Lightweight Steel Framing Design Manual

2nd Edition

Canadian Sheet Steel Building Institute

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Canadian Sheet Steel Building Institute

Errata #1 – May 23, 2006

1. On page 1-15 line 2, replace "The allowable web crippling strength … " with "The factored web crippling resistance …"

2. On page 2-15 line 5, replace "Weld group allowable moment (stud material governs)" with "Weld group factored moment resistance (stud material governs)."

3. On page 3-3 top, add Figure 3-2 (see below)

4. On page 4-20 line 14, replace "load bearing stud above" with "jack stud below".

5. On page 4-22 bottom, add the following sentence:

   "The angle below will connect to a track section (not shown in Figure 4-12) which forms a box section with the jack stud."

6. On page 4-23 line 14 from the bottom, replace "required" with "factored" twice.

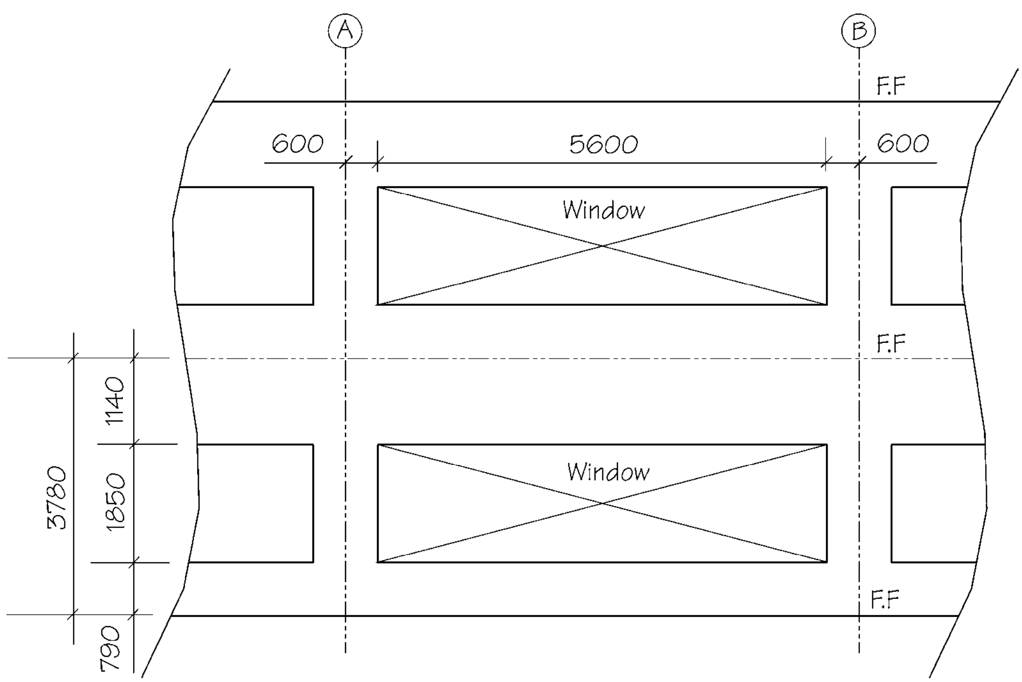

FIGURE 3-2

Lightweight Steel Framing Design Manual, 2nd Edition
Canadian Sheet Steel Building Institute

<u>Errata #2 – May 14, 2007</u>

1. On page 2-5, 2nd last line, replace "0.877" with "0.887".

2. On page 2-23, under "Screw input values":
   - Replace "Clip angle" with "Stud"
   - Replace "Bridging channel" with "Track"

3. On page 2-25, replace from "See Figure 2-18 …" to end of Step 5(e) with the following:

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

$$q_f = \sqrt{\left(\frac{M_f}{S_{weld}}\right)^2 + \left(\frac{V_f}{A_{weld}}\right)^2}$$

$$\begin{aligned} S_{weld} &= I_{weld}/c \\ &= 2\,[(1/12)(25)^3 + 25(62.5)^2\,]\,/\,75 \\ &= 2640\ \text{mm}^2 \end{aligned}$$

$$A_{weld} = L = 2(25) = 50\ \text{mm}$$

$$q_f = \sqrt{\left(\frac{15700}{2640}\right)^2 + \left(\frac{1850}{50}\right)^2}$$

$$= 37\ \text{N/mm}$$

$$\begin{aligned} q_r &= \phi P_n/L = \phi 0.75 t F_u \\ &= 0.40(0.75)(1.146)(310) \\ &= 107\ \text{N/mm} > 37\ \text{N/mm} \end{aligned}$$

OK

4. On page 2-38, 12th line, replace "0.532 kN" with "0.532 kN.m"

5. On page 2-40, Figure 2-31, reverse the direction of force T_f.

6. On pages 2-42 & 2-43, Figures 2-32 & 2-33, remove the T_f label from the bottom force acting at "a". *(The magnitude of the force at "a" does not equal T_f)*

7. On page 2-43, last line, replace "2.61 kN" with "4.00 kN".

8. On page 2-47, 9th line from bottom, replace "A_g" with "A_n".

9. On page 2-47, 6th line from bottom, replace "8(f)" with "8(d)".

10. On page 2-48, 5th line, replace"8(f)" with "8(d)".

11. On page 3-17, the tributary area for the P_{DL} calculation is shown incorrectly. Replace with:

$$P_{DL} = (\text{stud spacing})(W_D)(H_{FLR/FLR})$$
$$= (0.600)(0.8)(3.78)$$
$$= 1.814 \text{ kN (specified)}$$

Note that this error, affects subsequent calculations in Steps 7(d) through 7(i). These subsequent calculations have <u>not</u> been revised to reflect this higher 1.814 kN dead load.

12. On page 4-15, 5th line from bottom, replace "PD_L" with "P_{DL}".

13. On page 4-20, 14th line, replace "load bearing stud above" with "jack stud below".

14. On page 4-22, add a sentence at the end of the last paragraph: "The angle at the bottom of the box header will connect to a short piece of track which in turn connects to the jack stud – not shown on Figure 4-12."

15. On page 4-23, reword as follows:

    i) Bridging axial load

    Bridging factored axial load = 0.02 x stud factored axial load x number of studs braced (n).

16. On page 4-28, replace "P_{Ex} =" with "P_{Ey} =" 2nd occurrence only.

17. On page 4-34, 6th line from bottom, replace "9010 N" with "9010 N > 5040 N ***OK***"

18. On page 4-35, 2nd line, replace "7550 N" with "7550 N > 5040 N ***OK***"

19. On page F-1, Note F-1, replace "greater than" with "less than".

20. On Page H-1, 2nd paragraph, replace "particular", with "particularly".

21. On page J-1, 4th line from bottom, add quotation mark after 600S162-54 (50).

Lightweight Steel Framing Design Manual

2nd Edition

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CSSBI
Canadian Sheet Steel Building Institute

Lightweight Steel Framing Design Manual

Preface

This publication is intended as a guide for designers of lightweight steel framing (LSF) systems for buildings. LSF 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 pre-assembled 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 Canadian Sheet Steel Building Institute, its Members nor T.W.J. Trestain Structural Engineering warrant or assume liability for the suitability of the material for any general or particular use.

Scope and Purpose of the Manual

This manual has been prepared to assist practicing structural engineers to design lightweight steel framing (LSF) systems. This is the second edition of the Manual – the first was published in 1991.

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 the 2001 North American Specification for the Design of Cold Formed Steel Structural Members – CAN/CSA-S136-01 *(CSA 2001a)*, the Supplement 2004 to the North American Specification – S136S1-04 *(CSA 2004a)* and the National Building Code of Canada 2005 *(NRC 2005)*. The examples show how to translate the information available in the CSSBI Load Tables and LSF manufacturers' literature into complete structural systems. Both screwed and welded connection details are included with an emphasis on screwed. 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 *CAN/CSA-S136-01*. These include a rational method to check the warping torsional stresses in channel members, 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 Lightweight Steel Framing (LSF) members has been used throughout the Manual. This product identification method is described in Appendix I.

Changes from the 1st Edition of the Design Manual

The first edition of the Design Manual *(CSSBI 1991)* has been completely rewritten to reflect the substantial improvements that have occurred in the design of LSF members and connections.

- The design examples have been revised to conform to the latest design standards including CAN/CSA-S136-01 *(CSA 2001a)*, Supplement S136S1-04 *(CSA 2004a)*, and the NBC 2005 *(NRC 2005)*. In addition, the new AISI/COFS standards have been used where applicable *(COFS 2004a & 2004c)*.
- The universal designator system *(STUF nomenclature)* for identifying members has been used.
- The new standard thicknesses adopted by the CSSBI in their load tables *(CSSBI 2004)* have been used.
- Powder actuated fasteners have been added to the examples.
- A single outer top track deflection detail has been added.
- A slide clip detail for connecting wind bearing jamb studs has been added.
- A design methodology for flat strap blocking-in has been provided.
- Design Example #2 has been expanded to include both welded and screwed connections.
- The design method for checking cantilevering stud deflections has been revised.
- The connection details in Design Example #4 have been converted from welded to screwed to reflect the more common practice.

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 Manual or available at the following websites:

- Canadian Sheet Steel Building Institute (CSSBI) – www.cssbi.ca
- American Iron and Steel Institute (AISI) – www.steel.org
- Steel Stud Manufacturers Association (SSMA) – www.ssma.com
- Light Gauge Steel Engineers Association (LGSEA) – www.lgsea.com
- Center for Cold-Formed Steel Structures (CCFSS) – www.umr.edu/~ccfss
- North American Steel Framing Alliance (NASFA) – www.steelframingalliance.com

Acknowledgements

The Canadian Sheet Steel Building Institute would like to acknowledge the contribution of Mr. Tom Trestain, P. Eng. of T.W.J. Trestain Structural Engineering, Toronto, Canada who was retained for the preparation of this publication. Mr. Trestain is experienced in the design and erection of LSF products and is an active member on the CAN/CSA-S136-01 Committee as well as other voluntary industry committees.

The development of this Manual has been greatly assisted by the Dietrich Design Group Inc. who volunteered the CAD linework for the drawings.

A number of individual engineers have also added their expertise to this project. In the USA, Rob Madsen at Devco Engineering provided many helpful interpretations of industry practice on the west coast. Ed DiGirolamo of the Steel Network and John Matsen of Matsen Ford Design Associates were also very helpful regarding practices in the east. In Canada, Scot McCavour at McCavour Engineering provided much practical and useful advice along with one of their in-house details *(Figure 3-17)*. Lastly, the willingness of Raymond van Groll at Atkins & van Groll Inc. to share his expertise has been gratefully received.

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Introduction

1. Design Manual Focus

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

It was necessary to focus on fundamentals because the versatility of LSF construction makes an all inclusive design manual virtually impossible. By following the examples provided, the engineer should gain the confidence necessary to execute his 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 lightweight steel framing.

An intimate knowledge of CAN/CSA-S136-01 *(CSA 2001a)* and its supplement *(CSA 2004a)*, while desirable, is not essential. The examples focus on those areas of the Standard that require the designer's attention. Much of the work has already been done during the preparation of LSF product literature with section properties and load tables calculated and ready to use in tabular form.

In the Design Manual, 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 LSF systems.

Note that the Design Manual is almost entirely dedicated to hand calculation methods. Hand calculation is useful for illustration purposes but may not be the most efficient approach for routine design. Experienced practitioners automate the design process as much as possible typically by writing their own spreadsheet type programs or purchasing commercial cold-formed steel software packages or both. CAN/CSA-S136-01 is a North America standard and software developed in the USA can be applicable to Canada provided a Limit States Design option is included with Canadian load and resistance factors.

2. Loads

2.1 Wind, Earthquake and Gravity Specified Loads

The necessary information to calculate wind, earthquake and gravity loads is contained in the National Building Code of Canada 2005 *(NRC 2005)* and the accompanying Commentaries. See Item 4.

For wind load calculations, studs are assumed to be structural members rather than cladding elements.

2.2 Load Factors

Load factors and load combination factors are contained in the National Building Code of Canada 2005 *(NRC 2005)*.

3. Factored Resistances for Lightweight Steel Framing Elements

3.1 Factored Resistances for Members

Member capacities in the form of moment, shear, and web crippling factored resistances and moments of inertia for checking deflection are generally available in product literature. This literature also typically contains load tables for wind and axial load bearing studs and roof and floor joists.

The allowable spans, loads and section properties in this manual have been taken from the Canadian Sheet Steel Building Institute generic load tables *(CSSBI 2004)* using the following yield and tensile strength values for both stud and track.

- For thicknesses less than or equal to 1.146 mm, F_y = 230 MPa (33 ksi) and F_u = 310 MPa (45 ksi.)
- For thicknesses greater than 1.146 mm, F_y = 345 MPa (50 ksi) and F_u = 450 MPa (65 ksi.)

These assumed yield and tensile strength values are common for stud and standard track. Specialty items such as long legged deflection track, bridging channel and connection accessories are not included in the CSSBI tables. Check with the local LSF manufacturers before specifying the geometry and yield strength of these products.

Note that the member capacities in the CSSBI tables conservatively neglect the increase in strength due to the cold work of forming. This approach is preferred for welded construction because welding can reduce the effect of cold work. See CAN/CSA-S136-01 *(CSA 2001a)* Section A7.2 (c).

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 the CAN/CSA-S136-01 *(CSA 2001a)* and its supplement *(CSA 2004a)* which include provisions for local buckling, members in tension, bending, compression and combined axial load and bending. A commentary is also available, S136.1-01 *(CSA 2001b)*. For axially loaded wall studs clad with imperfect sheathing (i.e. sheathing that does not completely restrain the studs), refer to COFS 2004a. Lastly, the American Iron and Steel Institute publishes the AISI Manual, Cold-Formed Steel Design *(AISI 2002b)* which contains additional helpful information including computational aids, supplementary formulae, worked examples for beams columns and connections, properties of steels, and test procedures. Although this is an American publication, its contents are relevant to Canada because CAN/CSA-S136-01 is a North American standard.

3.2 Member Design Strength as a Function of Bracing

LSF 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.

3.2.1 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 that can occur with an old style wrap around type brick tie or with sheathing attachment when the screw is in tension.

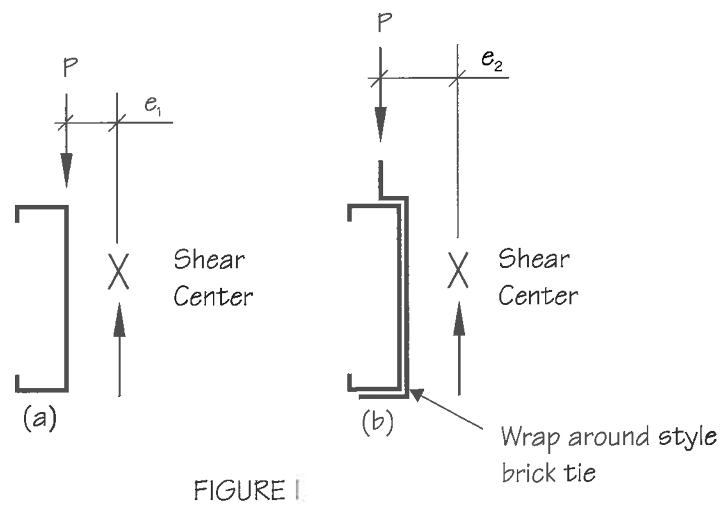
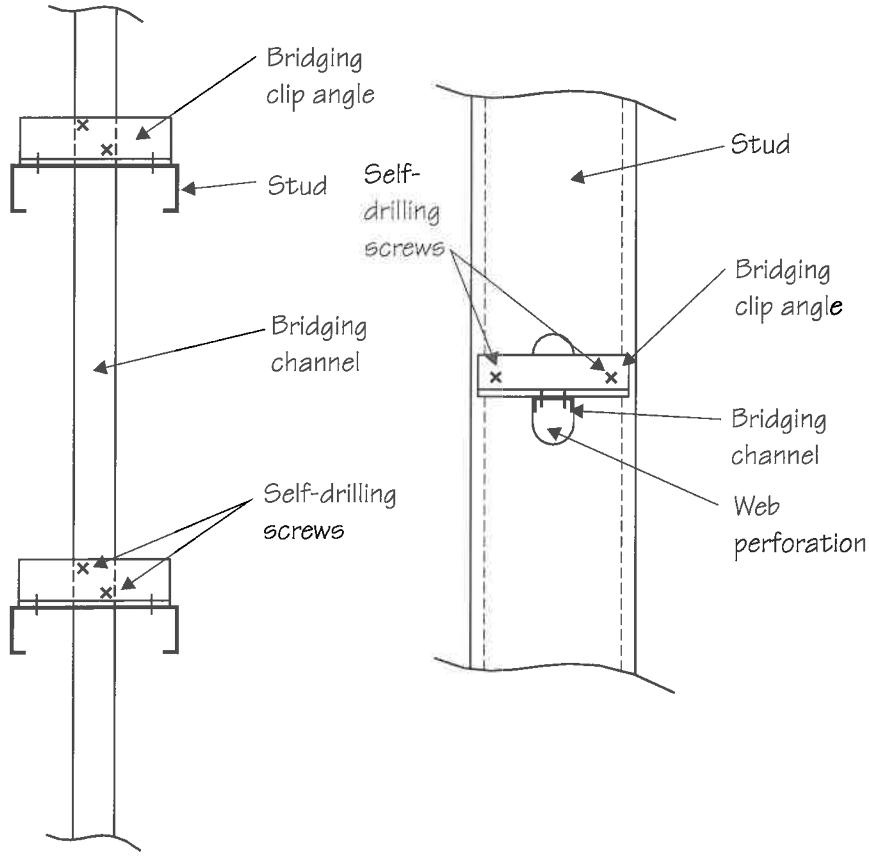


FIGURE I

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 neighbouring studs.



