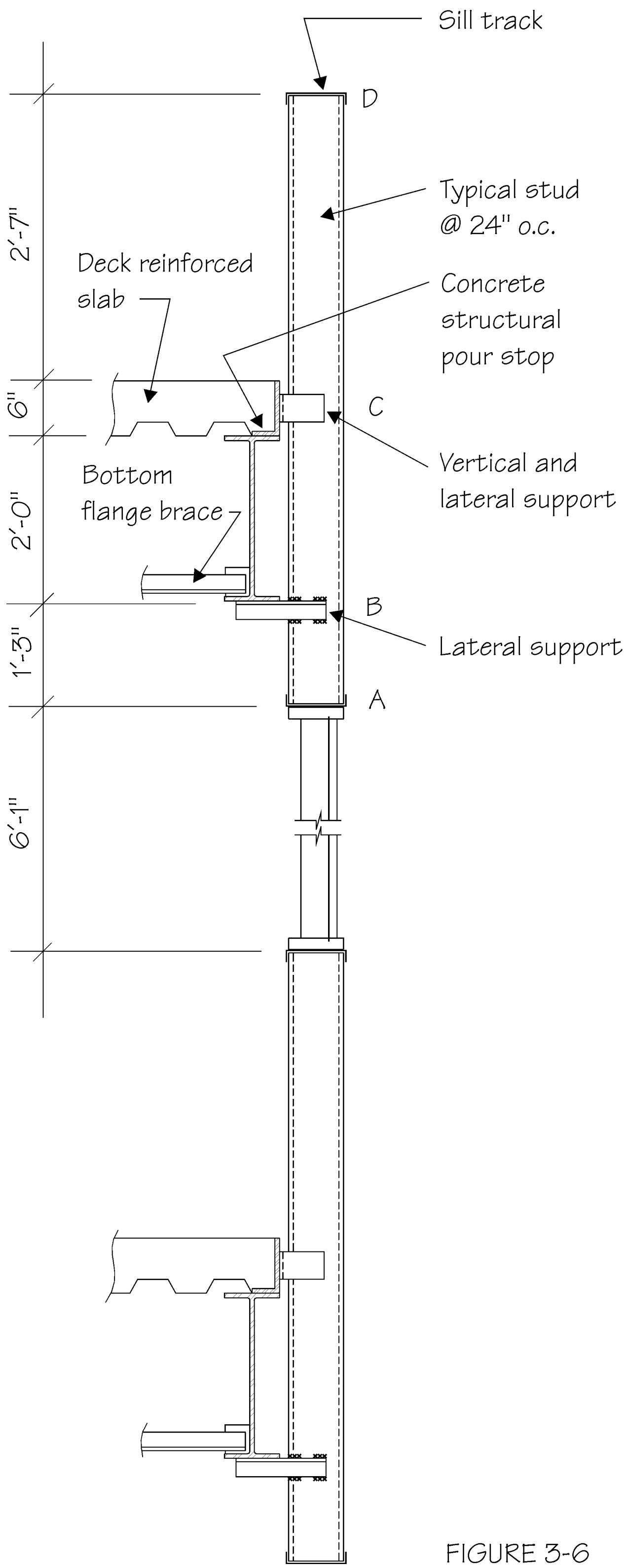

Sill track
D
Typical stud
@ 24" o.c.
2'-7"
Deck reinforced
slab
Concrete
structural
pour stop
6"
C
Vertical and
lateral support
Bottom
flange brace
2'-0"
B
Lateral support
1'-3"
A
6'-1"

FIGURE 3-6

Step 5 – Typical Stud Design

Step 5(a) – Wind Loading

The wind load diagram on the typical stud is shown in Figure 3-7. Dimensions are to center lines of supports at B and C assuming a 2" × 2" angle at B.

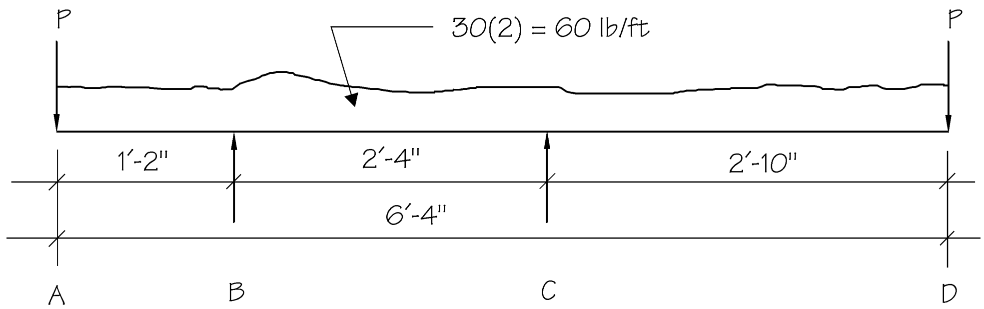


FIGURE 3-7

> *Note 3-1*
>
> *Coordination with the hot-rolled steel design is required to ensure that adequate bottom flange braces are provided to pick up the stud reaction at B.*

P = (6.08/2)(30)(2) = 182 lb per stud

From moments about B:
$2.33R_C + 1.17(182) + 60(1.17)^2/2 - 60(5.17)^2/2 - 5.17(182) = 0$

R_C = 639 lb.

R_B = 6.33(60) + 2(182) - 639 = 105 lb.

Maximum moment at C:

$$M_{req} = [60(2.83)^2/2 + 182(2.83)](12/1000)$$
$$= 9.06 \text{ in.kips}$$

Maximum shear to the right of C:

$$V_{req} = 182 + 60(2.83)$$
$$= 352 \text{ lb.}$$

Maximum deflection at D by computer analysis with all spans loaded as in Figure 3-7 except:

w = 60(0.70) = 42 lb/ft

gives

δ_c = 0.1520/ I in. at the end of the cantilever

Use $2L_c$ for cantilever deflection limit *(See Note 2-12):*

$\delta_{allowable}$ = (2)(34)/360 = 0.189"

I_{req} = 0.1520/0.189 = 0.804 in^4

Try 600S162-43 (33) stud:

M_{all} = 14.46 in.kips (distortional) > 9.06 in.kips ***OK***

V_{all} = 1.24 kips (punched) > 0.352 kips ***OK***

$I_{x(def)}$ = 2.32 in^4 > 0.804 in^4 ***OK***

Combined bending and shear:

By *AISI S100* Section C3.3.1

$$\sqrt{\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2} = \sqrt{\left(\frac{M}{M_{all}}\right)^2 + \left(\frac{V}{V_{all}}\right)^2}$$

where:
$M = M_{req}$ at C = 9.06 in.kips
$V = V_{req}$ at C = 0.352 kips
M_{all} = 16.68 in.kips V_{all} = 1.24 kips (use punched shear)

Substituting:

$$\sqrt{\left(\frac{9.06}{16.68}\right)^2 + \left(\frac{0.352}{1.24}\right)^2} \le 1.00$$

$0.61 \le 1.00$ ***OK***

> *Note 3-2*
>
> *AISI S100, Section C3.1.1 uses the yield moment, M_{nxo}/Ω (often called $M_{a\text{-}l}$ in manufacturers' literature) for combined bending and shear. Distortional buckling allowable moment is not considered for this interaction.*
>
> *No web crippling check is required here because the connection transfers shear directly from the web of the stud. For an example of a combined bending and web crippling calculation, see Example #2, Step 8(b).*

Check lateral instability:

OK by inspection. See Design Example #2, Step 1(b) for procedure to follow.

> *Note 3-3*
>
> *Based on the above design checks, a 600S162-33 (33) stud would be a possible alternative selection. The 600S162-43 stud, with a design thickness of 0.0451", has been chosen to facilitate welding. See Design Example #2, Step 6 for further discussion.*

Step 5(b) – Earthquake Loading

Wind loads are applied normal to the wall surface while earthquake loads can act in any horizontal direction.

Earthquake loads acting in the plane of the wall, including earthquake forces on the windows, are transferred to the wall through the connectors. These forces can be distributed by the weak axis strength of the studs, the shear diaphragm strength of the finishes, or flat strap cross-bracing when present.

To be consistent with the all-steel design approach used for this example, the earthquake forces are assumed to be distributed by the weak axis bending strength and stiffness of the studs.

> *Note 3-4*
>
> *When the diaphragm stiffness of the sheathings substantially exceeds the weak axis bending stiffness of the studs, and if the sheathings and their connectors lack the necessary diaphragm strength and ductility, then the studs will only be mobilized once the sheathings are damaged. To avoid sheathing damage, provide adequate diaphragm strength or add flat strap cross bracing.*

Assume moments and reactions due to earthquake can be found by proportioning the wind load effects.

From ASCE/SEI 7-10 ASD, the governing earthquake load combination is D + 0.7E.

Earthquake load acting on exterior walls with $F_p = 0.288W_p$
$= 0.7(0.288)W_p = 0.7(0.288)(16)$
$= 3.23$ psf

Wind load = 30 psf (Factored for ASD load combinations)

By proportioning the wind load moment at C, the applied weak axis earthquake moment is given by:

$M_{req} = (3.23/30)(9.06) = 0.975$ in.kips

From load tables, choose the lesser of the weak axis moment with the lips or the web in compression.

$M_{all} = 2.13 > 0.975$ in.kips ***OK***

A 600S162-43 stud has adequate strength for in-plane earthquake forces without relying on the shear diaphragm strength of the finishes or additional cross-bracing.

Therefore, for both wind and earthquake loadings, use a 600S162-43 (33) stud at 24" o.c.

Step 6 – Typical Track

It is industry standard practice to match track thickness and stud thickness unless there is a structural requirement for heavier track. In this example, 600T125-43 (33) would satisfy both industry standard practice and structural requirements. However, in the interest of providing additional resistance to construction abuse, the heavier 600T125-54 (50) has been selected instead. Some window systems also have minimum track thickness requirements for attaching the windows. This should be coordinated with the window contractor.

Step 7 – Typical Stud Connections

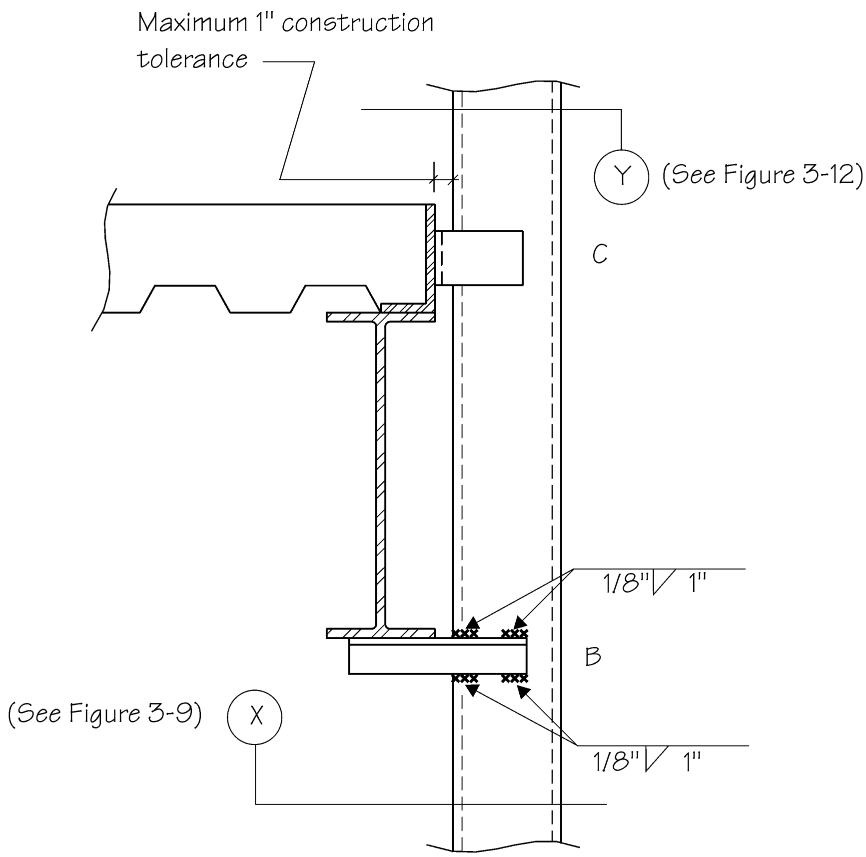


FIGURE 3-8

Step 7(a) – Lateral Support Connection at B Under Wind Load – Body of the Connector

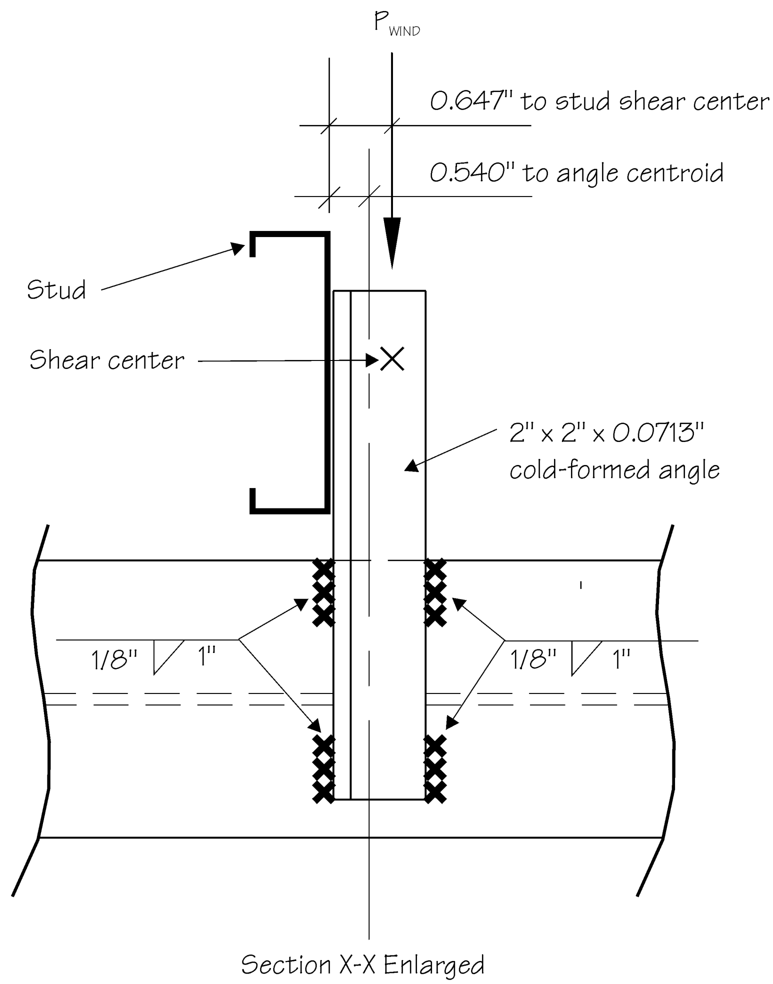


FIGURE 3-9

Note 3-5

1. *This connection resists the lateral wind reaction and is assumed to act as a torsional restraint for the stud. A rigorous analysis of this connection detail would be quite complex but this has been avoided here through the use of a number of conservative and simplifying assumptions.*

2. *Section properties for the 2" × 2" x 0.0713" are not available in the product literature and can be obtained from the formulas in AISI D100 Part I or by interpolation as done here.*

Try 2" × 2" × 0.0713" cold-formed angle.

For properties, linearly interpolate between properties for 2" × 2" × 0.060" and 0.075" angles as provided in AISI D100 Part I Table I-7. *(Linear interpolation is acceptable for the properties shown here but may not be for other properties.)*

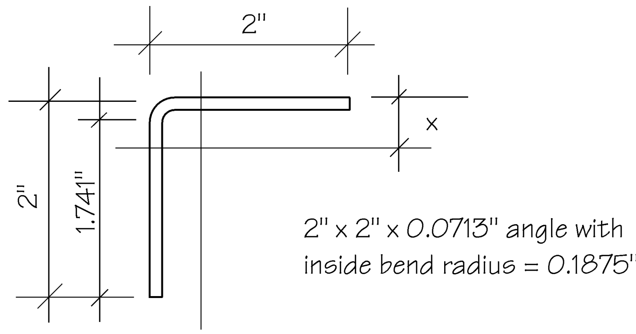


FIGURE 3-10

Angle	Area (in²)	S_x (in³)	x = y (in.)
2LU2×075	0.287	0.0796	0.541
2LU2×060	0.231	0.0642	0.535
2LU2×0713 (by interpolation)	0.273	0.0758	0.540

A = Area = 0.273 in²
x = distance to centroid = 0.540 in.
S_x = section modulus about horizontal axis = 0.0758 in³

and the stud shear center to the back of the stud is given by:
m - t/2 = 0.670 - 0.0451/2 = 0.647"

The loading on the angle is based on the following:

- The connection between the angle and the stud transfers the torsional restraint component *Rm - Kawm (discussed in Appendix I)*. For the design of the angle, it is conservative to neglect this torsional component.
- The stud shear is transferred from the web of the stud to the angle with an eccentricity about the vertical axis of the angle.
- The axial load in the angle is transferred to the bottom flange of the hot-rolled beam with an eccentricity about the horizontal axis of the angle.
- Conservatively assume the axial load in the angle is applied with an eccentricity about both axes on both ends.
- Use the simplified analysis method proposed in Appendix H for fully effective behavior.

Use the simplified analysis method proposed in Appendix H. For fully effective behavior, compressive stresses are limited to:

$$f = \frac{5190}{(w/t)^2}$$

$$= \frac{5190}{(1.741/0.0713)^2} = 8.70 \text{ ksi}$$

By inspection, overall stability effects can be neglected. Check interaction for strength only by AISI S100 Section C5.2.1:

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \leq 1$$

where:

P = required wind load at B
= 105 lb.

$M_x = M_y = Pe = 105(0.540)$
= 56.7 in.lb

$P_{no} = Af = (0.273)(8700) = 2375$ lb.

$M_{nx} = M_{ny} = Sf = 0.0758(8700) = 659$ in.lb

$\Omega_b = 1.67$
$\Omega_c = 1.80$

Substituting:

$$\frac{1.80(105)}{2375} + \frac{1.67\,(56.7)}{659} + \frac{1.67\,(56.7)}{659}$$

$= 0.37 < 1.00$ ***OK***

Step 7(b) – Later al Support Connection at B Under Earthquake Load – Body of the Connector

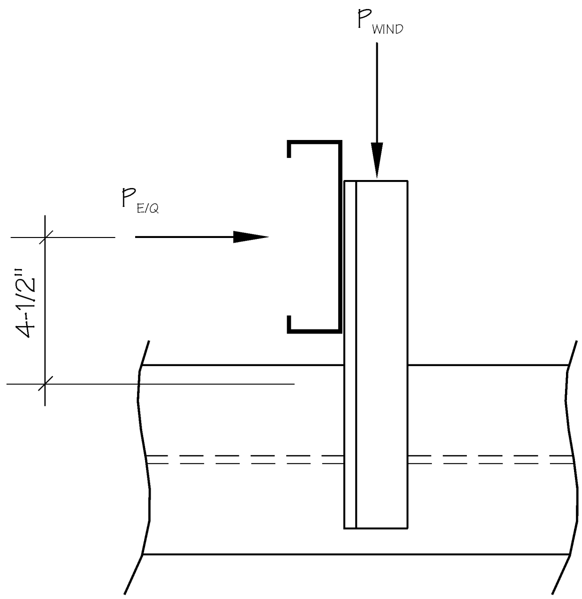


FIGURE 3-11

Assume the earthquake reaction at B can be found by proportioning the wind load effect.

For D + 0.7E load case and $F_p = 0.288W_p$:

E/Q load = $(0.7)(0.288)W_p = (0.7)(0.288)(16) = 3.23$ psf

Wind Load = 30 psf

$P_{WIND} = R_B = 105$ lb *(Step 5a)*

$P_{E/Q} = 105(3.23/30)$
$= 11.3$ lb.

$P_{E/Q}$ induces a moment in the angle with a lever arm of, say, 4-1/2" to the center line of the 1" weld *(Figure 3-11)*.

$M_{req} = 4.5(11.3) = 50.9$ in.lb

Using the limiting stress from Step 7(a) for fully effective behavior:

$M_{all} = Sf/\Omega_b$
$= 0.0758(8700)/1.67$
$= 395$ in.lb > 50.9 in.lb **OK**

Therefore, a 2" × 2" × 0.0713" angle is adequate for both wind and earthquake loading.

Note 3-6

The 2" × 2" × 0.0713" angle has a large strength reserve. This reserve is useful for miscellaneous parts since they can be engineered with a minimum effort without too much concern for precise eccentricities and loads. Also, the welding position for these parts is often awkward and the thicker 0.0713" material is less susceptible to welding damage.

Step 7(c) – Lateral Support at B Under Wind or Earthquake Load – Welds at Either End

The welds have substantial strength reserve and are ***OK*** by inspection.

Step 7(d) – Vertical and Lateral Support at C Under Dead and Wind Load – Body of the Connector

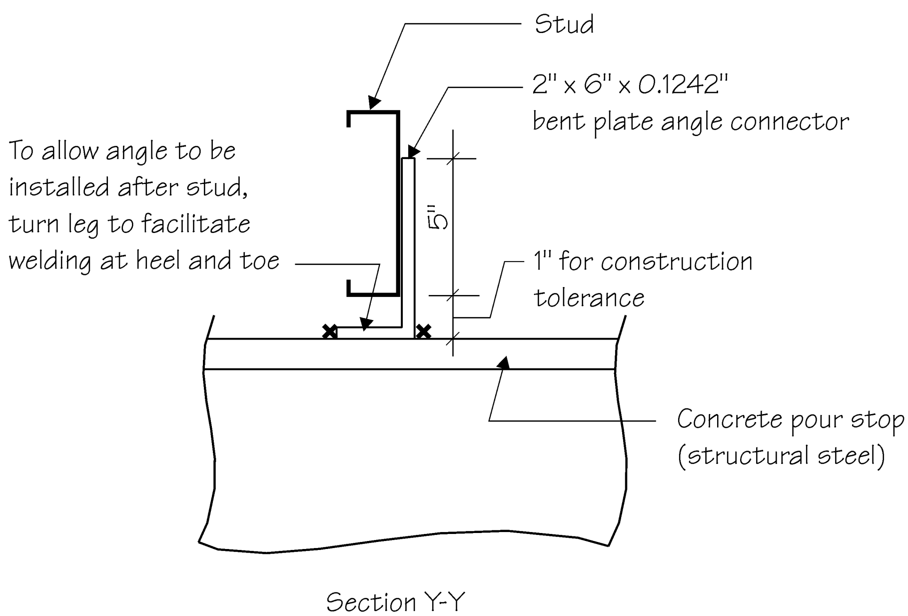


FIGURE 3-12

$P_{WIND} = R_c = 639$ lb

P_{DL} = (stud spacing)(W_D)($H_{FLR/FLR}$)
= (2)(16)(12.42)
= 397 lb

See Figure 3-13.

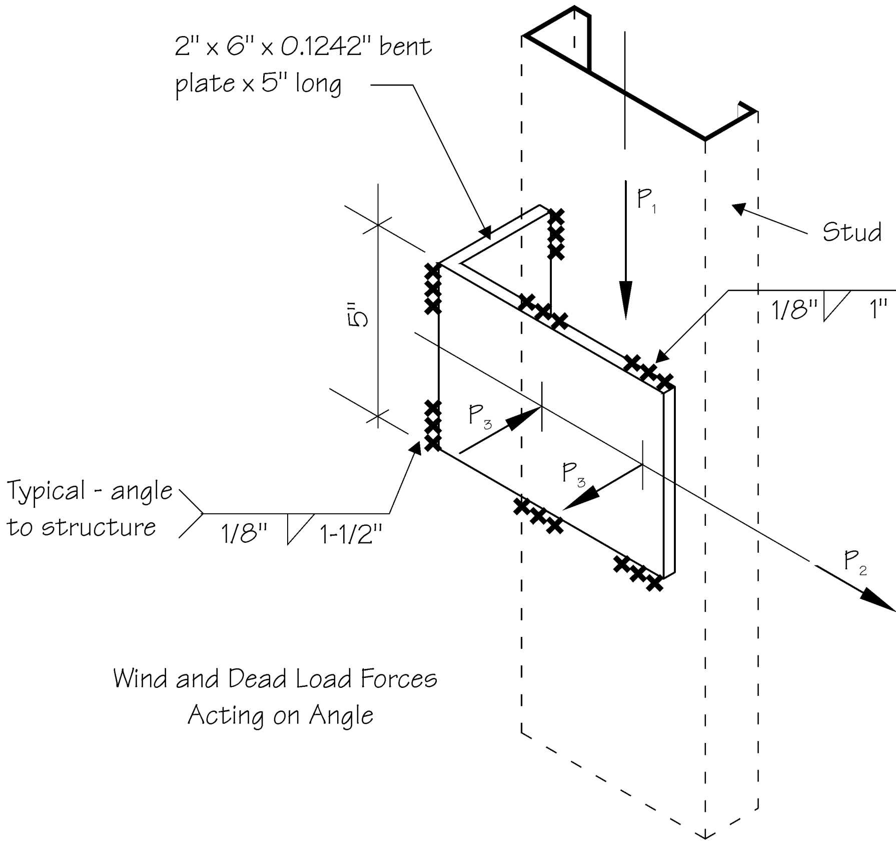


FIGURE 3-13

$P_1 = P_{DL} = 397$ lb

$P_2 = P_{WIND} = 639$ lb

The couple, P_3, is the torsional restraint component at C.

The required torsional restraint is given by *Rm - Kawm* from Appendix I. *(Unlike the previous design examples, the component Kawm is included here.)*

gives

$$P_3 = \frac{Rm - Kawm}{d}$$

where:

d = 4" for center-to-center spacing of 1" long welds when studs are 1" from face of structure

m = centerline of web to shear center of stud = 0.670 in.

$R = R_c = P_2 = 639$ lb.

$K = 0.67$ *(Appendix I)*

a = average distance to adjacent torsional braces
= (2.83 + 2.33)/2 = 2.58 ft.

w = 2(30) = 60 lb/ft

Substituting:

$$P_3 = \frac{639(0.670) - 0.67(2.58)(60)(0.670)}{4}$$

= 89.7 lb.

Calculate thickness of angle, t:

$A = 5t$
$S_x = (1/4)t(5)^2 = 4.17t$
$Z_y = (1/4)(5)t^2 = 1.25t^2$ *(The plastic section modulus is used here)*

$M_x = P_1(4) = 397(4)$
= 1588 in.lb

$M_y = P_3(4) = 89.7(4)$
= 359 in.lb

$T = P_2$
= 639 lb.

Combined stresses – stability effects are negligible. *(The following stress check is equivalent to checking AISI S100 Section C5.1.1 for strength only.)*

$$\frac{M_X}{S_X} + \frac{M_Y}{Z_Y} + \frac{T}{A} = \frac{F_y}{1.67}$$

For $F_y = 50$ ksi

$$\frac{1588}{4.17t} + \frac{359}{1.25t^2} + \frac{639}{5t} = \frac{50000}{1.67}$$

Gives:
$29940t^2 - 509t - 287 = 0$

Solving the quadratic gives:

t = 0.107 in.

Use the next standard cold-formed steel framing thickness:
t = 0.1242" *(design thickness for 10 ga.)*

Step 7(e) – Vertical and Lateral Support at C Under Dead and Earthquake Load – Body of the Connector

The distribution of earthquake forces to the reaction points B and C in Figure 3-6 is similar (i.e., proportional) to the distribution of wind forces. The earthquake reaction can, therefore, be found by proportioning the wind load reaction. See Figure 3-14.

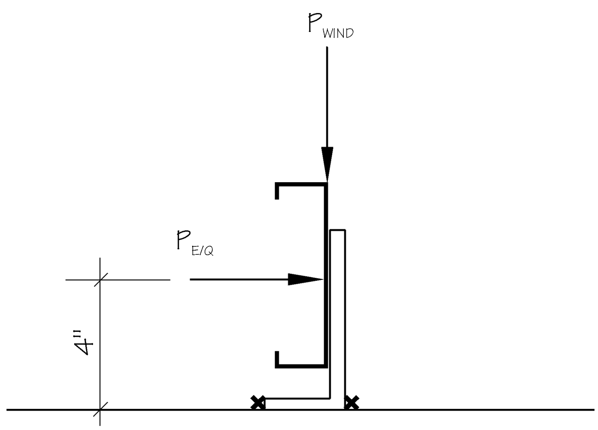


FIGURE 3-14

For the load case, D + 0.7E and $F_p = 0.288W_p$
E/Q load = 0.7(0.288)(16) = 3.23 psf

Wind load = 30 psf

$P_{WIND} = R_C$ = 639 lb (Step 5a)

$P_{E/Q}$ = 639(3.23/30) = 68.8 lb.

Due to dead load:
M_X = 1588 in.lb (Step 7d)

Due to earthquake:
$M_Y = (P_{E/Q})(4) = (68.8)(4)$
= 275 in.lb

Compared with the wind and dead load case, the earthquake moments are less severe and t = 0.1242" is ***OK*** by inspection.

Therefore, based on dead, wind and earthquake load cases, use t = 0.1242" *(10 ga)* bent plate 2" × 6" angle in 50 ksi steel × 5" long.

Step 7(f) – Vertical and Lateral Support at C Under Dead and Wind Load – Angle to Stud Welds

See Figure 3-13.

For the D + 0.6W load case:

Each weld is loaded by three orthogonal forces. The required forces from Step 7(d) are given by:

P_1 = 397 lb.
P_2 = 639 lb.
P_3 = 89.7 lb.

Assuming a uniform stress along the length of each weld *(see Note 3-7)*, the required resultant shear per weld is given by:

$$V_{req} = \sqrt{(P_1/4)^2 + (P_2/4)^2 + (P_3/2)^2}$$
$$= \sqrt{(397/4)^2 + (639/4)^2 + (89.7/2)^2}$$
$$= 193 \text{ lb/weld}$$

Note 3-7

The weld stresses resulting from the torsional restraint forces, P_3, have been treated in an approximate and possibly unconservative fashion here by assuming that the resulting weld stresses are uniform along the length of the weld. A more rigorous solution, consistent with the assumptions in Appendix G, would be:

- *Calculate the linear section properties for the weld group, S_{weld}*
- *Using the linear method, calculate the maximum required load per inch of weld length $q_{req} = M_{req} / S_{weld}$ (where $M_{req} = 4P_3$ in this example). Convert the other components to required loads per inch of weld length and find the resultant maximum, q_{req}, using the square root function.*
- *Compare the resultant q_{req} with the allowable weld capacity per inch of length as given by $q_{all} = P_n/(L\Omega) = 0.75\ tF_u/\Omega$*

The "exact" procedure has been demonstrated in Design Example #2 Step, 5(e) and Design Example #3, Steps 7(h) and 7(i). Given the strength reserve in this weld group, the extra design time for the "exact" procedure is not justified.

Step 7(g) – Vertical and Lateral Support at C Under Dead and Earthquake Load – Angle to Stud Welds

See Figures 3-13 and 3-14.

> *Note 3-8*
>
> *The earthquake load for weld design is higher than the earthquake load for the design of the body of the connector. See Step 3.*

For the D + 0.7E load case and $F_p = 0.900W_p$ *(See Note 3-8)*.

Earthquake Load = 0.7(0.900)(16) = 10.1 psf

Wind load = 30 psf

$P_{WIND} = R_c$ = 639 lb. (Step 5a)

Assume the earthquake reaction can be found by proportioning the wind load reaction at C.

$P_{E/Q}$ = 639(10.1/30) = 215 lb.

$P_1 = P_{DL}$ = 397 lb. (Step 7d)

Assuming $P_{E/Q}$ is distributed equally to all four welds, the required resultant shear per weld is given by:

$$V_{req} = \sqrt{(P_1/4)^2 + (P_{E/Q}/4)^2}$$
$$= \sqrt{(397/4)^2 + (215/4)^2}$$

= 113 lb/weld < 193 lb/weld due to dead + wind

Therefore, dead + wind from Step 7(f) controls with:

V_{req} = 193 lb/weld

The weld strength is governed by the steel properties for the stud:

$V_{all} = 0.75tLF_u/\Omega$
= 0.75(0.0451)(1)(45000)/3.05
= 499 lb/weld > 193 lb/weld **_OK_**

Therefore, use the angle to stud welds illustrated in Figure 3-13.

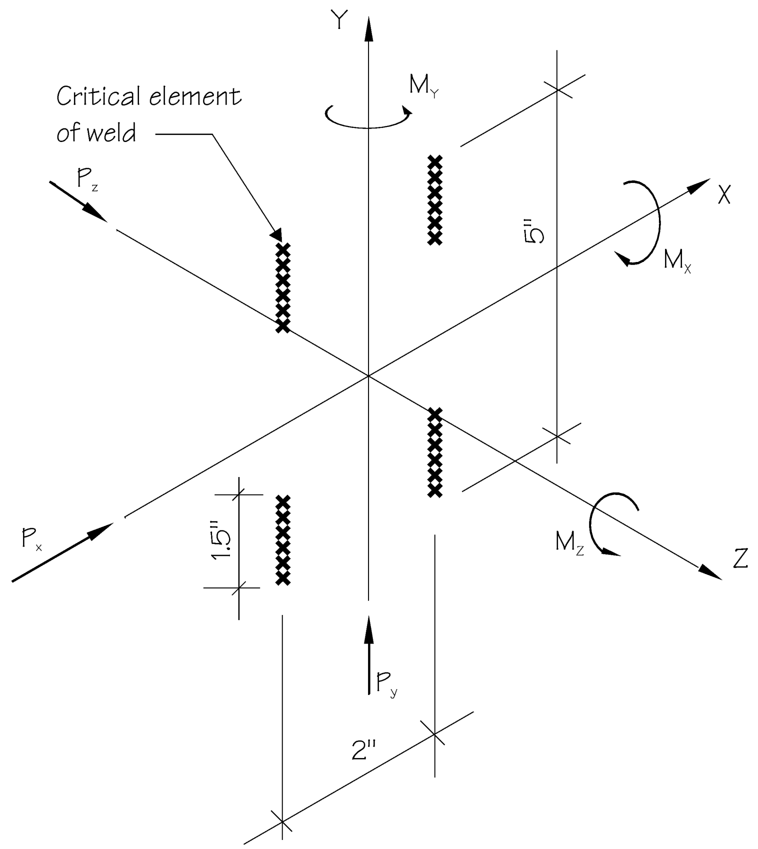
Step 7(h) – Vertical and Lateral Support at C Under Dead and Wind Load - Angle to Concrete Pour Stop Welds

The calculation of resultant shears acting on the weld group is somewhat complex and reference is made to the general method outlined in Appendix G.

Note 3-9

1. *The sign convention in Appendix G can be confusing. In the appendix and Figure 3-15, all forces, moments and stresses are shown in the positive direction. Coordinates x, y and z are positive or negative in the usual sense. While the Appendix G sign convention has been followed here, in some cases it may be simpler to just handle the sign of resulting stresses intuitively.*

2. *The critical element of weld has been selected where stresses are maximum and also where no weld stress relief is possible due to the direct bearing between parts.*



Weld Configuration with Axes Through the Centroid of the Weld Group

FIGURE 3-15

See Figures 3-13 and 3-15.

For the D + 0.6W load combination, the required forces are given in Step 7(d).

P_1 = 397 lb.
P_2 = 639 lb.
P_3 = 89.7 lb.

Try 1-1/2" long welds.

$P_X = 0$

$P_Y = -P_1 = -397$ lb.

$P_Z = P_2 = 639$ lb.

$M_X = 4P_1 = 4(397) = 1588$ in.lb

$M_Y = 4P_3 + 1P_2$
$= 4(89.7) + 1(639)$
$= 998$ in.lb

$M_Z = 1(P_1) = 1(397)$
$= 397$ in.lb

Linear properties with t=1:

$A = 4(1.5) = 6.0$ in.

$I_X = 4[(1/12)(1.5)^3 + 1.5(1.75)^2]$
$= 19.5\ in^3$

$I_Y = 4(1.5)(1)^2$
$= 6.0\ in^3$

$I_Z = I_X + I_Y = 25.5\ in^3$

For the critical element of weld in Figure 3-15 with due regard to signs *(see Note 3-9)*:

$q_x' = P_X/A = 0$

$q_y' = P_Y/A = -397/6.0 = -66.2$ lb/in

$q_z' = P_Z/A = 639/6.0 = 106.5$ lb/in

$q_x'' = M_Z y/I_Z = 397(2.5)/25.5 = 38.9$ lb/in

$q_y'' = M_Z x/I_Z = 397(-1)/25.5 = -15.6$ lb/in

$q_z'' = M_X y/I_X - M_Y x/I_Y = 1588(2.5)/19.5 - (998)(-1)/6.0 = 370$ lb/in

$q_x = q_x' - q_x'' = 0 - 38.9 = -38.9$ lb/in

$q_y = q_y' + q_y'' = -66.2 - 15.6 = -81.8$ lb/in

$q_z = q_z' + q_z'' = 106.5 + 370 = 476.5$ lb/in

$$q_{req} = \sqrt{q_x^2 + q_y^2 + q_z^2}$$
$$= \sqrt{(38.9)^2 + (81.8)^2 + (476.5)^2}$$
$$= 485\ \text{lb/in}$$

Step 7(i) – Vertical and Lateral Support at C Under Dead and Earthquake Load – Angle to Concrete Pour Stop Welds

See Figures 3-13, 3-14 and 3-15.

For the D + 0.7E load case and $F_p = 0.900W_p$:

From Step 7(g):
$P_{E/Q} = \pm 215$ lb.

From Step 7(d):
$P_{DL} = 397$ lb.

$P_X = 215$ lb. *(assuming PE/Q acts in the positive X direction)*

$P_Y = -P_1 = -397$ lb.

$P_Z = 0$

$M_X = 4P_1 = 4(397) = 1588$ in.lb

$M_Y = 4P_{E/Q} = 4(215)$
$= 860$ in.lb

$M_Z = 1(P_1) = 1(397)$
$= 397$ in.lb

Linear properties from Step 7(h):
A = 6.0 in.
$I_X = 19.5$ in^3
$I_Y = 6.0$ in^3
$I_Z = 25.5$ in^3

For the critical element of weld in Figure 3-15 with due regard to signs *(see Note 3-9)*:

$q_x' = P_X/A = 215/6.0 = 35.8$ lb/in

$q_y' = P_Y/A = -397/6.0 = -66.2$ lb/in

$q_z' = 0$

$q_x'' = M_Zy/I_Z = 397(2.5)/25.5 = 38.9$ lb/in

$q_y'' = M_Zx/I_Z = 397(-1)/25.5 = -15.6$ lb/in

$q_z'' = M_Xy/I_X - M_Yx/I_Y = 1588(2.5)/19.5 - 860(-1)/6.0 = 347$ lb/in

$q_x = q_x' - q_x'' = 35.8 - 38.9 = -3.10$ lb/in

$q_y = q_y' + q_y'' = -66.2 - 15.6 = -81.8$ lb/in

$q_z = q_z' + q_z'' = 0 + 347 = 347$ lb/in

$$\begin{aligned} q_{req} &= \sqrt{q_x^2 + q_y^2 + q_z^2} \\ &= \sqrt{(3.1)^2 + (81.8)^2 + (347)^2} \\ &= 357 \text{ lb/in} < 485 \text{ lb/in} \end{aligned}$$

Therefore, dead + wind from Step 7(h) controls:

$q_{req} = 485$ lb/in

The strength of welded joints with t > 0.10" is limited by the approximate method in Appendix A or shear through the effective throat of the weld itself.

Weld design data:

$t_{min} = 0.1242"$
$F_y = 50$ ksi for angle and 33 ksi for concrete pour stop
$F_u = 65$ ksi for angle and 45 ksi for concrete pour stop
Electrode $F_{xx} = 60$ ksi

Allowable strength for a nominal 1/8" fillet weld *(A fillet weld occurs at the toe of the connector angle and a flare-bevel groove weld at the heel.)*

From Appendix A:

Conservatively, use t = 0.1242" in combination with $F_u = 45$ ksi:

$$\begin{aligned} q_{all} &= P_n/(L\Omega) = 0.75tF_u/\Omega \\ &= 0.75(0.1242)(45)/3.05 \\ &= 1.37 \text{ kips/in} \end{aligned}$$

From *AISI S100* Section E2.5:

The effective throat of the fillet weld is limited to 0.707 times the thickness of the angle connector = 0.1242"

$$\begin{aligned} q_{all} &= P_n/(L\Omega) = 0.75t_wF_{xx}/\Omega \\ &= 0.75(0.707)(0.1242)(60)/2.55 \\ &= 1.55 \text{ kips/in} \end{aligned}$$

Use $q_{all} = 1.37$ kips/in > 0.485 kips/in ***OK***

Note that this q_{all} is also valid for the flare-bevel groove weld at the heel of the connector angle *(same effective throat if flare-bevel grove weld not filled flush to surface - see AISI S100 Section E2.6.)*

Therefore, use the angle to concrete pour stop welds illustrated in Figure 3-13.

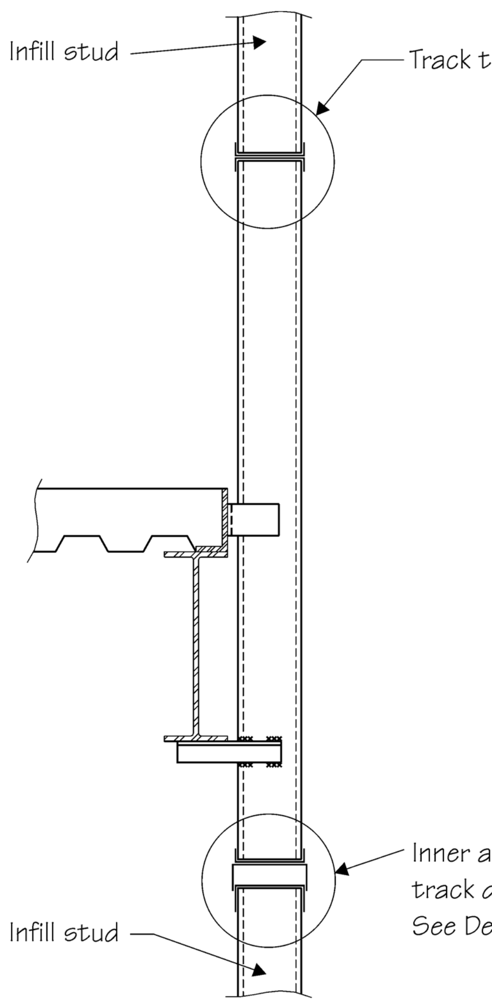
Step 8 – Stud Infill

The strip windows are interrupted periodically and replaced with a full-height stud wall. *(See elevation Figure 3-2.)* This full-height stud wall can be achieved by continuing the strip window details *(Figure 3-6),* but adding stud infill to replace the window. A deflection gap detail such as the inner and outer top track may be required at the top of the stud infill. This is illustrated in Figure 3-16.

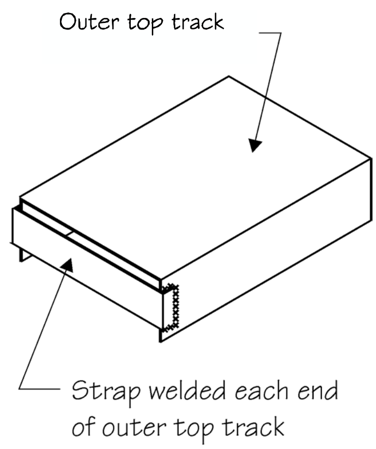
The deflection gap detail will not be required if all of the following conditions are satisfied:

- The stud infill is located at column lines where little or no relative slab deflection occurs.
- The accumulative effect of column axial shortening is not significant.
- Thermal expansion and contraction is not expected to be significant.
- Seismic drift is not accommodated via a sliding joint at this location.

Where the deflection gap detail is used and seismic drift is not accommodated with a sliding joint, adding welded retainer straps (per Figure 3-16, Detail A) at each end of the outer top track may be required to provide racking resistance for the infill studs. For wider segments of infill, the sheathing may provide adequate racking resistance as a cantilevered shearwall element and retainer straps may not be required.



Section Between Strip Windows



Detail A - Enlarged

FIGURE 3-16

Step 9 – Alternative Detail for Shop-Applied Finishes

An alternative deflection gap detail for the stud infill with shop-applied exterior insulation and finish system is shown in Figure 3-17.

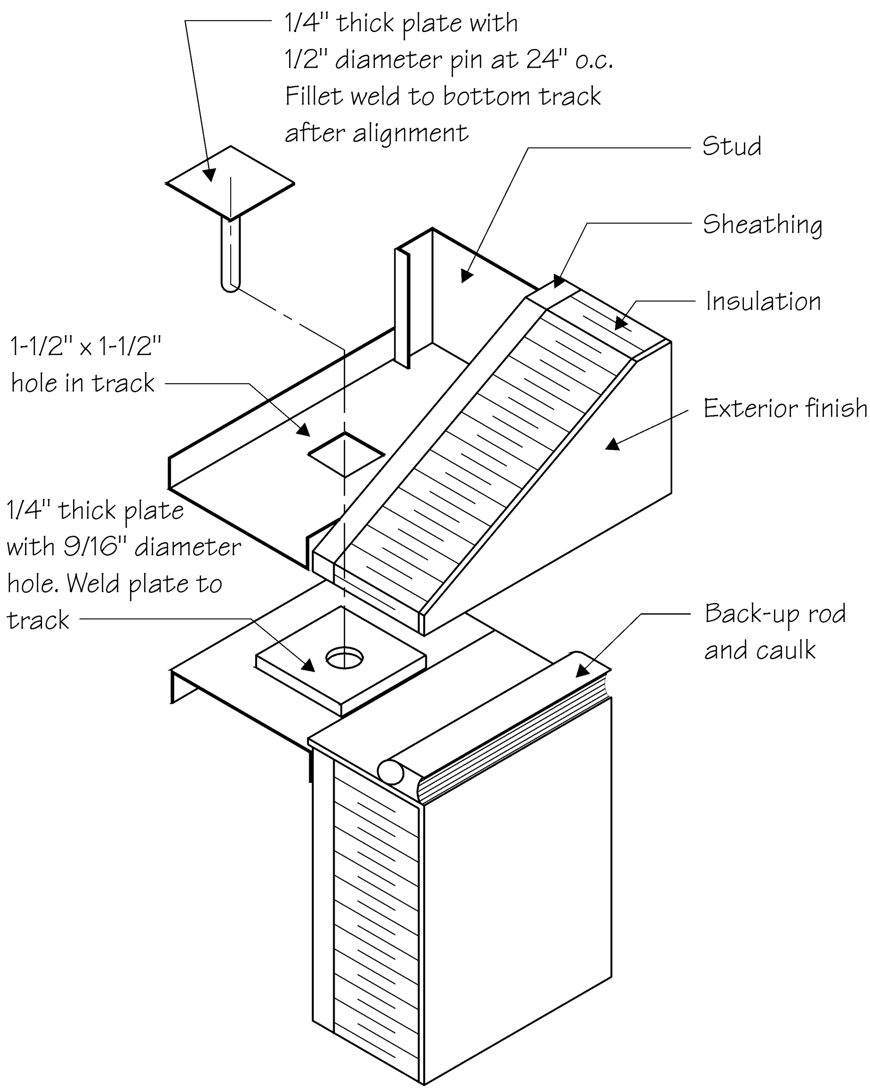


FIGURE 3-17

DESIGN EXAMPLE #4
COLD-FORMED STEEL FRAMING FLOOR AND AXIAL LOAD BEARING STUD WALL

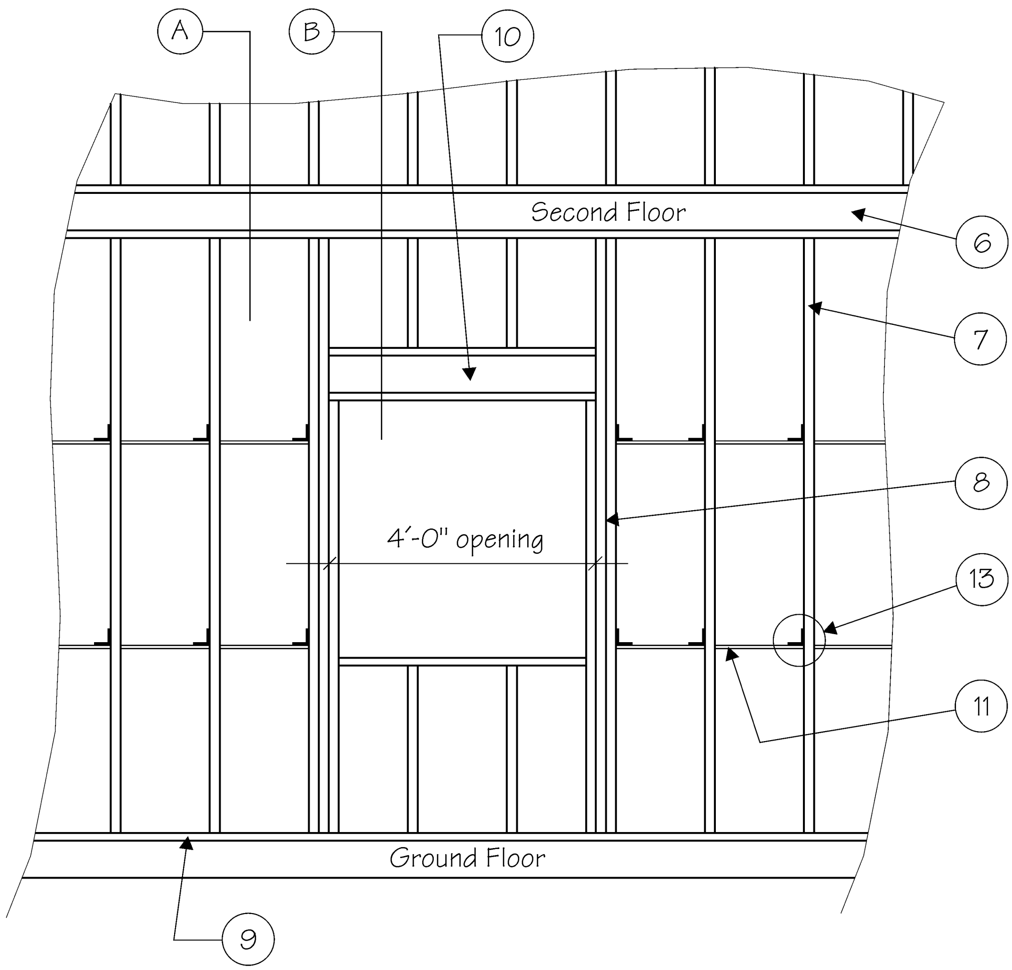


FIGURE 4-1

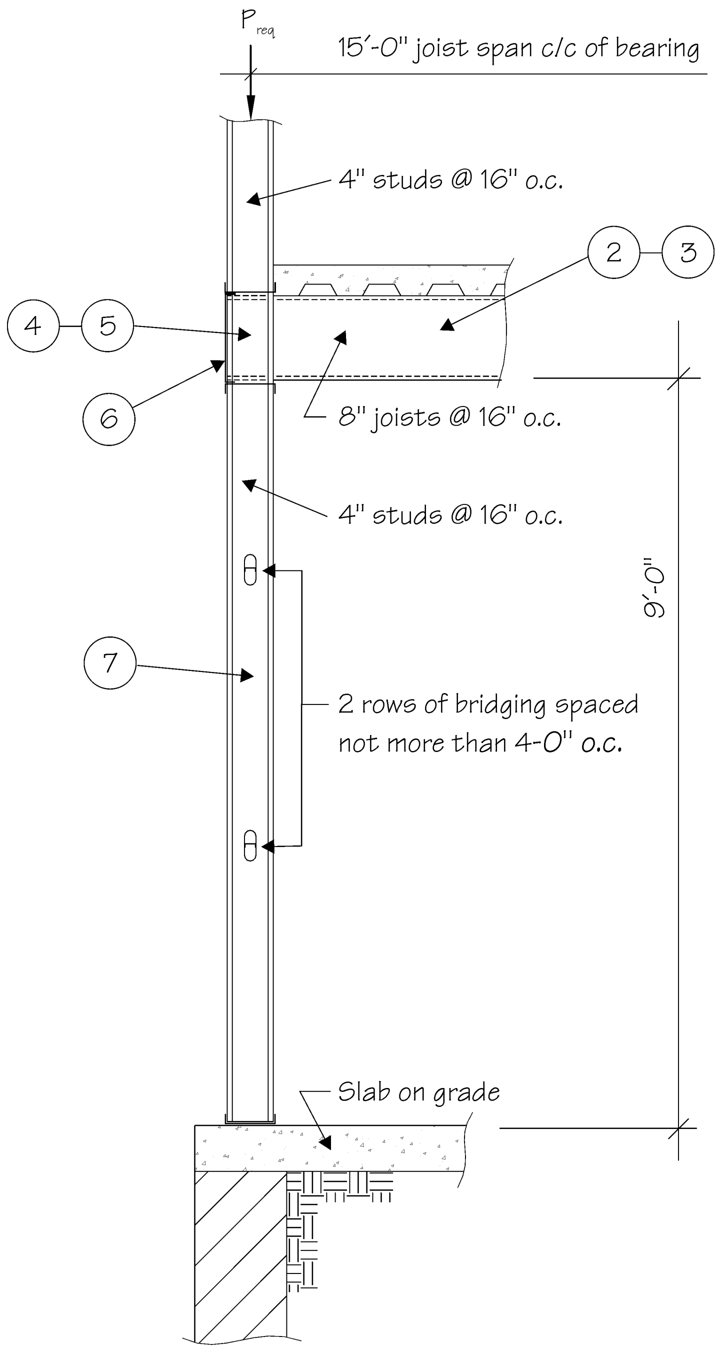
Introduction

This example covers the design of a cold-formed steel framing floor system bearing on a steel stud wall with a window opening. Detailed calculations are included for all elements, including the stud bridging and its anchorage.

The section numbers for the design of individual components are identified in Figures 4-1 and 4-2. Refer also to Step 12, Bridging Anchorage.

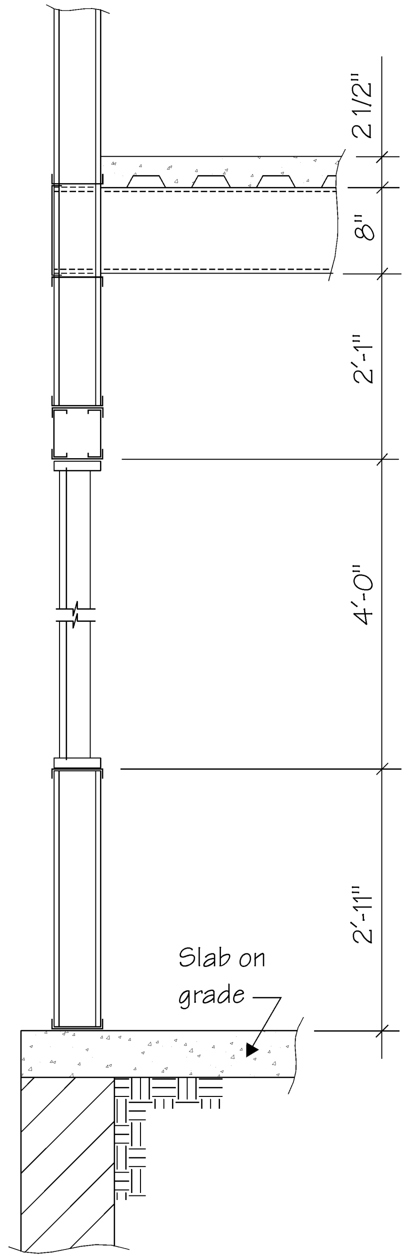
Step 1 – Given

- Design wind load, 0.6W = 25 psf based on Main Wind Force-Resisting System (MWFRS) wind loads as required by AISI S240 Section B1.1.2. The 0.6W indicates that the load has been factored for ASD load combinations.
- Floor design live load = 40 psf
- Floor partition allowance = 0 psf
- Wall loads from above:
  P_{LL} = 1.33 kips
  P_{DL} = 0.67 kips
  (No snow, rain or roof live load in this example)
- Wall deflection limit = L/360
- Floor deflection limit = L/360 for live load and L/240 for total load
- Vibration criteria = none
- Screwed connections
- Platform construction
- Required fire rating = none
- Lateral stability for the building as a whole will be provided by reinforced concrete elevator shaft and stairwells.
- Depth of stud to meet architectural requirements = 4"



Section A

FIGURE 4-2


2 1/2"
8"
2'-1"
4'-0"
2'-11"
Slab on grade
Section B

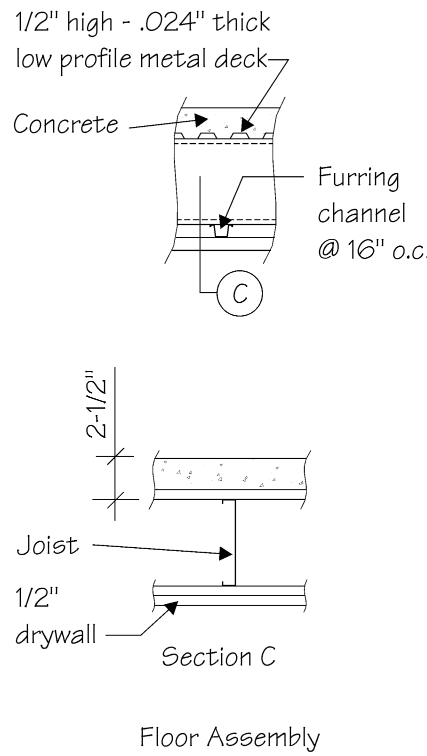

1/2" high - .024" thick low profile metal deck
Concrete
Furring channel @ 16" o.c.
C
2-1/2"
Joist
1/2" drywall
Section C
Floor Assembly

FIGURE 4-3

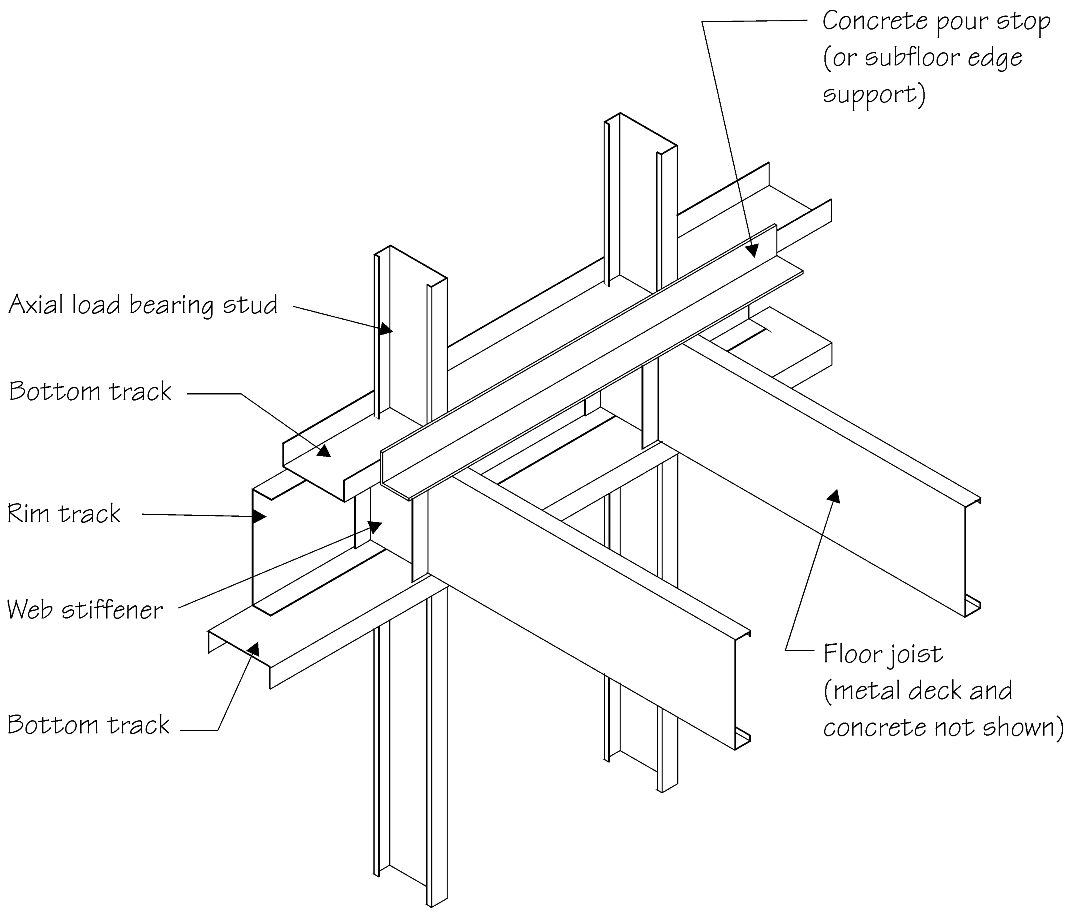


FIGURE 4-4

Step 2 – Floor Joist Selection

Live load		40 psf
Dead load		
2-1/2" concrete	30.2 psf	
0.024" metal deck	1.3	
800S200-54 joists @ 16" o.c.	1.8	
Furring + drywall	2.1	
Floor finish and misc.	4.0	39.4 psf
Total Load		79.4 psf

Span length L = 15'-0" single span c/c of bearing *(Figure 4-2)*

Deflection limit = L/360 for live load and L/240 for total load

Vibration criteria = none

Try 800S200-54 (50) joist @ 16" o.c.

From load tables for 40 psf live load and 40 psf dead load:

Allowable span length = 15'-3" > 15'-0" ***OK***

Step 3 – Floor Joist Bridging

Floor joist selection has been based on the assumption that the concrete deck and the ceiling below provide adequate torsional restraint for loads not applied through the shear center and for lateral instability. In addition to this restraint, it is standard practice in the industry to supply a minimum amount of bridging to align members during erection and to provide structural integrity during construction as well as in the completed structure. See AISI S240 Section B2.5 for minimum blocking-in requirements. Appropriate details are shown in Figure 4-5.