THROUGH THE KNOCKOUT BRIDGING

FIGURE II

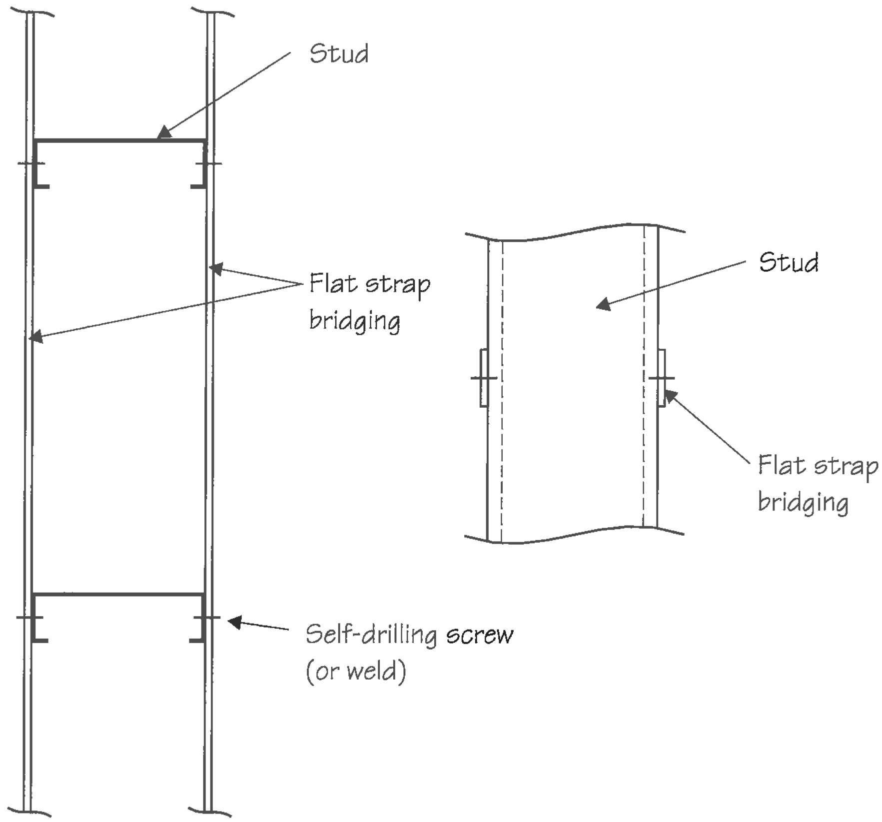
Advantages:

- 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.

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 bridging forces will accumulate over a number of studs. 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

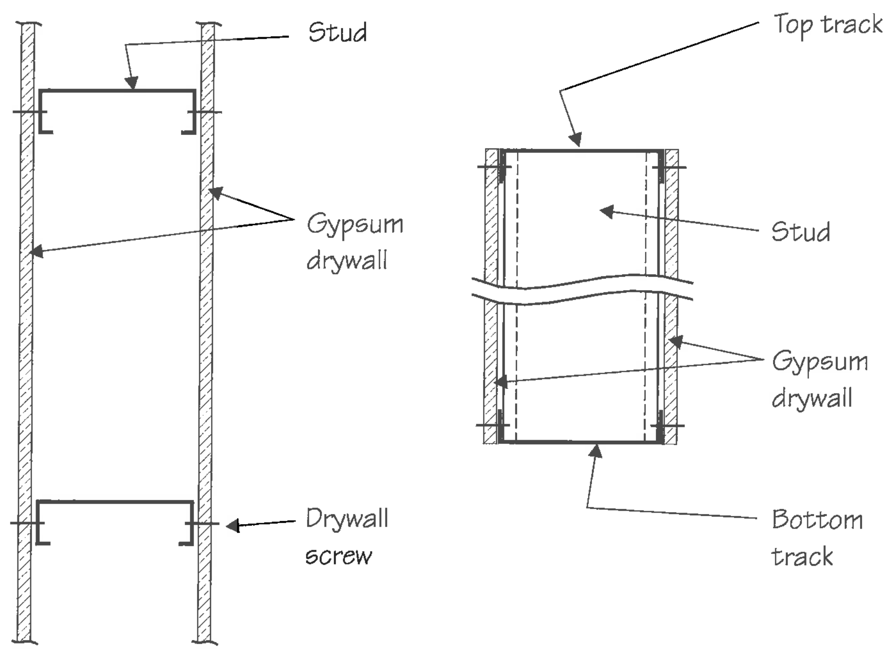
Advantages:

- Stiffest form of bridging even if installed with some initial slackness *(Miller 1989 & 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, waferboard 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).

- Based on limited experimental evidence, 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 typical sheathed wind bearing load tables can still be used provided there is sufficient steel bridging. Appropriately spaced bridging will reduce the effects of lateral-torsional buckling and the secondary stresses due to torsion such that they can be neglected.*

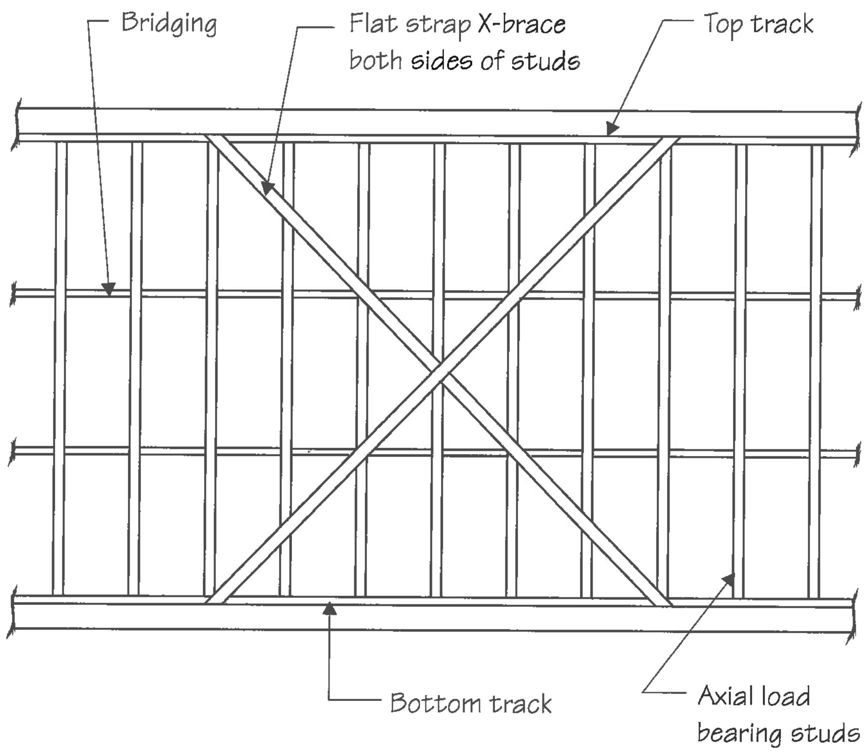
3.2.2 Bracing for Axial Load Bearing Studs

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 3.2.1)* 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 one 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 the 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

The Standard for Cold-Formed Steel Framing – Wall Stud Design *(COFS 2004a)* 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 moisture and load cycle sensitive 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 insure 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 the Standard for Cold-Formed Steel Framing – Wall Stud Design *(COFS 2004a)*

3.2.3 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.

3.3 Design Strengths for Connections

3.3.1 Welds

The unit strengths of fillet and flare groove welds are defined in CAN/CSA-S136-01 *(CSA 2001a)* Sections E2.4 and E2.5. 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.

3.3.2 Screws

The design strength for connections with sheet metal and self-drilling screws is defined in the CAN/CSA-S136-01 *(CSA 2001a)*. Analytical expressions are provided in the Specification with the exception of the shear and tensile strength of the screw itself. These tensile and shear strengths are provided in Appendix A.

3.3.3 Anchors and Fasteners

Suggested design values for three proprietary types of anchors are presented in Appendix B.

4. NBC 2005 Structural Commentaries

The Structural Commentaries on the National Building Code of Canada 2005 were not available during preparation of this document. As a result, there may be assumptions made in the Design Manual that are not consistent with the intent of the NBC 2005 as provided in the Commentaries.

Where appropriate, some of these uncertainties have been noted in the design examples including:

- Where powder actuated fasteners are used in tension and where earthquake design is a consideration, there may be restrictions on the use of these fasteners – see NBC 2005 Clause 4.1.8.17 (8) (d).
- The partial wind load provisions in the NBCC 2005 Clause 4.1.7.3 are assumed not to apply to the design of structural elements with small tributary areas such as studs.

Design Example #1

Wind Bearing Infill Wall with Screwed Connections and a Sheathed Design Approach

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 an unsheathed design approach with both welded and screwed connections, 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 & 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 = 400 mm o.c.
- Stud height = 4000 mm
- 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 = 152 mm for architectural considerations

Step 2 – Design Wind Load

As determined by the National Building Code of Canada *(NRC 2005)*:

Specified (unfactored) wind load for strength	= 1.32 kPa
Specified (unfactored) wind load for deflection	= I_w (1.32) = 0.75(1.32) = 0.99 kPa

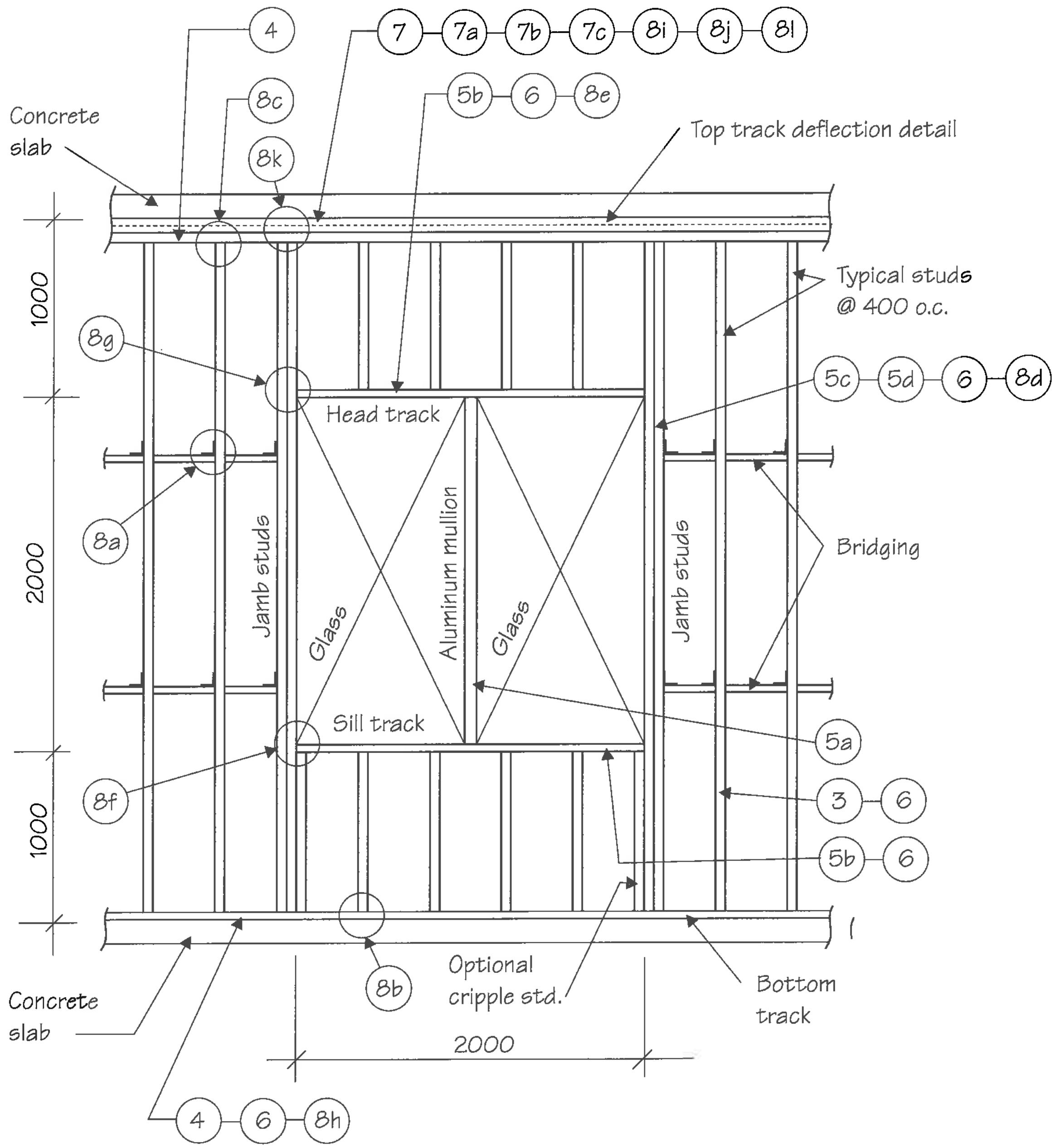


FIGURE 1-1

Step 3 – Typical Stud Selection

Refer to a CSSBI Wind Bearing Stud Allowable Height Tables *(CSSBI 2004)* with the following details *(see Note 1-1)*:

- Height = 4 m
- Spacing = 400 mm o.c.
- Wind load for strength = 1.32 kPa (specified)
- Wind load for deflection = 0.99 kPa (specified)
- Deflection limit = L/360

> *Note 1-1*
>
> *The current CSSBI wind bearing tables (CSSBI 2004) are based on the 1995 National Building Code of Canada (NRC 1995). These CSSBI tables can be used for wind bearing stud selections conforming to the 2005 NBC (NRC 2005) provided the design procedure here in Step 3 is followed.*

Try 600S162-43 stud (F_y = 230 MPa).

(Note: 600S162-43 is a universal designator system adopted by LSF manufacturers for their products. For a description of the system, see Appendix J.)

For strength (moment & shear):

For a load factor = 1.4
Factored wind w_f = 1.4(1.32) = 1.85 kPa

From CSSBI tables *(CSSBI 2004)*, choose next highest factored wind load = 2.15 kPa (although not used here interpolation between factored wind loads is acceptable when required)

H_{MAX} = 4.88 m > 4.0 m ***OK***

For web crippling *(See Note 1-2)*:

No asterisk appears on the strength allowable height, therefore, web crippling does not control.

> *Note 1-2*
>
> *For a typical stud to track connection (welded or screwed), the web crippling strength of the stud is adequately predicted assuming a nominal bearing length of 25 mm 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. The CSSBI load tables limit the distance from the centerline of the last punchout to the end of the stud to 300 mm minimum. Punchouts closer than 300 mm to the end of the stud may or may not result in a reduction to web crippling factored resistance. Refer to CAN/CSA-S136-01 web crippling provisions.*

For deflection:

Specified wind load w_s = 0.99 kPa

From CSSBI tables *(CSSBI 2004)*, choose next highest specified wind load = 1.20 kPa (although not used here interpolation between specified wind loads is acceptable when required).

H_{MAX} = 4.42 m > 4.0 m **_OK_**

Final typical stud selection

The 600 S162-43 stud at 400 mm o.c. is satisfactory for both strength and deflection. See Note 1-3.

> *Note 1-3*
>
> *Note that the thinner 600S162-33 stud is satisfactory for strength and deflection but not web crippling. A 600S162-33 could still be used provided web stiffeners were added to each stud top and bottom. This web stiffener solution is generally considered uneconomic and in addition, the thinner stud material can complicate connection design.*

Step 4 – Bottom and Inner Top Track

The Standard for Cold-Formed Steel Framing - Wall Stud Design *(COFS 2004a)* provides a design procedure for checking local failure (tear through) of the track. The COFS standard 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 track with Fy = 230 MPa.

For a typical stud the factored end reaction with a wind load factor of 1.4 is given by:

$P_f = 1.4(0.4)(4/2)(1.32) = 1.48$ kN

Check track tear through using COFS 2004a C4.2(b)

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

where:

P_{nst}	= nominal resistance for stud to track connection when subjected to transverse loads
t_t	= design track thickness
w_{st}	= $20\ t_t + 0.56\alpha$ (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
ϕ = 0.80 for Canada

For 600T125-33

$t_t = 0.879$ mm
$F_{ut} = 310$ MPa

$P_{nst} = 0.6(0.879)[20(0.879) + 0.56(25.4)](310)/1000$
$= 5.20$ kN

$P_{rst} = \phi P_{nst} = 0.80(5.20)$
$= 4.16 \text{ kN} > 1.48 \text{ kN}$ ***OK***

Therefore, 600T125-33 track is acceptable for typical stud tear though. 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 LSF 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:

$\frac{\text{Height}}{\text{Width}} = \frac{2000}{1000} = 2.00$ for glass and the two way distribution is appropriate.

See Note 1-4.