A maximum bridging spacing of 8'-0" o.c. is commonly used in this situation.
With one line of mid-span bracing, spacing = 15/2 = 7'-6" < 8'-0" ***OK***

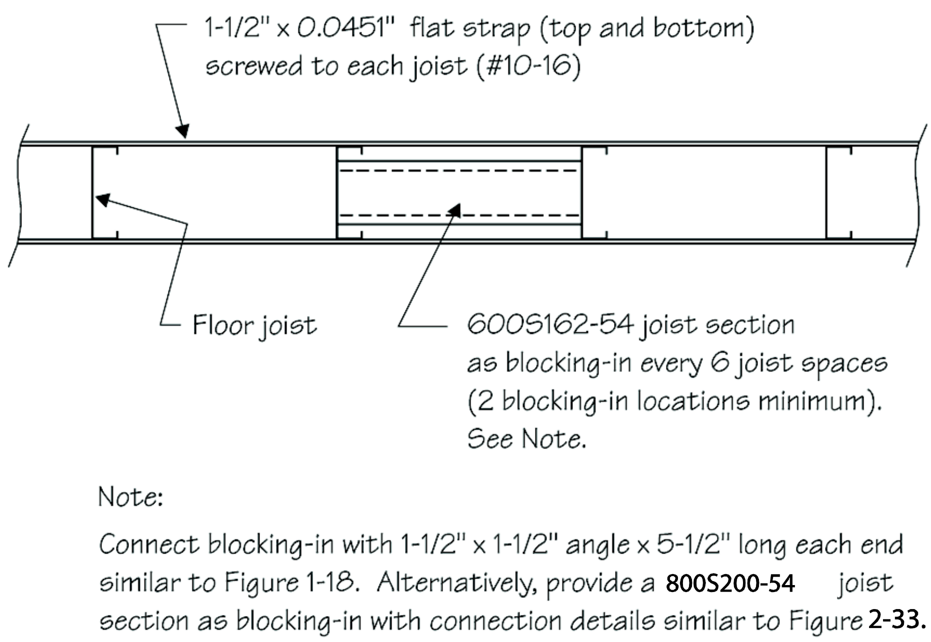


FIGURE 4-5

Step 4 – Floor Joist Web Stiffener

Floor joists typically require web stiffeners to resist the joist end reactions and to transfer the axial load from the studs above. These web stiffeners are designed in accordance with the requirements of *AISI S100* Section C3.7. A two-flange loading case is used for both the joist end reaction and the stud load above.

In the absence of a structural load distribution member at the floor level, in-line framing is required to provide load transfer through the floor assembly to the studs below. The cold-formed steel framing industry considers framing aligned when the center lines of the studs above, the floor joists and the studs below all line up vertically. This alignment is illustrated in Figure 4-4. Tolerances on in-line framing are provided in AISI S240 Section B1.2.3.

The stiffeners can be either inside or outside the joist. Figure 4-4 shows stiffeners outside. Note that the definition of in-line framing does not change with the stiffener location, but the allowable tolerances as defined in AISI S240 do. Tighter tolerances are required for the case of stiffeners outside.

AISI S240 Section B2.5.1 provides an alternative design method using clip angle bearing stiffeners.

Stiffeners outside the joist can be full-height, whereas stiffeners inside must be cut short to fit. *AISI S100* Section C3.7 specifies that the length of stiffeners shall not be less than the outside depth of the joist minus 3/8". Other requirements also apply – see AISI S100 Section C3.7.

For stiffener details on this project, see Figures 4-4 and 4-6.

Governing load combination:

D + L

P_{LL} = stud load above + floor joist reaction
= 1.33 + (15/2)(16/12)(40)/1000
= 1.73 kips/stud

P_{DL} = stud load above + floor joist reaction
= 0.67 + (15/2)(16/12)(39.4)/1000
= 1.06 kips/stud

$P_{req} = P_{LL} + P_{DL} = 1.73 + 1.06$
= 2.79 kips

Check web crippling capacity of stiffened joist.

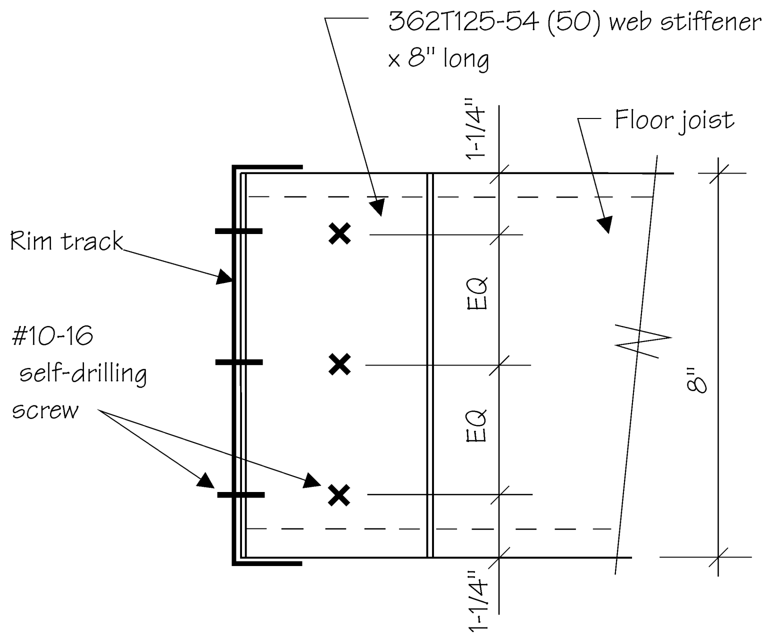


FIGURE 4-6

By *AISI S100* Section C3.7.2:

$P_n = 0.7(P_{wc} + A_eF_y) \geq P_{wc}$

where:

$P_{wc} = P_n$ = nominal web crippling for unstiffened joist by *AISI S100* Eq. C3.4.1-1 with bearing length = stud depth = 4". Use web crippling coefficients for fastened to support two-flange loading *(AISI S100 Table C3.4.1-2.)*

$$P_n = C t^2 F_y \sin\theta \left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right)$$

where:

R	= 0.0849"
t	= 0.0566"
Depth	= 8"
h	= Depth - 2t - 2R = 7.717"
N	= 4"
F_y	= 50 ksi
θ	= 90 degrees
C	= 7.5
C_R	= 0.08
C_N	= 0.12
C_h	= 0.048

Substituting:

$P_{wc} = P_n = 0.957$ kips

A_eF_y = stub column strength of stiffener

For 362T125-54 (50) stiffener, the term A_eF_y is available in AISI D100 Table III-3: $A_eF_y = 11.2$ kips

$P_n = 0.7(0.957 + 11.2)$
$= 8.51$ kips

$P_{all} = P_n / \Omega = 8.51/1.70$
$= 5.01 \text{ kips} > 2.79 \text{ kips}$ ***OK***

Note 4-1

As an alternative detail to Figure 4-4, the concrete floor finish could be carried to the outside face of the studs with the bottom track of the wall above bearing on the concrete. With this alternative, care is required to ensure that the voids in the concrete created by the corrugations in the floor deck do not create a weak link in the transmission of axial load through the floor system. There is the additional disadvantage that the erection of steel above cannot proceed until the concrete has been poured and at least partially cured. However, this approach is beneficial in that the alignment of the framing may not be so critically important.

Step 5 – Joist to Web Stiffener Connection

The connection of the stiffener to the joist is described in AISI S100 Section C3.7.2. A minimum of three fasteners are required and spaced such that the distance from the joist flanges to the first fasteners shall not be less than the depth of the joist/8.

Thus, depth/8 = 8/8 = 1"
Use 1-1/4" – see Figure 4-6.

Note that Section C3.7.2 does not prescribe any forces that the fasteners are required to resist. In this design example, any end torsional effects are assumed to be resisted by the attached sheathings. However, significant torsional resistance is also provided by the connection of the joist to the stiffener, the stiffener to rim track, and the top and bottom flange of the joist to the track (not shown).

Step 6 – Rim Track

Step 6(a) – Section Size

Use 800T125-54 (50) rim track *(thickness to match thickness of floor joists)*. See Figure 4-4 and Note 4-2.

> *Note 4-2*
>
> *It is common practice to supply rim track with narrow flanges (1-1/4" in this example). This type of detail implies that the axial loads in the wall studs above and below the rim track are applied eccentrically through the outside flange of the studs. However, the rim track narrow flange detail might be beneficial in that the end rotation of the floor joist is less likely to transmit an end moment into the wall studs below.*
>
> *In any case, appropriate design end eccentricities for this connection detail have not been researched and engineering judgement is required. Currently, it is common practice in the cold-formed steel framing industry to design the studs in Figure 4-4 as concentrically loaded. The weakening effect (if any) of this end eccentricity is assumed to be offset by conservative assumptions for end fixity. (These end fixity assumptions are reviewed in Step 7.)*

Step 6(b) – Screws

Provide nominal screw connection to match the stiffener to joist detail. See Figure 4-6.

Step 7 – Typical Stud

The following design approach is recommended for axial load bearing steel studs. Refer also to the discussion on bracing in Section 4.2.2 of the Introduction.

1. Use an all-steel (i.e. unsheathed) design approach with steel bridging at regular intervals to resist the torsional component of the load and the tendency for the studs to buckle laterally. The bridging will require periodic anchorage to the primary structure.

2. Conservatively assume $K_x = K_y = K_t = 1.0$. This assumption is common in published load tables and is consistent with AISI S240 Section B3.2.1.1.

3. Published load tables usually assume concentric axial loads and it is common practice to use this assumption in design.

Try 400S162-54 (50) stud and bridging spaced at 4'-0" o.c. maximum. The load tables used for stud selection in this *guide* are based on the assumption that the 4'-0" bridging spacing can be located anywhere along the length of the stud. See Note 4-3.

Note 4-3

When using combined axial and lateral load tables, care is required to ensure that the basic assumptions used to derive the allowable loads are understood.

1. *In the past, the output in load tables typically included a 0.75 load combination factor such that the designer only needed nominal loads to use the tables. The effect of this approach was to provide an automatic check on two load cases, L + D and 0.75(D + L + W). However, this approach is not consistent with the many different load combinations required by current standards such as ASCE/SEI 7-10 (ASCE 2010) and, as a result, newer tables typically do not have embedded load factors (except for Item 2 that follows). The designer is now required to apply the load combination factors before entering the tables and this approach is demonstrated here in the guide.*

2. *For checking wind load deflections, 0.7W (Tables are typically based on ASD level wind pressures so the 0.7 is consistent. If ASCE/SEI 7-10 wind loads are used unfactored, the wind loads are multiplied by 0.42 for deflection) may be embedded in the tables. This approach is assumed for the design examples provided in this guide.*

3. *There are two different assumptions in common use regarding bridging spacing. One assumption (which is used here) allows a maximum bridging spacing of, say, 4'-0" to occur anywhere over the length of the stud. For calculating allowable axial loads, the unsupported length (4'-0") is assumed to be in the worst possible location (typically in the middle). An alternative approach is to specify a maximum bridging spacing of again, say, 4'-0" o.c., but to also require that the bridging be equally spaced. For the 9'-0" stud length used here, the first assumption results in a bridging spacing of 4'-0" o.c. and the second 3'-0" o.c. This difference will have a significant impact on the allowable axial load capacity of the wall studs.*

For this example:

L_r (roof LL) = 0
R (rain load) = 0
S (snow load) = 0

From ASCE 2010, the remaining load combinations are (for strength):

D + L
D + 0.6W
D + 0.75L + 0.75(0.6W)

And for deflection:

0.7(0.6W) = 0.42W

Step 7(a) – Check Web Crippling

Web crippling can be checked from wind bearing allowable height tables *(if web crippling is flagged)* or from the allowable web crippling capacities typically published in load tables:

P_{req} = 25(16/12)(9/2) = 150 lb.

From load tables with 1" of bearing length and the end one flange fastened condtion:

P_{all} = 628 lb. > 150 lb. ***OK***

See also Step 9, 5th bullet for further discussion on the transfer of end shear in axial load bearing studs.

Step 7(b) – Check Deflection

A deflection check based on a load of 0.7W (0.42W where nominal wind load is per ASCE/SEI 7-10) is typically built into axial load bearing stud tables. For the allowable axial loads listed under 25 psf ASD wind load, the deflection check is actually done at 0.7(25) = 17.5 psf and subscripts in the load tables indicate that an L/360 deflection limit does not control.

Step 7(c) – Axial Load Capacity

Loads from the stud above plus the floor joist reaction *(from Step 4):*
P_{LL} = 1.73 kips/stud
P_{DL} = 1.06 kips/stud.

<u>D + L load case:</u>

W = 0
P_{req} = P_{LL} + P_{DL} = 2.79 kips

From load tables for 400S162-54 (50) stud at 16" o.c. and 0 psf wind *(Conservativly use 5 psf wind if 0 psf is not available):*

P_{all} = 4.63 kips > 2.79 kips ***OK***

<u>D + 0.6W load case:</u>

0.6W = 25 psf
P_{req} = P_{DL} = 1.06 kips/stud

From load tables for 400S162-54 (50) stud at 16" o.c. and 25 psf wind:

P_{all} = 3.06 kips > 1.06 kips ***OK***

D + 0.75(0.6W + L) load case

W = 0.75(25)
= 18.75 psf

$P_{req} = P_{DL} + 0.75P_{LL} = 1.06 + 0.75(1.73)$
= 2.36 kips

From load tables for 400S162-54 (50) stud at 16" o.c. and the next highest wind = 20 psf

P_{all} = 3.34 kips > 2.36 kips ***OK***

(From a software solution for 0.75(25) = 18.75 psf, P_{all} = 3.41 kips)

Use 400S162-54 (50).

Step 8 – Jamb Studs

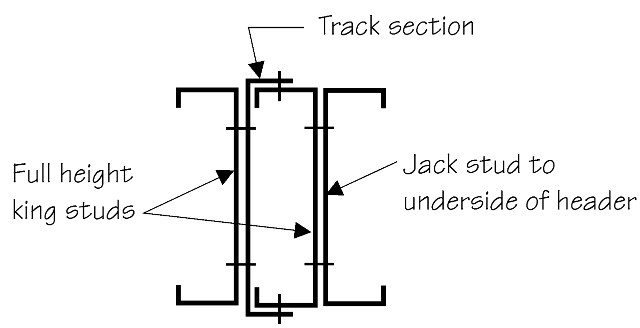


FIGURE 4-7

For this example, a built-up section consisting of 2 - 400S162-54 (50) king studs and 1-400S162-54 (50) jack stud is adequate for the jamb (by inspection). See Figure 4-7. This built-up section provides two studs (one jack and one king stud) to resist gravity loads and two full-height king studs for wind loads. Thus, each of the jamb studs will have the same tributary loading area as the typical studs for both gravity and wind loads. The track section is used as a connection device and its flexural strength is ignored. Note that the track section is cut short at the top and bottom tracks and is not available to participate in resisting axial loads.

The studs should be connected together to form a built-up section to resist wind load. A #10-16 screw spacing of 16" o.c. is recommended.

For axial load, the capacity of the built-up section can significantly exceed the sum of the capacity of the individual studs. However, the capacity of the jamb is adequate in this example when treated as individual studs. Any uncertainties such as the eccentrically applied gravity load from the header to the king stud can be accommodated within the strength reserve inherent in the partial built-up behavior. *(Note: AISI S100 addresses full built-up behavior in Sections D1.1 and D1.2. The fastening requirements in D1.1 and D1.2 are quite onerous and are usually seen as uneconomical compared with the simpler approach of summing the capacity of the individual studs in the jamb. AISI S240 Section B1.3 relaxes the interconnection requirements for two studs back-to-back with certain end conditions).*

Step 9 – Track Selection

The following design approach is recommended for the selection of track:

- In load bearing construction, it is recommended that the thickness of track be equal to or greater than the thickness of the stud.

- Axial loads are transferred in bearing between the end of the stud and the web of the track. The stud-to-track screws are not designed to transfer any axial load.

- The bearing stresses between the track and concrete should be checked using the approximate design expression proposed in Appendix F.

- Track should not be used as a beam to spread gravity loads at floor levels where studs or joists above do not align with studs below. Where misalignment is expected, a section with higher bending strength such as a hot-rolled angle or hollow structural section is required. With concrete floors, a concrete haunch is sometimes used which completely encloses the cold-formed steel floor members over each load bearing wall. See also the discussion on in-line framing in Step 4. CFSEI Tech Note W104-10 also provides guidance on top track load distribution.

- As for wind bearing studs, shear between the stud and the track is transferred by the stud bearing against the upstanding leg of the track except that there is the additional benefit of friction due to end bearing. Refer to Design Examples #1 and #2 for the design methodology for the track and the stud-to-track connection to resist wind loads.

- Stud-to-track connections should be pre-loaded before screwing in order to eliminate the bend radius gap. See Figure 4-8 that follows. Pre-loading deforms the track locally to allow the stud to seat. A maximum gap between the end of the stud and the track (after pre-loading) of 1/8" is permitted by AISI S240 Section C3.4.3.

- Track may also be subjected to axial tension and compression as a result of system lateral loads. Where axial loads are incurred, the track sections including splices between track sections must be designed accordingly.

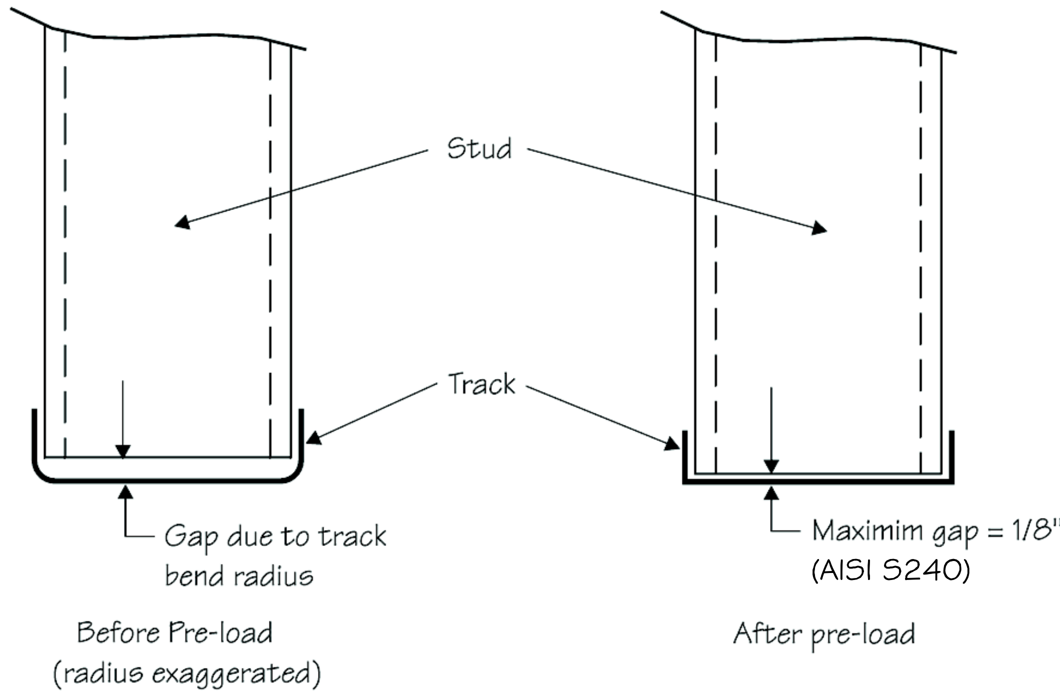


FIGURE 4-8

Check concrete bearing under the bottom track using the approximate method in Appendix F.

Try 400T125-54 (50) track *(Track with F_y = 50 ksi may require a special order.)*

From Step 4:
P_{LL} = 1.73 kips/stud
P_{DL} = 1.06 kips/stud.

For L + D load case:

$P_{req} = P_{LL} + P_{DL} = 2.79$ kips

From Appendix F, assuming concrete f_c' = 3 ksi:

$$x = 0.938 t_t \sqrt{\frac{F_y}{f_c'}} = 0.938(0.0566)\sqrt{\frac{50}{3}}$$
$$= 0.217"$$

$A_{brg} = (B + 2x)(C + x)(2) + [A - 2(C + x)][t_s + 2x]$

where:
$A = 4"$
$B = 1.625"$
$C = 0.500"$
$t_s = 0.0566"$
$x = 0.217"$

Substituting:
$A_{brg} = 4.21 \text{ in}^2$

$P_{all} = A_{brg} 0.34 f_c'$
$= 4.21(0.34)(3)$
$= 4.29$ kips > 2.79 kips and 400T125-54 (50) track ***OK***

Step 10 – Header

A box header detail is proposed – see Figure 4-10. AISI S240 includes special provisions for the design of this member, but these provisions have not been used here.

- The upturned track on top of the header means that AISI S240 Sections B3.3.1.3 and B3.3.2.3 do not apply. *(See Commentary on AISI S240.)*

Given the above, the header design here is based on the requirements of *AISI S100* instead of AISI S240. For certain header configurations, the provisions of AISI S240 provide design advantages, particularly with regard to web crippling.

The header load condition is shown in Figure 4-9.

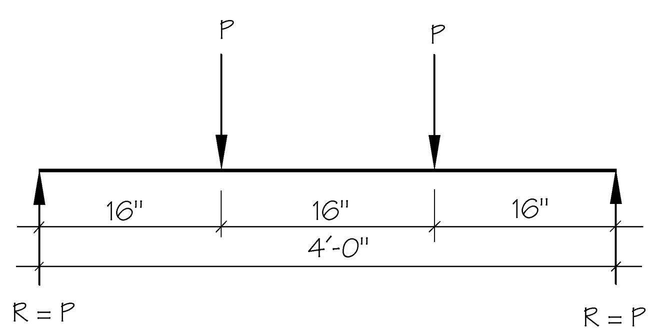


FIGURE 4-9

From Step 4:
P_{LL} = 1.73 kips/stud
P_{DL} = 1.06 kips/stud.

For the D + L load case:
$P = P_{DL} + P_{LL}$ = 2.79 kips

Try 2 - 800S162-68 (50) unperforated joist sections with 2 - 400T125-54 (50) track sections. The proposed built-up header configuration is shown in Figure 4-10.

Design the joist sections to carry gravity loads and the track to carry wind loads. Refer to Design Examples #1 and #2 for wind loaded track design methodology. Note that the top track is also assumed to provide resistance to lateral buckling such that the full moment capacity of the joist sections is available to resist the gravity loads.

The joist sections should be specified as unpunched as discussed in Step 10(b). Note, however, that unpunched moment and shear values may not be available in the load tables and in this case, punched values may be used as a conservative substitution. Punched values are used here.

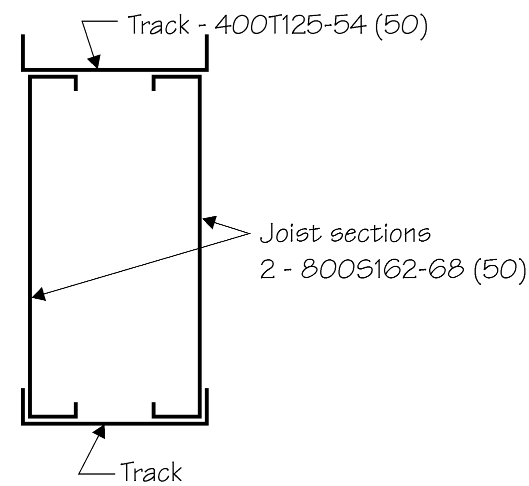


FIGURE 4-10

From the load tables for 2 - 800S162-68 (50):

M_{all} = 2(45.1) = 90.2 in.kips (distortional buckling controls)
V_{all} = 2(3.37) = 6.74 kips
$I_{x(def)} = 2(7.07) = 14.14\ in^4$
m = 0.586"

Step 10(a) – Moment Capacity (Gravity Loads)

M_{req} = 16P = 16(2.79)
= 44.6 in.kips < 90.2 in.kips ***OK***

Step 10(b) – Interior Web Crippling (Gravity Loads)

Derive the allowable web crippling at the location of load P for interior one flange condition. Assume an unfastened condition and bearing length equal to the flange width of the load bearing stud above = 1.625". From *AISI S100* Section C3.4 and Table C3.4.1-2:

$$P_{all} = \frac{C t^2 F_y \sin\theta}{\Omega}\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right)$$

where:

R = 0.10695"
t = 0.0713"
Depth = 8"
h = Depth - 2t - 2R = 7.6435"
N = 1.625"
F_y = 50 ksi
θ = 90 degrees
C = 13
C_R = 0.23
C_N = 0.14
C_h = 0.01
Ω = 1.65

substituting for 2 sections:

P_{all} = 2(2.15)
= 4.30 kips > 2.79 kips ***OK***

For this allowable web crippling capacity to be valid, web punchouts are not permitted in the vicinity of the point loads. The header member has therefore been specified as unperforated.

Step 10(c) – Combined Web Crippling and Bending (Gravity Loads)

Check combined bending and web crippling at the location of load P *(AISI S100 Section C3.5.1):*

$$0.91\left(\frac{P}{P_n}\right) + \left(\frac{M}{M_{nxo}}\right) \le \frac{1.33}{\Omega}$$

where:
$P = P_{req} = 2.79$ kips
$M = M_{req} = 44.6$ in.kips
$P_n = \Omega_w P_{all} = 1.65(4.30) = 7.10$ kips
$M_{nxo} = \Omega_b M_{a\text{-}l} = 1.67(2)(49.8) = 166.3$ in.kips
$\Omega = 1.70$

Substituting:

$$0.91\left(\frac{2.79}{7.10}\right)+\left(\frac{44.6}{166.3}\right)\le\frac{1.33}{1.70}$$

$0.626 \le 0.782$ ***OK***

Note that in the above equation, M_{nxo} is the nominal moment capacity for yield moment per AISI S100 Section C3.1.1. Distortional buckling allowable moment need not be considered in this interaction.

Step 10(d) – Combined Bending and Shear (Gravity Loads)

Combined bending and shear:

By *AISI S100* Section C3.3.1:

$$\sqrt{\left(\frac{\Omega_b M}{M_{nxo}}\right)^2+\left(\frac{\Omega_v V}{V_n}\right)^2}=\sqrt{\left(\frac{M}{M_{a-l}}\right)^2+\left(\frac{V}{V_{all}}\right)^2}$$

where for 2 sections:
$M = M_{req} = 44.6$ in.kips
$V = P_{req} = 2.79$ kips
$M_{a\text{-}l} = 99.6$ in.kips (local buckling allowable moment)
$V_{all} = 6.74$ kips

Substituting:

$$\sqrt{\left(\frac{44.6}{99.6}\right)^2+\left(\frac{2.79}{6.74}\right)^2}\le 1.00$$

$0.61 \le 1.00$ ***OK***

Step 10(e) – Deflection (Gravity Loads)

See Figure 4-9.

Check: LL for L/360
TL for L/240

$$\delta = \frac{Pa}{24EI}\left(3L^2 - 4a^2\right)$$

where:
$P = P_{LL} = 1.73$ kips
$P = P_{TL} = P_{LL} + P_{DL} = 1.73 + 1.06 = 2.79$ kips
$E = 29500$ ksi
$L = 48"$
$a = 16"$

LL Check:

$$\delta_{LL} = \frac{1.73(16)}{24(29500)I}\left[3(48)^2 - 4(16)^2\right]$$
$$= \frac{0.2302}{I} \text{ in.}$$

for $\delta_{all} = L/360 = 48/360 = 0.1333"$

$I_{req} = 0.2302/0.1333$
$= 1.73 \text{ in}^4$

TL Check:

$$\delta_{TL} = \frac{2.79(16)}{24(29500)I}\left[3(48)^2 - 4(16)^2\right]$$
$$= \frac{0.3712}{I} \text{ in.}$$

for $\delta_{all} = L/240 = 48/240 = 0.200"$

$I_{req} = 0.3712/0.200$
$= 1.86 \text{ in}^4$

Total load governs:

For 2 - 800S162-68 (50)
$I_{x(def)} = 14.14 \text{ in}^4 > 1.86 \text{ in}^4$ ***OK***

Step 10(f) – Track to Joist Connection (Gravity Loads)

The box header track to joist connection is required to provide torsional restraint at the locations of load P and at the supports for the header. See Figure 4-11.

Torsional restraint forces, P_L, by *AISI S100* Section D3.2.1. See also Figure 2-8.

$P_L = (m/d)P$

where:
$P = 2.79/2 = 1.395$ kips/joist section
$d = 8"$
$m = 0.586"$

$P_L = (0.586/8)(1.395)(1000)$
$= 102$ lb.

For #10-16 self-drilling screw in shear use $V_{all} = 370$ lb. from Example #2, Step 2(c). *(This allowable strength is based on 2 sheets at $t = 0.0566"$ and $F_u = 45$ and 65 ksi and is conservative here.)*

$V_{all} = 370$ lb. > 102 lb. ***OK***

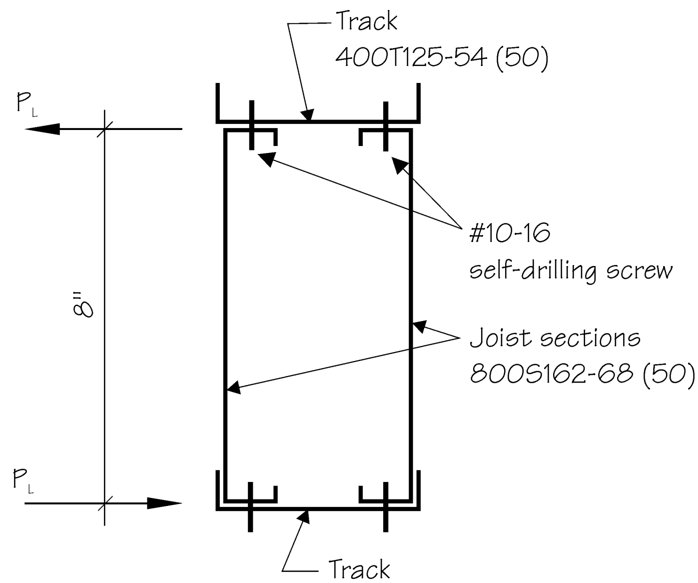


FIGURE 4-11

Step 10(g) – Built-Up Header to Jamb Connection

There are a number of acceptable ways to connect a header to the jamb studs. The design procedure used here is as follows:

- The allowable web crippling capacity (end one flange) of the box header is calculated assuming a bearing length equal to the flange width of the jack stud.
- The jack stud is assumed to carry this web crippling load.
- The residual end reaction for the box header is calculated and is given by the total end reaction less the allowable web crippling capacity from above.
- This residual portion of the reaction is assumed to be transferred to the first king stud via a shear connection detail consisting of a short piece of track.

See Figures 4-12 and 4-13.

Exterior one flange web crippling for box header:

Assume an unfastened condition and bearing length equal to the flange width of the jack stud below = 1.625". From *AISI S100* Section C3.4 and Table C3.4.1-2:

$$P_{all} = \frac{C\,t^2\,F_y\,\sin\theta}{\Omega}\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right)$$

where:

R	= 0.10695"
t	= 0.0713"
Depth	= 8"
h	= Depth - 2t - 2R = 7.6435"
N	= 1.625"
F_y	= 50 ksi
θ	= 90 degrees
C	= 4
C_R	= 0.14
C_N	=0.35
C_h	= 0.02
Ω	= 1.85 (unfastened - conservative)

Substituting for 2 - 800S162-68 (50) sections:

P_{all} = 2(0.964) = 1.93 kips

(For this allowable web crippling capacity to be valid, web punchouts are not permitted in the vicinity of the point loads. The header member has therefore been specified as unpunched.)

The web crippling load P_{all} = 1.93 kips is carried by the jack stud. The capacity of the jack stud is OK by inspection.

The balance of the header reaction is carried by a shear connection to the king stud. This required force is given by:

V_{req} = 2.79 -1.93 = 0.860 kips

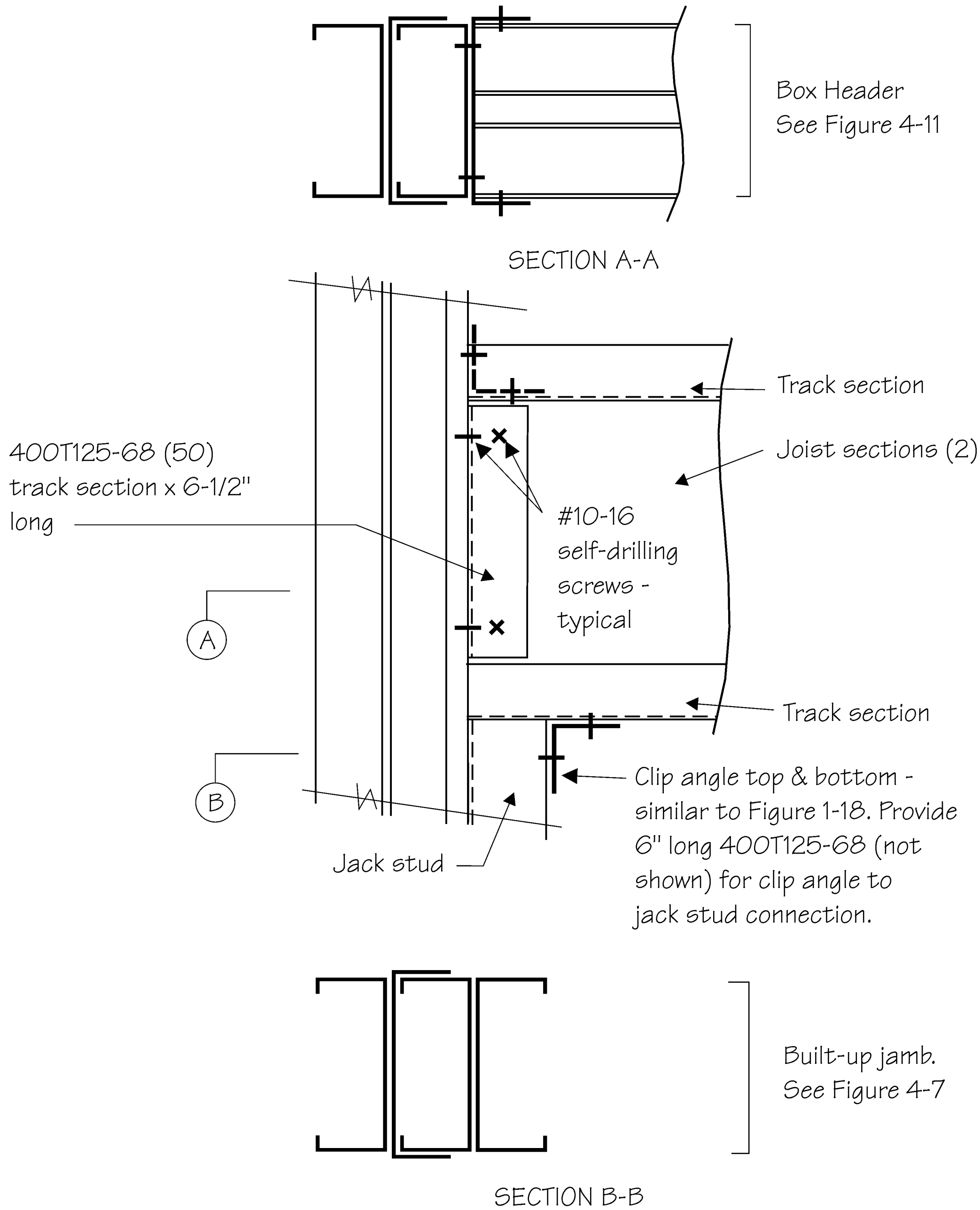

Box Header
See Figure 4-11
SECTION A-A
Track section
Joist sections (2)
400T125-68 (50)
track section x 6-1/2"
long
#10-16
self-drilling
screws -
typical
A
B
Track section
Clip angle top & bottom -
similar to Figure 1-18. Provide
6" long 400T125-68 (not
shown) for clip angle to
jack stud connection.
Jack stud
Built-up jamb.
See Figure 4-7
SECTION B-B

FIGURE 4-12

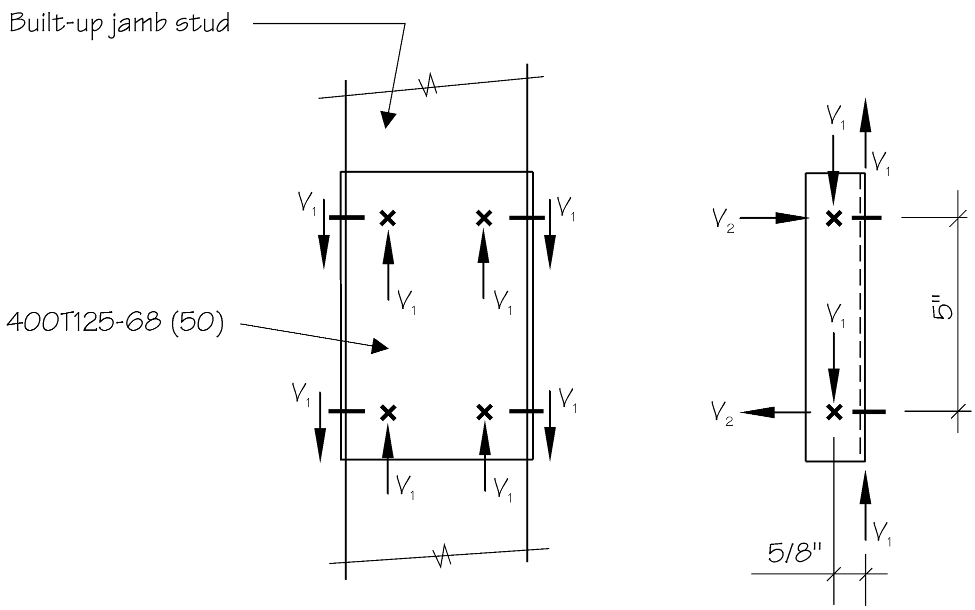


FIGURE 4-13

Provide a short piece of track (6-1/2" long) to act as a shear connector. Use t = 0.0713" to match header joist section.

Calculate screw forces assuming #10-16 self-drilling screws:

From Figure 4-13:

V_1 = 860/4 = 215 lb.

V_2 = [860(0.625)/5]/2 = 54 lb.

$$\text{Resultant } V_{req} = \sqrt{V_1^2 + V_2^2} = \sqrt{215^2 + 54^2}$$
$$= 222 \text{ lb/screw}$$

For #10-16 self-drilling screw in shear, use V_{all} = 370 lb from Example #2 Step 2(c). *(This allowable shear is based on 2 sheets at t = 0.0566" and F_u = 45 and 65 ksi and is conservative here.)*

Gives V_{all} = 370 lb/screw > 222 lb/screw ***OK***

Note the clip angle connection details at the top and bottom of the box header in Figure 4-12. These angles transfer the lateral wind loads from the header to the built-up jamb. *(Only the track portion of the box header is assumed to carry wind.)*

Step 11 – Frequency of Bridging Anchorage

Design bridging to resist torsion induced in the studs by wind load (*AISI S100 Section D3.2.1)* and to resist the weak axis buckling of the studs. Forces will accumulate in the bridging channel, and the design check here is to determine the number of studs that can be braced without exceeding the capacity of the bridging channel. Where the bridging channel is at the limit of its capacity, anchorage is required.

The stud torsional effect which induces major axis bending moments in the bridging channel was previously reviewed in Design Example #2, Step 2.

AISI S100 Section D3.3 provides a method for determining the required brace force and stiffness for axially loaded compression members. However, a simplified method is available in AISI S240 Section B3.4 where a bracing force equal to 2% of the design compression load in the stud is specified. The Commentary *to AISI S240* further states that the 2% bracing force is accumulative between bracing points. A bracing stiffness requirement is assumed not to apply. See Note 4-4.

> *Note 4-4*
>
> *This 2% approach to bracing design is based on historical practice. More sophisticated approaches that include both strength and stiffness requirements are available. See Galambos 1998, Green 2004b, and AISI S100, Section D3.3. However, determination of the actual stiffness of common bracing assemblies is not well researched and workable calculation methods do not generally exist.*

The bridging channel will be subjected to axial load and both major and minor axis bending moment. The capacity of the channel is checked using the beam-column provisions in *AISI S100* Section C5.2.1.

Step 11(a) – Applied Loads

i) Bridging axial load

Required bridging axial load *(tension or compression)*
P_{req} = 0.02 × required stud axial load x number of studs braced (n).

ii) Bridging major axis moment, M_x

Bridging major axis moment is taken from Figures 2-5, 2-6, 2-7 and 2-8.

The outside span is critical and is shown with the moment coefficients in Figure 2-7. The moment, M, is derived from the top and bottom flange brace requirements given in *AISI S100* Section D3.2.1.

$P_L = 1.5(m/d)W$

where:
a = average bridging spacing
= (4 + 2.5)/2 = 3.25 ft.
(assumes 4-ft unbraced length at mid-height)
w = load/ft on stud
= (16/12)(25) = 33.3 lb/ft
W = wa
m = stud web center line to shear center = 0.754"
d = 4"

Substituting:
$P_L = 1.5(0.754/4)(33.3)(3.25)$
= 30.6 lb.

Then the moment resisted by the bridging channel is given by the flange brace couple with a lever arm equal to the depth of the stud. See Figure 2-8.

$M = P_L\ d = 30.6(4) = 122$ in.lb and the resulting moment values in the outside span are illustrated in Figure 4-14.

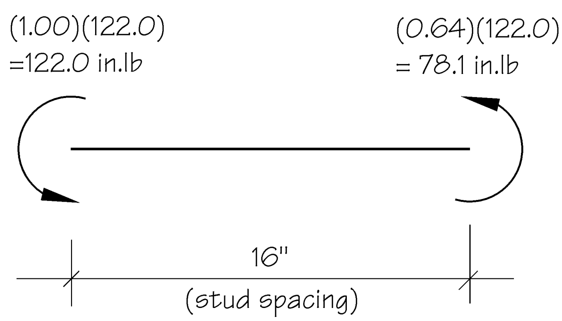


FIGURE 4-14

iii) Bridging minor axis moment, M_y

Bridging minor axis moment is illustrated in Figure 4-15. See Note 4-5.

$M_y = (X_{cg})(\text{Bridging axial load}) = X_{cg}P_{req}$
$= 0.126P_{req}$

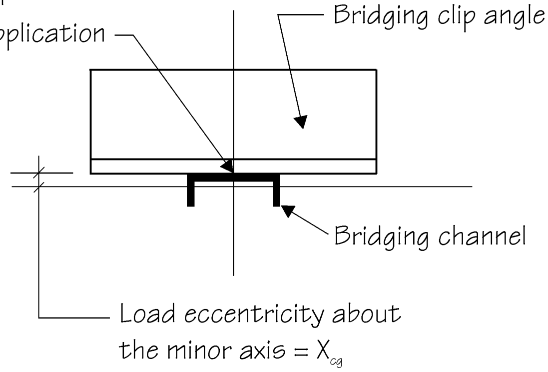


FIGURE 4-15

Note 4-5

The axial load in the bridging channel is incremented at every stud and accumulates over the number of studs between bridging anchorage points. While each increment of axial load is applied with a minor axis eccentricity, the accumulated axial load is assumed to be concentric. Significant minor axis eccentricity does occur in this example at the bridging anchorage point.

Step 11(b) – Allowable Design Strengths

Use 150U50-54 (33) bridging channel. See Figure 4-16.

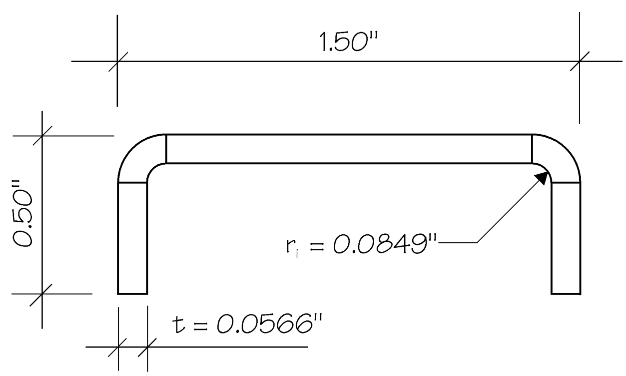


FIGURE 4-16

Allowable design strengths will be checked using the combined compressive axial load and bending provisions in *AISI S100* Section C5.2.1.

i) Section properties

The following bridging channel section properties are taken from load tables or can be calculated from the formulas in AISI D100 Part I. *(Note that the section is fully effective at a uniform stress of F_y = 33 ksi - i.e. $\lambda \le 0.673$ for all elements at $f = F_y$ AISI S100 Section B2.1 – calculations not shown here. Effective properties are therefore not required for either bending or axial load.)*

t = 0.0566 in.
r_i = 0.0849 in.
A = fully effective (unreduced) area = 0.130 in^2
r_x = 0.549 in.
r_y = 0.145 in.
x_0 = 0.254 in.
$r_0 = \sqrt{r_x^2 + r_y^2 + x_0^2}$ = 0.622 in.
I_x = fully effective (unreduced) inertia = 0.0390 in^4
I_y = fully effective (unreduced) inertia = 0.00274 in^4
X_{cg} = location of fully effective (unreduced) centroid = 0.126 in.
C_w = 0.00104 in^6
J = 0.000138 in^4
j = 0.787 in.

S_{fx} = fully effective (unreduced) major axis section modulus = 0.0520 in^3

S_{fy} = fully effective (unreduced) minor axis section modulus
= I_y / (0.5 - X_{cg})
= 0.00733 in^3

ii) Nominal axial strength, P_n (*AISI S100 C4.1 and C4.1.2*)

Assume $K_xL_x = K_yL_y = K_tL_t$ = 16"

Determine the controlling critical elastic buckling stress, F_e:

$$\sigma_{ey} = \frac{\pi^2 E}{\left(\dfrac{K_y L_y}{r_y}\right)^2} = \frac{\pi^2 (29500)}{\left[\dfrac{16}{0.145}\right]^2} = 23.91 \text{ ksi}$$

$$\sigma_{ex} = \frac{\pi^2 E}{\left(\dfrac{K_x L_x}{r_x}\right)^2} = \frac{\pi^2 (29500)}{\left[\dfrac{16}{0.549}\right]^2} = 342.8 \text{ ksi}$$

$$\sigma_t = \frac{1}{Ar_0^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right]$$

$$= \frac{1}{(0.130)(0.622)^2}\left[11300(0.000138) + \frac{\pi^2(29500)(0.00104)}{16^2}\right]$$

$$= 54.52 \text{ ksi}$$

$$\beta = 1 - (x_0 / r_0)^2$$

$$= 1 - (0.254/0.622)^2 = 0.8332$$

From *AISI S100* Section C4.1.1, the flexural critical elastic buckling stress is given by:
F_e = the lesser of σ_{ex} or σ_{ey}
= 23.91 ksi

From *AISI S100* Section C4.1.2, F_e may also be limited by the torsional-flexural critical elastic buckling stress given by:

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

Substituting gives:

F_e = 52.91 ksi

F_e = 23.91 ksi governs.

From AISI S100 Section C4.1:

$P_n = A_e F_n$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{33}{23.91}} = 1.175$$

For $\lambda_c \leq 1.5$

$$F_n = (0.658)^{\lambda_c^2} F_y = (0.658)^{1.175^2}(33)$$

= 18.52 ksi

$P_n = A_e F_n$ = 0.130(18.52) *(No local buckling)*
= 2.41 kips

iii) Nominal flexural strength, M_{nx}

Check lateral-torsional buckling by *AISI S100* Section C3.1.2.1:

From Design Example #2, Step 2(a) there is no reduction in allowable moment for lateral instability.

$M_{nx} = F_c S_x = 33(0.0520)$ *(No local buckling)*

$= 1.716$ in.kips

iv) Nominal flexural strength, M_{ny}

Lateral buckling associated with bending about the y-axis can be checked using AISI S100 Section C3.1.2.1 with the critical elastic stress defined by Equation C3.1.2.1-10.

This expression applies to bending about the centroidal axis perpendicular to the symmetry axis. For typical cold-formed steel framing members, it is the weaker axis by a significant margin, lateral buckling does not occur, and $F_c = F_y$.

That is,

$M_{ny} = S_{fy} F_y = 0.00733(33)$ *(No local buckling)*

$= 0.242$ in.kips

v) Nominal axial strength, P_{no}

$P_{no} = A_e F_y = 0.130(33)$ *(No local buckling)*

$= 4.29$ kips

vi) P_{Ex} and P_{Ey}

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} = \frac{\pi^2 (29500)(0.0390)}{(16)^2}$$

$= 44.4$ kips

$$P_{Ey} = \frac{\pi^2 EI_y}{(K_y L_y)^2} = \frac{\pi^2 (29500)(0.00274)}{(16)^2}$$

$= 3.12$ kips

vii) C_{mx} and C_{my}

$C_{mx} = 0.6 - 0.4(M_1/M_2)$

For M_1 and M_2, see Figure 4-14.

$C_{mx} = 0.6 - 0.4(0.64M/M)$

= 0.344

To calculate C_{my}, assume a concentric axial load on one end and an eccentric axial load on the other with $e_y = X_{cg}$. This gives:

$C_{my} = 0.6$

Summarizing:

$P_n = 2.41$ kips
$M_{nx} = 1.716$ in.kips
$M_{ny} = 0.242$ in.kips
$P_{no} = 4.29$ kips
$P_{Ex} = 44.4$ kips
$P_{Ey} = 3.12$ kips
$C_{mx} = 0.344$
$C_{my} = 0.6$

$\Omega_c = 1.80$
$\Omega_b = 1.67$

Step 11(c) – Interaction Checks

By *AISI S100* Section C5.2.1,
Interaction Equation #1 (C5.2.1-1):

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\left(1 - \frac{\Omega_c P}{P_{Ex}}\right)} + \frac{\Omega_b C_{my} M_y}{M_{ny}\left(1 - \frac{\Omega_c P}{P_{Ey}}\right)} \le 1.00$$

Substituting allowable design strengths from Step 11(b):

$$\frac{1.80P}{2.41} + \frac{1.67(0.344)M_x}{1.716\left(1 - \frac{1.80P}{44.4}\right)} + \frac{1.67(0.6)M_y}{0.242\left(1 - \frac{1.80P}{3.12}\right)} \le 1.00$$

Interaction Equation #2 (C5.2.1-2):

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \le 1.00$$

Substituting allowable design strengths from Step 11(b):

$$\frac{1.80P}{4.29} + \frac{1.67M_x}{1.716} + \frac{1.67M_y}{0.242} \le 1.00$$

Try anchoring bridging every 11 studs.

The required bridging loads are from Step 11(a) and are multiplied by the appropriate load combination factor in the calculations that follow.

Load Case I: D + L

P_{req}/stud = 2.79 kips (from Step 7c)

P for bridging channel with n = 11
= 2.79(0.02)(11)
= 0.614 kips

$M_x = 0$

$M_y = 0.126P = 0.126(0.614)$
= 0.0774 in.kips

Substituting in Interaction Equation #1:

0.46 + 0.00 + 0.50 = 0.96 < 1.00 ***OK***

Substituting in Interaction Equation #2:

0.26 + 0.00 + 0.53 = 0.79 < 1.00 ***OK***

Load Case II: 0.6W + D

P_{req}/stud = 1.06 kips (from Step 7c)

P for bridging channel with n = 11
= 1.06(0.02)(11)
= 0.233 kips

M_x = 0.122 in.kips

$M_y = 0.126P = 0.126(0.233)$
= 0.0294 in.kips

Substituting in Interaction Equation #1:

0.17 + 0.04 + 0.14 = 0.35 < 1.00 ***OK***

Substituting in Interaction Equation #2:

0.10 + 0.12 + 0.20 = 0.42 < 1.00 ***OK***

<u>Load Case III:</u> D + 0.75(0.6W + L)

P_{req}/stud = 2.36 kips (from Step 7c)

P for bridging channel with n = 11
= 2.36(0.02)(11)
= 0.519 kips

M_x = 0.122(0.75)
= 0.0915 in.kips

M_y = 0.126P = 0.126(0.519)
= 0.0654 in.kips

Substituting in Interaction Equation #1:

0.39 + 0.03 + 0.39 = 0.81 < 1.00 ***OK***

Substituting in Interaction Equation #2:

0.22 + 0.09 + 0.45 = 0.76 < 1.00 ***OK***

Therefore, from Load Cases I, II and III interaction checks, anchoring bridging every 11 studs is ***OK***. See Note 4-6.

Note 4-6

1. *Flat strap tension bridging (see Introduction Fig. III) is also an acceptable brace for axial load bearing steel studs. Note that the accumulated force in flat strap bridging includes 2% of the axial load in each stud plus the force necessary to restrain torsion in every stud. The accumulation of the torsional component can be reduced with periodic blocking-in between the studs.*

2. *The spacing of bridging anchorage is based on a strength criterion only. To help control the stiffness of the bridging, arrange the bridging anchorage so that no stud is more than 6 stud spaces away from an anchorage location.*

Step 12 – Bridging Anchorage

From Step 11(c), the bridging must be anchored every 11 studs. See Figure 4-17 for a suggested anchorage detail using flat strap X-bracing.

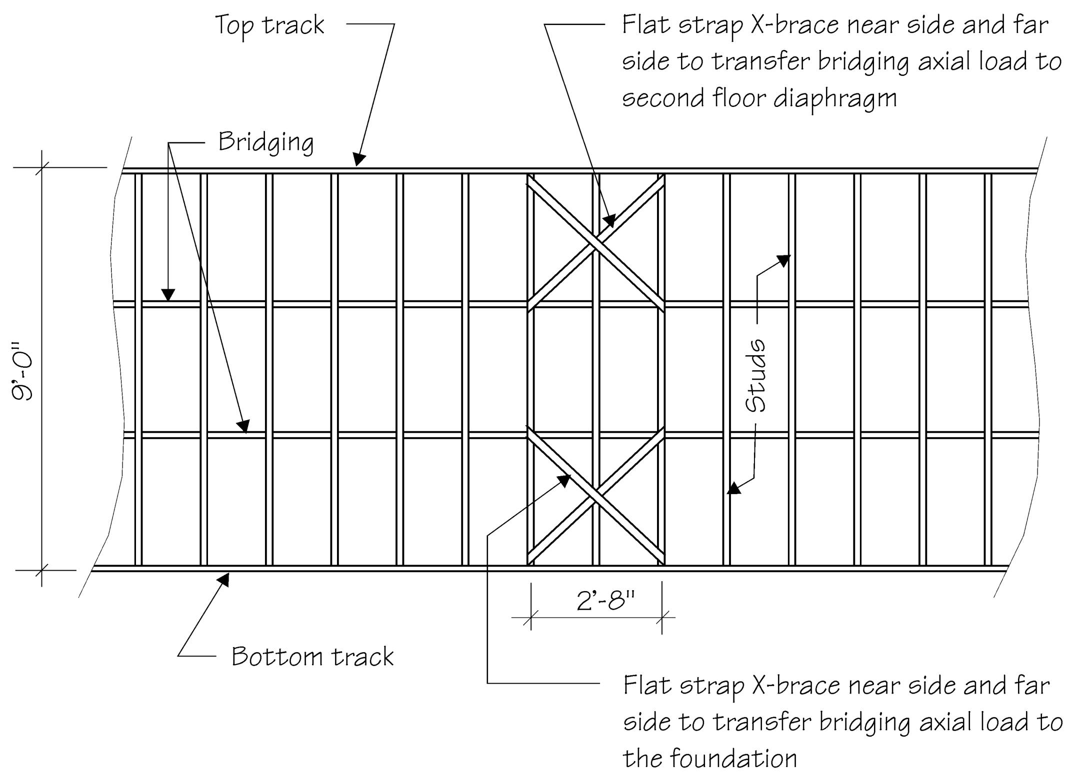


FIGURE 4-17

Introduction Figure V shows another acceptable detail. Other anchorage arrangements are commonly used including anchoring bridging to shear wall elements or built-up members such as jambs. Wherever the bridging is anchored, the anchorage point must have sufficient strength and stiffness. CFSEI Tech Note W400-15 provides additional information regarding anchorage of bridging of axially loaded cold-formed steel studs.

Step 12(a) – Flat Strap X-Bracing

See Figure 4-18. The distance between the top or bottom track and a line of bridging is assumed to be 2'-6" with the 4'-0" maximum bridging spacing at mid-height.

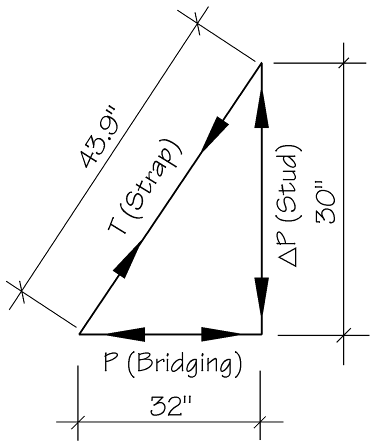


FIGURE 4-18

Input data from previous steps:

From Step 11(c):

$P_{bridging}$ = 614 lb.	D + L
= 233 lb.	D + 0.6W
= 519 lb.	D + 0.75(0.6W + L)

From Step 7(c):

P_{req} /stud = 2.79 kips	D + L
= 1.06 kips	D + 0.6W
= 2.36 kips	D + 0.75(0.6W + L)
P_{all} /stud = 4.63 kips	D + L
= 3.06 kips	D + 0.6W
= 3.34 kips	D + 0.75(0.6W + L)

Try 1-1/2" x 0.0451" flat strap with F_y = 33 ksi.

i) X-bracing vertical reaction

The vertical component of force in the flat straps increases the stud axial load *(for the studs serving as anchorage points)*:

The vertical component from two levels of straps is given by:

$\Delta P = 2P_{bridging}(30/32)$

D + L load case
$\Delta P = 2(614)(30/32)$
$= 1151$ lb.

P_{req}/stud = 2.79 + ΔP = 2.79 + 1.15
= 3.94 kips

P_{all} = 4.63 kips > 3.94 kips ***OK***

<u>D + 0.6W load case:</u>

ΔP = 2(233)(30/32)
= 437 lb.

P_{req}/stud = 1.06 + ΔP = 1.06 + 0.44
= 1.50 kips

P_{all} = 3.06 kips > 1.50 kips ***OK***

<u>D + 0.75(0.6W + L) load case:</u>

ΔP = 2(519)(30/32)
= 973 lb.

P_{req}/stud = 2.36 + ΔP = 2.36 + 0.97
= 3.33 kips

P_{all} = 3.34 kips > 3.33 kips ***OK***

ii) Number of screws required to connect flat straps to studs
D + L load case governs. See Figure 4-19.

For X-bracing on both sides of studs:
T_{req}/strap = ($P_{bridging}$/2)(43.9/32)
= (614/2)(43.9/32)
= 421 lb.

For detailing #10-16 screw locations, assume the following distances:

- End distance = 3d = 3(0.190) = 0.570"
- Minimum center-to-center spacing = 3d = 3(0.190) = 0.570"
- Minimum edge distance = 1.5d = 1.5(0.190) = 0.285"

Screw design input values:

Strap	t_1 = 0.0451"	F_{u1} = 45 ksi
Stud	t_2 = 0.0566"	F_{u2} = 65 ksi
Screw	Size = #10-16	d = 0.190" *(Appendix A, Table A-2)*

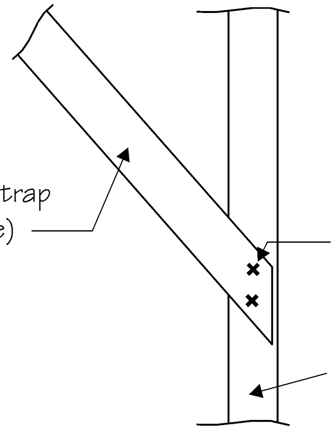


FIGURE 4-19

Allowable screw capacity (V_{all} /screw) is given by the following *(calculations not shown – see Design Example #2, Step 2(c) for typical procedure):*

AISI S100 Section E4.3.1 tilting and bearing:
V_{all} = 347 lb.

AISI S100 Section E4.3.2 shear through the screw itself:
V_{all} = 467 lb.

V_{all} = 347 lb. governs

Required number of screws = T_{req} / V_{all}
= 421/347
= 1.2

Use 2 screws at each end of each strap.

iii) Number of screws required to transfer flat strap horizontal reaction into top and bottom track

Flat strap imposes a horizontal load near the end of the stud. The stud transfers this load into the track (top or bottom) through the stud to track screw connection.

For the D + L load case, the horizontal reaction is given by:
P_{req} = 614 lb.

Screw design input values:

Track	t_1 = 0.0566"	F_{u1} = 65 ksi
Stud	t_2 = 0.0566"	F_{u2} = 65 ksi
Screw	Size = #10-16	d = 0.190" *(Appendix A, Table A-2)*

V_{all} /screw = 467 lb. with shear through the screw itself governing. *(Calculations not shown – see Design Example #2, Step 2(c) for typical procedure.)*

V_{all} = 2(467) for one screw each side
= 934 lb. > 614 lb. ***OK***

iv) Flat strap size

For 1-1/2" x 0.0451" flat strap with F_y = 33 ksi and F_u = 45 ksi:

T_{req} /strap = 421 lb. from above

Check the gross section *AISI S100* Section C2.1:
$T_{all} = T_n/\Omega = A_gF_y/\Omega$
= (1.50)(0.0451)(33)/1.67
= 1.34 kips > 0.421 kips ***OK***

Check the net section:
Use AISI S100 Equation C2.2-2.

Assume screws are aligned perpendicular to the tensile force:

$T_{all} = T_n/\Omega = A_nF_u/\Omega$
= [1.50 - 2(0.190)](0.0451)(45)/2.00
= 1.14 kips > 0.421 kips

Therefore, 1-1/2" x 0.0451" flat strap with F_y 33 ksi and 2 - #10-16 screws each end. ***OK***

Step 12(b) – Bridging Clip Angle at Bridging Anchorage Point

To ensure a stiff connection detail, in axial load bearing construction, size clip angles as per Note 2-4, except that it is recommended that the thickness of the bridging clip angle be the greater of 0.0566" or one thickness heavier than the thickness of the stud.

For clip angle with 400S162-54 stud, use 1-1/2" x 1-1/2" clip angle with t = 0.0713", F_y = 50 ksi and 3-1/2" long.

i) Connection of bridging channel to bridging clip angles at anchorage point

See Figure 4-20.

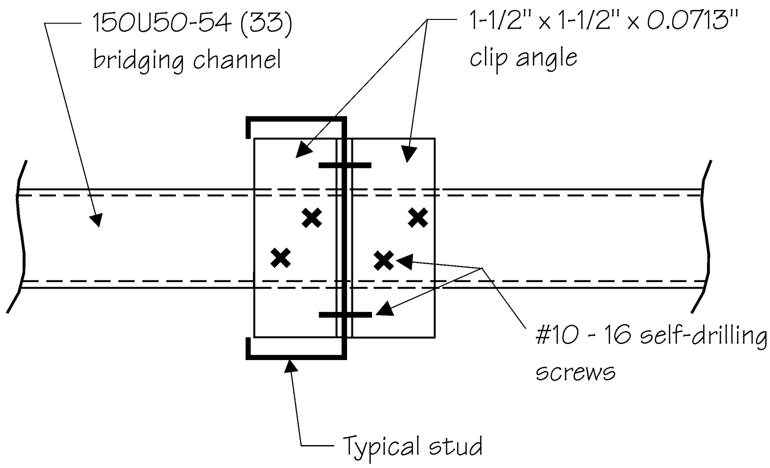


FIGURE 4-20

From Step 11(c), the maximum axial load in the bridging channel is given by D + L Load Case:

Required shear per screw:

$P_{bridging}$ = 614 lb. *(tension or compression)*

Assuming all load is transferred through one clip angle:
V_{req} /screw = 614/2 = 307 lb.