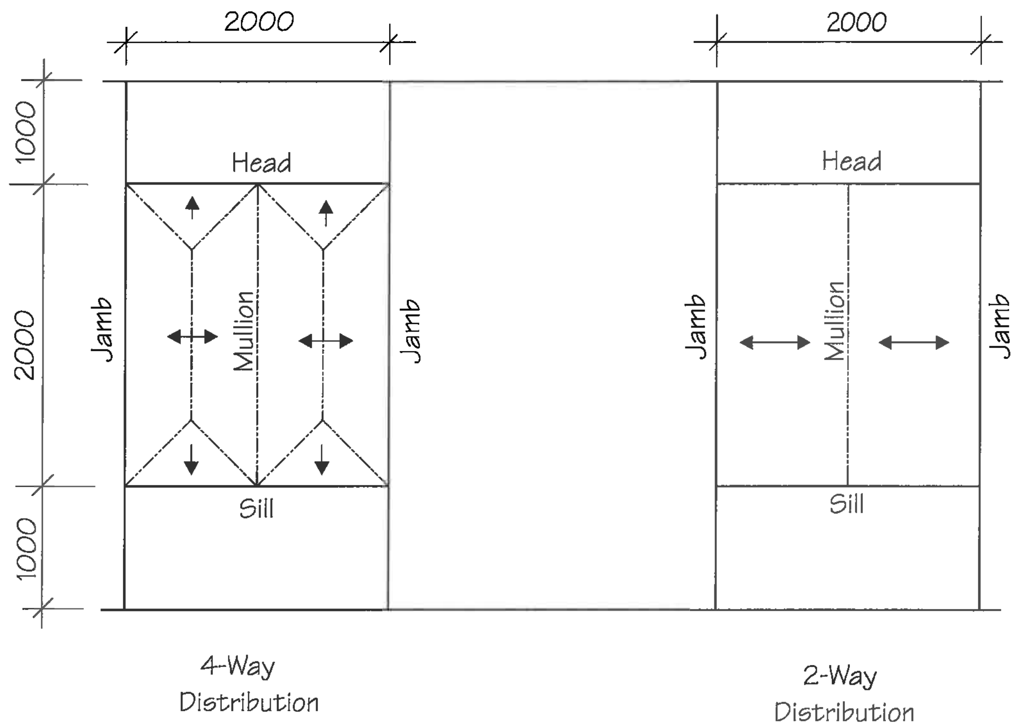


FIGURE 1-2

Note 1-4

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 LSF 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 LSF 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

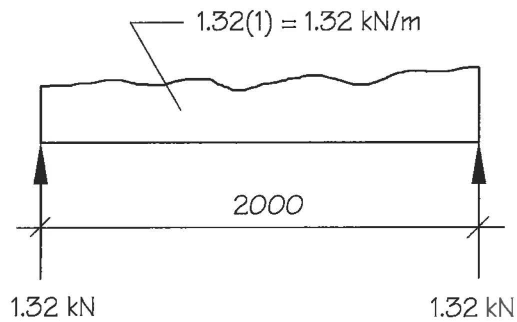


FIGURE 1-3

Step 5(b) – Sill and Head Track Design

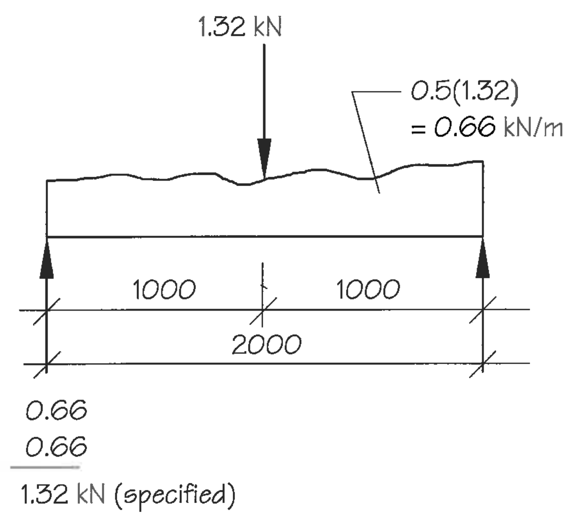


FIGURE 1-4

Factored Moment (with 1.4 load factor)

$$M_f = 1.4\left[\frac{PL}{4} + \frac{wL^2}{8}\right]$$

$$= 1.4\left[\frac{1.32(2)}{4} + \frac{0.66(2)^2}{8}\right]$$

$$= 1.39 \text{ kN.m}$$

Factored Shear (with 1.4 load factor)

$$V_f = 1.4(1.32) = 1.85\,kN$$

Required Inertia for Deflection (with I_w = 0.75)

$$\delta = (0.75)\left[\frac{PL^3}{48EI} + \frac{5wL^4}{384EI}\right]$$

$$= (0.75)\left[\frac{1320\,(2000)^3}{48\,(203000)\,I} + \frac{5(0.66)(2000)^4}{384(203000)\,I}\right]$$

$$= \frac{1.321x10^6}{I}\,mm$$

For deflection limit = L/360

$$\delta = 2000/360 = 5.56\,mm$$

$$I_{req} = \frac{1.321x10^6}{5.56} = 238000\,mm^4$$

From CSSBI track section properties, try 600T125-43 with F_y = 230 MPa

M_r	= 1.55 kN.m > 1.39 kN.m	***OK***
V_r	= 7.83 kN > 1.85 kN	***OK***
$I_{x(def)}$	= 716000 mm^4 > 238000 mm^4	***OK***

Reinforced window head for gravity loads (dead load sag) – see Figure 1-6A & 1-6B.

The 600T125-43 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)* 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 = 0.4 kPa

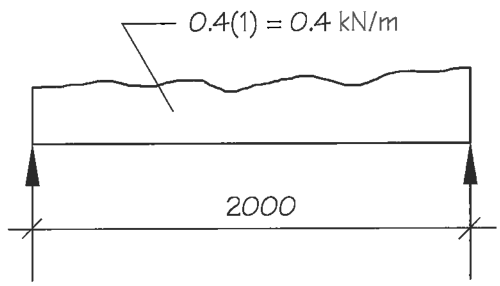


FIGURE 1-5

Factored moment (with 1.4 load factor)

$$M_f = 1.4\frac{wL^2}{8}$$
$$= 1.4\left[\frac{0.4\ (2)^2}{8}\right] = 0.280\,\text{kN.m}$$

Required Inertia (dead load only)

$$\delta = \frac{5wL^4}{384EI}$$
$$= \frac{5(0.4)(1)(2000)^4}{384(203000)\ I}$$
$$= \frac{411000}{I}\,\text{mm}$$

For deflection limit = L/360 say

$$\delta = 2000/360 = 5.56\,\text{mm}$$
$$I_{req} = \frac{411000}{5.56} = 73900\ \text{mm}^4$$

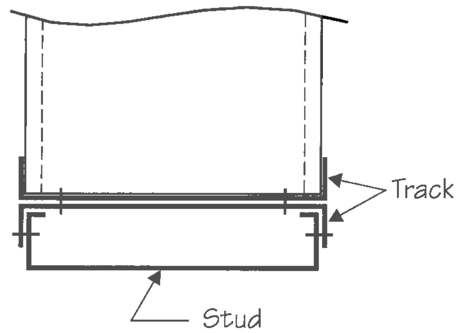


FIGURE 1-6A

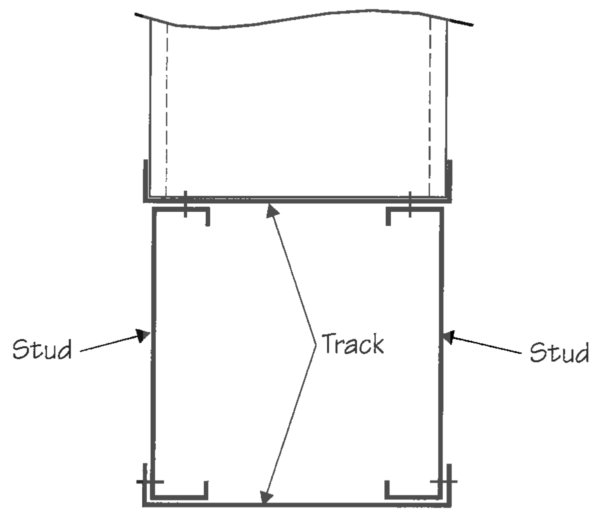


FIGURE 1-6B

Try 600T125-43 track plus 600S162-43 stud (Figure 1-6A configuration)

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

M_{ry} = 0.401 kN.m (lips in compression) > 0.280 kN.m ***OK***

$I_{y(def)} \approx$ fully effective weak axis inertia for stud and track
= (0.0616 + 0.0181) x 10^6 mm^4
= 79700 mm^4 > 73900 mm^4 ***OK***

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

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

$$I_{y(def)} = 2(0.966) \times 10^6 \text{ mm}^4$$
$$= 1932000 \text{ mm}^4 \qquad >> 73900 \text{ mm}^4 \qquad \textbf{\textit{OK}}$$

Step 5(c) – Jamb Stud Design

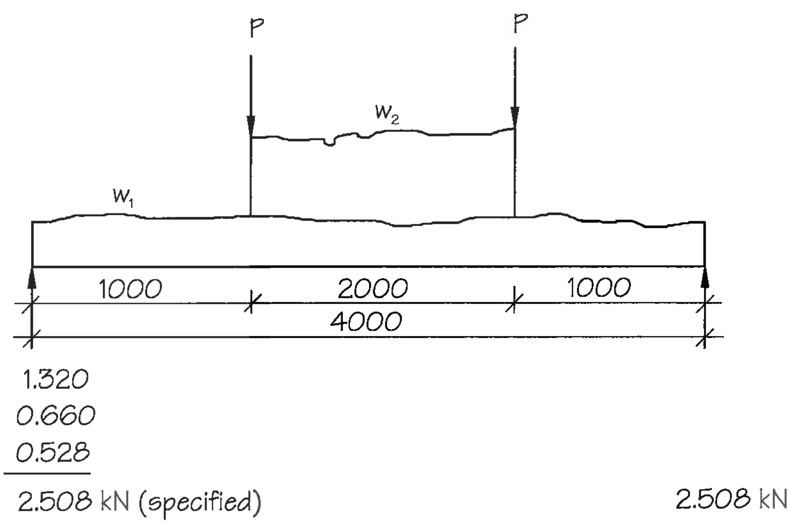


FIGURE 1-7

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

$w_1 = (0.4/2)(1.32) = 0.264$ kN/m (specified)
$w_2 = (0.5)(1.32) = 0.66$ kN/m (specified)
$P = 1.32$ kN (head/sill reaction - specified)

Factored Moment (with 1.4 load factor)

Maximum at midspan
$$M_f = 1.4[(2.51)(2) - 1.32(1) - 0.264(2)^2/2 - 0.66(1)(0.5)]$$
$$= 3.98 \text{ kN.m}$$

Factored Shear & Web Crippling with 1.4 load factor

$V_f = 1.4(2.51) = 3.51$ kN

Required Inertia with $I_w = 0.75$

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

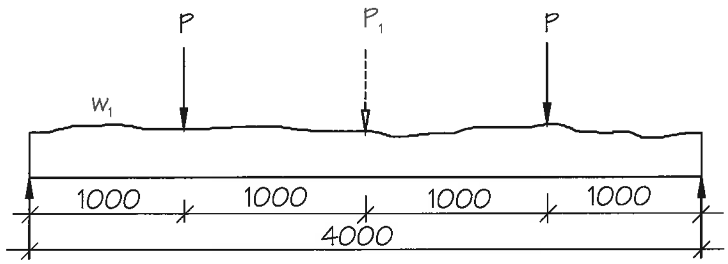


FIGURE 1-8

P = 1.32 kN (specified)
$P_1 = 0.66(2) = 1.32$ kN (specified)
$w_1 = 0.264$ kN/m (specified)

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

$$= \frac{0.75}{203000I}\left[\frac{5(0.264)(4000)^4}{384} + \frac{1320(4000)^3}{48} + \frac{1320(1000)\left[3(4000)^2 - 4(1000)^2\right]}{24}\right]$$

$$= \frac{18.69(10^6)}{I}$$

Note that by computer

$$\delta_{exact} = \frac{17.98(10^6)}{I}$$

Therefore, replacing the partial UDL with a point load is conservative by 3.9%.

For a deflection limit of L/360, δ = 4000/360 = 11.1 mm.

Using the exact computer solution:

$$I_{req} = \frac{17.98(10^6)}{11.1} = 1.62(10^6)\ \text{mm}^4$$

Possible Built-up Member Configurations – see Figures 1-9, 1-10 and 1-11.

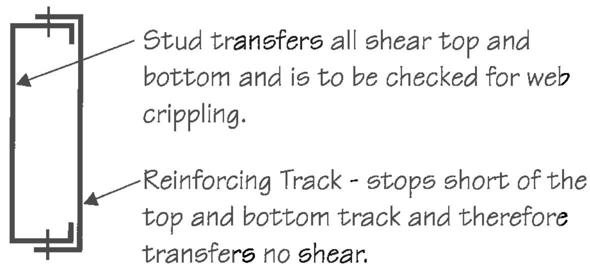


FIGURE 1-9

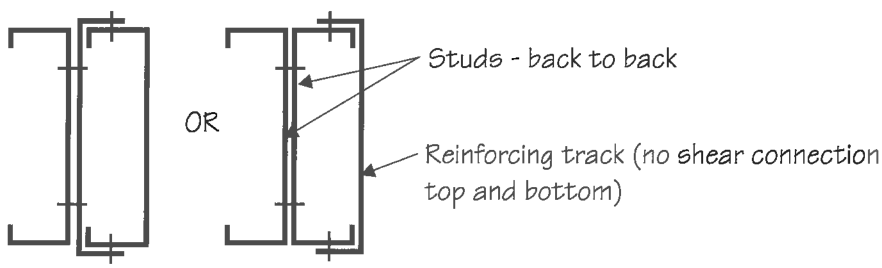


FIGURE 1-10

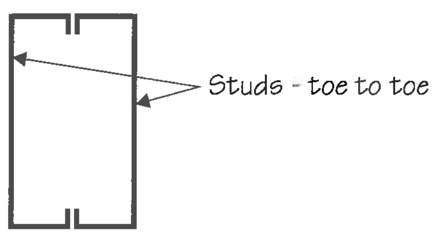


FIGURE 1-11

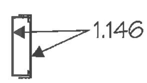
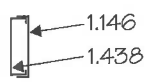
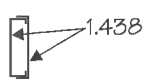
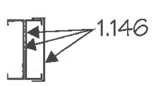
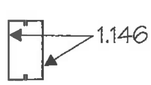
Step 5(d) – Jamb Selection

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

This table is based on the design approximation that the moment resistance and inertia of the built-up sections are the simple addition of the component parts. See Note 1-5 for an alternative approach.

Note that the track section used as part of the built-up member will exceed the standard pre-cut length of 3 m (10'-0") and may require a special order. Check with the local manufacturers.

Table 1-1 Jamb Stud Selection Table

		Factored Moment (kN.m)	Inertia (10^6 mm^4)	Factored Web Crip-pling (kN)	Factored Shear (kN)
	Required →	3.98	1.62	3.51	3.51
Built-Up Section	Component Sections				
A 	Track Stud Sum	1.55 2.59 4.14	0.716 0.966 1.682	0 2.26 2.26	0 7.03 7.03
B 	Track Stud Sum	1.55 4.71 6.26	0.716 1.190 1.906	0 5.16 5.16	0 11.10 11.10
C 	Track Stud Sum	3.01 4.71 7.72	0.912 1.190 2.102	0 5.16 5.16	0 11.10 11.10
D 	Track Stud Stud Sum	1.55 2.59 2.59 6.73	0.716 0.966 0.966 2.648	0 2.26 2.26 4.52	0 7.03 7.03 14.06
E 	Stud Stud Sum	2.59 2.59 5.18	0.966 0.966 1.932	2.26 2.26 4.52	7.03 7.03 14.06

<u>Built-up Section A:</u> Unsatisfactory for web crippling.

<u>Built-up Section B:</u> ***OK***

<u>Built-up Section C:</u> ***OK***

Built-up Section D: ***OK***

- The allowable web crippling strength, P_{ext}, of two studs connected back to back may be greater than two times the web crippling strength for a single stud. Refer to CAN/CSA-S136-01 Table C3.4.1-1 for a web crippling design expression specific to back to back studs.
- Note that the track reinforcing for the built-up jamb stud may not be necessary in order to satisfy strength or stiffness requirements (as is the case here) 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 single stud tear through can be conservatively applied to the back to back case to check the track thickness.

  From Step 4 for 600T125-33,
  $P_{rst} = \phi P_{nst} = 4.16 \text{ kN} > 3.51 \text{ kN}$
  Therefore by inspection 600T125-43 ***OK***

Built-up Section E: ***OK***

- 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 1-5

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 formulae 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 may still be 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 all of the load for this case.

Step 6 – Final Stud and Track Member Selection

Note 1-6

It is generally 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 will result in the wrong thickness in the wrong place on site.*
- *Manufacturers do not stock stud but rather they roll to order. By specifying one type of stud the delivery time is reduced and the cost premium for small production runs is eliminated. The same logic may or may not apply to track since some manufacturers carry rolled track in inventory but typically only in 3 m (10 foot) lengths.*

There is little justification for mixing thicknesses of stud and track on this project. See Note 1-6. 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 = 1.146 mm.
• 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 ± 6 mm. (The ± 6mm implies considerably better than average concrete tolerances on this project.)