Allowable shear per screw:

Screw design input values:

Clip angle	t_1 = 0.0713"	F_{u1} = 65 ksi
Bridging channel	t_2 = 0.0566"	F_{u2} = 45 ksi
Screw	Size = #10-16	d = 0.190" *(App. A, Table A-2)*

V_{all} /screw = 370 lb. with AISI S100 Equation E4.3.1-1 governing *(Calculations not shown – see Design Example #2, Step 2(c) for typical procedure).*

V_{all} = 370 lb. > 307 lb. ***OK***

Therefore, for transfer of forces between the bridging channel and the clip angle, a single clip angle is sufficient.

ii) Connection of bridging clip angle to studs at anchorage point

The bridging channel can be in tension or compression. The load is transferred to the clip angle, then to the stud, and finally to the flat strap X-bracing.

First check if a single clip angle is sufficient for the connection between the clip angle and the stud. The screws will be in tension. Again, the D + L load case governs.

Required pullout/screw:

T_{req} /screw = 614/2 = 307 lb.

Allowable pullout/screw:

Screw design input values:

$t_c = t_2 = 0.0566$" and $F_u = 65$ ksi
Screw = #10-16 d = 0.190" *(App., A Table A-2)*

T_{all} /screw = 198 lb. *(Calculations not shown – see Design Example #2, Step 2(c) for typical procedure.)*

T_{all} = 198 lb. < 307 lb. ***UNSATISFACTORY***

Therefore, a single clip angle is not sufficient because the clip angle to stud screws acting in tension do not have sufficient capacity.

Add a second clip angle as illustrated in Figure 4-20. With this configuration, the load transfer between the clip angle and the stud will be in bearing whether the bridging channel is in tension or compression.

Step 13 – Bridging to Typical Stud Screwed Connection

The bridging channel to stud connection detail is required to transfer the torsional component of the wind load plus 2% of the axial load in the stud.

See Figure 4-21.

M_{req} for the torsional restraint of the stud under the full wind load 0.6W.

From Step 11(a)ii:

M_{req} = 122 in.lb

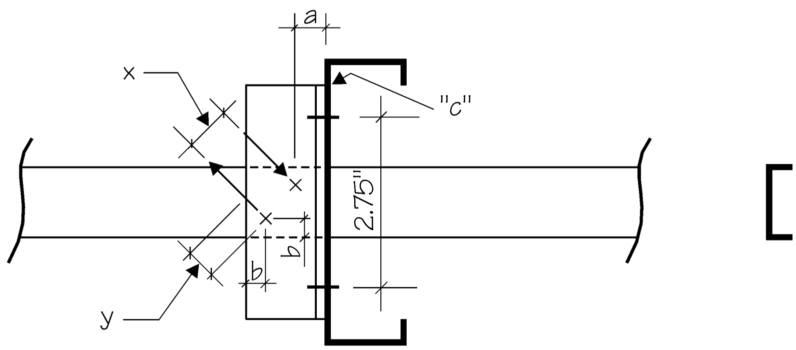


FIGURE 4-21

P_{req} for translation restraint of stud:

For full live load L *(with PLL from Step 7c):*

$P_{req} = 0.02P_{LL} = 0.02(1.73)(1000)$
$= 34.6$ lb.

For dead load D *(with PDL from Step 7c):*

$P_{req} = 0.02P_{DL} = 0.02(1.06)(1000)$
$= 21.2$ lb.

Step 13(a) – Bridging Channel to Bridging Clip Angle Screws

From Design Example #2, Step 2(c) and Figures 2-12 and 4-21, the spacing between the screws is given by:
x = 1.101"

Required shear per screw:

Load Case I: D + L

For translational restraint only
$V_{req} = (34.6 + 21.2)/2$
$= 28$ lb/screw

Load Case II: 0.6W + D

For translational restraint + torsional restraint
$V_{req} = 21.2/2 + 122/1.101$
$= 121$ lb/screw

(The V_{req} /screw total conservatively assumes the two contributing components are directly additive even though they act in different directions.)

Load Case III: D + 0.75(0.6W + L)

For translational restraint + torsional restraint
V_{req} = 21.2/2 + 0.75(34.6)/2 + 0.75(122)/1.101
= 107 lb/screw

Load Case II governs and V_{req} = 121 lb/screw.

Allowable shear per screw:

Screw design input values:

Clip angle	t_1 = 0.0713"	F_{u1} = 65 ksi
Bridging channel	t_2 = 0.0566"	F_{u2} = 45 ksi
Screw	Size = #10-16	d = 0.190" *(App. A Table A-2)*

V_{all} /screw = 370 lb.

With AISI S100 Equation E4.3.1-1 governing *(Calculations not shown – see Design Example #2, Step 2(c) for typical procedure),* gives:
V_{all} = 370 lb. > 121 lb. ***OK***

Step 13(b) – Bridging Clip Angle to Stud Screws

From Figure 4-21 with a 3-1/2" long bridging clip angle, the spacing between the screws is assumed to be 2.75".

Required pullout per screw:

Load Case I: D + L

For translational restraint only
T_{req} = (34.6 + 21.2)/2
= 28 lb/screw

Load Case II: 0.6W + D

For translational restraint + torsional restraint *(Moments about "c" Fig. 4-21)*
T_{req} = 21.2/2 + 122/(2.75 + 0.375)
= 50 lb/screw

<u>Load Case III:</u> D + 0.75(0.6W + L)

For translational restraint + torsional restraint *(Moments about "c" Fig. 4-21)*
V_{req} = 21.2/2 + 0.75(34.6)/2 + 0.75(122)/(2.75 + 0.375)
= 53 lb/screw

Load Case III governs and V_{req} = 53 lb/screw.

Allowable pullout per screw:

Screw design input values:

$t_c = t_2$ = 0.0566" and F_u = 65 ksi
Screw = #10-16 d = 0.190" *(App. A, Table A-2)*

T_{all} /screw = 198 lb. *(Calculations not shown – see Design Example #2, Step 2(c) for typical procedure),* gives:
T_{all} = 198 lb. > 53 lb. ***OK***

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DESIGN EXAMPLE #5
COLD-FORMED STEEL FRAMING FLOOR AND AXIAL LOAD BEARING WALL—LEDGER FRAMING

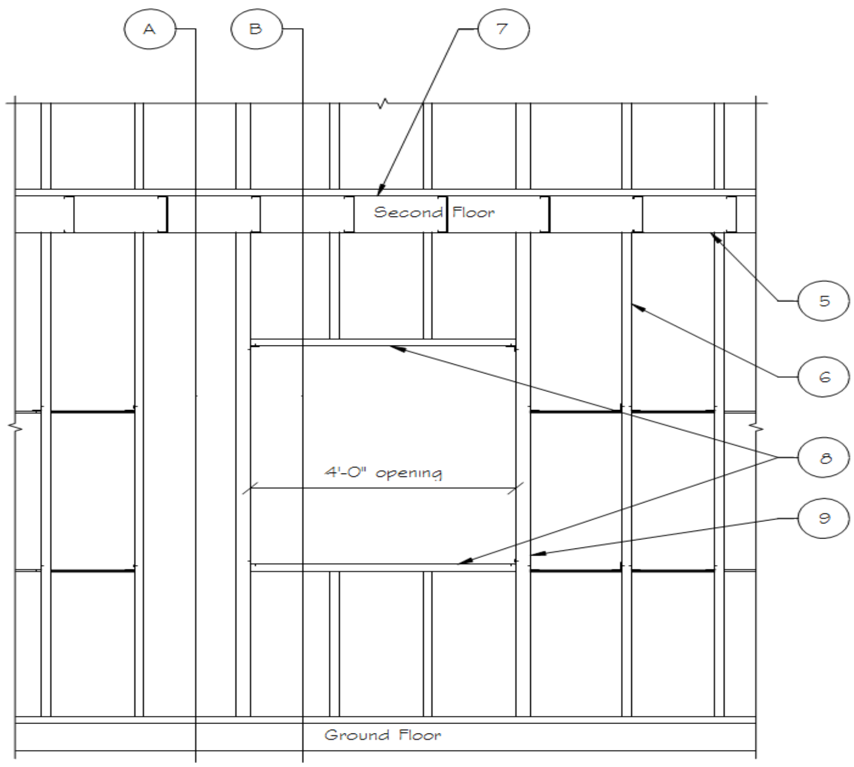

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Figure 5 - 1

Introduction

This example covers the design of a cold-formed steel framing floor system supported by a steel stud wall, including a window opening using the ledger framing method. The example will mimic Example #4 so that the user can compare the two methods. The detailed calculation of bridging forces is not included in this example. Refer to Example #4 for design of bridging, bridging connections and anchorage. The section numbers for the design of individual components are identified in Figures 5-1 and 5-2.

Step 1 – Given

- Design wind load, 0.6W = 25 psf (factored for ASD load combinations)
- Floor design live load = 40 psf
- Floor partition allowance = 0 psf
- Wall loads from above:
  P_{LL} = 1.33 kips
  P_{DL} = 0.67 kips
  (No snow, rain or roof live load in this example)
- Wall height at 2nd Floor = 9'0" to bottom of joist
- Wall deflection limit = L/360
- Floor deflection limit = L/360 for live load and L/240 for total load
- Vibration criteria = none
- Screwed connections
- Ledger construction
- Required fire rating = none
- Lateral stability for the building as a whole will be provided by reinforced concrete elevator shaft and stairwells
- Depth of stud to meet architectural requirements = 4"

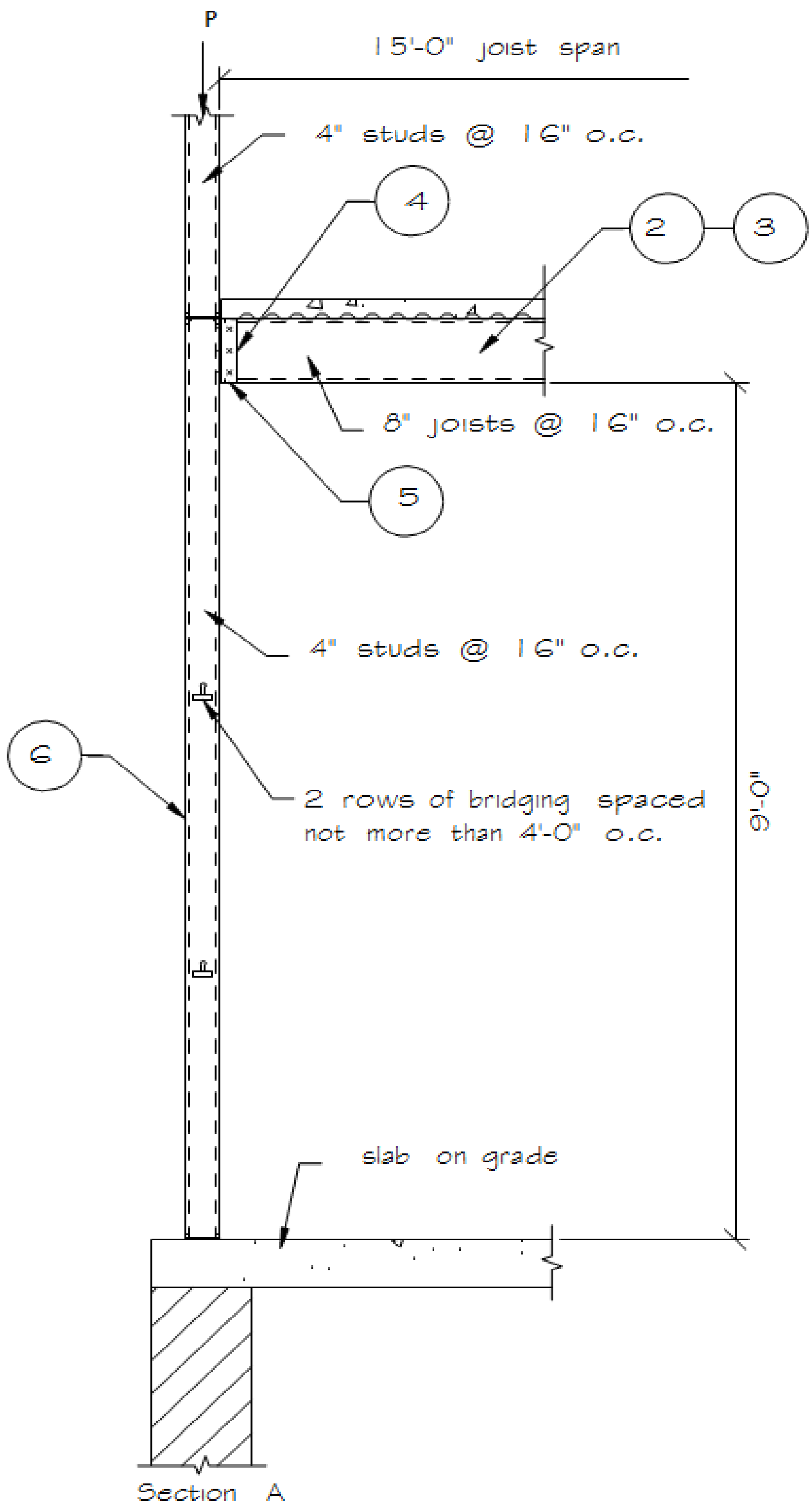
P
15'-0" joist span
4" studs @ 16" o.c.
4
2
3
8" joists @ 16" o.c.
5
4" studs @ 16" o.c.
6
2 rows of bridging spaced not more than 4'-0" o.c.
9'-0"
slab on grade
Section A

Figure 5 - 2

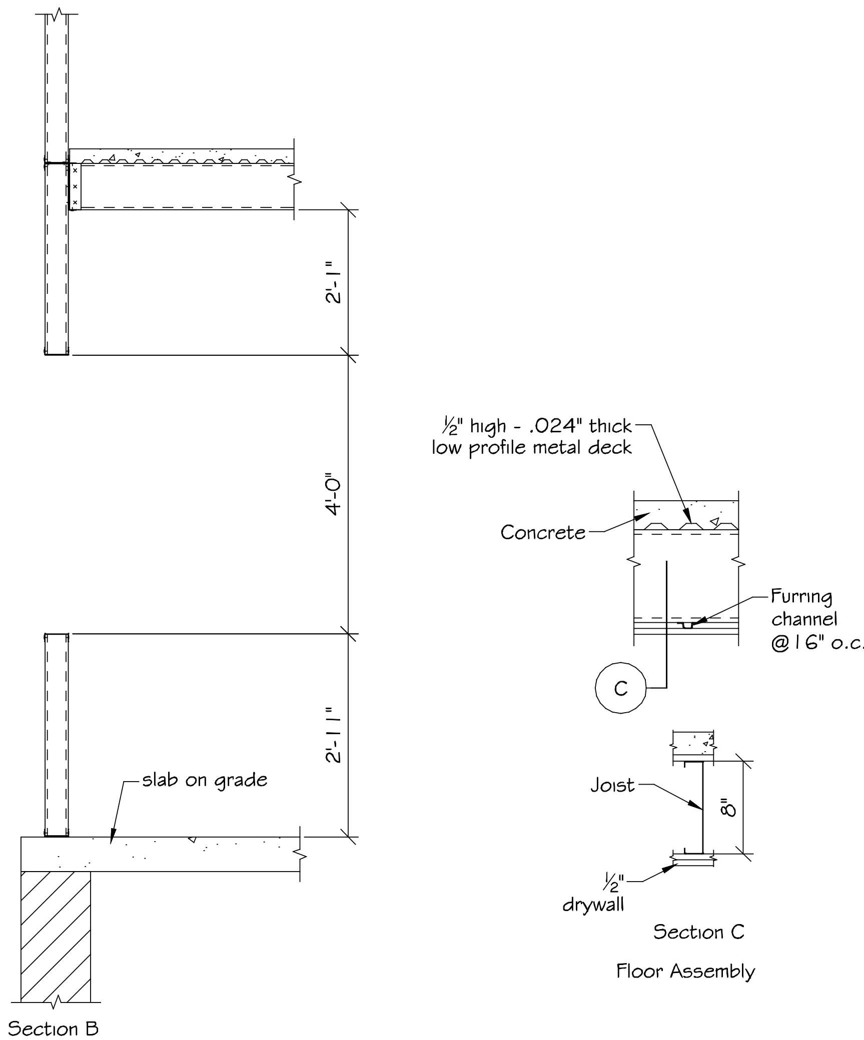
2'-1"
4'-0"
2'-11"
slab on grade
Section B
½" high - .024" thick
low profile metal deck
Concrete
Furring
channel
@16" o.c.
C
Joist
8"
½"
drywall
Section C
Floor Assembly

Figure 5 - 3

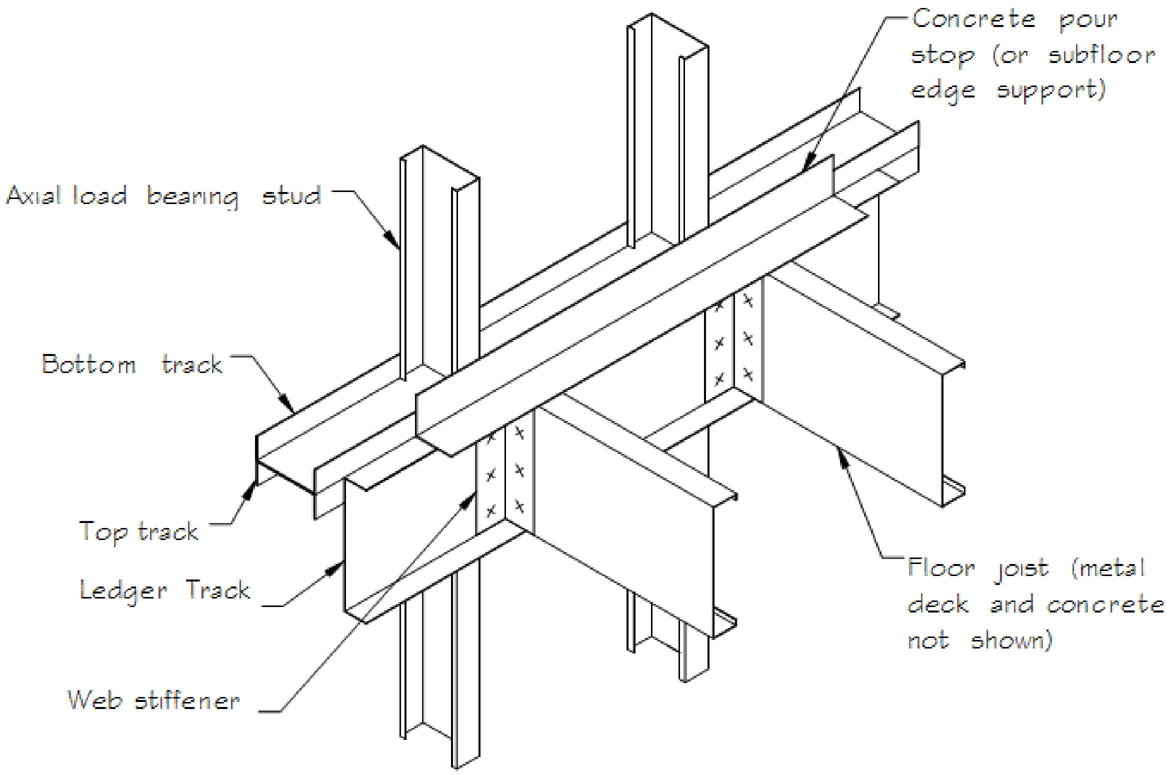

Figure 5 - 4

Step 2 – Floor Joist Selection

Live load:	40 psf
Dead load (Example #4):	39.4 psf
Total load:	79.4 psf

Span length L = 15'-0" single span (overall length – slightly conservative compared to center line of bearing)

Deflection limit = L/360 for live load and L/240 for total load

Vibration criteria = none

Try 800S200-54 (50) joist @ 16" o.c.:

From load tables for 40 psf live load and 40 psf dead load:

Allowable span length = 15'-3" > 15'-0" ***OK***

Step 3 – Floor Joist Bridging

See Example #4, Step 3 for floor bridging design.

Step 4 – Floor Joist Web Stiffener

Floor joists typically require web stiffeners. These web stiffeners are designed in accordance with the requirements of *AISI S100* Section C3.7. Gravity loads from the wall above do not contribute to joist web crippling when ledger framing is used.

Governing load combination:

D + L

P_{LL} = floor joist reaction
= (15/2)(16/12)(40)/1000
= 0.40 kips/stud

P_{DL} = floor joist reaction
= (15/2)(16/12)(39.4)/1000
= 0.39 kips/stud

$P_{req} = P_{LL} + P_{DL} = 0.40 + 0.39$
= 0.79 kips

For web crippling strength of the joist alone, an end one flange loading case is used with the bearing length equal to the leg length of the track.

The end one flange web crippling capacity for an 800S200-54 joist with 1.5" bearing from manufacturers' tables:

P_a = 0.651 kips ***STIFFENERS REQUIRED***

Note that the provisions of AISI S100 Section C3.7.2 apply to C-Section flexural members with two-flange loading. For the relatively modest one-flange loading of this example, a more simplistic design using a clip angle to transfer the joist end reaction to the ledger track in shear can be used.

For shear transfer details on this project, see Figures 5-4 and 5-5.

For a joist end reaction of 0.79 kips and #10 screws in shear having a capacity from Example #2, Step 2(c) of 370 lb/screw, the number of screws required to transfer the shear can be estimated as:

N_{screws} = 790/370 = 2.14

Therefore, use L2×2×54-mil x 0′ 8″ with (3) #10 screws each leg.

Note 5-1

The above analysis ignores the eccentricity on the fasteners. See Example #1, Figure 1-19 for a more detailed analysis of clip angle connections including eccentricity. For this example, by observation, the number of screws specified is adequate based on the minor eccentricity and the relatively wide screw spacing.

The angles can be either inside or outside the joist. Figure 5-4 shows the angles outside. Angles outside the joist also allow easier connection to the ledger track.

AISI S100 Section C3.7 specifies that the length of web stiffeners shall not be less than the outside depth of the joist minus 3/8". While the angles here are not acting as web stiffeners in the context of the provisions of AISI S100, it is prudent to use angles as long as reasonably possible to provide a strong, stiff connector.

The ledger track acts as a load distribution member. As such, it is not required for the joists to align with the studs. The angle noted above would only attach the joist to the ledger track. See Step 5 for design of the ledger track and its connection to the wall studs.

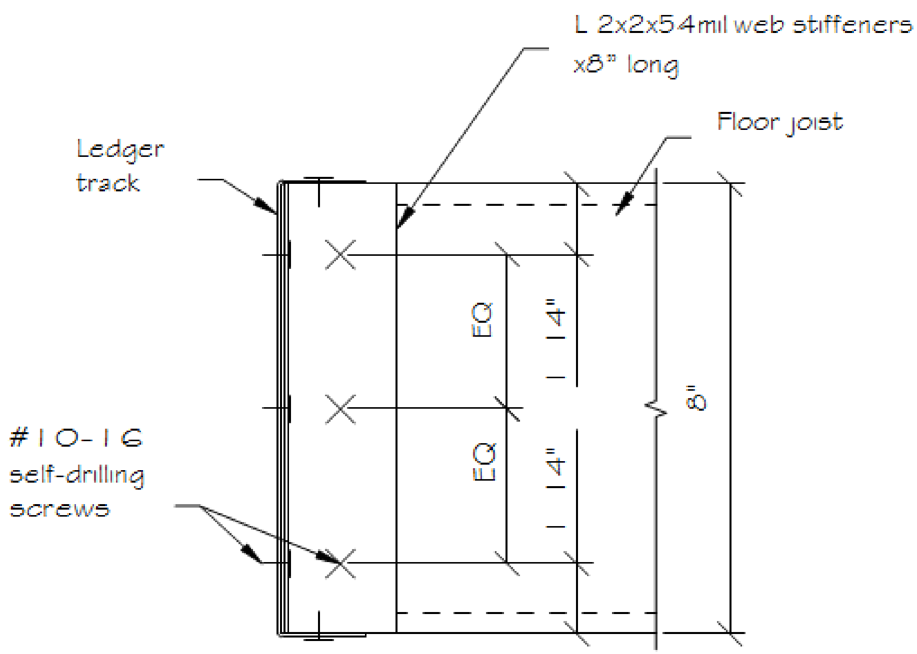

Figure 5 - 5

Step 5 – Ledger Track

Step 5(a) – Ledger Track Section Size

The ledger track transfers vertical loads to the wall studs and depending on the diaphragm details, may also transfer lateral loads from the wall studs into the diaphragm. In Figure 5-6A, lateral loads from the wall are transferred directly into the diaphragm via the attachment of the wall tracks to the ply. In Figure 5-6B, the diaphragm stops short and the load path for wall lateral loads must go through the ledger track to the joist and then into the diaphragm.

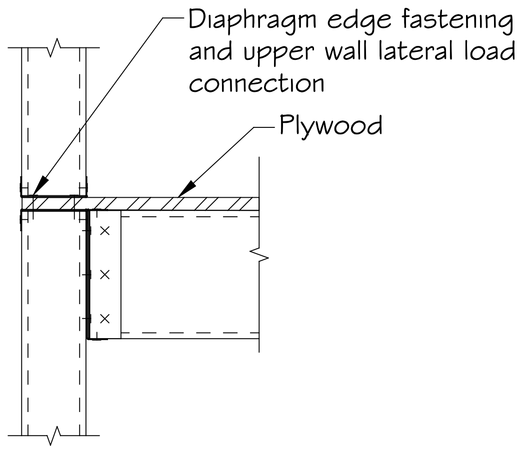

Figure 5 - 6A

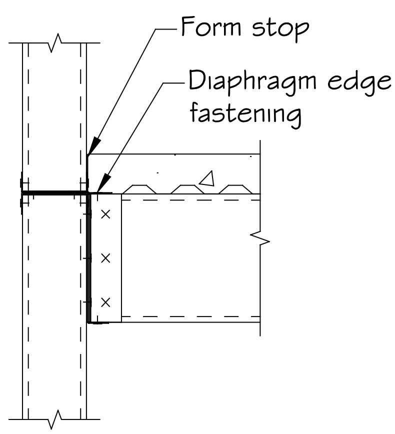

Figure 5 - 6B

Because the ledger track can act as a load distribution member, joists and studs need not be aligned. For sizing the track, the most conservative location, where the joists are centered between studs, is used. As a conservative simplification, simple span moments are also used.

Try 800T150-68 (50 ksi):

From manufacturer's data:

$M_{a\text{-}x}$ = 37.6 in-k (strong-axis bending)
$M_{a\text{-}y}$ = 1.80 in-k (weak-axis bending)

Check Load Combinations:

D + L:

$P_{DL} + P_{LL} = 0.79$ kips
Span = 16" (Stud Spacing)

Moment from vertical loads:
$M_v \leq 0.79(16/4) = 3.16$ in-k < 37.6 in-k ***OK***

D + 0.75(0.6W + L):

Moment from vertical loads:
$P_{DL} + 0.75\,P_{LL} = 0.39 + 0.75(0.40) = 0.69$ kips
$M_{D+L} \leq 0.69(16/4) = 2.76$ in-k

Moment from wind load:
$P_w = 0.75[25(16/12)(9.67/2 + 9.67/2)] = 242$ lb (0.242 kips)
$M_w \leq 0.242(16/4) = 0.968$ in-k

$M_{D+L}/M_{a\text{-}x} + M_w/M_{a\text{-}y}$
$2.76/37.6 + 0.968/1.80 = 0.61 < 1.0$ ***OK***

D + 0.6W:

Moment from vertical load:
$P_{DL} = 0.39$ kips
$M_D = 0.39(16/4) = 1.56$ in-k

Moment from wind load:
$P_w = 25(16/12)(9.67/2 + 9.67/2) = 322$ lb (0.322 kips)
$M_w \leq 0.322(16/4) = 1.29$ in-k

$M_{D+L}/M_{a\text{-}x} + M_w/M_{a\text{-}y}$
$1.56/37.6 + 1.29/1.80 = 0.76 < 1.0$ ***OK***

Therefore, 800T150-68 (50) ledger track is adequate. However, a heavier track may be desirable to eliminate the need for boxed headers. See Step 7 for design of the ledger track to act as a header.

Note 5-2

The published allowable weak-axis bending strength for track is most often given as the minimum value based on compression in the web or compression in the flange tip. For the case of a continuous track resisting both positive and negative wind pressures, both of these conditions will occur.

It is likely that the track will span across many studs, resulting in moments lower than the PL/4 moments used above. A more rigorous analysis may in some cases result in a lighter track section. However, the added stiffness and flexural strength of heavier wall track are beneficial for framing at wall openings (see Step 9).

Step 5(b) – Connection Ledger Track to Studs

The connection of the ledger track to the studs must be capable of transferring the loads from Step 5(a). Gravity loads are transferred in shear from the track to the wall studs, and lateral wind and seismic loads are transferred from the wall studs into the ledger track.

Check Load Combinations:

D + L:

Shear Forces:

$P_{DL} + P_{LL} = 0.79$ kips

#10 screws in min 68-mil track and min 54-mil studs (conservatively applied from previous examples with 54-mil studs and track)

$V_{all} = 370$ lb/screw;

$T_{all} = 109$ lb/screw:

$N_{D+L} = 790/370 = 2.14$ screws

D + 0.75(0.6W + L):

Shear Forces:

$P_{DL} + 0.75\ P_{LL} = 0.39 + 0.75(0.40) = 0.69$ kips

Tension Forces:

$P_w = 0.75[25(16/12)(9.67/2 + 9.67/2)] = 242$ lb (0.242 kips)

Combined Shear and Pullout (*AISI S100 E4.5.2.1*)

Try (4) #10: $690/(4x370) + 242/(4x109) = 1.02 < 1.15$ **OK**

D + 0.6W:

Shear Forces:

P_{DL} = 0.39 kips
Tension Forces:
P_w = 25(16/12)(9.67/2 + 9.67/2) = 322 lb (0.322 kips)

Try (4) #10: 390/(4x370) + 322/(4x109) = 1.00 < 1.15 **OK**

Therefore, use (4) #10 for each stud (minimum 54-mil studs, max 16" on center).

Note 5-3

Note that walls parallel to joists will attach directly to the edge joist. The associated gravity load shear in the connections will be much reduced. The design process otherwise is the same.

Consideration should also be given to the rotational stability of the edge joist under lateral load. The analysis above assumes lateral loads are shared between all of the fasteners. If the rim track or edge joist is not rotationally stiff, load may be concentrated in the fasteners nearest the diaphragm. Additional screws near the diaphragm could be used to ensure adequate fastening. Careful detailing and field observation would be required to ensure fasteners were installed as required. Solid blocking could also be used to provide rotational restraint to the edge joist, allowing the straightforward analysis shown above.

Step 6 – Typical Stud

Step 6(a) – Loading Combinations

The following design approach is recommended for axial load bearing steel studs in ledger framing systems. Refer also to the discussion on bracing in Section 4.2.2 of the Introduction.