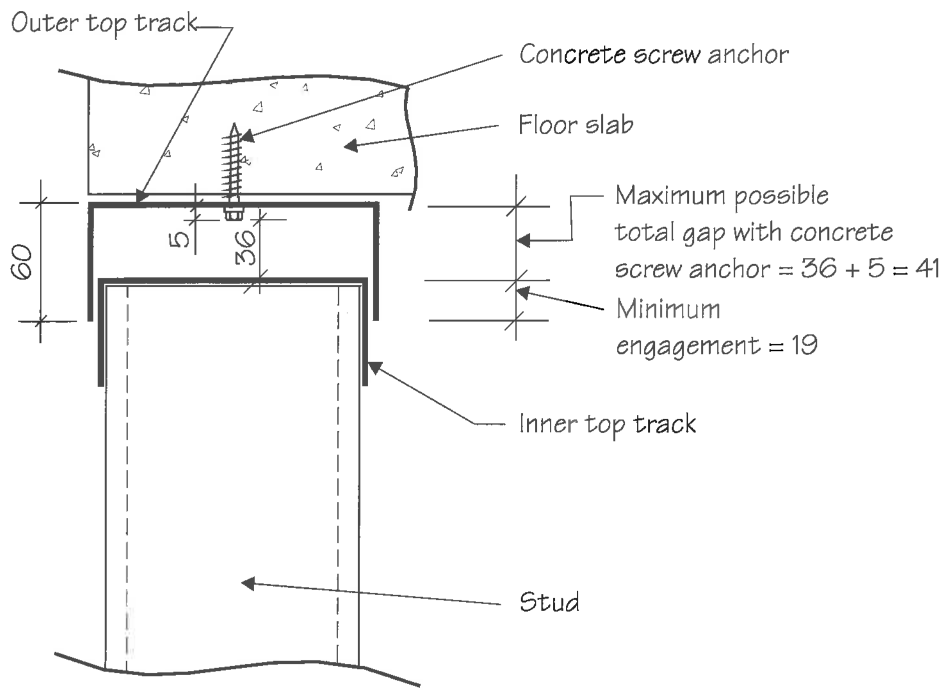


FIGURE 1-12

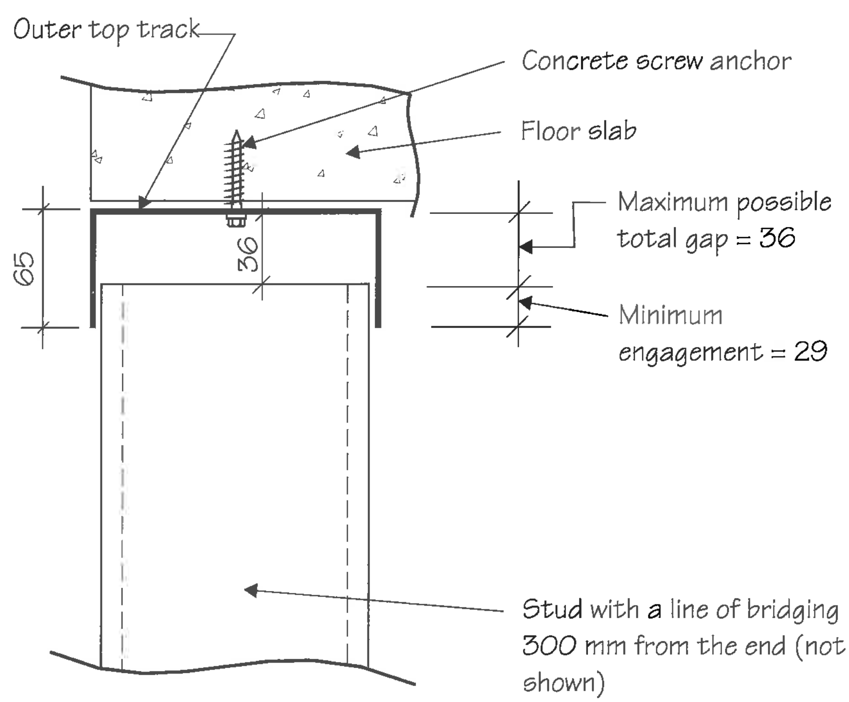


FIGURE 1-13

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 = 12 mm 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 18 mm plus or minus the construction tolerance of 6 mm. This results in a minimum possible gap at the time of installation of 12 mm which as adequate to accommodate slab deflections above assuming the slab below does not deflect. Conversely, if the slab below deflects 12 mm and the slab above does not deflect then the maximum possible gap is 18 + 6 + 12 = 36 mm.

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. See Figure 1-12. The maximum total gap is given by: 36 + 5 = 41 mm. *(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. See Note 1-7.)*

> *Note 1-7*
>
> *The previous edition of the Manual (CSSBI 1991) accommodated wedge type expansion anchors by installing them at punchout locations in the web of the inner top track. The punchouts eliminated bolt head interference and were useful as a post installation inspection ports. Currently, few manufacturers have the capability of supplying punchouts in the inner top track and no manufacturer does so routinely.*

Assuming a minimum engagement of 19 mm, the leg of the outer top track must be: 36 + 5 + 19 = 60 mm.

The 41 mm maximum total gap is used in the calculations that follow to determine the thickness of the outer top track.

Deflection gap summary with concrete screw anchors *(See Step 7(b) for dimensional requirements with powder actuated fasteners.)*

Tolerance	= ± 6 mm
Deflection	= 12 mm
Concrete screw anchor head	= 5 mm
Minimum installation gap	= Deflection + Screw Head = 12 + 5 = 17 mm
Maximum installation gap	= Deflection + Screw Head + 2 x Tolerance = 12 + 5 + 2(6) = 29 mm
Maximum possible gap	= Maximum installation gap + Deflection = 29 + 12 = 41 mm

One leg of the outer top track is assumed to be loaded uniformly by the inner top 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).

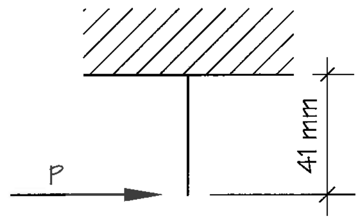


FIGURE 1-14

Figure 1-14 illustrates the cantilever design assumption for the outstanding leg of the outer top track. Check the required track thickness (with a load factor of 1.4). See Note 1-8.

$$
\begin{aligned}
P_f &= \left(\frac{\text{Stud Height}}{2}\right) w_f \\
&= (4/2)(1.4)(1.32) \\
&= 3.70\,\text{kN/m of length for strength} \\
&= 3.70\,\text{N/mm}
\end{aligned}
$$

$$
\begin{aligned}
M_f &= 41P_f = 41(3.70) \\
&= 152 \text{ N.mm per mm of length}
\end{aligned}
$$

Assuming an elastic section modulus, F_y = 345MPa and a 1 mm length of track:

$$
\begin{aligned}
M_r &= (1/6)bt^2\phi F_y \\
&= (1/6)(1)t^2(0.9)(345) \\
&= 51.8t^2 \text{ N.mm per mm of length}
\end{aligned}
$$

For $M_f = M_r$

$$t = \sqrt{\frac{152}{51.8}} = 1.738\,\text{mm}$$

Use next standard design thickness = 1.811 mm with F_y = 345 kPa.

Note 1-8

1. *The outer top tack is sized using an elastic section modulus. If a plastic section modulus, $Z=(1/4)bt^2$, is used, the factored moment resistance, M_r, is 50% higher.*

   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.

2. *Some manufacturers sell outer top track in F_y = 230 MPa material which will increase the required thickness accordingly.*

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

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

where:

$$P = \left(\frac{\text{Stud Height}}{2}\right) w_s l_w$$
$$= (4/2)(1.32)(0.75)$$
$$= 1.98\,\text{kN/m} = 1.98\,\text{N/mm}$$

L_1 = 152 mm (track width)
L_2 = 41 mm (maximum gap)
E = 203000 MPa
$I = bt^3/12 = 1(1.811)^3/12 = 0.495\ \text{mm}^4$ per mm of length

Substituting into the above equation and solving gives:

$\delta \geq 1.08$ mm *(See Note 1-9)*

Note 1-9:

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

2. *Horizontal movement in the top track detail should be expected and accounted for in the architectural detailing.*

Use 1.811 mm thick outer top track with 60 mm leg length and F_y = 345 MPa. See Note 1-10.

Note 1-10

Sheathings are used in this design example to brace the studs to resist the torsional component of 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 and transfer of the bracing forces to the structure requires special detailing. One choice is illustrated in Figure 2-23 with the last stud anchored to the shearwall or column. Alternatively, the inner and outer top track can be connected together (screws or welds) adjacent to shearwalls or columns where relative slab deflection and/or where the accumulative effect of column axial shortening is not expected to occur. Detail A on Figure 3-16 (or some variation) is another possible choice.

This periodic anchorage of the inner top track is also worthwhile for racking resistance in the plane of the wall to resist seismic forces and construction abuse.

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

Powder 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 powder actuated fasteners:

Tolerance	= ± 6 mm
Deflection	= 12 mm
Concrete screw anchor head	= 0 mm
Minimum installation gap	= Deflection + Screw Head
	= 12 + 0 = 12 mm
Maximum installation gap	= Deflection + Screw Head + 2 x Tolerance
	= 12 + 0 + 2(6) = 24 mm
Maximum possible gap	= Maximum installation gap + Deflection
	= 24 + 12 = 36 mm

And assuming a minimum engagement of 19 mm, the leg of the outer top track must be 36 + 0 + 19 = 55 mm.

Reworking the track thickness calculations from Step 7(a) but with a cantilever leg length of 36 mm gives the following:

t = 1.602 mm
Use next standard design thickness = 1.811 mm with F_y = 345 kPa.

and reworking the outer top track horizontal movement calculations with t = 1.811 mm and L_2= 36 mm gives:

$\delta \geq 0.79$ mm

See Note 1-10.

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.

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-knockout style bridging is used the bridging is typically located 300 mm from the end of the stud. Flat strap bridging can be closer to the end because there is no knockout 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 29 mm, the leg of the outer top track must be 36 + 0 + 29 = 65 mm.

The thickness of the single top track can be checked using the provisions of the Standard for Cold-Formed Steel Framing - Wall Stud Design *(COFS 2004a)*.

From Section C4.3 *(COFS 2004a)*

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

and

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

where:

P_{ndt} = nominal strength of the deflection track when subjected to transverse loads
w_{dt} = effective track length
S = centre to centre 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
ϕ	= 0.45 for Canada

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

Stud Section	
Design Thickness	1.14 mm to 1.81 mm
Design Yield Strength	228 MPa to 345 MPa
Nominal Depth	88.9 mm to 152.4 mm
Nominal Flange Width	41.3 mm to 63.5 mm
Stud Spacing	305 mm to 610 mm
Stud bearing Length	19.1mm minimum

Track Section	
Design Thickness	1.14 mm to 1.81 mm
Design Yield Strength	228 MPa to 345 MPa
Nominal Depth	88.9 mm to 152.4 mm
Nominal Flange Width	50.8 mm to 76.3 mm

In addition, the clear distance from the stud to the end of the track must be greater than or equal to $w_{dt}/2$ *(Commentary COFS 2004a)*.

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

For a typical stud with a load factor of 1.4 and S=400 mm

$$P_f = (1/2)(\text{stud height})(1.4)(1.32)(0.4)$$
$$= (1/2)(4)(1.4)(1.32)(0.4)$$
$$= 1.478 \text{ kN per stud (factored)}$$

Try t = 2.282 mm with e = 36 mm (no fastener head clearance required) and F_y = 345 MPa

$$w_{dt} = 0.11(25.4)^2(36^{0.5} / 2.282^{1.5}) + 5.5(25.4)$$
$$= 263.2\,\text{mm} \leq 400\,\text{mm}$$

$$P_{ndt} = \frac{263.2(2.282)^2(345)}{4(36)}$$
$$= 3284\,\text{N}$$

$$P_r = \phi P_{ndt} = 0.45(3284)/1000$$
$$= 1.478 \text{ kN} = P_f \qquad \textbf{\textit{OK}}$$

Use the next standard design thickness = 2.583 mm

Note that this design thickness exceeds the 1.81 mm limit in COFS 2004a. This extrapolation of the design equations is deemed acceptable under the rational analysis provisions of CAN/CSA-S136-01 Section A1.1 and the $\phi = 0.45$ is more conservative than the resistance factor requirements of that section.

Recheck stud web crippling in the top track.

The CSSBI load tables use the COFS web crippling equations (where the range of parameters is satisfied). See COFS 2004a Section C4.2. This COFS web crippling 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 CAN/CSA-S136-01 for unfastened end one flange loading. See CAN/CSA-S136-01 Table C3.4.1-2.

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

where:

R	= 1.808 mm
t	= 1.146 mm
Depth	= 152.4 mm
h	= Depth - 2t - 2R = 146.5 mm
N	= 19 mm or 29 mm
F_y	= 230 MPa
θ	= 90 degrees
C	= 4
C_R	= 0.14
C_N	= 0.35
C_h	= 0.02
ϕ	= 0.70 (unfastened)

substituting

P_r = 1.31 kN with N = 19 mm
= 1.49 kN with N = 29 mm

and

P_f = 1.4(1.32)(0.400)(4/2)
= 1.48 kN > 1.31 kN with N = 19 mm — ***UNSATISFACTORY***
< 1.49 kN with N = 29 mm — ***OK***

Therefore use minimum engagement of 29 mm for the single top track deflection detail.

For a jamb stud, assume the single top track design provisions apply.

From Step 5(c) with a 1.4 load factor
$P_f = (1.4)(2.508) = 3.51$ kN

$t = 2.583$ mm
$e = 36$ mm
$F_y = 345$ MPa

Substituting gives:

$w_{dt} = 242$ mm
$P_{ndt} = 3.87$ kN

$P_r = \phi P_{ndt} = (0.45)(3.87)$
$= 1.74 \text{ kN} < 3.51 \text{ kN}$ ***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 explicit 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 manufacturer's recommendations. A maximum spacing of 1500 mm o.c. is common and is used here. Therefore, for a 4 m span, two rows of bridging are required at third points.

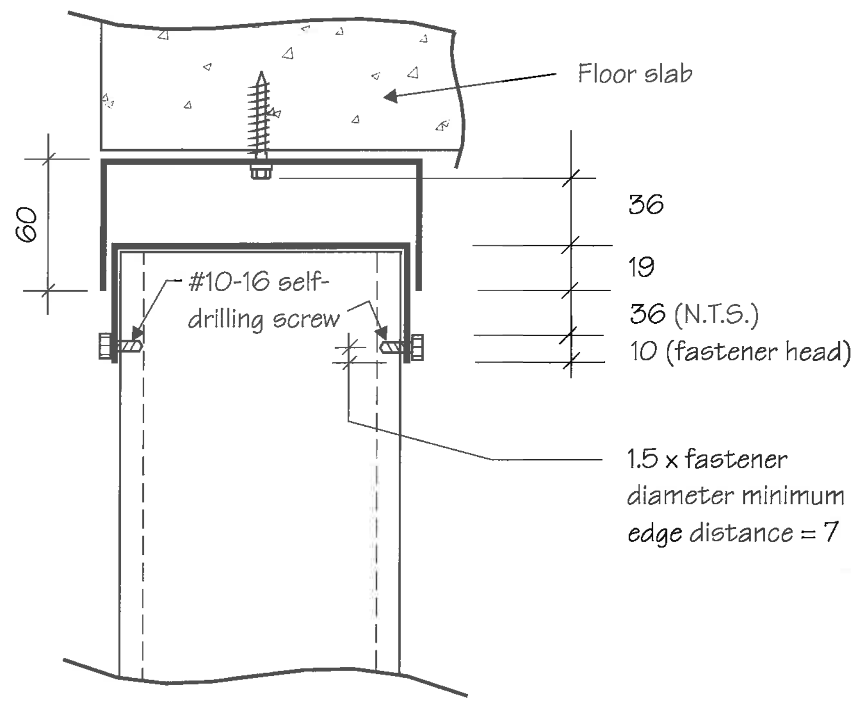
Use 150U50-54 continuous through-the-punchout bridging channel with 38 x 38 x 1.438 x 140 mm long clip angles at each stud. Connect bridging channel to clip angles and clip angles to studs with 2 - #10 self-drilling screws. The screw locations in the clip angles may be defined by pre-drilled pilot holes 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 Step 7(a) for derivation of the deflection gap requirements.



Required inner top track leg length
= 19 + 36 + 10/2 + 7 = 67 mm
Use 70 mm

FIGURE 1-15

Note 1-11

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.*

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

The connection requirements for a track and stud jamb member are not defined in CAN/CSA-S136-01. Experience in the field indicates that a connection spacing of 600 mm o.c. is adequate. The details are shown in Figure 1-16.

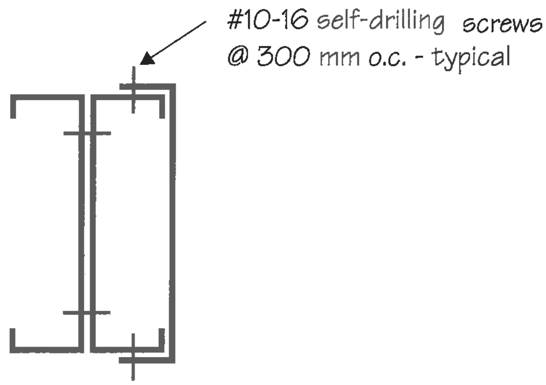


FIGURE 1-16

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.

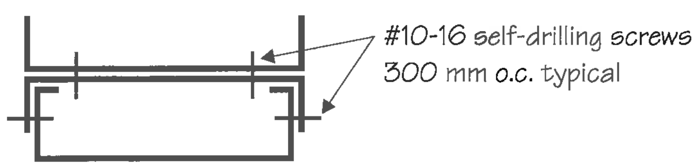


FIGURE 1-17

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

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

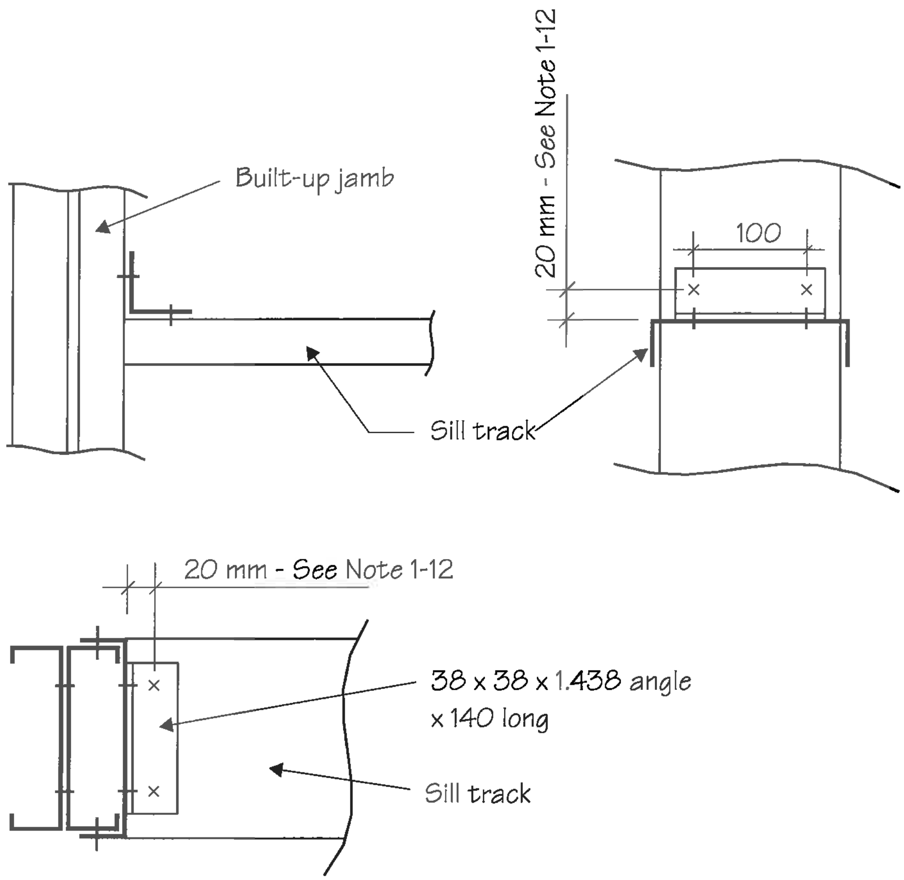


FIGURE 1-18

Note 1-12

1. *Minimum dimension for screw gun clearance varies depending on the manufacturer of the screw gun. A minimum of 16 mm is generally adequate.*
2. *Choose angle one thickness heavier than the connected members but not less than 1.438 mm. This rule of thumb is intended to control deformation in the angle connector.*
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 factored load and the factored resistance for all four screws is the same.