1. Use an all-steel (i.e., unsheathed) design approach with steel bridging at regular intervals to resist the torsional component of the load and the tendency for the studs to buckle laterally. The bridging will require periodic anchorage to the primary structure.

2. Conservatively assume $K_x = K_y = K_t = 1.0$. This assumption is common in published load tables and is consistent with AISI S240 Section B3.2.1.1. For distortional buckling, assume $K_\phi = 0$. See CFSEI Tech Note G100-08 for information regarding the selection of K_ϕ.

3. Published load tables usually assume concentric axial loads. That is not the case in a ledger framing system. Eccentric moment should be accounted for explicitly.

Try 400S162-54 (50) stud and bridging spaced at 4'-0" o.c. maximum.

For this example:

L_r (roof LL) = 0
R (rain load) = 0
S (snow load) = 0

From ASCE 2010, the remaining load combinations are (for strength):

D + L
D + 0.6W
D + 0.75(0.6W + L)

For deflection, the 0.6W term above may be reduced by the 0.7 factor in the last two combinations:

D + 0.7(0.6W)
D + 0.75[0.7(0.6W) + L]

Step 6(b) – Check Web Crippling

Web crippling can be checked from wind bearing allowable height tables *(if web crippling is flagged)* or from the allowable web crippling capacities typically published in load tables.

P_{req} = 25(16/12)(9/2) = 150 lb.

From load tables with 1" of bearing length and the end one flange fastened condition:

P_{all} = 628 lb. > 150 lb. ***OK***

Step 6(c) – Check Deflection

Eccentric axial loads result in deflection in addition to those resulting from wind loads. For the axial load applied to the inside flange of the stud as it is for ledger framing, the deflection will be outward and thus be additive to negative wind pressures.

The deflection of a stud with eccentric axial loads (concentrated moment one end) is given by the equation:

$$\Delta e = -\frac{Me}{6EI}\left(\frac{x^3}{H} - Hx\right)$$

where:
M_e = Applied End Moment (in-lb)

E = Modulus of Elasticity (29,500 ksi)
I = Moment of inertia for deflection (in^4)
x = Location along stud (inches)
H = Total stud height (inches)

Based on the above, the maximum deflection due to concentrated end moment occurs at

$$x = \frac{H}{\sqrt{3}} = 0.577H \quad \text{and is equal to} \quad \Delta_{e\,max} = -\frac{MeH^2}{6\sqrt{3}EI}\left(\frac{1}{\sqrt{3}} - 1\right)$$

Note that the maximum deflection from wind occurs at H/2 which does not coincide with the location for the maximum deflection due to eccentric moment. Closed-form solutions for the true combined maximum deflection do not exist. For this example, the total maximum deflection will be conservatively taken as the sum of the maximum deflection due to eccentric moment and the maximum moment due to wind.

D + 0.7(0.6W):

Only the axial load added at a given level contributes to the eccentric moment for the stud below. The axial load from levels above are transferred directly from the bottom of the upper stud to the top of the lower stud with no significant eccentricity. Therefore, for calculating deflection:

P_{DL} = 0.39 kips.
e = 4/2 = 2 inches to center line of stud
$M_e = P_{DL}e$ = 0.39(2) = 0.78 in-k
H = 9′ 8″ = 116 inches (conservatively taken to the top of stud)
I = 1.098 in^4

$\Delta_{e\,max}$ = 0.0208″

Deflection due to uniform loads can be found using the standard equation:

$$\Delta_{w\,max} = \frac{5w\ell^4}{384EI}$$

w = 0.7(25 psf)(16/12) = 23.3 lb/ft = 1.94 lb/in
ℓ = H = 116 inches

$\Delta_{w\,max}$ = 0.1415″

$\Delta_{max} = \Delta_{e\,max} + \Delta_{w\,max}$ = 0.162″ = L/715 ***OK***

D + 0.75[0.7(0.6W)+L]:

$P_{DL} + 0.75\,P_{LL}$ = 0.39 + 0.75(0.40) = 0.69 kips

$M_e = (P_{DL} + 0.75P_{LL})e = 0.69(2) = 1.38$ in-k

$\Delta_{e\ max} = 0.0368''$

$w_w = 0.75(0.7)(0.6W) = 17.5$ lb/ft $= 1.46$ lb/in

$\Delta_{w\ max} = 0.1061''$

$\Delta_{max} = \Delta_{e\ max} + \Delta_{w\ max} = 0.143'' = L/811$ ***OK***

> *Note 5-4*
>
> *The deflections due to eccentric moment look relatively small in the example above. However, they account for 13% and 26% respectively of the total deflection. Where deflection-sensitive finishes or tall wall studs are used, that could be enough to require a heavier or deeper stud than would be required if the deflection from eccentric moment were ignored.*
>
> *As discussed above, combining the maximum deflection from end moment and wind load is somewhat conservative. However, a detailed analysis shows that the conservatism is slight. For this load case, the difference between the maximum deflection calculated above versus the true maximum deflection is less than 1%.*

Step 6(d) – Combined Bending and Axial Load Capacity

Loads from the stud above plus the floor joist reaction *(from Steps 1 and 4 above):*

$P_{LL} = 1.33 + 0.40 = 1.73$ kips/stud

$P_{DL} = 0.67 + 0.39 = 1.06$ kips/stud.

<u>D + L</u>:

$P_{req} = P_{LL} + P_{DL} = 2.79$ kips/stud

$M_e = (0.40 + 0.39)(2) = 1.58$ in-k/stud *See Step 6(b)*

From load tables for 400S162-54 (50) stud at 16" o.c. and 0 psf wind *(Conservatively use 5 psf wind if 0 psf is not available):*

Interaction - AISI S100 Eq. C5.2.1-1

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny}\alpha_y} \le 1.0$$

$P_n/\Omega_c = P_{all} = 4.48$ kips

$M_{nx}/\Omega_b = M_{all} = 14.9$ in-k
$C_{mx} = 1.0$ conservatively

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{EX}} \qquad \text{(AISI S100 Eq. C5.2.1-4)}$$

where:

$$P_{EX} = \frac{\pi^2 E I_x}{(K_x L_x)^2} \qquad \text{(AISI S100 Eq. C5.2.1-6)}$$

$$P_{EX} = \frac{\pi^2 29500 \times 1.098}{(116)^2} = 23.76 \text{ kips}$$

$$\alpha_x = 1 - \frac{1.8(2.79)}{23.76} = 0.789$$

Interaction: $\frac{2.79}{4.48} + \frac{1.58}{14.9(0.789)} = 0.76$ ***OK***

D + 0.6W:

0.6W = 25 psf
Uniform bending load, w= 33.3 lb/ft for studs 16″ oc

$P_{req} = P_{DL} = 1.06$ kips/stud

$M_e = (0.39)(2) = 0.78$ in-k/stud (at end of stud)

For studs with concentrated moment at the top and uniform load along their length, the bending moment is given by:

$$M(x) = \frac{M_e x}{H} + \frac{wHx}{2} - \frac{wx^2}{2}$$ where x is measured from the bottom of stud

It can be shown that the above equation is maximized at $x = \frac{M_e}{wH} + \frac{H}{2}$

Using the above equations with consistent units, the maximum bending moment for this load case is:

$M_{max} = 5.07$ in-k at x = 5.04 ft from the bottom of stud

Interaction check—AISI S100 Eq. C5.2.1-1.

Using values from the D + W load case where appropriate:

$$\alpha_x = 1 - \frac{1.8(1.06)}{23.76} = 0.920$$

Interaction: $\frac{1.06}{4.48} + \frac{5.07}{14.9(0.920)} = 0.61$ ***OK***

D + 0.75(0.6W + L):

W = 0.75(25) = 18.75 psf
w = 25 lb/ft for studs 16″ oc

$P_{req} = P_{DL} + 0.75P_{LL} = 1.06 + 0.75(1.73)$
$= 2.36$ kips

$M_e = [0.75(0.40) + 0.39](2) = 1.38$ in-k/stud (at end of stud)

$M_{max} = 4.23$ in-k at x = 5.31 ft. from the bottom of stud

Using values from the D + W load case where appropriate:

$$\alpha_x = 1 - \frac{1.8(2.36)}{23.76} = 0.821$$

Interaction: $\frac{2.36}{4.48} + \frac{4.23}{14.9(0.821)} = 0.87$ ***OK***

Use 400S162-54 (50).

Step 7 – Gravity Header

Step 7(a) – Gravity Header Selection

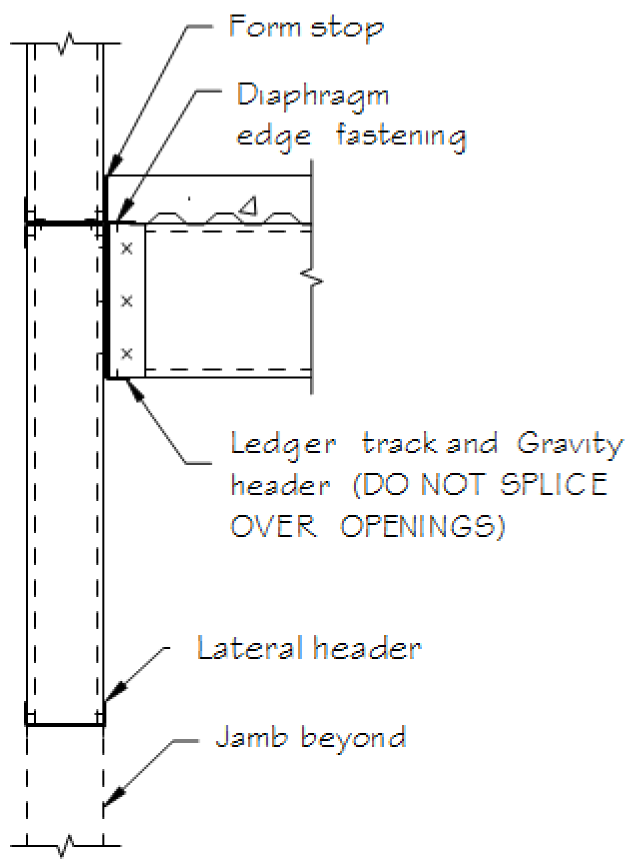

Figure 5 - 7

The ledger track can often be sized to carry gravity loads over the openings without the addition of a boxed header. When the ledger track is used for this purpose, care must be taken to not allow splices in the track over the openings.

Size Ledger Track for Gravity Loads:

D + L (controls for gravity load by inspection):

$P_{DL} + P_{LL} = 2.79$ kips/stud

For the above span and loading condition:

$M_{req} = P(16) = 44.64$ in-k

Try 800T150-97 (50) ledger track:

M_{all} = 65.6 in-k **OK**

Deflection can be calculated or determined using software for this condition to be 0.039 inches, or L/1228, which easily meets the L/360 deflection criteria.

Therefore, use 800T150-97 (50) track over openings. Do not splice over openings.

Step 7(b) – Cripple Studs to Gravity Header

To ensure load transfer from the cripple studs to the gravity header (ledger track), the connection must resist gravity loads from the wall studs in shear in addition to any tension loads created by gravity load eccentricities in combination with wind. For this case, the load case D + 0.75(0.6W + L) controls.

D = 0.67 kips, $P_{eD} = 0.67 \times 2/(33 - 8/2) = 46$ lb/stud
L = 1.33 kips, $P_{eL} = 1.33 \times 2/(33-8/2) = 92$ lb/stud
$P_w = 25 \times (16/12) \times (9.67/2 + 2.75/2) = 207$ lb/stud

Therefore, V = 0.67 + 0.75(1.33) = 1668 lb/stud
T = 46 + 0.75(92 + 207) = 270 lb/stud

Try (7) #10-16 screws using allowable shear and tension values from the previous examples:

$$1668/(7\times370) + 270/(7\times109) = 1.00 < 1.15$$ **OK**

Therefore, Use (7) #10 for each cripple stud.

Step 7(c) – Lateral Header and Sill Selection

A single track is often adequate to carry out-of-plane lateral loads over typical openings. The track must carry wind loads in addition to any lateral forces resulting from eccentric dead and live loads above the opening. As in Step 7(b), the eccentricity is from the center of the supported cripple studs to the ledger that is carrying the gravity loads.

All pertinent load combinations must be checked to determine the critical loads on the header and sill track. Lateral loads on the header track generated by eccentric gravity loads act inward. Therefore, for sizing the header the total lateral header lateral load is maximized when these loads are combined with positive wind load.

For the header, it can be shown that D + 0.75(0.6W + L) results in the maximum total lateral force:

Lateral Load from Wind: P_w = 25(16/12)×(2.75 + 4)/2 = 113 lb @ 16"
Lateral Load from Dead Load Eccentricity: P_{eD} = 46 lb @ 16" from Step 7(b)
Lateral Load from Live Load Eccentricity: P_{eL} = 92 lb @ 16" from step 7(b)

Therefore, P_{max} = 46 + 0.75(113 + 92) = 200 lb

Sill:
P_s = 25(16/12)(4+2.92)/2 = 115 lb

Therefore, header controls:
M_{req} = P(16) = 3.20 in-k

Try 400T150-54 (50) header and sill track:

M_{all} = 11.2 in-k ***OK***

Deflection can be calculated or determined using software for this condition to be 0.029 inches, or L/1667, neglecting the 0.7 factor on P_w which easily meets the L/360 deflection criteria.

Step 8 – Lateral Header and Sill to Jamb Connection

The header and sill track to jamb connections transfer only horizontal forces as determined above.

By inspection, use 54-mil clip angles x 0′4″ with (2) #10 each leg.

Step 9 – Jamb Studs

Step 9(a) – Jamb Stud Selection

Jamb studs are sized to carry the accumulated gravity and lateral loads from the opening plus any additional tributary gravity and lateral loads from above and from adjacent stud bays.

The lateral loads applied to the header due to eccentric gravity loads from above act inward. Therefore, the resulting moment on the jamb due to these loads opposes the eccentric moment caused by the connection of the ledger track/gravity header to the inside flange of the jamb stud.

For this example, analysis shows that the more critical load case is negative wind in combinations with ledger gravity loads, including eccentricity. It is conservative for the design of the jamb studs in this case to ignore the header lateral loads due to eccentric gravity loads over the opening determined in step 7(c).

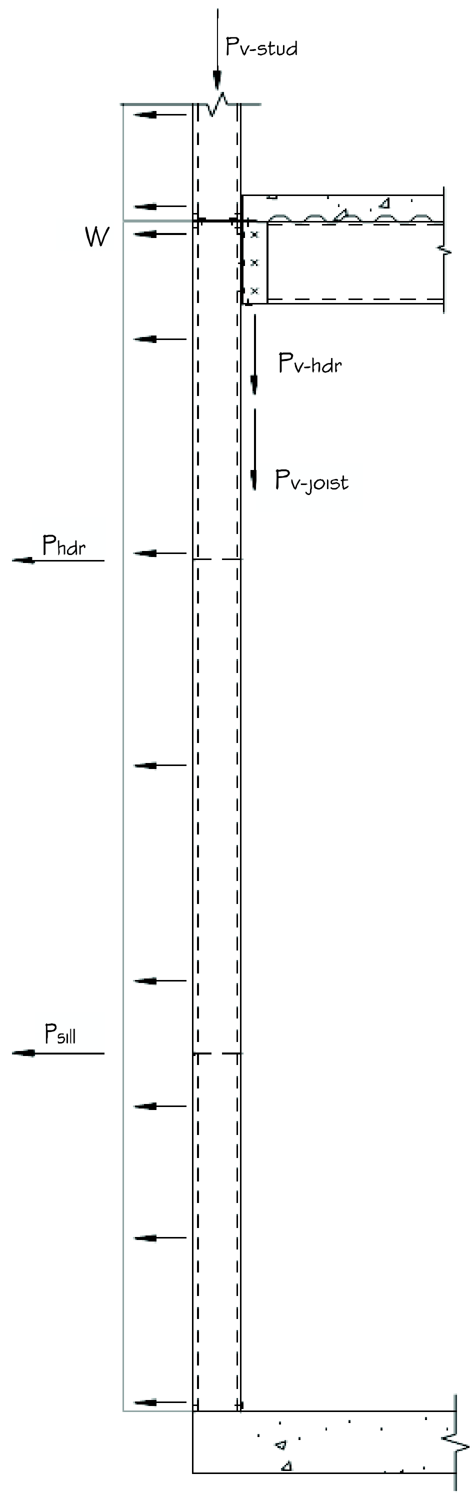

Figure 5 - 8

D + L:

Gravity Loads:

$P_{v\text{-}hdr}$ = 2.79 kips (header vertical reaction)
$P_{v\text{-}stud}$ = 2.00 kips (from stud above jamb)
$P_{v\text{-}joist}$ = 0.79 kips (from joist adjacent to opening)

$\Sigma P_v = 2.79 + 2.00 + 0.79 = 5.58$ kips

See Figure 5-8 for application of loads to the jamb. Both $P_{v\text{-}hdr}$ and $P_{v\text{-}joist}$ are transferred through the inboard stud flange (i.e., from the ledger track). $P_{v\text{-}stud}$ is not applied eccentrically to the jamb. Therefore,

$M_e = (2.79 + 0.79)(2) = 7.16$ in-k

Try Single 400S200-97 Jamb:

From computer analysis following Step 6c:
Maximum Interaction = 0.84 ***OK***
Maximum Deflection = L/1196 (neglecting the 0.7 factor for wind)

Note 5-5

Single jamb studs are sometimes preferable to built-up members due to the cost associated with welding of boxed jambs and the thickness build-up of steel and fasteners associated with stud/track built-up section. Design Example #4 illustrates the design of a stud/track jamb. For this example, a single jamb is contemplated.

The ability of CRC to provide rotational stability to the jamb stud is questionable. In this example, the header and sill provide effective 'blocking' that results in rotational stability. The header and sill are spaced 48" apart. As such, for this example, the jamb is designed for an unbraced length, K_yL_y and K_tL_t of 48". Note that the weak-axis (K_yL_y) bracing requires a complete load path in order to provide bracing as assumed. See Design Example #4, Steps 11 and 12 for additional information.

D + 0.6W:

Gravity Loads:

$P_{v\text{-}hdr}$ = 1.06 kips (header vertical reaction)
$P_{v\text{-}stud}$ = 0.67 kips (from stud above jamb)
$P_{v\text{-}joist}$ = 0.39 kips (from joist adjacent to opening)

$\Sigma P_v = 1.06 + 0.67 + 0.39 = 2.12$ kips

Both $P_{v\text{-}hdr}$ and $P_{v\text{-}joist}$ are transferred through the inboard stud flange (i.e., from the ledger track). $P_{v\text{-}stud}$ is not applied eccentrically to the jamb. Therefore,

$M_e = (1.06 + 0.39)(2) = 2.90$ in-k

<u>Lateral Loads</u>:
$P_{hdr} = 113$ lb (Step 7c)
$P_{sill} = 115$ lb (Step 7c)

Both P_{hdr} and P_{sill} are based on 16″ tributary widths. Therefore, the jamb sees a full 16″ tributary width of wind pressure:

$w = 25(16/12) = 33.3$ lb/ft

Computer analysis for the above loading indicates a maximum total bending moment,

$M = 10.1$ in-k

Try Single 400S200-97 Jamb:

From computer analysis following Step 6c:
Maximum Interaction = 0.56 ***OK***
Maximum Deflection = L/498 (neglecting the 0.7 factor for wind)

<u>D + 0.75(0.6W + L)</u>:

<u>Gravity Loads</u>:
$P_{v\text{-}hdr} = 0.67 + 0.39 + 0.75(1.33 + 0.40) = 2.36$ kips (header vertical reaction)
$P_{v\text{-}stud} = 0.67 + 0.75(1.33) = 1.67$ kips (from stud above jamb)
$P_{v\text{-}joist} = 0.39 + 0.75(0.40) = 0.69$ kips (from joist adjacent to opening)

$\Sigma P_v = 2.36 + 1.67 + 0.69 = 4.72$ kips

Both $P_{v\text{-}hdr}$ and $P_{v\text{-}joist}$ are transferred through the inboard stud flange (i.e., from the ledger track). $P_{v\text{-}stud}$ is not applied eccentrically to the jamb. Therefore,

$M_e = (2.36 + 0.69)(2) = 6.10$

<u>Lateral Loads</u>:
$P_{hdr} = 0.75(113) = 85$ lb (See Step 7c)
$P_{sill} = 0.75(115) = 86$ lb (See Step 7c)

Both P_{hdr} and P_{sill} are based on 16″ tributary widths. Therefore, the jamb sees a full 16″ tributary width of wind pressure:

$w = 0.75(25)(16/12) = 25$ lb/ft

Computer analysis for the above loading indicates a maximum total bending moment,

M = 10.1 in-k

Try Single 400S200-97 Jamb:

From computer analysis following Step 6c:
Maximum Interaction = 0.86 ***OK***
Maximum Deflection = L/511 (neglecting the 0.7 factor for wind)

Therefore, use Single 400S200-97 Jamb.

Step 9(b) – Gravity Header/Jamb Connection

The ledger track accumulates gravity load over the opening that is transferred to the jamb studs as a shear connection. In addition, the jamb top lateral reaction is transferred in fastener tension.

Fastener Design:

D + L:

$V = P_{v\text{-hdr}} + P_{v\text{-joist}}$ from Step 9a
$= 2.79 + 0.79 = 3.58$ kips

T = 0.064 kips [from eccentric moment Me on stud: 3.58(2)/(108+8/2)]

#10 screws in 97-mil jamb and 97-mil track (conservative from previous example):
$V_{all} = 370$ lb; $T_{all} = 109$ lb

Try (10) #10-16 screws for combined shear and tension per AISI S100 Section E4.5.3.1:

Check Shear Dead Alone: 3580/370 = 9.68 ***OK***

$$\frac{3580}{370(10)} + \frac{64}{109(10)} = 1.03 \le 1.15$$ ***OK***

D + 0.6W:

$V = P_{v\text{-hdr}} + P_{v\text{-joist}}$ from Step 9a
$= 1.06 + 0.39 = 1.45$ kips

T = 0.252 kips (from computer analysis, Step 9a)

#10 screws as noted above:

V_{all} = 370 lb; T_{all} = 109 lb

Try (10) #10-16 screw for combined shear and tension per AISI S100 Section E4.5.3.1:

$$\frac{1450}{370(10)} + \frac{252}{109(10)} = 0.62 \le 1.15 \qquad \textbf{\textit{OK}}$$

D + 0.75(0.6W + L):

$V = P_{v\text{-}hdr} + P_{v\text{-}joist}$ from Step 9a

= 2.36 + 0.69 = 3.05 kips

T = 0.155 kips (from computer analysis, Step 9a)

#10 screws in minimum 54-mil jamb and 97-mil track:

V_{all} = 370 lb; T_{all} = 109 lb

Try (10) #10-16 screw for combined shear and tension per AISI S100 Section E4.5.3.1:

$$\frac{3050}{370(10)} + \frac{155}{109(10)} = 0.97 \le 1.15 \qquad \textbf{\textit{OK}}$$

Therefore, use (10) #10-16 screws each of end of ledger track/header to jamb.

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Part II – Beam Design
Part III – Column Design
Part IV – Connection Design
Part V – Supplementary Information
Part VI – Test Procedures

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APPENDIX A
DESIGN VALUES FOR SELF-DRILLING SCREWS AND WELDS

There are a variety of acceptable fasteners for connecting cold-formed steel framing members. This appendix provides design data only for welds and self-drilling screws.

A.1 Welds

The strengths of fillet and flare groove welds are defined in *AISI S100* Sections E2.5 and E2.6. The strength is a function of the weld type, weld length, material thickness, material tensile strength, and the direction of loading.

The design examples in this guide use a simplified conservative approach as follows *(The terms are defined in AISI S100):*

- The allowable strength of all fillet and flare-bevel groove welds irrespective of the length to thickness ratio or direction of loading is set equal to $0.75tLF_u / \Omega$ with $\Omega = 3.05$. This expression is valid for the welding of metallic coated or uncoated material provided the effective throats of welds are not less than the thickness of the thinnest connected part.

- In addition, if $t > 0.10$ inch, the allowable strength determined above shall not exceed $0.75t_wLF_{xx} / \Omega$ with $\Omega = 2.55$.

- For welded connections in which the thickness of the thinnest connected part is greater than 3/16 in., reference is made to the AISC "Specification for Structural Steel Buildings” (AISC 2010)."

In the design examples, the drawings show a nominal weld size of 1/8". Where this approach is used on engineering drawings, it should be accompanied by a note: "For material less than or equal to 0.10" thick, drawings show nominal weld leg sizes. For such material, the effective throat of welds shall not be less than the thickness of the thinnest connected part."

A.2 Self-Drilling Screws

For the purposes of this guide, the design strength of self-drilling screw connections is calculated in accordance with the requirements of *AISI S100* Sections E4 and C2. The relevant subsections are as follows:

- Section E4.3.1 – Tilting and bearing failure modes
- Section E4.3.2 – Shear in the screw itself
- Section E4.4.1 – Pull-out
- Section E4.4.2 – Pull-over
- Section E4.4.3 – Tension in the screw itself.

- Section E4.5.1 - Combined shear and pull-over
- Section E4.5.2 – Combined shear and pull-out
- Section E4.5.3 – Combined shear and tension in screws
- Section C2.2 – Rupture of net section

It is assumed that pull-over, Sections E4.4.2 and E4.5.1, do not govern for typical cold-formed steel framing screwed connections.

The sections covering tension and shear in the screw itself require the use of test values. The ultimate strengths defined in Table A-1 will be used in the design examples.

Table A-1
Self-Drilling Screw Ultimate Strengths

Screw Size	Ultimate Screw Tensile Strength (lb.)	Ultimate Screw Shear Strength (lb.)
8 - 18	1545	1000
10 - 16	1936	1400
10 - 24	2702	1500
12 - 14	2778	2000
12 - 24	3020	2100
1/4 - 14	4060	2600

Note A.1-1

1. *The shear and tensile strengths in Table A-1 have been taken from the 2005 product catalogue by ITW Construction Products for Buildex TEKS self-drilling self-tapping screws and may not be appropriate for other screw types or products from other screw manufacturers. Other screw types are acceptable provided the shear and tensile strengths are available from the manufacturer or from test.*

2. *AISI S100 allows the use of test values in lieu of the design expressions in E4.*

3. *See CFSEI Tech Note F701-12 for additional information regarding screw strength.*

In addition, the design of screwed connections requires the nominal hole diameter. Appropriate values for design are provided in Table A-2 *(taken from reference (AISI 2012b))*.

Table A-2
Nominal Diameters for Screws

Number Designation for Screw	Nominal Diameter (in.)
6	0.138
8	0.164
10	0.190
12	0.216
1/4	0.250

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APPENDIX B
ANCHOR DESIGN VALUES

There are a variety of acceptable fasteners for connecting cold-formed steel framing members to either concrete or steel structures. This appendix provides guidance for three types of anchors: wedge type expansion anchors (concrete), self-tapping concrete screw anchors (concrete) and low velocity pins (concrete and steel).

B.1 Wedge and Screw Type Concrete Anchors

The determination of the strength of wedge and screw type concrete anchors is generally governed by ACI 318-11, Appendix D. The required calculations are complex and outside the scope of AISI D110.

Wedge and screw type anchors are almost exclusively designed using software developed by the anchor manufacturer. Critical inputs include:

- Concrete type (normal weight or light weight)
- Concrete strength, f_c'
- Anchor plate geometry
- Anchor layout (i.e., spacing in each direction)
- Anchor edge distance
- Load magnitudes for tension and shear forces, including any prying forces
- Special inputs for seismic loads
- Anchor type and size

Based on user inputs, the software checks critical spacing and edge distances, anchor shear and tension capacity, and any appropriate shear + tension interactions.

In addition to the software, anchors typically have evaluation reports that may have additional requirements relative to installation and inspection. It is important that all pertinent requirements be understood by the engineer and communicated to the builder via the construction documents.

B.2 Power-Actuated Fasteners Into Concrete

The design values in this appendix have been taken from ICC Evaluation Service Report No. ESR-2269 (issued December 1, 2012). The design values are for Hilti X-U low-velocity power-actuated fasteners (PAFs) and will not be appropriate for other PAF types or PAFs of similar type by other manufacturers. Other PAF types are acceptable provided the design values are available from the manufacturer or from test.

Only a part of the design data is included here and the user is referred to the report for additional information such as detail on the fasteners themselves, installation, other fastener types, identification requirements and limits of applicability for the design values.

Table B.2-1
Power-Actuated Fasteners
Allowable Tension and Shear Values
Driven Into Normal Weight Uncracked Concrete

Fastener Type & Diameter (in.)	Embed-ment Depth (in.)	f_c' = 2000 psi		f_c' = 4000 psi		f_c' = 6000 psi	
		Tension (lb.)	Shear (lb.)	Tension (lb.)	Shear (lb.)	Tension (lb.)	Shear (lb.)
X-U *0.157 (shank)* *0.32 (head)*	3/4	100	125	100	125	105	205
	1	165	190	170	225	110	280
	1-1/4	240	310	280	310	180	425
	1-1/2	275	420	325	420	-	-

Notes B.2

These notes apply to Table B.2-1:

1. The tabulated allowable loads utilize a factor of safety that is greater than or equal to 5.

2. Minimum edge distance is 3 inches and minimum center-to-center spacing is 4 inches.

3. For combined tension and shear:

$$\left(\frac{P}{P_a}\right)+\left(\frac{V}{V_a}\right)\leq 1.0$$

where:
P = applied tension load
V = applied shear load
P_a = allowable tension load
V_a = allowable shear load

4. Uncracked concrete is assumed.

5. The use of fasteners in concrete to resist seismic loads is outside the scope of ICC ESR-2269 except for use with certain nonstructural components and with strict limitations on allowable loads. See ICC ESR 2269 for more information.

6. Deeper fastener embedments of 1-1/4 inch and 1-1/2 inch may require specific Power-Actuated Tool (PAT) types and cartridge booster settings. Consult with the manufacturer.

B.3 Power-Actuated Fasteners Into Steel

Common practice is to use ICC-ESR values for PAF capacity, edge distance and spacing requirements. As such, that method is followed in the examples herein. The design values in this appendix have been taken from ICC Evaluation Service Report ESR-2269 (issued December 1, 2012). The design values are for Hilti X-U knurled low-velocity power-actuated fasteners (PAFs) and will not be appropriate for other PAF types or PAFs of similar type by other manufacturers. Other PAF types are acceptable provided the design values are available from the manufacturer or from test. *(Note that the test data for power-actuated fasteners into steel can be transposed into allowable loads using AISI S100 Section F. This AISI approach can be used in lieu of the allowable loads provided in the ICC reports.)*

Only a part of the design data is included here and the user is referred to the report for additional information such as detail on the fasteners themselves, installation, inspection, other fastener types, identification requirements and limits of applicability for the design values.

Table B.3-1
Power-Actuated Fasteners
Allowable Tension and Shear Values
Driven Into Steel

Fastener Type & Diameter (in.)	Steel Thickness (in.)	Tension (lb.)	Shear (lb.)
X-U *0.157 (shank)* *0.32 (head)*	3/16	500	720
	1/4	775	720
	3/8	935	720
	1/2	900	720
	3/4	350 [6] / 275 [7]	375 [6] / 350 [7]

Notes B.3:

These notes apply to Table B.3-1:

1. The tabulated allowable loads utilize a factor of safety that is greater than or equal to 5.
2. Base Steel Requirements:

   For X-U fasteners:
   Base material must comply with the minimum requirements of ASTM A36.
3. Minimum edge distance is 1/2 inch and minimum center-to-center spacing is 1 inch.
4. For combined tension and shear:

$$\left(\frac{p}{P_a}\right) + \left(\frac{V}{V_a}\right) \le 1.0$$

where:
p = applied tension load
V = applied shear load
P_a = allowable tension load
V_a = allowable shear load

5. The use of fasteners to resist earthquake loads is outside the scope of ICC ESR-2269.

6. Allowable load for 1/2 inch embedment in 3/4 inch steel.

7. Allowable load for 3/8 inch embedment in 3/4 inch steel.

AISI S100 Section E5 may also be used to determine the capacity of PAFs in concrete and steel. Provisions for tension strength, pull-out strength, pull-over strength, shear strength, bearing and tilting strength, pull-out strength in shear and net section rupture strength are included. Minimum spacing and edge distance requirements are also provided.

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APPENDIX C
SIMPLIFIED APPROXIMATE METHOD FOR THE CALCULATION OF WARPING TORSIONAL STRESSES

Winter 1950 outlines an approximate method for the calculation of warping torsional stresses. The following has been taken directly from the original paper with the exception of some added comments in brackets and minor simplifications in the algebra.

"... The performance of a channel loaded in the plane of the web *(and therefore eccentric with respect to the shear center)* can be visualized most simply by thinking of a single load P at mid-span *(of a single span beam)* and considering the displacement of the mid-span section as proceeding in successive stages depicted in Figure C-1. The section, then, is thought of as being first displaced downward in simple translation *(with the load P through the shear center)*. The stresses introduced would be those of simple beam theory and are indicated in character by the appropriate signs at the corners of the section (Fig. C-1b). Next, the channel is considered as cut and the two halves displaced *(by the forces F, where Fh = Pe)* much like two individual beams resulting in the appropriate indicated corner stresses (Fig. C-1c). To fit the two halves together they are next rotated about their individual shear centers, giving rise to ordinary shear stresses of the St. Venant character (Fig. C-1d). In this inclined position, finally, the component of the vertical load parallel to the major axis, βP, causes additional bending about the minor axis, with its corresponding normal stresses (Fig. C-1e)...... It is evident that under such a stress distribution cross sections distort out of their original plane; for this reason the stresses associated, in particular, with the displacement stage (c) of Figure C-1 are generally known as warping stresses......

(By comparison with a more precise theoretical model, it was demonstrated that warping torsional stresses could be predicted with reasonable accuracy by adding the stresses for Figures C-1b and C-1c only. In addition, the term $\beta h/2$ was found to be small, such that F could be approximated by Pe/h. Lastly, the analysis was extended to channels with intermediate braces as depicted in Figure C-2.)

... The action of intermediate braces is now easily visualized. It prevents horizontal displacement of the fictitious half-beams at the points of bracing; consequently, these half-beams are converted from simple beams of span length equal to that of the entire channel to continuous beams with individual spans equal to the distances between braces *(L versus l_0 in Figure C-2 for the particular case of bracing at third points)*. The resulting maximum horizontal bending moment on the 'half-beam' and the corresponding stresses of Figure C-1c are less than one quarter of those obtained without bracing, as can be verified easily by continuous beam analysis ...

For horizontal bending the cross section of each half-beam is regarded as consisting of the flange, lip and one quarter of the web. *(The one-quarter web assumption was verified by comparison with a more accurate theoretical model.)* This beam is loaded horizontally at all points where vertical loads P act on the channel by the corresponding horizontal loads F

= Pe/h ... For distributed vertical load p the corresponding distributed horizontal load, of course, is f = pe/h. Each half-beam so loaded represents a continuous beam supported at the braces, as shown for one particular case in Figure C-2. Stresses from this horizontal bending (Figure C-3b) are computed in the usual manner and superimposed on those from vertical bending (Figure C-3a) to result in the maximum corner stresses. *(For a sample calculation, see Design Example #2.)*

... For design purposes, it is now possible to take one of two positions. Conservatively, one can stipulate that the maximum corner stress shall not exceed the yield point ... Here the use of a single channel with discrete bracing will always be less economical than one with continuous bracing since the corner stress in the former always exceeds that of the latter for the same load. This difference decreases with decreasing spacing of braces. Alternatively, one can take advantage of the reserve strength by plastic stress redistribution ...*(such that)* the difference between the maximum corner stress *(due to warping and simple bending superimposed)* shall not exceed a specified fraction of ...*(the maximum stress due to simple bending alone)*. This fraction must be so specified that it shall not adversely affect the carrying capacity i.e. such that its effect would be obliterated by plastic redistribution.

On the basis of the experimental evidence *(a series of tests were run as part of this study)*, it is seen that a 15% overstress does not affect the carrying capacity of the channels significantly... It would seem, therefore, that within the limits of our test evidence, a theoretical overstress of about 15% can be disregarded in practical design. The problem then, merely, to locate braces such that no more than this overstress will occur.

Note C-1

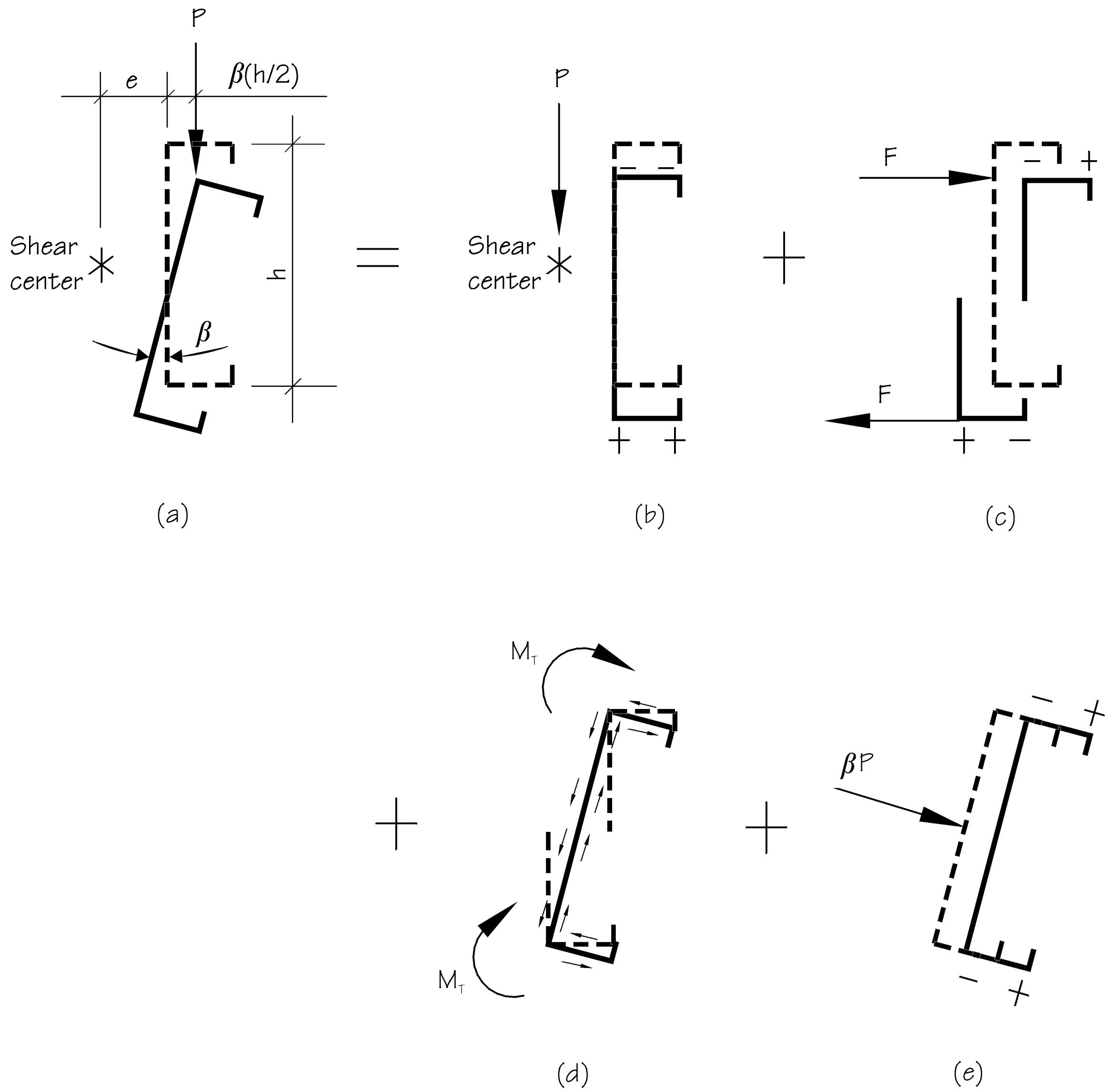
1. *For the design examples in this document, the 15% overstress has not been utilized and the corner stress has been limited to the yield stress. This approach has been taken because:*
   - *The tests that were part of this study only included the case where maximum additive compressive stress due to warping and bending occurred at the flange/web junction. Other studies (Bogdan 1999) indicate that the lip/flange case is more critical and the 15% allowance may not be justified.*
   - *The effect of web punchouts on the torsional strength of the stud is not well understood.*
   - *Some of the bridging details used in standard stud construction are somewhat flexible and allow some twisting to occur at the bridging points. This twist may magnify the warping torsional stresses.*
   - *There is some interaction between lateral instability and warping torsion not accounted for in this procedure.*
   - *In the design examples, the procedure has been extended to loading cases not confirmed by testing.*

2. *AISI S100 Section C3.6 allows the factor R to be increased by 15% if the maximum combined stress occurs at the web/flange junction. However, the value of R cannot exceed 1.0.*

Note C-2

More accurate methods for calculating warping torsional stresses are available. Refer to Seaburg 1997 and Moore 2002. These references contain the solution to 12 torsional loading cases but require torsional section properties for the cross-section, including J, C_w and W_{ns}. W_{ns} is the normalized warping function at point s on the cross-section and is not usually available in published load tables for lightweight steel framing members. Refer to Galambos 1968 for a method of calculation.

Some commercially available software will calculate the required section properties as well as torsional stresses and the factor R from AISI S100 Section C3.6. It should be noted that while the calculations are rigorous, they are based on a limited set of loading and support conditions that may only approximate the actual physical conditions of a given design.



$Fh + 2M_T = P(e + \beta h/2)$

- is compression
+ is tension

FIGURE C-1

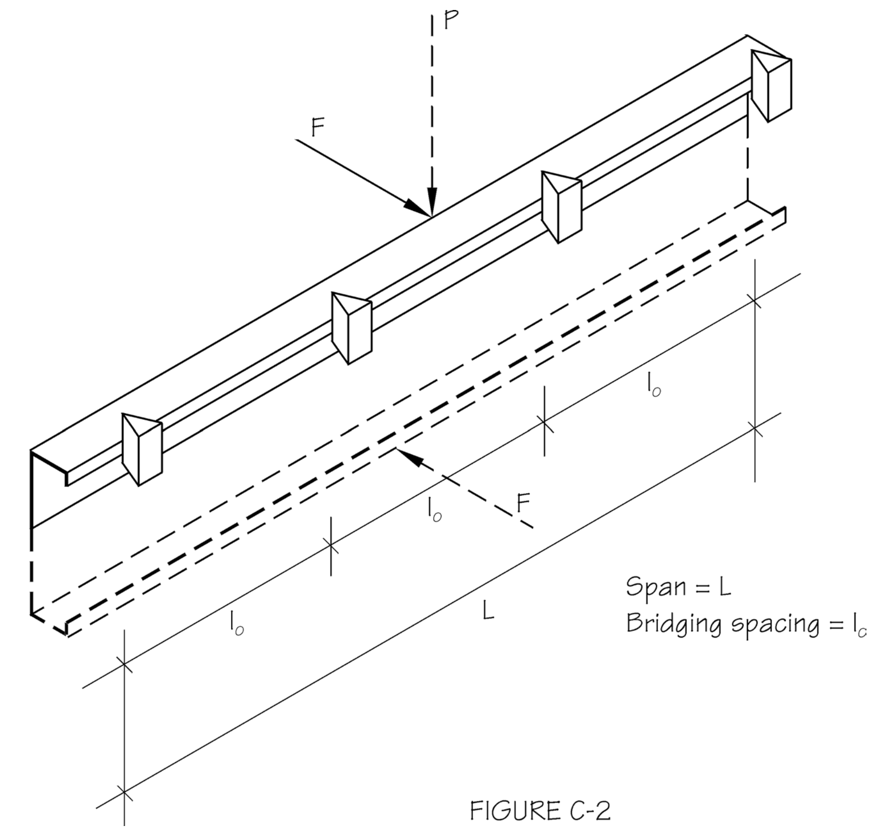


FIGURE C-2

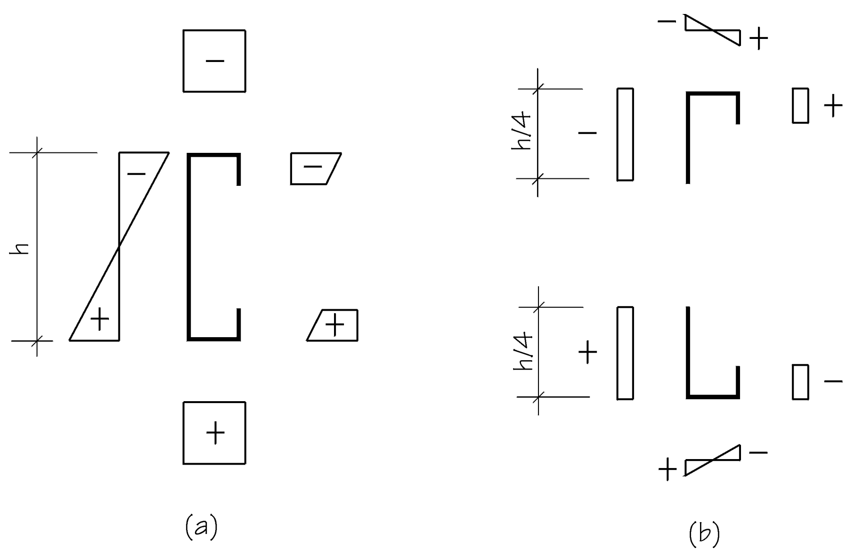


FIGURE C-3

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APPENDIX D
OUTER TOP TRACK FLEXIBILITY FORMULAS

To connect wind bearing studs to the structure, inner and outer top track details are useful for accommodating floor deflections due to gravity loads, building lateral drift due to wind or seismic loads, and construction tolerances.

The detail is, however, inherently flexible. Some horizontal movement occurs in the track whenever the studs are loaded by wind or seismic forces.

The following approximate formulas provide a lower bound estimate of the movement to be expected assuming uniform loading along the length of the track. Local deformations in the vicinity of fasteners and overall torsional deformations between fastener locations have been neglected. See also Appendix E where it is shown that localized increases in deflection can occur due to discontinuities in the inner top track and due to locally heavily loaded studs (such as jambs).

Two different fastening conditions have been examined:

Figure D-1: Outer top track to concrete with an expansion anchor
Figure D-2: Outer top track welded to a steel beam

For both figures:

P = horizontal load from inner top track
L_2 = maximum gap

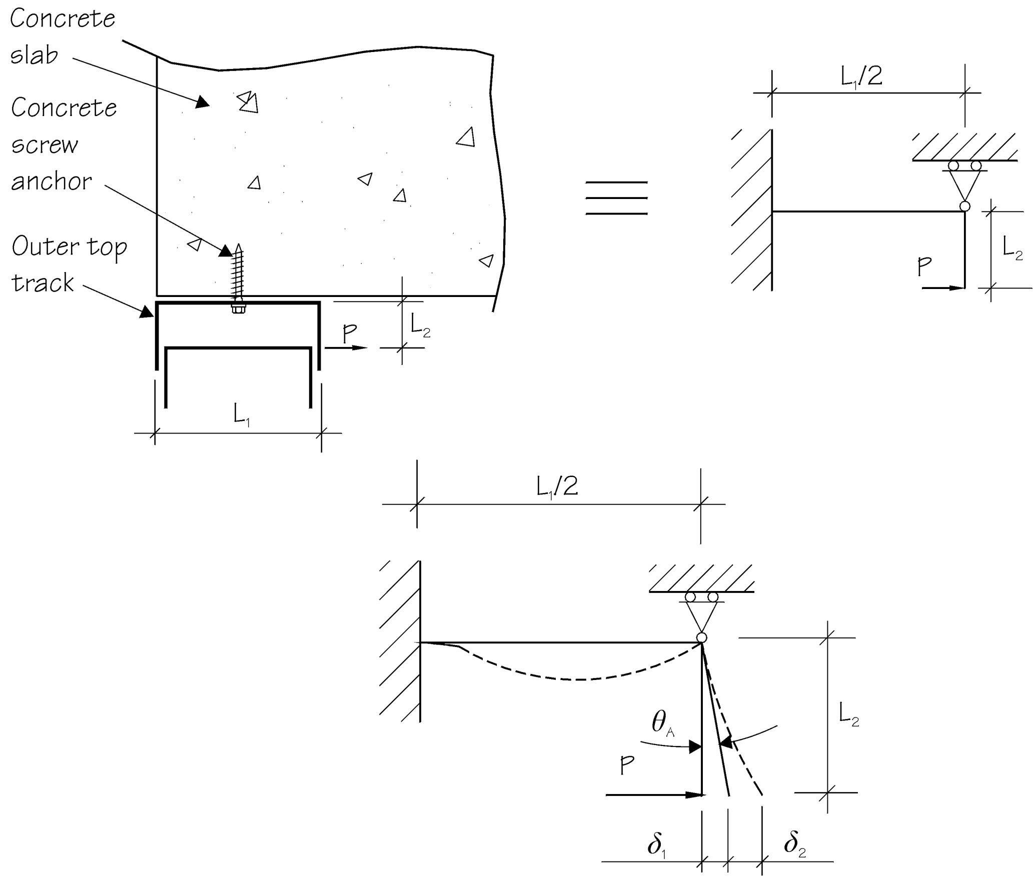

FIGURE D-1

From Figure D-1:

$$\begin{aligned}\delta_{total} &= \delta_1 + \delta_2 \\ &= \theta_A L_2 + \delta_2 \\ &= \frac{PL_2^2 L_1}{8EI} + \frac{PL_2^3}{3EI} \\ &= \frac{P}{EI}\left(\frac{L_2^2 L_1}{8} + \frac{L_2^3}{3}\right)\end{aligned}$$

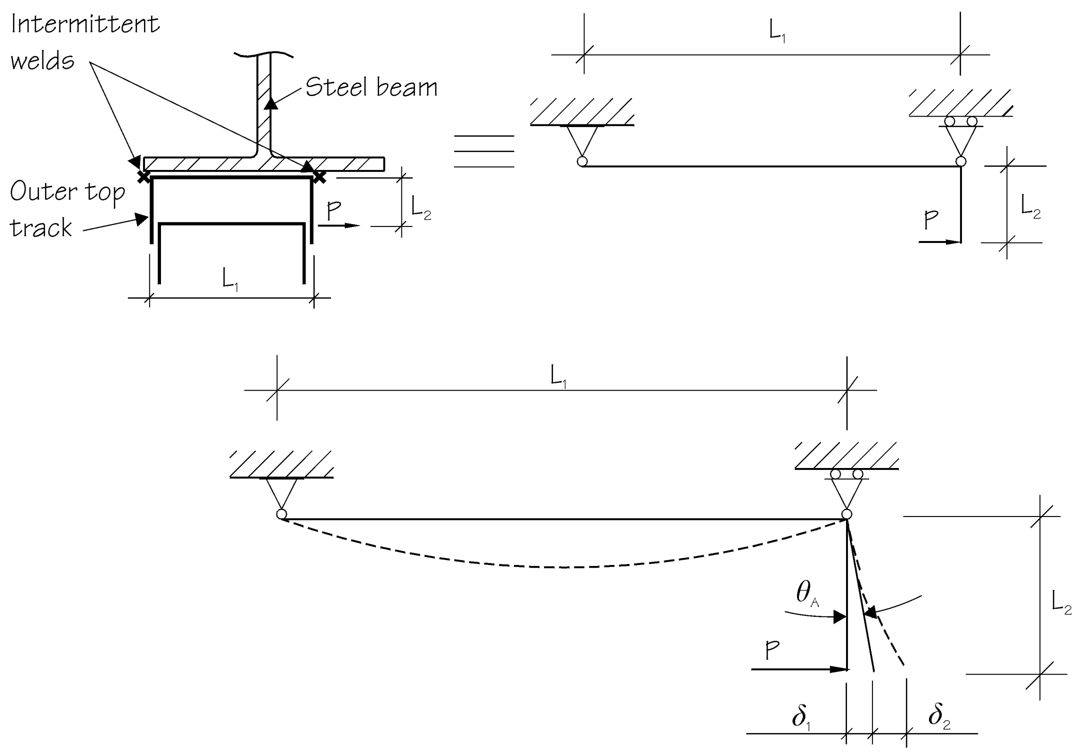

FIGURE D-2

From Figure D-2:

$$
\begin{aligned}
\delta_{total} &= \delta_1 + \delta_2 \\
&= \theta_A L_2 + \delta_2 \\
&= \frac{PL_2^2 L_1}{3\,EI} + \frac{PL_2^3}{3\,EI} \\
&= \frac{P}{3\,EI}\left(L_2^2 L_1 + L_2^3\right)
\end{aligned}
$$

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APPENDIX E
INNER TOP TRACK AS A BEAM ON AN ELASTIC FOUNDATION

The outer top track is typically designed as if uniformly loaded by the inner top track. The validity of this assumption can be reviewed by treating the inner top track as a beam supported by the outer top track which in turn functions as an elastic foundation.

While it may seem intuitively obvious that the inner top track will effectively distribute loads from typical studs spaced at 16" or 24" o.c., it is not so clear that large reactions from window jamb studs will be effectively distributed. A further complication is the case of buildings with short pieces of stud wall interrupted by full-height windows and shear walls. This condition is common in condominium type projects.

The basic equations for finite length beams on elastic foundations are taken from Roark 1975:

$\beta = (k/4EI)^{1/4}$

$C_2 = \cosh \beta L \sin \beta L + \sinh \beta L \cos \beta L$

$C_3 = \sinh \beta L \sin \beta L$

$C_4 = \cosh \beta L \sin \beta L - \sinh \beta L \cos \beta L$

$C_{11} = \sinh^2 \beta L - \sin^2 \beta L$

$C_{A1} = \cosh \beta(L - a) \cos \beta(L - a)$

$C_{A2} = \cosh \beta(L - a) \sin \beta(L - a) + \sinh \beta(L - a) \cos \beta(L - a)$

$F_2 = \cosh \beta x \sin \beta x + \sinh \beta x \cos \beta x$

$F_1 = \cosh \beta x \cos \beta x$

$F_{A4} = \cosh \beta(x - a) \sin \beta(x - a) - \sinh \beta(x - a) \cos \beta(x - a)$

If $x \leq a$ then $F_{A4} = 0$

$$\theta_A = \frac{W}{2EI\beta^2} \frac{C_2 C_{A2} - 2C_3 C_{A1}}{C_{11}}$$

$$y_A = \frac{W}{2EI\beta^3} \frac{C_4 C_{A1} - C_3 C_{A2}}{C_{11}}$$

y = local horizontal deflection in outer top track leg

$$= y_A F_1 + \frac{\theta_A F_2}{2\beta} - \frac{W F_{A4}}{4EI\beta^3}$$

where:
L = Beam length, inches
a = Distance from left end to point load, inches
x = Distance from left end to deflection location, inches
k = Spring constant for outer top track, lbs/inch per inch of deflection
I = Inner top track major axis beam inertia, $inches^4$
W = Point load, lbs

Example E1

Check the inner and outer top track design from Design Example #1 as shown in Figure E-1.

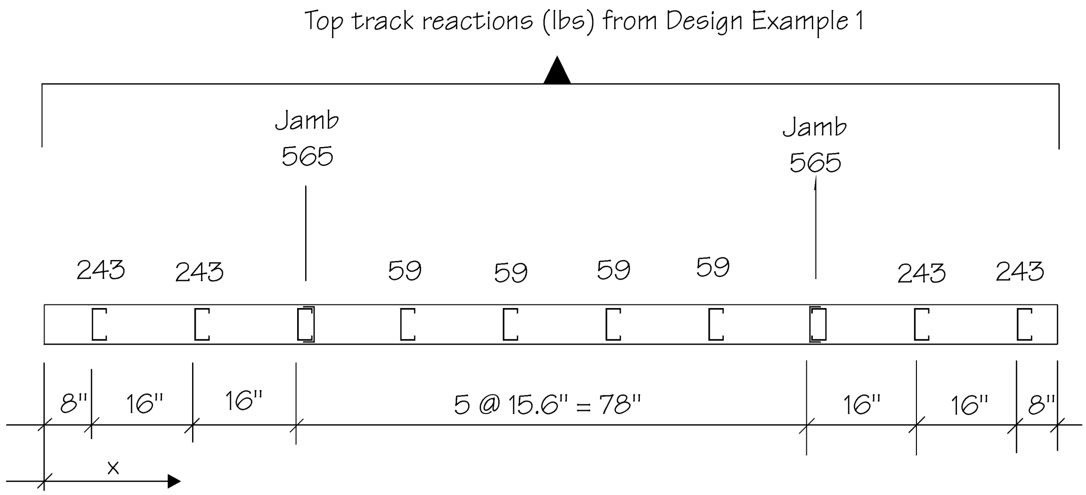


FIGURE E-1

Assume inner top track length = 158"

Approximate inertia of long-legged inner top track by using the deflection inertia for 600T125-43 track section with F_y = 33 ksi:

$I_{x(def)} = 1.75\ in^4 \pm$

From Appendix D *(Figure D-1)* for outer top track leg:

$$\delta_{total} = \frac{P}{EI}\left(\frac{L_2^2 L_1}{8} + \frac{L_2^3}{3}\right)$$

The spring constant k @ δ = 1" is given by:

$$k = \frac{EI}{\frac{L_2^2 L_1}{8} + \frac{L_2^3}{3}}$$

$$= \frac{24EI}{3L_2^2 L_1 + 8L_2^3}$$

For t = 0.0713" outer top track:

L_1 = 6"
L_2 = 1.5"
I = (1/12)bt^3
= (1/12)(1)$(0.0713)^3$
= 30.21 x 10^{-6} in^4/in
E = 29.5 x 10^6 psi

Substituting and solving for k:

k = 316.9 lb/in per inch of deflection

See Table E-1 for calculations of outer top track horizontal local deflections at x = 0" and at x = 40" due to stud reactions. *(These deflections are found by solving for y = local horizontal deflection in outer top track leg using the formulas on Page E-1.)*

Check βL

βL = 0.0352(158) = 5.6 < 6.0 ***OK***

Note E-1

1. *Roark 1975 restricts the beam on an elastic foundation to βL ≤ 6 because of potential round-off errors when two nearly equal large numbers are subtracted.*
2. *If programmed on a computer, the equations are easy to use. Double precision calculations will extend the βL ≤ 6 limit somewhat. See Roark 1975 for alternative equations when βL > 6.*
3. *Theory has not been confirmed by test.*

Table E-1			
Stud Reaction W		$\delta_{(horizontal)}$ (inches)	
Location a (in)	Magnitude (lb)	@ x = 0"	@ x = 40" (at jamb stud)
8	243	-0.0391	-0.0052
24	243	-0.0154	-0.0110
40	565	-0.0050	-0.0328
55.6	59	+0.0007	-0.0027
71.2	59	+0.0009	-0.0015
86.8	59	+0.0006	-0.0006
102.4	59	+0.0003	-0.0001
118	565	+0.0011	+0.0012
134	243	-0.0001	+0.0007
150	243	-0.0004	+0.0005
	Σ	-0.0564	-0.0515

δ = 0.0564 inches at x = 0 inches governs for the point-loaded beam (the inner top track) on an elastic foundation (the outer top track).

For a uniform load on the outer top track as a cantilever:
w = 6.5(28)/12 = 15.17 lb/in
δ = w/k = 15.17/316.9 = 0.0479 in.

Then:

$$\frac{\delta_{(\text{outer top track as an elastic foundation})}}{\delta_{(\text{outer top track as a uniformly loaded cantilever})}} = \frac{0.0564}{0.0479} = 1.18$$

Note that stresses in the cantilevering outer top track leg also increase (locally) by a factor of 1.18.

Example E2

Repeat Example E1 except with the inner top track cut-off 1" to the left of the jamb. See Figure E-2.

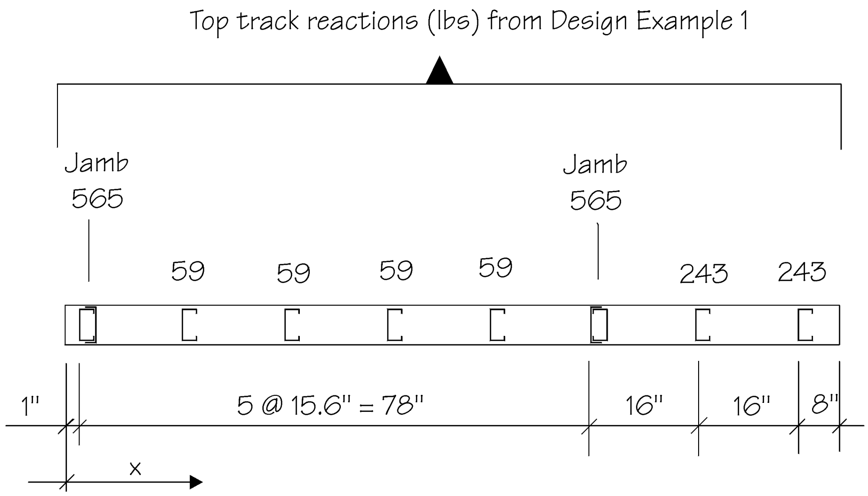


FIGURE E-2

Table E-2			
Stud Reaction W		$\delta_{(horizontal)}$ (inches)	
Location a (in)	Magnitude (lb)	@ x = 0"	@ x = 16"
1.0	565	-0.1212	-0.0596
16.6	59	-0.0061	-0.0048
32.2	59	-0.0018	-0.0029
47.8	59	+0.0003	-0.0013
63.4	59	+0.0009	-0.0003
79.0	565	+0.0076	+0.0016
95.0	243	+0.0018	+0.0011
111.0	243	+0.0002	+0.0011
	Σ	-0.1183	-0.0651

For this case at x = 0"

$$\frac{\delta_{(\text{outer top track as an elastic foundation})}}{\delta_{(\text{outer top track as a uniformly loaded cantilever})}} = \frac{0.1183}{0.0479} = 2.47$$

and at x = 16"

$$\frac{\delta_{(\text{outer top track as an elastic foundation})}}{\delta_{(\text{outer top track as a uniformly loaded cantilever})}} = \frac{0.0651}{0.0479} = 1.36$$

This example illustrates the locally high outer top track deflections (and stresses) that can develop if the inner top track joint occurs near a heavily loaded stud. For this case, the deflections (and stresses) will be 2.47 times those from the simple uniformly loaded assumption. Note, however, that the stresses will be localized and that at a distance of 16" from the end of the inner top track, the ratio has dropped to 1.36. It is likely that the high overstress implied by the 2.47 ratio will be alleviated somewhat by plastic redistribution.

See also Note 1-5 from Design Example #1 for a discussion of plastic versus elastic section modulus when checking the strength of the outer top track leg.