Sill track end shear = V = 1.32 kN (specified). See Step 5(b).

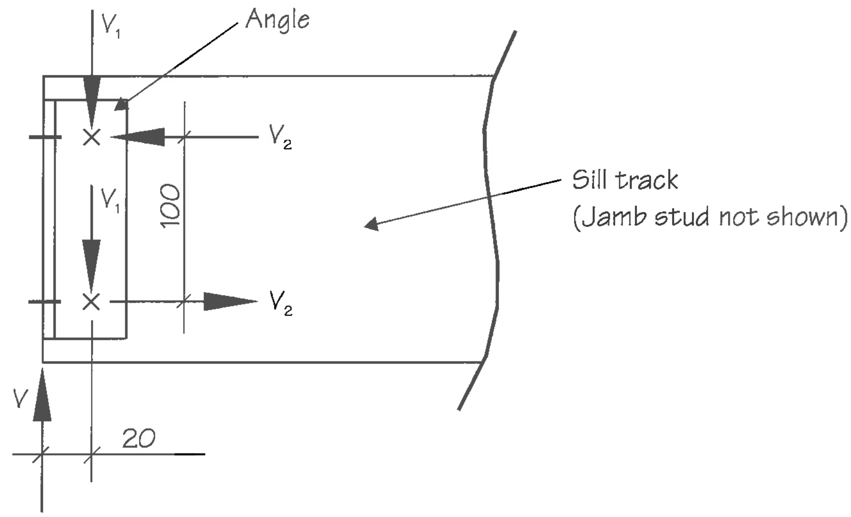


FIGURE 1-19

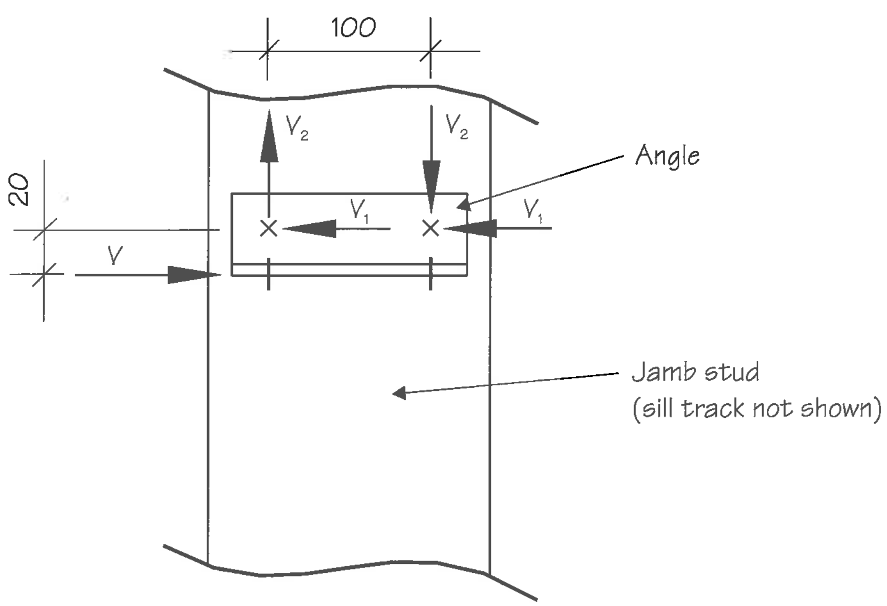


FIGURE 1-20

$V_1 = V/2 = 1.32/2 = 0.66$ kN (specified)
$V_2 = Ve/100 = 20V/100 = 20(1.32)/100 = 0.264$ kN (specified)

Factored shear resultant (1.4 load factor)

$$V_f = 1.4\sqrt{V_1^2 + V_2^2} = 0.995 \text{ kN}$$

Determine screw shear capacities in shear by CAN/CSA-S136-01 Section E4 assuming #10-16 self-drilling screws.

> *Note 1-13*
>
> *CAN/CSA-S136-01 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. CAN/CSA-S136-01allows the use of test values in lieu of the design expressions in E4.*

Screw design input values:

Angle	t_1 = 1.438 mm	F_{u1} = 450 MPa
Track	t_2 = 1.146 mm	F_{u2} = 310 MPa
Jamb	t_2 = 1.146 mm	F_{u2} = 310 MPa
Screw	Size = #10-16	d = 4.83 mm *(Appendix A Table A-2)*

Screw Shear Resistance

Screw shear resistance limited by E4.3.1 tilting and bearing

t_2/t_1 = 0.797 < 1.0 therefore choose the governing P_{ns} from CAN/CSA-S136-01 Equations E4.3.1-1, E4.3.1-2 and E4.3.1-3.

$$P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2} = 3510 \text{ N} - \text{governs}$$
$$P_{ns} = 2.7 t_1 d F_{u1} = 8440 \text{ N}$$
$$P_{ns} = 2.7 t_2 d F_{u2} = 4630 \text{ N}$$

Gives:

$V_r = \phi\, P_{ns}$ = (0.40)(3.51)
= 1.40 kN

Screw shear resistance limited by E4.3.2 end distance *(E4.3.2 in turn refers to C2.2. For amendments to C2.2 see the Supplement S136S1-04 (CSA 2004a))*

See Commentary *(CSA 2001b)* Figure B-C2.2-3 for failure path 5,2,3,6 which is the critical path for this geometry.

Sill track end distance will govern at e = 20 mm rather than the angle at e = 18 mm *(sill track is thinner and has lower Fu)*. Conservatively assume that the resultant shear acts perpendicular to the end of the sill track.

$L_c = 0.6L_{nv}$ = 1.2e - 0.6h *(h is diameter)*
= 1.2(20) - 0.6(4.83)

$= 21.1$ mm

Gives:

$$V_r = \phi_u A_n F_u = \phi_u L_c t F_u$$
$$= 0.75(21.1)(1.146)(310)/1000$$
$$= 5.62 \text{ kN}$$

Screw shear resistance limited by E4.3.3 shear in the screws themselves. Refer to the Supplement S136S1-04 *(CSA 2004a)*

$$P_{ns} = P_{ss}$$

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

$$V_r = \phi P_{ns} = \phi P_{ss} = 0.40(6.23)$$
$$= 2.49 \text{ kN}$$

The governing V_r is from E4.3.1 and is given by

$V_r = 1.40$ kN > 0.995 kN ***OK***

Note 1-14

1. *Add a stud under the connection to resist dead load and construction abuse at the time of window installation. See optional cripple stud 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 The Standard for Cold-Formed Steel Framing – Wall Stud Design (COFS 2004a) Section C4.2(c)*

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. See Figure 1-21.

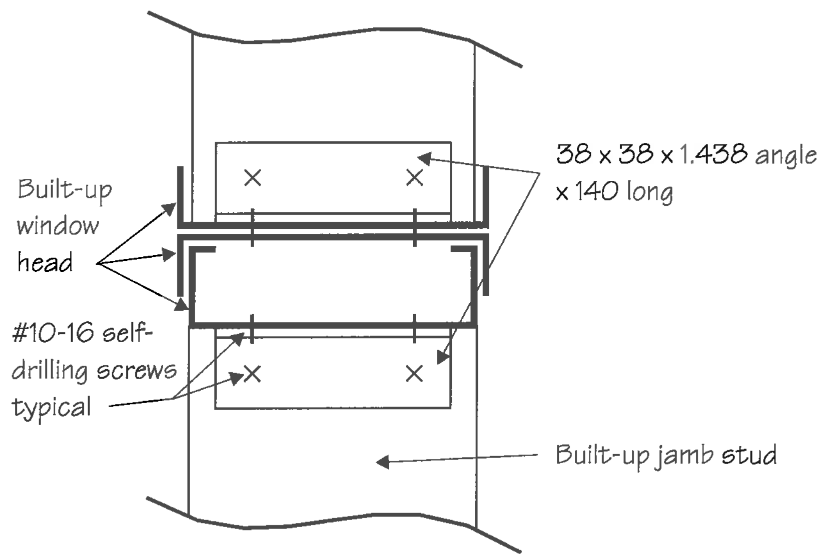


FIGURE 1-21

Step 8(h) – Bottom Track to Concrete Connection

The bottom track to concrete connection is designed for three types of concrete anchors – wedge type, screw type and powder actuated. See Note 1-15.

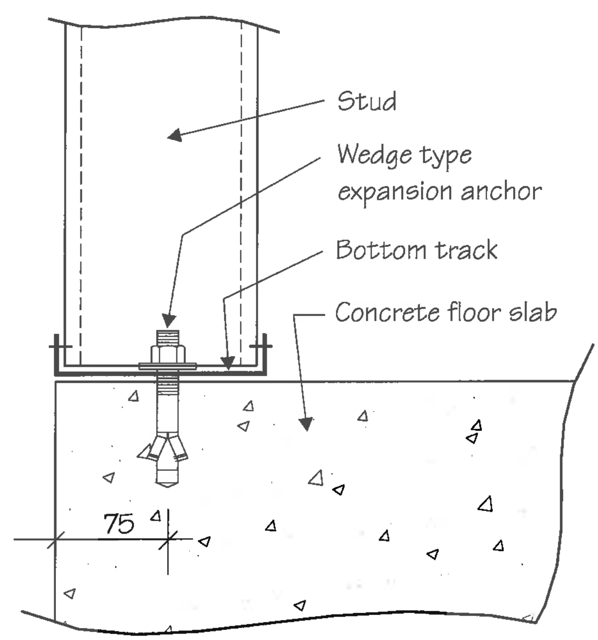


FIGURE 1-22

Note 1-15

1. *Reference Drysdale 1991 recommends an anchor spacing less than or equal to 800 mm o.c. regardless of the type of anchor used. This spacing is necessary to control local and overall track deformations.*
2. *The bottom track anchor is assumed to be loaded in shear only with negligible pull-out due to prying.*

Applied factored load to bottom track (with 1.4 load factor)

$$V_f = (1/2)(\text{Stud Height})(1.32)(1.4)$$
$$= (4/2)(1.32)(1.4) = 3.70 \text{ kN/m for strength}$$

Concrete floor slab strength $f_c' = 25$ MPa

i) Alternative (a) – 6.4 mm diameter wedge type expansion anchor

Factored shear resistance of anchor

For load data see Appendix B.1 Tables B.1-1 and B.1-2.

$f_c' = 25$ MPa
Edge distance = 75 mm

Choose standard embedment = $h_{nom} = 51$ mm

Edge distance reduction factor from Appendix B Table B.1-2 and B.1-3

$$f_{RV1} = \frac{c}{3h_{min}} = \frac{75}{3(29)} = 0.862$$

$$V_r = f_{RV1} R_r$$
$$= f_{RV1} (\phi_e \overline{X} / 1.33)$$

Interpolation for $f_c' = 25$ MPa. See Appendix B.1 Notes.

f_c' (MPa)	$\overline{X}$ (kN)
20.7	8.9
27.6	8.9
25	8.9 (by interpolation - trivial for this case)

Then $V_r = 0.862(0.4)(8.9)/1.33$
$= 2.31$ kN /fastener

Check Bearing by CAN/CSA-S136-01 Section E3.3.1 (without consideration of bolt hole deformation)

$P_n = m_f CdtF_u$

$d/t = 6.4/1.146 = 5.59 < 10$
Therefore, $C = 3$

$m_f = 0.75$ (without washer)

$P_n = 0.75(3)(6.4)(1.146)(310)/1000$
$= 5.12$ kN

$P_r = \phi P_n = 0.50(5.12)$
$= 2.56$ kN

$V_r = 2.31$ kN/fastener governs

Required fastener spacing
$= (V_r/V_f)(1000) = (2.31/3.70)(1000)$
$= 624$ mm o.c.

Use 600 mm o.c. < 800 mm o.c. *(See Note 1-15)* ***OK***

Fastener requirements at jamb studs

V_f = factored jamb bottom reaction (with 1.4 load factor)
$= 1.4(2.51)$ (from Step 5(c))
$= 3.51$ kN

Provide 2 fasteners adjacent to jamb

$V_r = 2(2.31)$
$= 4.62$ kN > 3.51 kN ***OK***

Space fasteners at $S_{crit} = 2.25h_{act} = 2.25(51) = 115$ mm o.c. say 60 mm each side of jamb.

Bottom track bending strength between fasteners

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

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

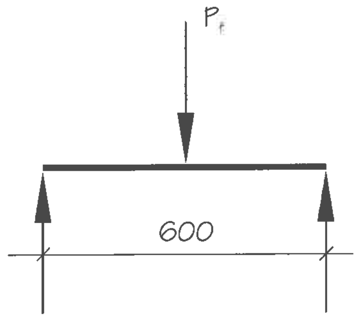


FIGURE 1-23

With 1.4 load factor for a typical stud
$P_f = 1.4(4/2)(0.4)(1.32)$
$= 1.48$ kN

$M_f = P_f L/4 = 1.48(0.6)/4$
$= 0.22$ kN.m

For 600T125-43 track with $F_y = 230$ MPa
$M_r = 1.55$ kN.m >> 0.22 kN.m ***OK***

ii) Alternative (b) – 9.5 mm diameter wedge type expansion anchor

Factored shear resistance of anchor

For load data see Appendix B.1 Tables B.1-1 and B.1-2.

$f_c' = 25$ MPa
Edge distance = 75 mm

Choose standard embedment = $h_{nom} = 64$ mm

Edge distance reduction factor from Appendix B Table B.1-2 and B.1-3

$$f_{RV1} = \frac{c}{3h_{min}} = \frac{75}{3(41)} = 0.610$$

$$V_r = f_{RV1} R_r$$
$$= f_{RV1} (\phi_e \overline{X} / 1.33)$$

Interpolation for f_c' = 25 MPa. See Appendix B.1 Notes.

f_c' (MPa)	$\overline{X}$ (kN)
20.7	21.9
27.6	21.9
25	21.9 (by interpolation - trivial for this case)

Then $V_r = 0.610(0.4)(21.9)/1.33$
$= 4.02$ kN /fastener

Check Bearing by CAN/CSA-S136-01 Section E3.3.1 (without consideration of bolt hole deformation)

$P_n = m_f C d t F_u$

$d/t = 9.5/1.146 = 8.29 < 10$
Therefore, C = 3

$m_f = 0.75$ (without washer)

$P_n = 0.75(3)(9.5)(1.146)(310)/1000$
$= 7.59$ kN

$P_r = \phi P_n = 0.50(7.59)$
$= 3.80$ kN

$P_r = 3.80$ kN/fastener governs

Required fastener spacing
$= (V_r/V_f)(1000) = (3.80/3.70)(1000)$
= 1030 mm o.c. > 800 mm o.c. *(See Note 1-15)* ***OK***

Use 800 mm o.c.

Fastener requirements at jamb studs

V_f = factored jamb bottom reaction (with 1.4 load factor)
= 1.4(2.51) (from Step 5(c))
= 3.51 kN

Provide 1 fastener adjacent to jamb

$V_r = 3.80$ kN > 3.51 kN ***OK***

Bottom track bending strength between fasteners

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

From standard beam diagrams – for 2 equal concentrated loads the moment is maximum with the loads located as in Figure 1-24.

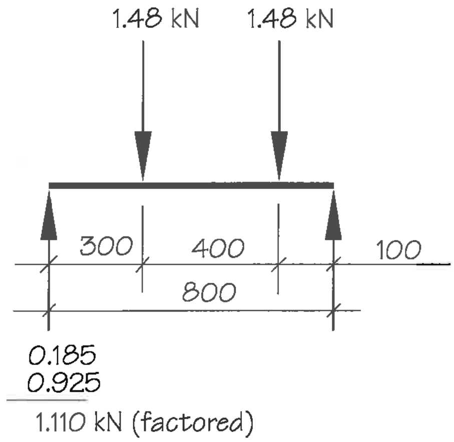


FIGURE 1-24

With 1.4 load factor for a typical stud
P_f = 1.4(4/2)(0.4)(1.32)
= 1.48 kN

M_f =1.110(0.300)
= 0.333 kN.m

For 600T125-43 track with F_y = 230 MPa
M_r = 1.55 kN.m >> 0.333 kN.m ***OK***

iii) Alternative (c) – 6.4 mm diameter concrete screw anchor

See Appendix B.2, Table B.2-1

f_c' = 25 MPa
Edge distance = 75 mm

Choose embedment depth = 32 mm

Factored shear resistance

For 100% anchor efficiency, required edge distance
= 10 diameters = 10 x 6.4 = 64 mm < 75 mm therefore no reduction.

$V_r = \phi_e \overline{X} / 1.33$

Interpolation for f_c' = 25 MPa. See Appendix B Notes B.2.

f_c' (MPa)	$\bar{X}$ (kN)
20.7	7.1
27.6	7.4
25	7.29 (by interpolation)

Then $V_r = 0.4(7.29)/1.33$
$= 2.19$ kN /fastener

Check Bearing by CAN/CSA-S136-01 Section E3.3.1 (without consideration of bolt hole deformation – assume this bolt bearing provision is applicable to concrete screw anchors)

$P_r = \phi P_n = 2.56$ kN *(from 8(h)(i))*

$V_r = 2.19$ kN/fastener governs

$V_f = 3.70$ kN/m

Required fastener spacing
$= (V_r/V_f)(1000) = (2.19/3.70)(1000)$
= 592 mm o.c. < 800 mm o.c. *(See Note 1-15)* ***OK***

Use 600 mm o.c.

Fastener requirements at jamb studs

V_f = factored jamb bottom reaction (with 1.4 load factor)
= 1.4(2.51) (from Step 5(c))
= 3.51 kN

Provide 2 fasteners adjacent to jamb

$V_r = 2.19(2) = 4.38$ kN > 3.51 kN ***OK***

Minimum spacing = 12 diameters = 12(6.4) = 77 mm o.c.

Space fasteners 40 mm either side of jamb stud.

iv) Alternative (d) – 3.68 mm diameter powder actuated fastener

See Appendix B.3, Table B.3-1

f_c' = 25 MPa
Edge distance = 75 mm

Choose embedment depth = 32 mm

Factored shear resistance

For 100% anchor efficiency, required edge distance
= 60 mm < 75 mm therefore no reduction.

$V_r = \phi_e \overline{X} / 1.33$

Interpolation for f_c' = 25 MPa. See Appendix B Notes B.2.

f_c' (MPa)	$\overline{X}$ (kN)
13.8	6.4
27.6	7.3
25	7.13 (by interpolation)

Then V_r = 0.32(7.13)/1.33
= 1.72 kN /fastener

Check Bearing by CAN/CSA-S136-01 Section E3.3.1 (without consideration of bolt hole deformation – assume this bolt bearing provision is applicable to powder actuated fasteners)

$P_n = m_f C d t F_u$

d/t = 3.68/1.146 = 3.21 < 10
Therefore, C = 3

m_f = 0.75 (without washer)

P_n = 0.75(3)(3.68)(1.146)(310)/1000
= 2.94 kN

$P_r = \phi P_n$ = 0.50(2.94)
= 1.47 kN

P_r = 1.47 kN/fastener governs

V_f = 3.70 kN/m

Required fastener spacing
= $(V_r/V_f)(1000)$ = (1.47/3.70)(1000)
= 397 mm o.c. < 800 mm o.c. *(See Note 1-15)* ***OK***

Use 400 mm o.c.

Fastener requirements at jamb studs

V_f = factored jamb bottom reaction (with 1.4 load factor)
= 1.4(2.51) (from Step 5(c))

= 3.51 kN

Provide 3 fasteners adjacent to jamb

V_r = 3(1.47) = 4.41 kN > 3.51 kN **_OK_**

Space fasteners 70 mm o.c.

Alternatively provide 1 - 9.5 mm diameter wedge type expansion anchor adjacent to jamb *(From 8(h)(ii))*.

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

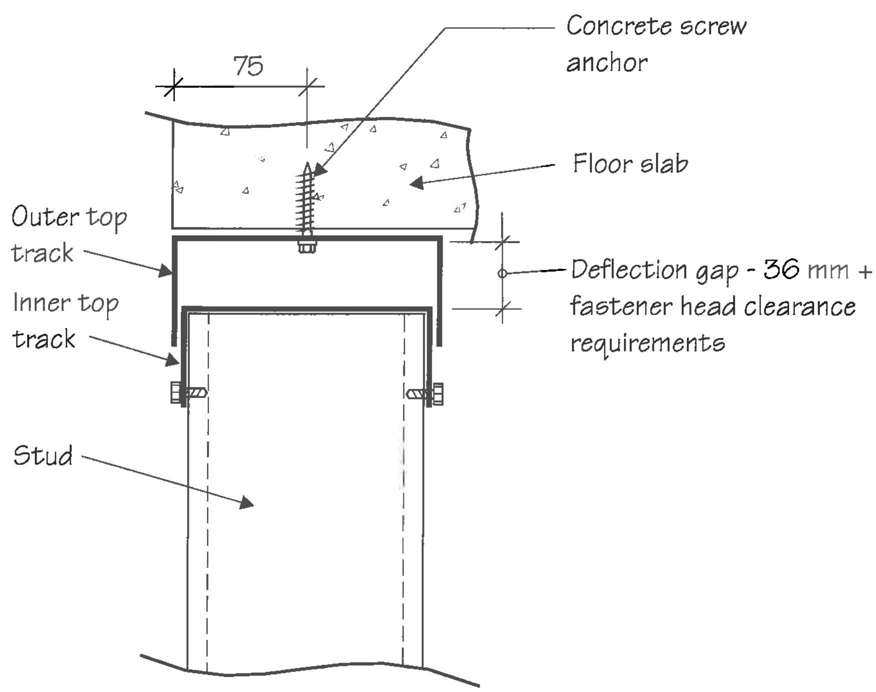


FIGURE 1-25

Note 1-16

1. *The maximum anchor spacing of 800 mm o.c. recommended in Drysdale 1991 applies to bottom track. For top track, this recommended maximum spacing is reduced to 600 mm to account for the absence of torsional restraint from the studs. This 600 mm 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.*

Applied load to top track (with a load factor of 1.4)

V_f = (1/2)(Stud Height)(1.32)(1.4)
= (1/2)(4)(1.32)(1.4)
= 3.70 kN/m for strength

i) Alternative (a) – 6.4 mm diameter concrete screw anchor

The total deflection gap including fastener head clearance is 36 + 5 = 41 mm. See Figure 1-26.

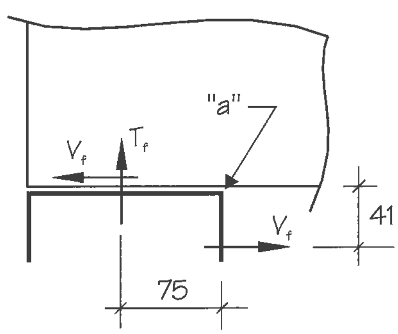


FIGURE 1-26

Factored loads

V_f = 3.70 kN/m

Taking moments about "a" Figure 1-26 gives:

$T_f = 41V_f/75 = 41(3.70)/75$
$= 2.02$ kN/m

Factored resistances for shear and tension

f_c' = 25 MPa
Edge distance = 75 mm

Choose embedment depth = 32 mm

Factored shear resistance

V_r = 2.19 kN/fastener *(from Step 8(h)iii)*

Factored tensile resistance

For 100% anchor efficiency, required edge distance
= 10 diameters = 10 x 6.4 = 64 mm < 75 mm therefore no reduction.

$$T_r = \phi_e \overline{X} / 1.33$$

Interpolation for f_c' = 25 MPa. See Appendix B Notes B.2.

f_c' (MPa)	$\overline{X}$ (kN)
20.7	5.0
27.6	5.5
25	5.31 (by interpolation)

Then T_r = 0.4(5.31)/1.33
= 1.60 kN /fastener

Check interaction using equation from Appendix B.2 Note #5 with s = fastener spacing in metres:

$$\left(\frac{sT_f}{T_r}\right) + \left(\frac{sV_f}{V_r}\right) \leq 1.0$$

Substituting and solving for "s":

$$\left(\frac{s2.02}{1.60}\right) + \left(\frac{s3.70}{2.19}\right) \leq 1.0$$

s = 339 mm o.c.

Use 300 mm o.c.

ii) Alternative (b) – 3.68 mm diameter powder actuated fastener

The total deflection gap including fastener head clearance is 36 + 0 = 36 mm. See Figure 1-27. *(Note that where earthquake design is a consideration, there may be restrictions on the use of powder actuated fasteners in tension - see NBC 2005 Clause 4.1.8.17 (8) (d))*

Factored loads

$V_f = 3.70$ kN/m

Taking moments about "a" Figure 1-27 gives:

$T_f = 36V_f/75 = 36(3.70)/75$
$= 1.78$ kN/m

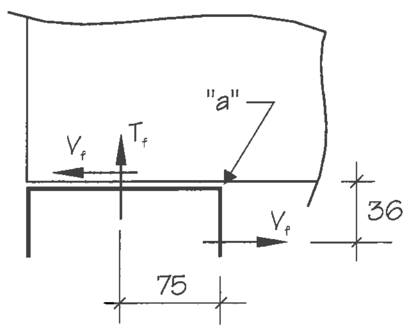


FIGURE 1-27

Factored resistances for shear and tension

$f_c' = 25$ MPa
Edge distance = 75 mm

Choose embedment depth = 32 mm

Factored shear resistance *(Note that for purposes of a shear/tension interaction check the bearing limit state for factored shear resistance does not apply)*

$V_r = 1.72$ kN/fastener *(from Step 8(h)iv)*

Factored tensile resistance

For 100% anchor efficiency, required edge distance
= 60 mm < 75 mm therefore no reduction.

$T_r = \phi_e \overline{X} / 1.33$

Interpolation for f_c' = 25 MPa. See Appendix B Notes B.2.

f_c' (MPa)	$\overline{X}$ (kN)
13.8	3.6
27.6	5.1
25	4.82 (by interpolation)

Then T_r = 0.32(4.82)/1.33
= 1.16 kN /fastener

Check interaction using equation from Appendix B.3 Note #3 with s = fastener spacing in metres:

$$\left(\frac{sT_f}{T_r}\right)+\left(\frac{sV_f}{V_r}\right)\leq 1.0$$

Substituting and solving for "s":

$$\left(\frac{s1.78}{1.16}\right)+\left(\frac{s3.70}{1.72}\right)\leq 1.0$$

s = 271 mm o.c.

Use 250 mm o.c.

Check bearing for fasteners at 250 mm o.c.

V_f = 3.70(0.25) = 0.925 kN/fastener

P_r = 1.47 kN/fastener *(from Step 8(h)iv)* > 0.925 kN **OK**

Note that powder actuated fasteners have relatively small head diameters and pull-over and the interaction of shear and pull-over should be checked as a possible limit states. Check this limit state for the 250 mm o.c. fastener spacing.

V_f = 3.70(0.25) = 0.925 kN/fastener
T_f = 1.78(0.25) = 0.445 kN/fastener

Assume the pull-over provisions for screws apply to powder actuated fasteners. See CAN/CSA-S136-01 Section E4.4.2.

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

where:
t_1 = 1.811 mm (outer top track)
d_w = 8.18 mm (fastener head diameter Table B.3-1)
F_{u1} = 450 MPa

$$P_{nov} = 1.5(1.811)(8.18)(450)/1000 = 10.00 \text{ kN}$$

$$P_r = \phi P_{nov} = 0.40(10.00) = 4.00 \text{ kN}$$

Check interaction for combined shear an pull-over using Supplement S136S1-04 *(CSA 2004a)* Section 4.5.

$$\frac{\overline{Q}}{P_{ns}} + 0.71\frac{\overline{T}}{P_{nov}} \leq 1.10\,\phi$$

$$\overline{Q} = V_f = 0.925 \text{ kN}$$
$$\overline{T} = T_f = 0.445 \text{ kN}$$
$$P_{ns} = 2.7t_1dF_{u1} = 2.7(1.811)(3.68)(450)/1000 = 8.10 \text{ kN}$$
$$P_{nov} = 10.00 \text{ kN}$$

$$\frac{0.925}{8.10} + \frac{(0.71)(0.445)}{10.00} \leq 1.10(0.55)$$

Gives $0.146 < 0.605$ ***OK***

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

i) Alternative (a) – 6.4 mm diameter concrete screw anchor

The concrete screw anchor spacing derived for the inner and outer top track case (339 mm o.c. actual rounded to 300 mm o.c.) could be used here. *(See Step 8(i)i)* . However, the fastener spacing can be increased because the deflection gap does not have to provide head clearance for the screw anchor. For a deflection gap of 36 mm from Figure 1-27 and Step 8(i)ii:

$$V_f = 3.70 \text{ kN/m}$$
$$T_f = 1.78 \text{ kN/m}$$

And from Step 8(i)i

$$V_r = 2.19 \text{ kN/fastener}$$
$$T_r = 1.60 \text{ kN/fastener}$$

Check interaction using equation from Appendix B.2 Note #5 with s = fastener spacing in metres:

$$\left(\frac{sT_f}{T_r}\right) + \left(\frac{sV_f}{V_r}\right) \leq 1.0$$

Substituting and solving for "s":

$$\left(\frac{s1.78}{1.60}\right)+\left(\frac{s3.70}{2.19}\right)\leq 1.0$$

s = 357 mm o.c.

Use 350 mm o.c. < 400 mm o.c. *(See Note1-17)* ***OK***

> *Note 1-17*
>
> *The single top deflection track design provisions in COFS 2004a 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.*

ii) Alternative (b) – 3.68 mm diameter powder actuated fastener

The analysis here will be the same as for the inner and outer top track from Step 8(i)ii. *(Note that where earthquake design is a consideration, there may be restrictions on the use of powder actuated fasteners in tension - see NBC 2005 Clause 4.1.8.17 (8) (d))*

Use 250 mm o.c. < 400 mm o.c. ***OK***

iii) Alternative (c) – 6.4 mm diameter wedge type expansion anchor

From Figure 1-27 and Step 8(i)ii:

V_f = 3.70 kN/m
T_f = 1.78 kN/m

For factored shear resistance see Step 8(h)i

V_r = 2.31 kN/fastener

Factored tensile resistance

For load data see Appendix B.1 Tables B.1-1 and B.1-2.

f_c' = 25 MPa
Edge distance = 75 mm

For standard embedment = h_{nom} = 51 mm

Edge distance reduction factor from Appendix B Table B.1-2 and B.1-3

$$f_{RN} = \frac{\frac{c}{h_{act}} + 2.00}{3.75} = \frac{\frac{75}{51} + 2.00}{3.75} = 0.925$$

$$T_r = f_{RN} R_r$$
$$= f_{RN} (\phi_e \overline{X} / 1.33)$$

Interpolation for f_c' = 25 MPa. See Appendix B.1 Notes.

f_c' (MPa)	$\overline{X}$ (kN)
20.7	12.0
27.6	13.3
25	12.8 (by interpolation)

Then T_r = 0.925(0.4)(12.8)/1.33
= 3.56 kN /fastener

Check interaction of shear and moment using the equation from Appendix B.1 Note #3 with S = fastener spacing in metres.

$$\left(\frac{ST_f}{T_r}\right)^{5/3} + \left(\frac{SV_f}{V_r}\right)^{5/3} \leq 1.0$$

$$\left(\frac{1.78\,S}{3.56}\right)^{5/3} + \left(\frac{3.70S}{2.31}\right)^{5/3} \leq 1.0$$

Solving by trial and error

S = 576 mm o.c. > 400 mm o.c.

Use 400 mm o.c. See Note 1-17.

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

Top or Bottom Track	Anchor Size & Type	Embedment Depth (mm)	Spacing (mm)	Number of Fasteners at Jambs
Bottom Track	6.4 mm diameter wedge anchor	51	600	2
	9.5 mm diameter wedge anchor	64	800	1
	6.4 mm diameter screw anchor	32	600	2
	3.68 mm diameter powder actuated anchor	32	400	3
Inner/Outer Top Track	6.4 mm diameter screw anchor	32	300	*Note 1*
	3.68 mm diameter powder actuated anchor	32	250	*Note 1*
Single Outer Top Track	6.4 mm diameter screw anchor	32	350	*Note 2*
	3.68 mm diameter powder actuated anchor	32	250	*Note 2*
	6.4 mm diameter wedge anchor	51	400	*Note 2*

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 powder actuated fasteners in tension - see NBC 2005 Clause 4.1.8.17 (8) (d))*

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 LSF 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 1-28.

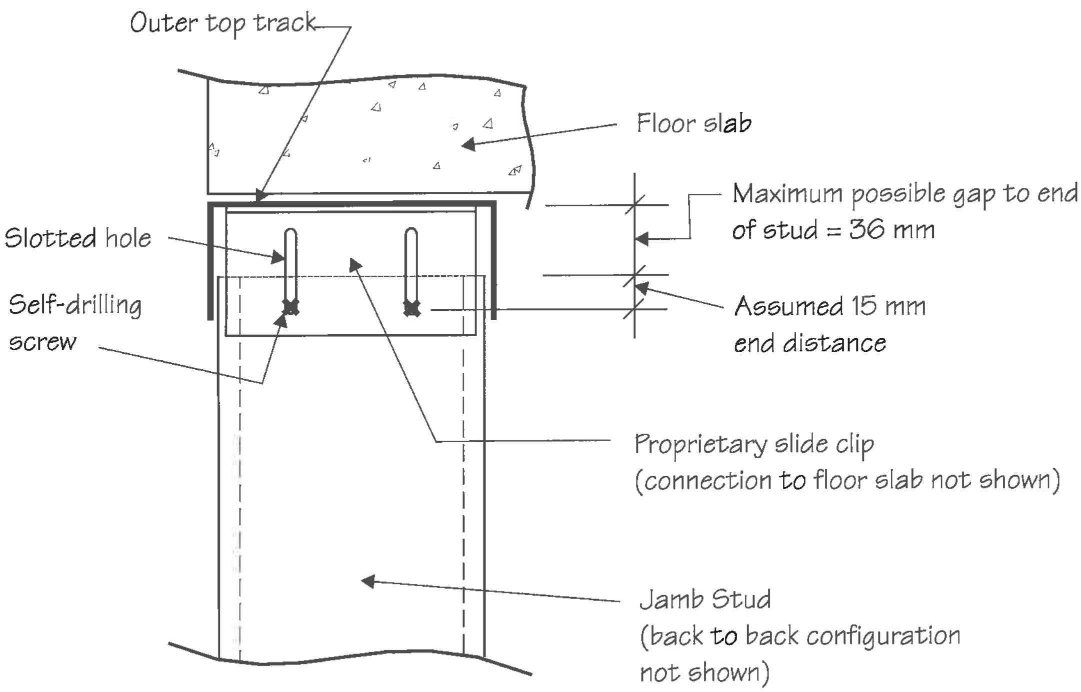


FIGURE 1-28

For this connection, the 9.5 mm diameter wedge type expansion anchor has been selected with 50 mm embedment.

From Table B.1-3 *(Appendix B)*:
h_{min} = 41 mm
h_{nom} = 51 mm

From Table B.1-2 *(Appendix B)* for $h_{min} \leq h_{act} \leq h_{nom}$

The minimum edge distance is governed by shear and is given by:
$c_{min} = 1.5h_{min} = 1.5(41) = 62$ mm.