Conclusions

1. The outer top track is subject to locally high stresses.

2. These locally high stresses are greater where the inner top track joint occurs near a heavily loaded stud.

APPENDIX F
BEARING STRESS DISTRIBUTION BETWEEN TRACK AND CONCRETE FOR AXIAL LOAD BEARING STUDS

The bearing stress distribution between the track and concrete for axial load bearing studs has not been researched with the exception of some preliminary testing at the University of Manitoba. *(A summary of this work has been published – see LGSEA 2001b).* This appendix proposes a method for calculating the bearing area that should be considered an approximation only. See Figure F-1.

The allowable bearing stress on concrete is taken from AISC 360 *(AISC 2010),* Section J8. Allowable bearing stress = $0.85f_c'/\Omega$

$= 0.85f_c'/2.50$

$= 0.34f_c'$

> *Note F-1*
>
> *Where the ratio of bearing area to the area of the concrete support is less than 1, a higher allowable bearing stress may be permitted. Refer to the relevant concrete specification.*

The width of track that can cantilever beyond the face of the stud is shown on Figure F-1 as "x" and is calculated as follows:

$$M_{req} = \frac{0.34f_c'x^2}{2}$$

$$M_{all} = ZF_y/\Omega$$

where:

Z = plastic section modulus

$= (1/4)bt_t^2$ with b = 1"

$\Omega = 1.67$

Set $M_{req} = M_{all}$ and solve for x

Gives:

$$x = 0.938t_t\sqrt{\frac{F_y}{f_c'}}$$

Then from Figure F-1:

$$A_{brg} = (B + 2x)(C + x)(2) + [A - 2(C + x)][t_s + 2x]$$

Note F-2

Among other approximations, this bearing area calculation does not take into account the beneficial effect of the flange of the track nor does it account for the detrimental influence of local buckling in the web of the stud.

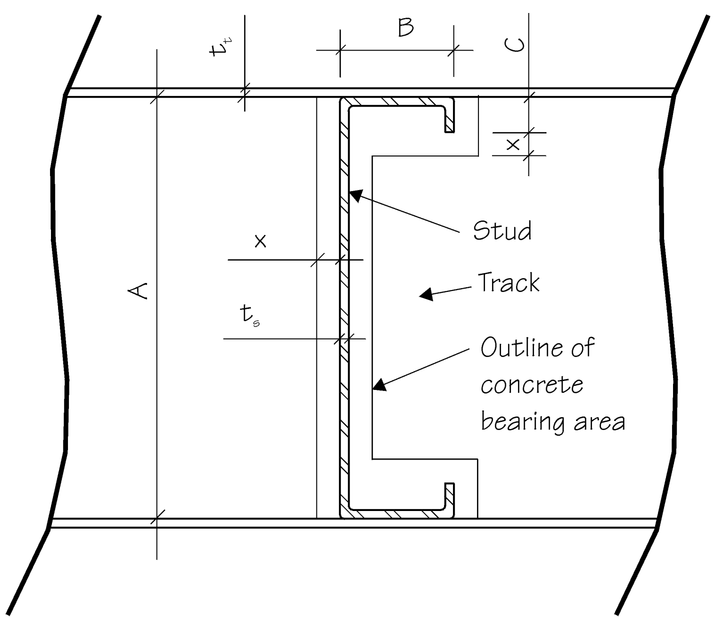
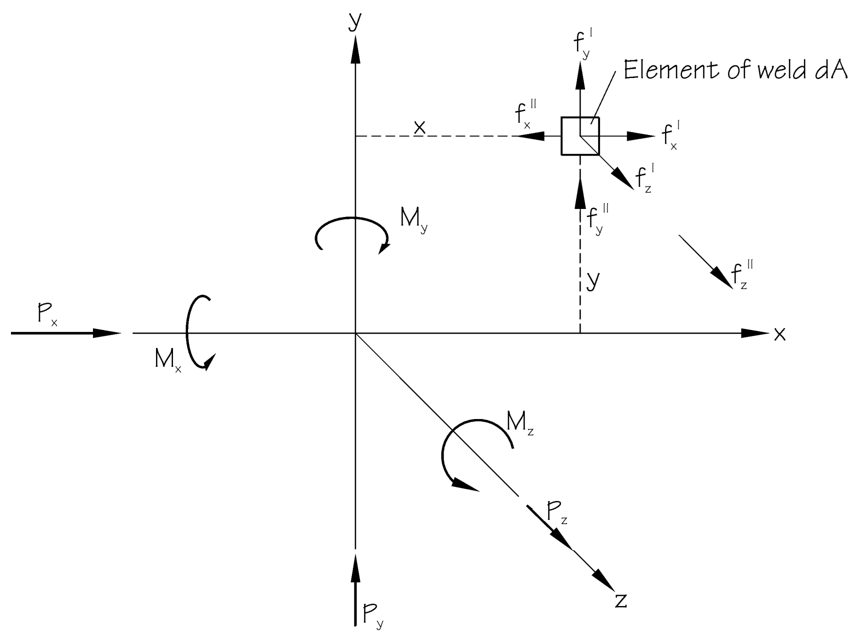


FIGURE F-1

APPENDIX G
GENERAL METHOD FOR DETERMINING STRESSES IN WELDED CONNECTIONS

The following method is taken from (*Bresler 1967)* except that the sign of M_y has been revised to conform to the usual convention for positive moments.



Stress Components on Weld Element

FIGURE G-1

At any point of the connection, the stress on the weld due to one single component of load can be computed from the conventional formulas (Equations 1, 2 and 3). In Figure G-1, the notation shows f_x and f_y as shearing stresses and f_z as normal stress.

Due to forces:

$$f_x^I = \frac{P_x}{A}, \qquad f_y^I = \frac{P_y}{A}, \qquad f_z^I = \frac{P_z}{A} \qquad (1)$$

Due to moments:

$$f_x^{II} = \frac{M_z}{I_z} y, \qquad f_y^{II} = \frac{M_z}{I_z} x, \qquad f_z^{II} = \frac{M_x}{I_x} y - \frac{M_y}{I_y} x \qquad (2)$$

where:

$$A = \int dA \qquad I_x = \int y^2 dA \qquad I_y = \int x^2 dA$$

and:

$$I_z = \int (x^2 + y^2)\, dA = I_x + I_y$$

Resultant components of stress with due regard to signs:

$$f_x = f_x^{I} - f_x^{II}, \qquad f_y = f_y^{I} + f_y^{II}, \qquad f_z = f_z^{I} + f_z^{II} \qquad (3)$$

For fillet welds, x, y, and z components of stress on a given leg of the weld are used to determine q_{req}, the maximum required resultant shear force per unit length of weld, and the latter is arbitrarily considered a "shear" force acting on the throat section as follows:

$$q_{req} = t f = t \sqrt{f_x^2 + f_y^2 + f_z^2}$$

where t is the effective throat dimension.

For welded connections with welds of uniform size, calculations may be simplified by considering t = 1 and computing q_{req} values directly without calculating stresses. In this method, all loads acting on a fillet weld are considered as shears, independent of their actual direction.

See Design Example No. 3, Steps 7(h) and 7(i) for a worked example using this approach.

APPENDIX H
SIMPLIFIED CONSERVATIVE DESIGN APPROACH FOR EQUAL LEG ANGLES WITHOUT LIPS

This appendix proposes a simplified method for calculating the axial capacity of equal leg angles without lips.

It is proposed to restrict compressive stresses such that local buckling does not occur either due to axial load or moment. This approach will substantially underestimate the true capacity of angles, particularly when the flat width to thickness ratio of the unstiffened flanges is large. However, where efficient use of material is less important than efficient use of a designer's time, this approach is useful.

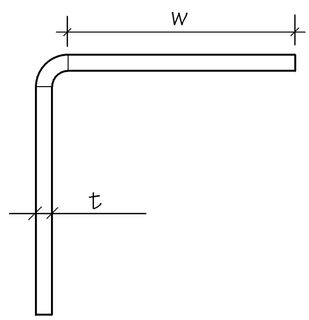


FIGURE H-1

From *AISI S100* Section B2.1:

$\lambda \leq 0.673$ for fully effective behavior (i.e. no local buckling)

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \leq 0.673$$

Substitute into the expression for λ the following:

$$F_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2$$

$\mu = 0.3$
$E = 29500$ ksi
$k = 0.43$

and setting equal to 0.673 and reworking gives:

$$f = \frac{5190}{(w/t)^2} \text{ ksi}$$

Thus, if bending and axial stresses are restricted to f, then local buckling can be neglected. Overall stability of the angle must, of course, still be checked.

APPENDIX I
REACTION FORCES AT END OF STUD

Figure I-1 is a free body diagram of a short piece of stud at the end support. An all-steel design approach is assumed – that is, sheathings are assumed to provide no torsional restraint to the studs. This free body diagram is appropriate for designing the restraint required for the end of the stud in order to transfer the stud end shear and torsion.

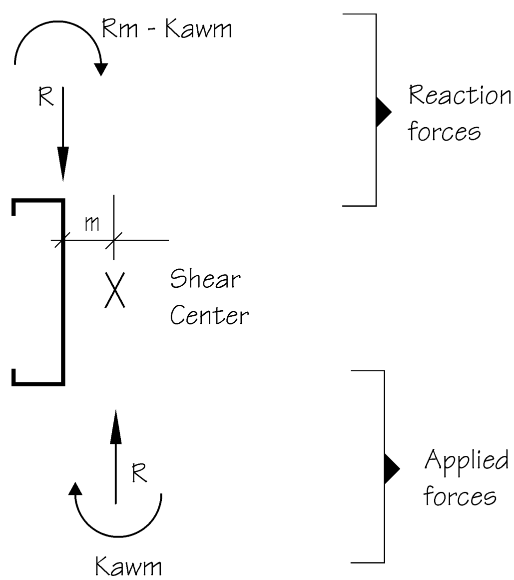


FIGURE I-1

<u>The applied forces consist of:</u>

- The resultant internal shear, *R*, at the end of the stud with a line of action through the stud shear center.
- The accumulated torsion between the end reaction and the first line of bridging given by *Kawm* with:
  - *a* = distance between the end reaction and the first line of bridging
  - *w* = wind load/unit length assumed to be applied through the web of the stud
  - *m* = distance from the center line of the stud web to the shear center
  - *K* = coefficient considering force distribution at supports. *(Where the accumulated torsion Kawm relieves the internal connection stresses, it is conservative to underestimate the value for the constant, K. AISI S100 Section*

D3.2.1 uses K = 1.5 for intermediate torsional brace points. A value of K = 0.50/1.5 = 0.33 at the end reaction would be a conservatively low assumption consistent with the conservatively high 1.5 value for an interior line of bridging. Alternatively, where the accumulated torsion adds to the internal connection stresses, K = 1.5(0.50) = 0.75 would be appropriate.)

<u>The reaction forces (and the forces applied to the end connection) consist of:</u>

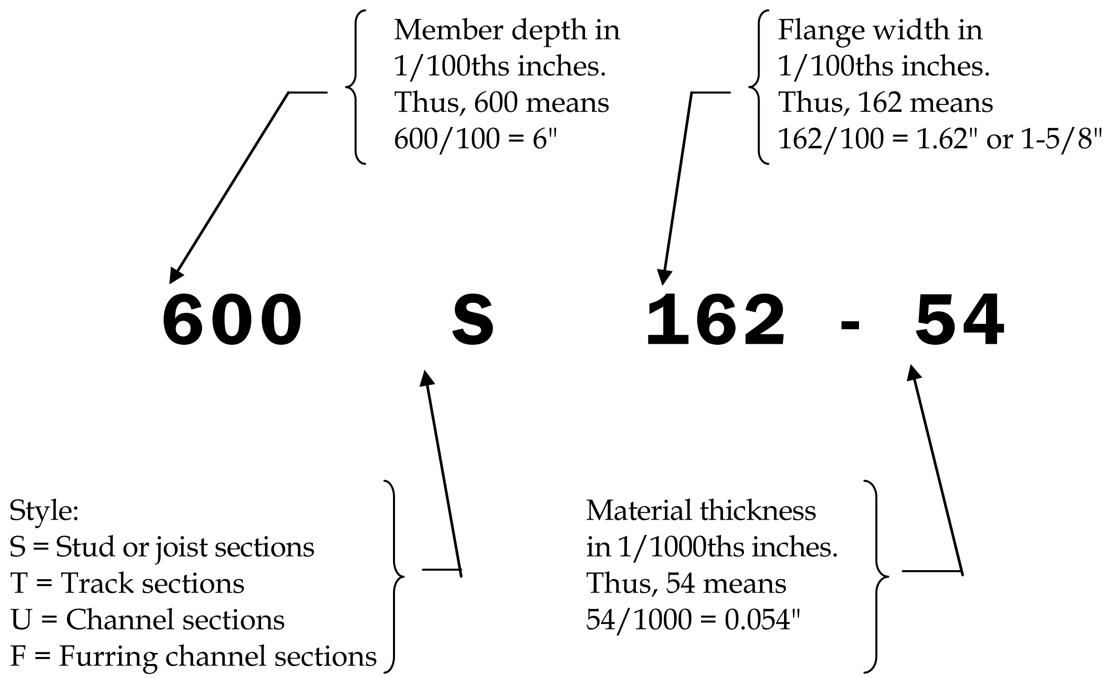
- The reaction force, *R*, which is assumed to be applied along the line of the stud web.
- The moment, *Rm - Kawm*, which is required for equilibrium.

Note that for continuous stud applications, the restraint force Rm - Kawm also applies at interior reaction points. Once again, where the torsional component Kawm relieves the internal connection stresses, it is conservative to underestimate the value of the constant, K. For this case, a value of K = 1.00/1.5 = 0.67 at the interior reaction would be a conservatively low assumption consistent with the conservatively high 1.5 value (AISI S100 Section D3.2.1) for an interior line of bridging.

APPENDIX J
PRODUCT IDENTIFICATION

AISI S201 *(AISI 2012c)* requires that cold-formed steel framing manufacturers use a universal designator system for their products. The designator is a four part code which identifies depth, flange width, member type and material thickness.

Example: 600S162-54



Notes:

1. *The designator remains the same in imperial and metric.*

2. *Material thickness is given as the minimum thickness exclusive of coatings and represents 95% of the design thickness. See AISI S100 Section A2.4.*

3. *For sections with a yield strength other than 33 ksi, the yield strength used in design needs to be identified on the contractual documents and when ordering the steel. [e.g., "600S162-54 (50 ksi)" for 50 ksi yield material. "600S162-54 (50)" is also acceptable.]*

4. *For track, "T", sections, depth is a nominal dimension. Track outside depth equals nominal depth + two times design thickness + one inside corner radius.*

5. *For "S" sections (studs and joists), lip lengths are standardized as follows:*

Flange Width Designation	Flange Width (in.)	Stiffening Lip (in.)
S125	1-1/4	0.188
S137	1-3/8	0.375
S162	1-5/8	0.500
S200	2	0.625
S250	2-1/2	0.625

6. *Section styles are defined in Figure J-1:*

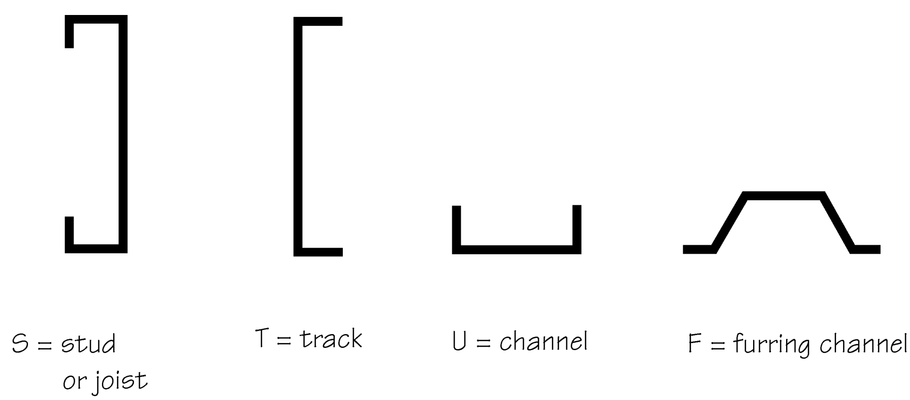


FIGURE J-1

APPENDIX K
SECTION REFERENCE BETWEEN AISI S240 AND AISI S200, S210, S211, S212, S213, AND S214

AISI S240 Section	Title	Source Standard	Section
A.	**GENERAL**	S200 to S214	A
A1	Scope	S200 to S214	A1
A1.1		S200	A1
A1.2		S200 to S214	A1
A1.2(a)		S210	A1
A1.2(b)		S211 and S212	A1
A1.2(c)		S213	A1
A1.2(d)		S214	A1
A1.3		new	
A1.4		S210 to S212	A1
A1.5		S200 to S214	A1
A1.6		S200 to S214	A1
A1.7		S200 to S214	A1
A2	Definitions	S200 to S214	A2
A2.1	Terms	S200	A2
A3	Material	S200	A3
A4	Corrosion Protection	S200	A4
A4.1		S200	A4.1
A4.2		S200	A4.2
A4.3		S200	A4.3
A4.4		S200	A4.4
A4.5		S200	A4.5
A4.6		S200	A4.6
A5	Products	S200	A5
A5.1	Base Steel Thickness	S200	A5.1
A5.2	Minimum Flange Width	S200	A5.2
A5.3	Product Designator	S200	A5.3
A5.4	Manufacturing Tolerances	S200	A5.4
A5.5	Product Identification	S200	A5.5
A6	Referenced Documents	S200 to S214	varies

AISI S240 Section	Title	Source Standard	Section
B.	**DESIGN**	S210 to S212	B
B1	General	new (editorial)	n/a
B1.1	Loads and Load Combinations	S210 to S214	varies
B1.1.1	Live Load Reduction on Wall Studs	new	n/a
B1.1.2	Wind Loading Considerations for Wall Studs in U.S. and Mexico	S211	A3.1
B1.2	Design Basis	S210 and S211	B
		S213 and S214	B1
B1.2.1	Floor Joists, Ceiling Joists and Roof Rafters	S210	B1
B1.2.2	Wall Studs	S211	B1
B1.2.3	In-Line Framing	S200	C1
B1.2.4	Sheathing Span Capacity	S200	C2
B1.2.4.1		S200	C2.3.2
B1.2.4.2		S200	C2.4.2
B1.2.4.3		S200	C2.5.2
B1.2.5	Load Path	S213	C1
B1.2.6	Principles of Mechanics	S213	B2
B1.3	Built-Up Section Design	S210	B1.5
		S211	B1.7
B1.3.1		S211	B1.7
B1.3.2		S211	B1.7
B1.4	Properties of Sections	S210 and S211	B1.1
B1.5	Connection Design	S211	B2.1
B1.5.1	Screw Connections	S200	D1
B1.5.1.1	Steel-to-Steel Screws	S200	D1.1
B1.5.1.2	Sheathing Screws	S200	D1.2
B1.5.1.3	Spacing and Edge Distance	S200	D1.5
B1.5.1.4	Gypsum Board	S200	D1.6
B1.5.2	Welded Connections	S200	D2
B1.5.3	Bolts	S200	D3.1
B1.5.4	Power-Actuated Fasteners	new	
B1.5.5	Other Connectors	S200	D3.2
B1.5.6	Connection to Other Materials	S200	D3.3
B2	Floor and Ceiling Framing	new (editorial)	n/a
B2.1	Scope	new (editorial)	n/a
B2.2	Floor Joist Design	new (editorial)	n/a
B2.2.1	Bending	new (editorial)	n/a
B2.2.1.1	Lateral-Torsional Buckling	S210	B1.2.1
B2.2.1.2	Distortional Buckling	new	n/a
B2.2.2	Shear	S210	B1.2.2
B2.2.3	Web Crippling	S210	B1.2.3
B2.2.4	Bending and Shear	S210	B1.2.4

AISI S240 Section	Title	Source Standard	Section
B2.2.5	Bending and Web Crippling	S210	B1.2.5
B2.3	Ceiling Joist Design	new (editorial)	n/a
B2.3.1	Tension	S210	B1.3.1
B2.3.2	Compression	new (editorial)	n/a
B2.3.2.1	Yielding, Flexural, Flexural-Torsional and Torsional Buckling	S210	B1.3.1
B2.3.2.2	Distortional Buckling	new	n/a
B2.3.3	Bending	new (editorial)	n/a
B2.3.3.1	Lateral-Torsional Buckling	S210	B1.3.2
B2.3.3.2	Distortional Buckling	new	n/a
B2.3.4	Shear	S210	B1.3.3
B2.3.5	Web Crippling	S210	B1.3.4
B2.3.6	Axial Load and Bending	S210	B1.3.5
B2.3.7	Bending and Shear	S210	B1.3.6
B2.3.8	Bending and Web Crippling	S210	B1.3.7
B2.4	Floor Truss Design	S210	B2
B2.5	Bearing Stiffeners	S210	B3.1
B2.5.1	Clip Angle Bearing Stiffeners	S210	B3.1.1
B2.6	Bracing Design	S210	B4
B2.7	Floor Diaphragm Design	S210	B5
B3	Wall Framing	new (editorial)	n/a
B3.1	Scope	new (editorial)	n/a
B3.2	Wall Stud Design	new (editorial)	n/a
B3.2.1	Axial Strength [Resistance]	new (editorial)	n/a
B3.2.1.1	Yielding, Flexural, Flexural-Torsional and Torsional Buckling	S211	B1.2
B3.2.1.2	Distortional Buckling	new	n/a
B3.2.2	Bending	new (editorial)	n/a
B3.2.2.1	Lateral-Torsional Buckling	S211	B1.3
B3.2.2.2	Distortional Buckling	MS09-3A	n/a
B3.2.3	Shear	S211	B1.4
B3.2.4	Axial Load and Bending	S211	B1.5
B3.2.5	Web Crippling	S211	B1.6
B3.2.5.1	Stud-to-Track Connection for C-Section Studs	S211	B2.2
B3.2.5.2	Deflection Track Connection for C-Section Studs	S211	B2.3
B3.3	Header Design	S212	B
B3.3.1	Back-to-Back Headers	S212	B1
B3.3.2	Box Headers	S212	B2
B3.3.3	Double L-Headers	S212	B3
B3.3.4	Single L-Headers	S212	B4
B3.3.5	Inverted L-Header Assemblies	S212	B5
B3.4	Bracing	S211	B3
B3.4.1	Intermediate Brace Design	S211	B3.1

AISI S240 Section	Title	Source Standard	Section
B3.5	Serviceability	S211	B4
B4	Roof Framing	new (editorial)	n/a
B4.1	Scope	new (editorial)	n/a
B4.2	Roof Rafter Design	new (editorial)	n/a
B4.2.1	Tension	S210	B1.4.1
B4.2.2	Compression	new (editorial)	n/a
B4.2.2.1	Yielding, Flexural, Flexural-Torsional and Torsional Buckling	S210	B1.4.1
B4.2.2.2	Distortional Buckling	new	n/a
B4.2.3	Bending	new (editorial)	n/a
B4.2.3.1	Lateral-Torsional Buckling	S210	B1.4.2
B4.2.3.2	Distortional Buckling	new	n/a
B4.2.4	Shear	S210	B1.4.3
B4.2.5	Web Crippling	S210	B1.4.4
B4.2.6	Axial Load and Bending	S210	B1.4.5
B4.2.7	Bending and Shear	S210	B1.4.6
B4.2.8	Bending and Web Crippling	S210	B1.4.7
B4.3	Roof Truss Design	S210	B2
B4.4	Bearing Stiffeners	S210	B3.1
B4.5	Bracing Design	S210	B4
B4.6	Roof Diaphragm Design	S210	B5
B5	Lateral Force-Resisting Systems	new (editorial)	n/a
B5.1	Scope	new (editorial)	n/a
B5.2	Shear Wall Design	S213	C
B5.2.1	General	S213	C1
B5.2.1.1	Type I Shear Walls	S213	C2
B5.2.1.2	Type II Shear Walls	S213	C3
B5.2.2	Nominal Strength [Resistance]		
B5.2.2.1	Type I Shear Walls	S213	C2
B5.2.2.2	Type II Shear Walls	S213	C3
B5.2.3	Available Strength [Factored Resistance]	S213	C2
B5.2.4	Collectors and Anchorage	S213	C3.3
B5.2.5	Design Deflection	S213	C2.1.1
B5.3	Strap Braced Wall Design	S213	C4
B5.4	Diaphragm Design	S213	D
B5.4.1	General	S213	D1
B5.4.2	Nominal Strength	S213	D2
B5.4.3	Available Strength	S213	D2
B5.4.4	Design Deflection	S213	D2
B5.4.5	Beam Diaphragm Tests for Non-Steel Sheathed Assemblies	S213	D4

AISI S240 Section	Title	Source Standard	Section
C.	**INSTALLATION**	S200 to S212	C
C1	General	S210 to S212	C
C2	Member Condition	S200	B2
C2.1	Web Holes	S200	B2.1
C2.2	Cutting and Patching	S200	B2.2.1
C2.3	Splicing	S200	B2.2.2
C3	Structural Framing	S200	C2
C3.1	Foundation	S200	C2.1
C3.2	Ground Contact	S200	C2.2
C3.3	Floors	S200	C2.3
C3.3.1	Plumbness and Levelness	S200	C2.3.1
C3.3.2	Alignment	S200	C2.3.2, C2.3.3
C3.3.4	Bearing Width	S200	C2.3.4
C3.3.5	Web Separation	S200	C2.3.5
C3.4	Walls		
C3.4.1	Straightness, Plumbness and Levelness	S200	C2.4.1
C3.4.2	Alignment	S200	C2.4.2, C2.4.3
C3.4.3	Stud-to-Track Connection	S211	C1
C3.4.4	Back-to-Back and Box Headers	S212	C1
C3.4.5	Double and Single L-Headers	S212	C2
C3.4.6	Inverted L-Header Assemblies	S212	C3
C3.5	Roofs and Ceilings	S200	C2.5
C3.5.1	Plumbness and Levelness	S200	C2.5.1
C3.5.2	Alignment	S200	C2.5.2, C2.5.3
C3.5.3	End Bearing	S200	C2.5.4
C3.6	Lateral Force-Resisting Systems		
C3.6.1	Shear Walls	S213	C2
C3.6.2	Strap Braced Walls	n/a	
C3.6.3	Diaphragms	S213	D2
C4	Connections		
C4.1	Screw Connections	S200	D1
C4.1.1	Steel-to-Steel Screws	S200	D1.1
C4.1.2	Installation	S200	D1.3
C4.1.3	Stripped Screws	S200	D1.4
C4.2	Welded Connections	S200	D2
C5	Miscellaneous	S200	E

AISI S240 Section	Title	Source Standard	Section
C5.1	Utilities	S200	E1
C5.1.1	Holes	S200	E1.1
C5.1.2	Plumbing	S200	E1.2
C5.1.3	Electrical	S200	E1.3
C5.2	Insulation	S200	E2
C5.2.1	Mineral Fiber Insulation	S200	E2.1
C5.2.2	Other Insulation	S200	E2.2
D.	**QUALITY CONTROL AND QUALITY ASSURANCE**	new	n/a
E.	**TRUSSES**	S214	A-G
E1	General	S214	A
E1.1	Scope and Limits of Applicability	S214	A1
E2	Truss Responsibilities	S214	B
E3	Loading	S214	C
E4	Truss Design	S214	D
E4.1	Materials	S214	D1
E4.2	Corrosion Protection	S214	D2
E4.3	Analysis	S214	D3
E4.4	Member Design	S214	D4
E4.5	Gusset Plate Design	S214	D5
E4.6	Connection Design	S214	D6
E4.7	Serviceability	S214	D7
E5	Quality Criteria for Steel Trusses	S214	E
E5.1	Manufacturing Quality Criteria	S214	E1
E5.2	Member Identification	S214	E2
E5.3	Assembly	S214	E3
E6	Truss Installation	S214	F
E6.1	Installation Tolerances	S214	F1
E7	Test-Based Design	new (editorial)	
E7.1	Component Structural Performance Load Test	S214	G1
E7.2	Full-Scale Confirmatory Load Test	S214	G2
E7.3	Full-Scale Structural Performance Load Test	S214	G3
F	**TESTING**	new	n/a
F2	Truss Components and Assemblies	new	n/a

AISI S240 Section	Title	Source Standard	Section
APPENDIX 1	**CONTINUOUSLY BRACED DESIGN FOR DISTORTIONAL BUCKLING RESISTANCE**	new	n/a
APPENDIX 2	**TEST METHODS FOR TRUSS COMPONENTS AND ASSEMBLIES**	new	
2.1	Component Structural Performance Load Test	S214	G1
2.2	Full-Scale Confirmatory Load Test	S214	G2
2.3	Full Scale Structural Performance Load Test	S214	G3

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APPENDIX L
CFSEI TECHNICAL NOTES

The following technical notes are available to download at www.CFSEI.org:

D001-13	Durability of Cold-Formed Steel Framing Members
D100-13	Corrosion Protection of Fasteners
D200-12	Corrosion Protection for Cold-Formed Steel Framing in Coastal Areas
F100-09	Design of Clip Angle Bearing Stiffeners
F101-12	Screws for Attachment of Steel-To-Wood and Wood-Steel
F102-11	Screw Fastener Selection
F140-10	Welding Cold-Formed Steel
F300-09	Pneumatically Driven Pins for Wood-Based Panel Attachment
F501-11	Cold-Formed Steel Truss to Bearing Connections
F701-12	Evaluation of Screw Strength Capacity
G000-08	Cold-Formed Steel Design Software
G100-07	Using Chapter F of the NA Specification for the Design of Cold-Formed Steel Structural Members
G101-08	Design Aids and Examples for Distortional Buckling
G102-09	Designing Cold-Formed Steel using the Direct Strength Method
G103-11a	Tabulated Local and Distortional Elastic Buckling Solutions for Standard Shapes
G104-14	Welded Boxed-Beam Design
G200-15	Chase the Loads - Load Path Considerations for Cold-Formed Steel Light-Frame Construction
G500-11	Guidelines for Inspecting Cold-Formed Steel Structural Framing in Low Rise Buildings
G800-12	ASTM Standards for Cold-Formed Steel
G801-13	ASTM A1003 - No Cause for Rejection
G802-13	AISI Section A2.2 - Other Steels
G900-15	Design Methodology for Hole Reinforcement of Cold-Formed Steel Bending Members
J100-11	Cold-Formed Steel Floor Joists
L001-10	Design of Diagonal Strap Bracing Lateral Force Resisting Systems for the 2006 IBC
L200-09	Roof Framing Anchorage Forces: MWFRS or C&C
L202-12	Diaphragm Design with Pneumatically Driven Pins
L300-09	Design of End Posts for Diaphragm Shear Walls
T001-09	Fire and Acoustic-Rated Assemblies for Multi-Unit Structures
T100-12	Fire-Rated Assemblies for Cold-Formed Steel Construction
W100-08a	Single Slip Track Design
W101-09	Common Design Issues for Deflection Track
W102-12	Introduction to Curtain Wall Design Using Cold-Formed Steel
W103-11	Design of By-Pass Slip Connectors in Cold-Formed Steel Construction
W104-10	Top Track Load Distribution Members
W105-13	Design of Nonstructural Members
W200-09	Header Design

W400-16	Mechanical Bridging and Bridging Anchorage of Axially Loaded Cold-Formed Steel Studs
W500-12	Construction Bracing for Walls
W106-15a	Design for Splicing of Cold-Formed Steel Wall Studs

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