The minimum fastener to fastener spacing is given by:
$s_{min} = h_{act} = 50$ mm.

Instead use 75 mm minimum edge distance to allow for some variation in the location of the slab edge along with the 50 mm minimum spacing. Using these distances, the concrete anchors will be asymmetrically placed as shown in Figures 1-29 and 1-30.

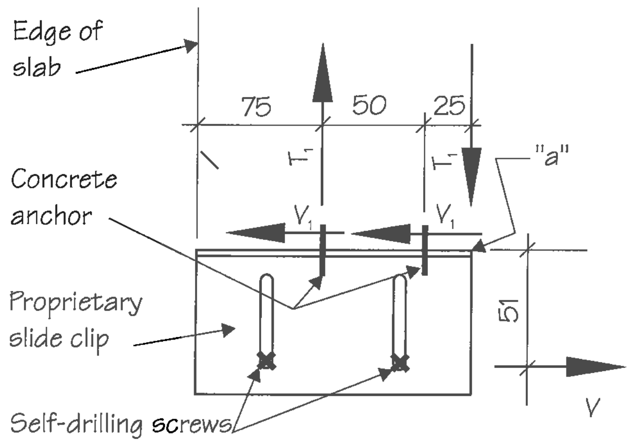


FIGURE 1-29

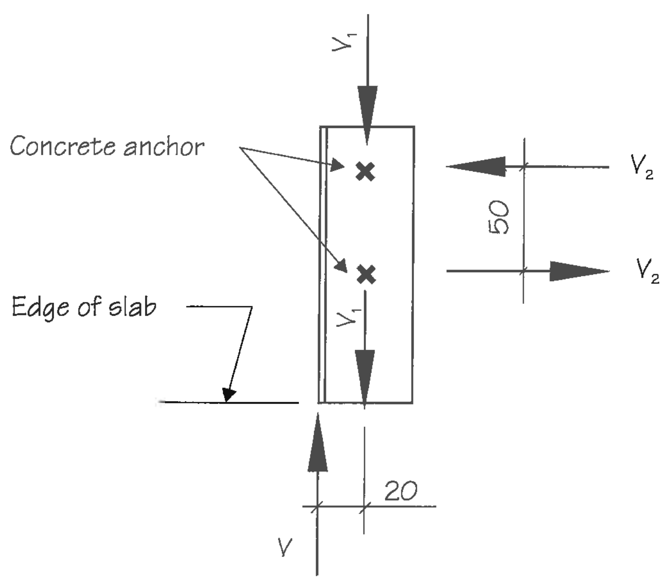


FIGURE 1-30

Factored shear and tension *(See Figures 1-29 & 1-30)*

V = jamb reaction = 2.51 kN (specified) from Step 5(c).

$V_1 = V/2 = 2.51/2$
$= 1.26$ kN (specified)

$V_2 = 20V/50 = 20(2.51)/50$
$= 1.00$ kN (specified)

Factored shear resultant with 1.4 load factor

$$V_f = 1.4\sqrt{V_1^{\,2} + V_2^{\,2}} = 1.4\sqrt{1.26^2 + 1.00^2}$$
$$= 2.25 \text{ kN}$$

Moments about "a" assuming 1 fastener only in tension

$T_1 = 51V/(25+50) = 51(2.51)/75$
$= 1.71$ kN (specified)

Factored tension with 1.4 load factor

$T_f = 1.4(1.71)$
$= 2.39$ kN

Factored shear resistance – double interpolation required – one for embedment depth and one for f_c'.

$$R_r = \phi_e \overline{X} / 1.33$$

Interpolation for embedment depth at f_c' = 20.7 MPa

Embedment (mm)	$\overline{X}$ (kN)
41	21.3
64	21.9
50	21.5 (by interpolation)

Interpolation for embedment depth at f_c' = 27.6 MPa

Embedment (mm)	$\overline{X}$ (kN)
41	21.9
64	21.9
50	21.9 (by interpolation)

Interpolation for f_c' = 25 MPa

f_c' (MPa)	$\overline{X}$ (kN)
20.7	21.5 (from above)
27.6	21.9 (from above)
25	21.7 (by interpolation)

Then R_r = 0.4(21.7)/1.33
= 6.53 kN /fastener

Shear edge distance reduction factor

$$f_{RV1} = \frac{c}{3h_{min}} = \frac{75}{3(41)} = 0.610$$

Shear centre/centre spacing reduction factor

$$f_{AV} = \frac{\frac{s}{h_{act}} + 10.25}{12.5} = \frac{\frac{50}{50} + 10.25}{12.5} = 0.900$$

Then

$V_r = f_{RV1} \,.\, f_{AV} \,.\, R_r$ = 0.610(0.900)(6.53)
= 3.58 kN

Factored tensile resistance – double interpolation required – one for embedment depth and one for f_c'.

$$R_r = \phi_e \overline{X} / 1.33$$

Interpolation for embedment depth at f_c' = 20.7 MPa

Embedment (mm)	$\overline{X}$ (kN)
41	15.2
64	25.9
50	19.4 (by interpolation)

Interpolation for embedment depth at f_c' = 27.6 MPa

Embedment (mm)	$\overline{X}$ (kN)
41	18.2
64	30.8
50	23.1 (by interpolation)

Interpolation for f_c' = 25 MPa

f_c' (MPa)	$\overline{X}$ (kN)
20.7	19.4 (from above)
27.6	23.1 (from above)
25	21.7 (by interpolation)

Then $R_r = 0.4(21.7)/1.33$
$= 6.53$ kN /fastener

Tension edge distance reduction factor

$$f_{RN} = \frac{\frac{c}{h_{act}} + 2.00}{3.75} = \frac{\frac{75}{50} + 2.00}{3.75} = 0.933$$

Tension centre/centre spacing reduction factor

$$f_{AV} = \frac{\frac{s}{h_{act}} + 0.88}{3.13} = \frac{\frac{50}{50} + 0.88}{3.13} = 0.601$$

Then

$T_r = f_{RN} \,.\, f_{AN} \,.\, R_r = 0.933(0.601)(6.53)$
$= 3.66$ kN

Check interaction of tension and shear using the equation from Appendix B.1 Note #3.

$$\left(\frac{T_f}{T_r}\right)^{5/3} + \left(\frac{V_f}{V_r}\right)^{5/3} \leq 1.00$$

$$\left(\frac{2.39}{3.66}\right)^{5/3} + \left(\frac{2.25}{3.58}\right)^{5/3} = 0.95 \leq 1.00$$ ***OK***

Step 8(l) – 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 1-31 and 1-33.

i) Alternative (a) - Field welding

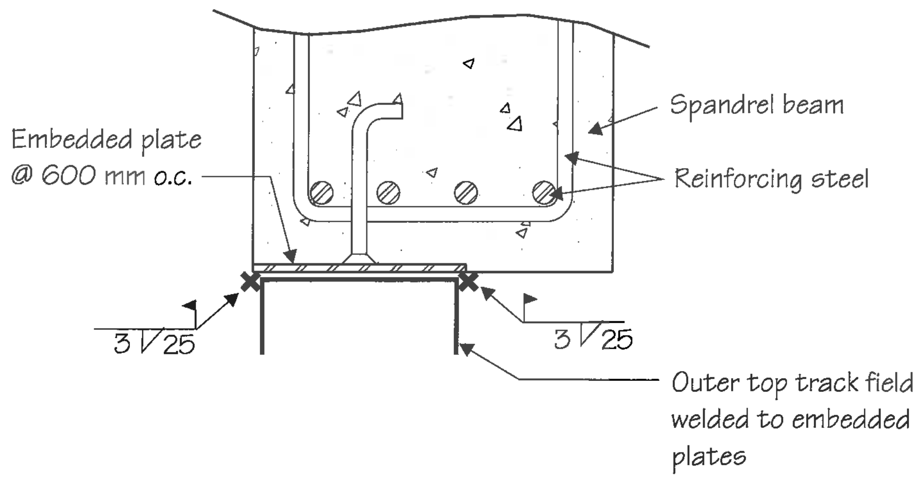


FIGURE 1-31

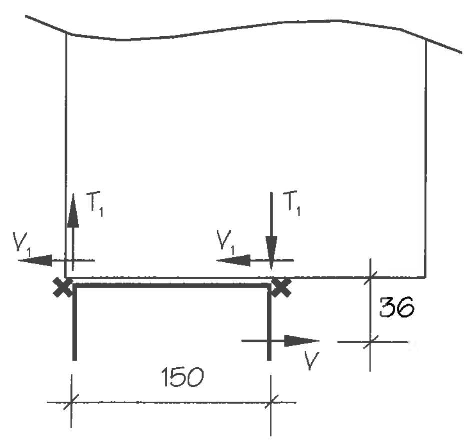


FIGURE 1-32

Assume embedded plates at maximum recommended spacing for anchoring outer top track = 600 mm o.c. *(Note 1-16)*.

Check weld as shown in Figure 1-32

Applied load to track = V = 2(1.32) = 2.64 kN/m (specified)

For welds at 600 mm o.c.

$V_1 = (0.600)(2.64)/2$
$= 0.792$ kN per weld (specified)

By taking moments
$T_1 = 36(0.600)V/150 = 36(0.600)(2.64)/150$
$= 0.380$ kN per weld (specified)

Factored resultant force on each weld with 1.4 load factor
Factored resultant $= 1.4\sqrt{V_1^2 + T_1^2} = 1.23$ kN per weld

Factored resistance for a weld length of L mm *(See Note 1-18)*

$V_r = 0.75\phi tLF_u$
$= 0.75(0.40)(1.811)L(450)$
$= 244L$ N

For resultant = V_r

L = 1.23/0.244 = 5.00 mm required length of weld

Use L = 25 mm as a minimum practical weld length

Note 1-18

1. *A simplified approach to weld strengths is used in this Manual. 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, $0.75\phi tLF_u$.*
2. *For this simplified method, the strength of fillet and flare groove welds in cold formed steel thicknesses less than or equal to 2.54 mm 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 3 mm accompanied by a note "For material less than or equal to 2.54 mm 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 2002b, Part I.*

ii) Alternative (b) – Powder actuated fasteners

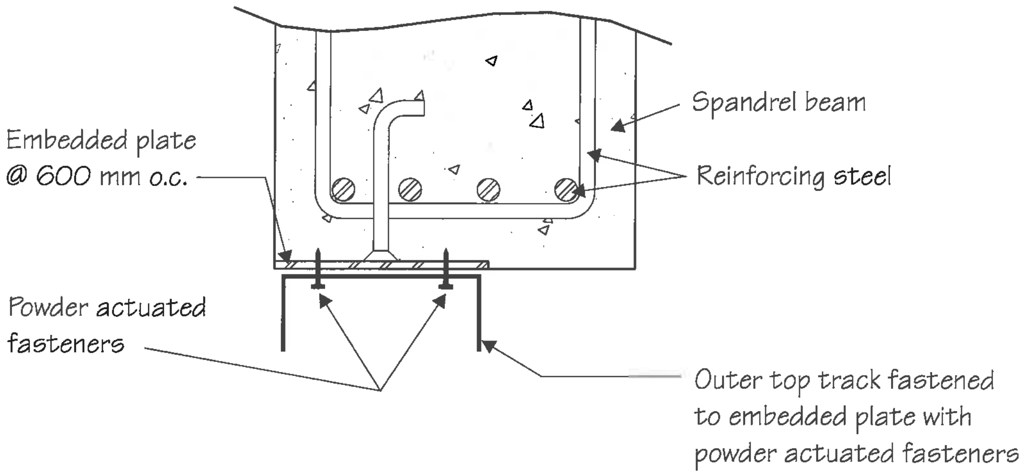


FIGURE 1-33

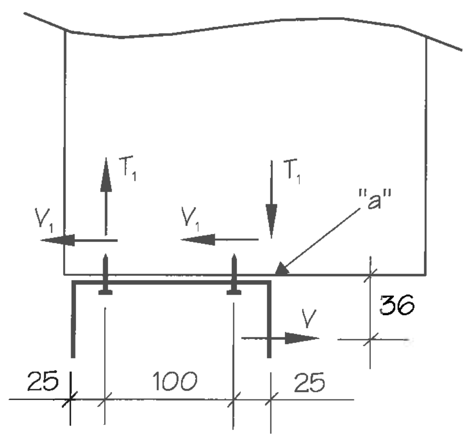


FIGURE 1-34

Factored loads on fasteners

V = 2.64 kN/m (specified) from previous step

With 1.4 load factor

V_{1f} = 1.4(0.6)(2.64)/2 kN/m
= 1.11 kN/fastener

and taking moments about "a"

T_{1f} = 1.4(0.6)(2.64)(36/(125)
= 0.639 kN/fastener

Factored resistances from Appendix B Table B.4-1

Minimum edge distance = 12 mm < 25 mm ***OK***

$$V_r = \phi_e \bar{x} / 1.33 = 0.32(13.8)/1.33$$
$$= 3.32 \text{ kN}$$

$$T_r = \phi_e \bar{x} / 1.33 = 0.32(15.0)/1.33$$
$$= 3.61 \text{ kN}$$

Interaction check

$$\left(\frac{T_f}{T_r}\right) + \left(\frac{V_f}{V_r}\right) \leq 1.0$$

$$\left(\frac{0.639}{3.61}\right) + \left(\frac{1.11}{3.32}\right) = 0.51 < 1.00$$ ***OK***

(Note that where earthquake design is a consideration, there may be restrictions on the use of powder actuated fasteners in tension - see NBC 2005 Clause 4.1.8.17 (8) (d))

Design Example #2
Wind Bearing Infill Wall with an Unsheathed Design Approach and Welded or Screwed Connections

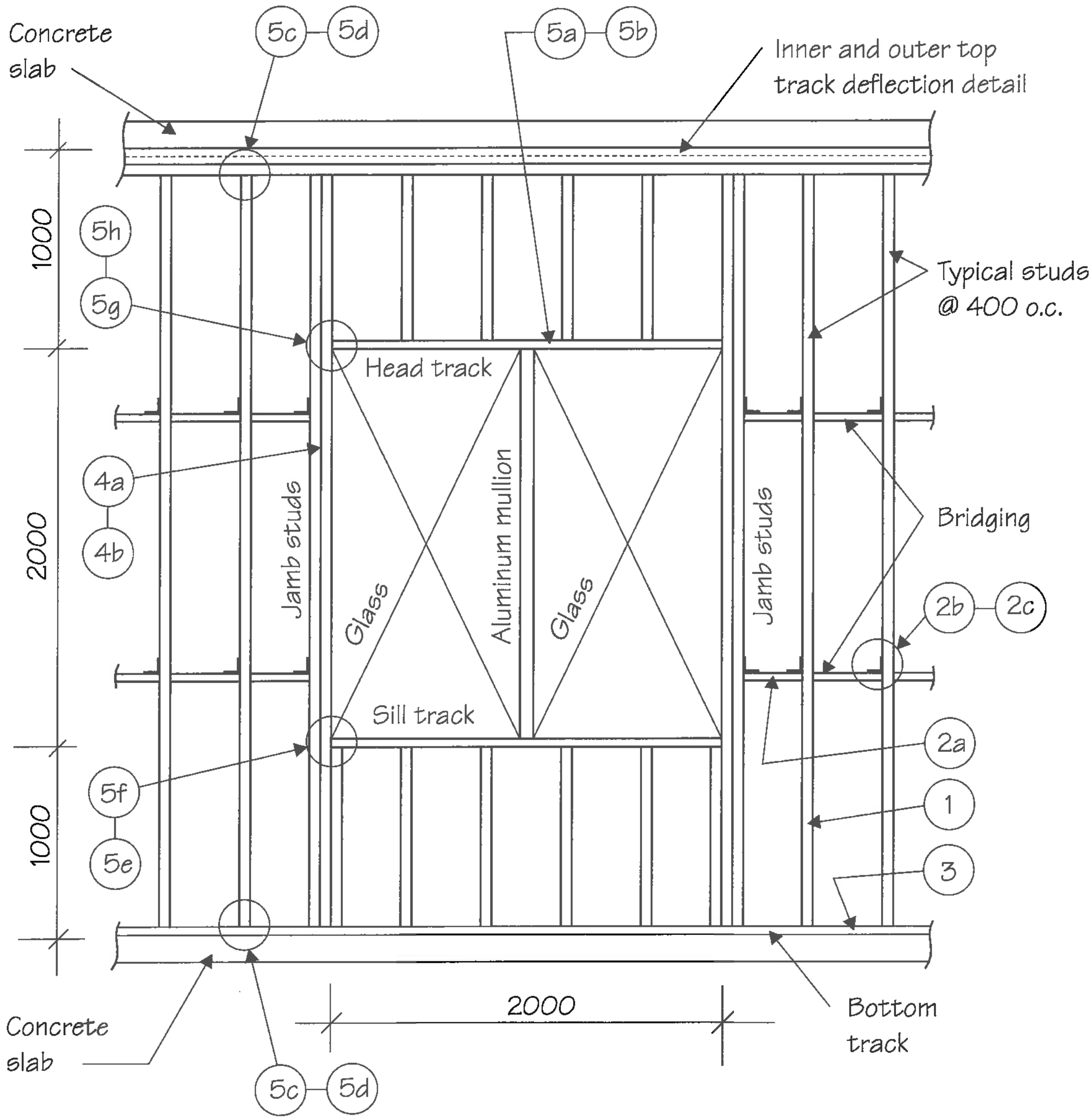


FIGURE 2-1

Introduction

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

Members are checked for lateral instability and for the torsional effects of loads not applied through the shear center. 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 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.

This design example uses a torsional eccentricity from the shear center to the centerline 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 centerline 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 = 1333 mm. The specified applied wind load = 1.32 kPa is assumed to act through the web centerline. 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.

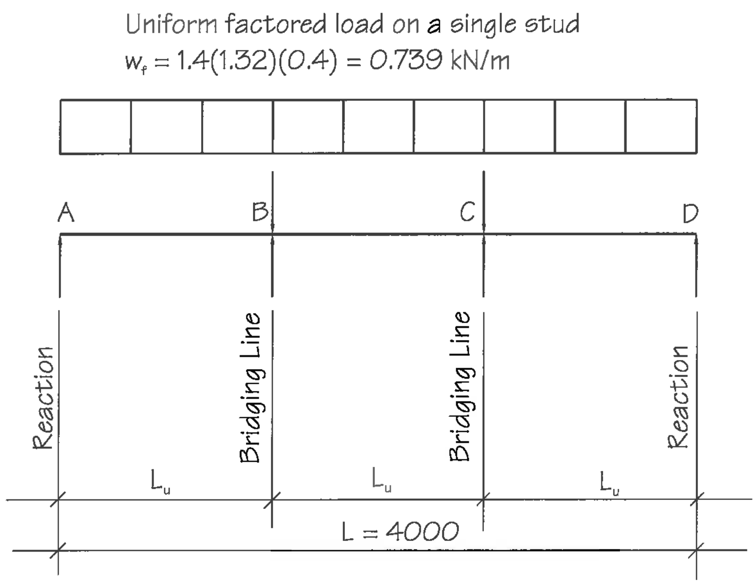


FIGURE 2-2

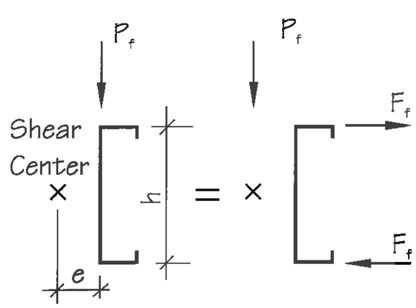


FIGURE 2-3A

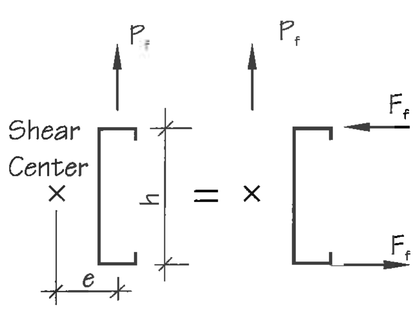


FIGURE 2-3B

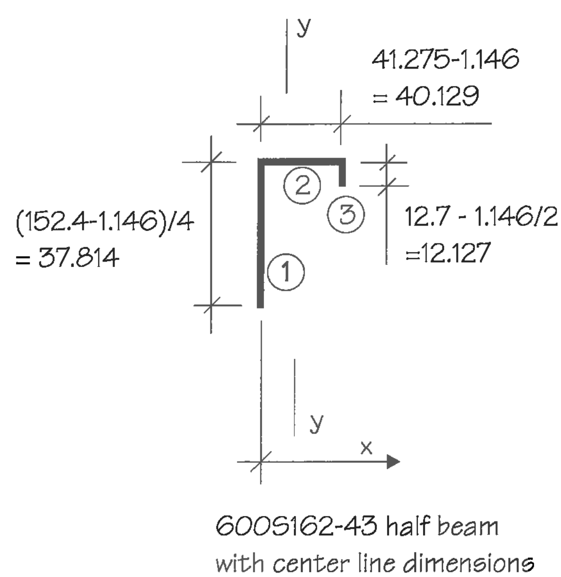


FIGURE 2-4

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 2002b, Part I, Section 3. For this example, neglect the corner radii.

Element	L	X	LX	LX^2	I_0/t
1	37.814	0.00	0.00	0.00	0.00
2	40.129	20.065	805.19	16156	5385
3	12.127	40.129	486.64	19529	0.00
Σ	90.070		1291.83	35685	5385

$$X_{cg} = \Sigma LX/\Sigma L$$
$$= 1291.83/90.07 = 14.343 \text{ mm}$$

$$I_{cg} = [\ \Sigma LX^2 + \Sigma I_0/t - \Sigma L(X_{cg})^2\ ]\ t$$
$$= [\ 35685 + 5385 - 90.07(14.343)^2][\ 1.1461\ ]$$
$$= 25830 \text{ mm}^4$$

$$S_{y(web)} = \frac{I}{X_{cg} + t/2} = \frac{25830}{14.343 + 1.146/2}$$
$$= 1732 \text{ mm}^3$$

$$S_{y(lip)} = \frac{I}{Flg - X_{cg} + t/2} = \frac{25830}{40.129 - 14.343 + 1.146/2}$$
$$= 980 \text{ mm}^3$$

Factored load on "half-beam" (with 1.4 load factor)

The torsional loads, F_f, illustrated in Figure 2-3A are derived as follows.

$$F_f = P_f(e/h)$$

$$P_f = w_f \text{ (stud spacing)} = 1.4(1.32)(0.4)$$
$$= 0.739 \text{ kN/m}$$

e = m = 17.0 mm (distance from shear center to web centerline)

h = centerline depth = 152.4 – 1.146
= 151 mm.

Substituting

$$F_f = 0.739(17/151)$$
$$= 0.0832 \text{ kN/m (factored)}$$

The factored 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:

$M_f = F_f L_u^2/10$
$= 0.0832(1333)^2/10$
$= 14800$ N.mm

(If the moment on the half-beam is to be checked at the midheight of the stud - i.e. mid-distance between bridging lines - then $M_f = F_f L_u^2/40$. See Note 2-1, Item 1.)

The compressive warping torsional stress is given by:

$\sigma_w = M_{req}/S_{y(lip)}$
$= 14800/980$
$= 15.1$ MPa (factored)

For combined bending and warping torsion calculate the moment reduction factor, R defined as follows:

R is given by the ratio of 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. See Note 2-1 Item 1. Stresses are calculated with full section properties for the torsional stresses and effective section properties for the bending stresses.

$S_{x(eff)}$ is not available in the CSSBI load tables but can be calculated from:

$M_{rx} = S_{x(eff)} F_y(0.9)$

$M_{rx} = 2.59 \times 10^6$ N.mm
$F_y = 230$ MPa

Substituting gives:

$S_{x(eff)} = 12500 \text{ mm}^3$

Also
$M_{fx} = w_f L^2/8$
$= 1.4(0.4)(1.32)(4)^2/8$
$= 1.48$ kN.m (factored)

Then
$\sigma_{bend} = M_{fx}/S_{x(eff)} = 1.48(10^6)/12500$
$= 118$ MPa

$$R = \frac{\sigma_{bend}}{\sigma_{bend} + \sigma_w} = \frac{118}{118 + 15.1}$$
$$= 0.887$$

Reduced factored moment resistance
$= R.M_{rx} = 0.877(2.59)$
$= 2.30$ kN.m

M_{fx} = 1.48 kN.m < 2.30 kN.m ***OK***

Note 2-1

1. *For this design example, the maximum warping torsional stress occurs at a line of bridging whereas the maximum primary bending moment stress occurs at midspan. As a design expediency, this example assumes that these warping stresses and primary bending stresses are additive even though they occur at two different locations along the length of the stud. A more rigorous design procedure would check M_r(reduced) both at the line of bridging and at midspan against M_f calculated for each of the two locations.*

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. *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.*

4. *For torsional eccentricity to the flange centerline:*

$$\begin{aligned} e &= m + (\text{centerline flange width})/2 \\ &= 17.0 + 40.129/2 \\ &= 37.1 \text{ mm} \end{aligned}$$

$F_f = 0.182$ *kN/m*

$M_f = 32300$ *N.mm*

$\sigma_w = 32.9$ *MPa*

$R = 0.782$

Reduced factored moment resistance = 2.03 kN.m

M_{fx} *= 1.48 kN.m < 2.03 kN.m* ***OK***

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

From Figure 2-2:

L_u = 1333 mm

(From CSSBI load tables – 600S162-43 section properties – the maximum unsupported length for no reduction in moment capacity is given by Lu = 1040 mm < 1333mm. Therefore, check for reduced moment.)

Assume:

$K_t = K_y = 1$

From CSSBI load tables:

$J = 126\ mm^4$
$C_w = 295 \times 10^6\ mm^6$
$A = 288\ mm^2$
$r_x = 57.9\ mm$
$r_y = 14.6\ mm$
$x_0 = 26.9\ mm$
S_f = full unreduced section modulus
$= 12.7 \times 10^3\ mm^3$
$M_{rx} = 2.59$ kN.m

$E = 203000$ MPa
$G = 78000$ MPa
$C_b = 1$ (Conservative – only a small benefit for CAN/CSA-S136-01 Equation C3.1.2.1-10 for the middle L_u)

From CAN/CSA-S136-01 Section C3.1.2:

$$\sigma_{ey} = \frac{\pi^2 E}{\left(\dfrac{K_y L_y}{r_y}\right)^2} = \frac{\pi^2(203000)}{\left[\dfrac{(1)(1333)}{14.6}\right]^2}$$
$$= 240.3\,\text{MPa}$$

$$r_0 = \sqrt{r_x^{\,2} + r_y^{\,2} + x_0^{\,2}}$$
$$= \sqrt{57.9^2 + 14.6^2 + 26.9^2}$$
$$= 65.49\,\text{mm}$$

$$\sigma_t = \frac{1}{Ar_0^{\,2}}\left[GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2}\right]$$
$$= \frac{1}{(288)(65.49)^2}\left[78000(126) + \frac{\pi^2(203000)(295)(10^6)}{[(1)(1333)]^2}\right]$$
$$= 277.2\text{ MPa}$$

$$F_e = \frac{C_b r_o A\sqrt{\sigma_{ey}\sigma_t}}{S_f}$$
$$= \frac{(1)(65.49)(288)\sqrt{(240.3)(277.2)}}{12700}$$
$$= 383.3\text{ MPa}$$

$2.78F_y = 639.4$ MPa
$0.56F_y = 128.8$ MPa

For $2.78F_y > F_e > 0.56F_y$

$$F_c = \frac{10}{9}F_y\left(1 - \frac{10F_y}{36F_e}\right)$$
$$= \frac{10}{9}(230)\left[1 - \frac{10(230)}{36(383.3)}\right]$$
$$= 213.0 \text{ MPa}$$

See Note 2-2.

Note 2-2

1. *The next step in CAN/CSA-S136-01 is to calculate a revised effective section modulus, S_c , with F_y replaced by F_c . The moment capacity reflecting lateral buckling is then given by $M_{rx} = \phi M_n = \phi S_c F_c$. For typical lightweight steel framing profiles, calculating S_c requires considerable work with little benefit. Use instead the following procedure:*

   $M_{rx(lateral\ buckling)} = M_{rx(fully\ braced)}$ x (F_c/F_y)

   This expression is always conservative.

2. *Where section capacity includes the effect of cold work of forming F_y is replaced by F_{ya} throughout Step 1(b). See CAN/CSA-S136-01 Section A7.2. Note that the CSSBI load tables (CSSBI 2004) have conservatively neglected cold work of forming and F_{ya} does not apply.*

$M_{rx(\text{lateral buckling})} = M_{rx(\text{fully braced})}$ x (F_c/F_y)
$= 2.59(213.0/230)$
$= 2.40$ kN.m

From Step 1(a):

$M_{fx} = 1.48$ kN.m < 2.40 kN. m **OK**

Therefore from Steps 1(a) and 1(b), the factored moment resistance of the typical stud is reduced by warping torsion and by lateral instability. There is sufficient strength reserve, however, such that the basic stud selection is unaffected.

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.

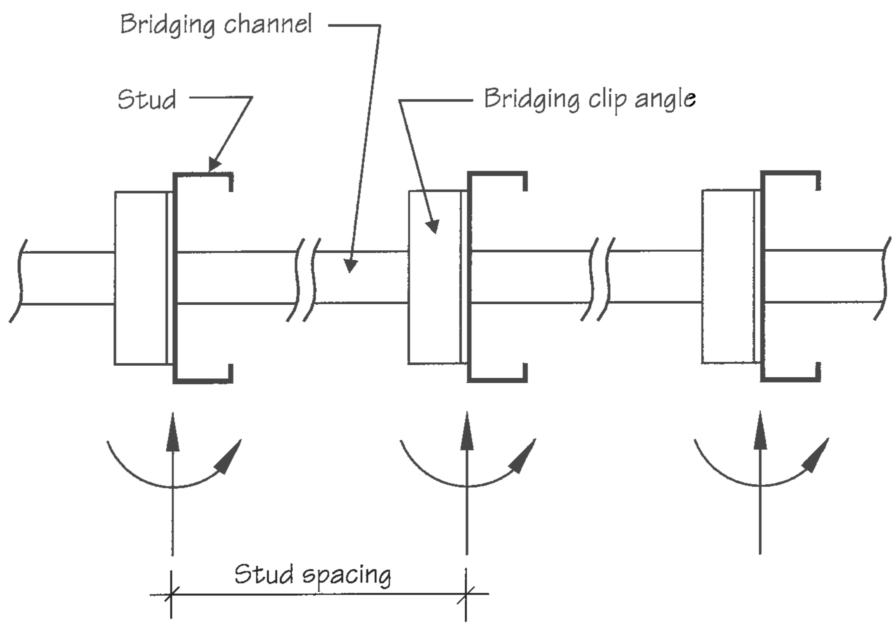
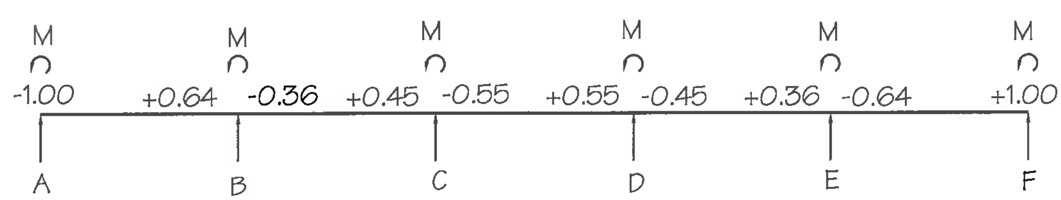


FIGURE 2-5



Moment = Coefficient x M
(+ve = tension bottom fiber)

FIGURE 2-6

FIGURE 2-7

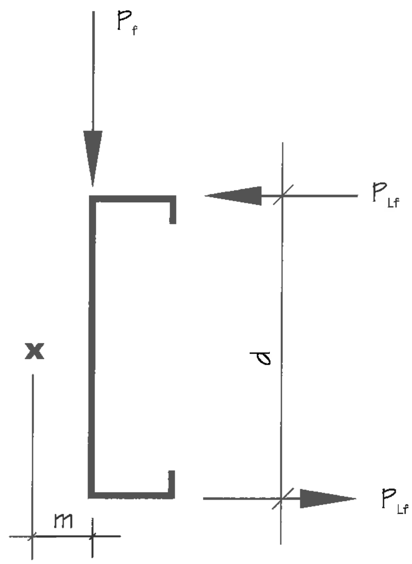


FIGURE 2-8

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 the Supplement S136S1-04 *(CSA 2004a)* Section D3.2.2. See also Figure 2-8.

$P_{Lf} = 1.5(m/d)W_f$

where:
a = bridging spacing = 1.333 m
w_f = factored load/m (with 1.4 load factor)
= 1.32(0.4)(1.4) = 0.739 kN/m
$W_f = aw_f$ = 1.333(0.739) = 0.985 kN
m = stud web center line to shear center = 17.0 mm
d = 152.4 mm

Substituting:
P_{Lf} = 1.5(17.0/152.4)0.985
= 0.165 kN

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.

$M_f = P_{Lf}\, d = 165(152.4) = 25100$ N.mm and the resulting moment values in the outside span are illustrated in Figure 2-9.

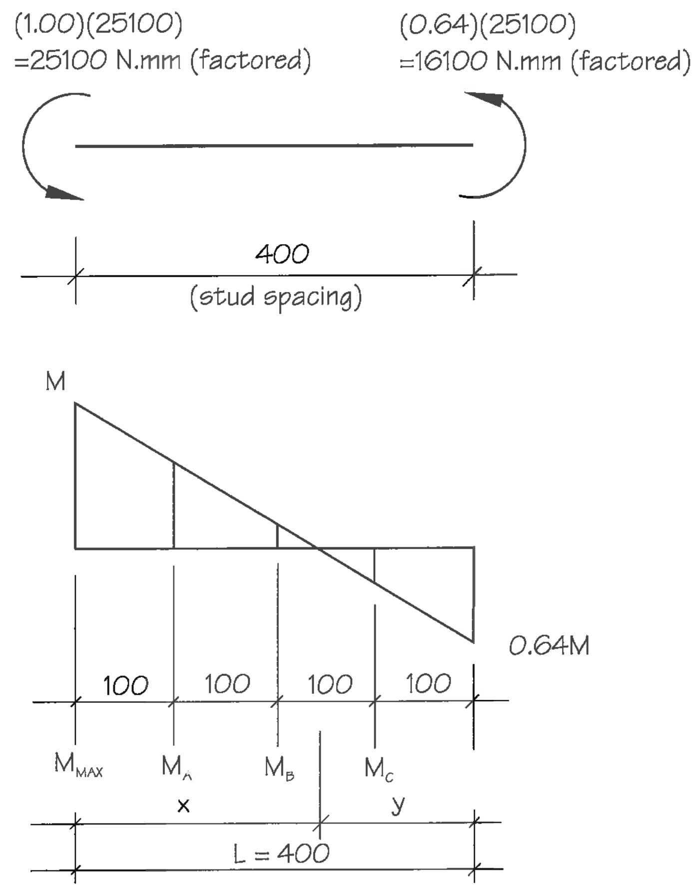


FIGURE 2-9

Try 150U50-54 (50) bridging channel with F_y = 345 MPa. *(Note that bridging channel with F_y = 345 MPa may require a special order. Check with local manufacturers.)*

The following bridging channel section properties are not available in the CSSBI load tables *(CSSBI 2004)* but have been derived using the formulae in AISI 2002b Part I. *(Note that the section is fully effective at a uniform stress of Fy = 345 MPa - i.e. $\lambda \leq 0.673$ for all elements at $f = Fy$ CAN/CSA-S136-01 B2.1 - not checked here.)*

Inside bend radius = 2.16 mm
A = 83.6 mm^2
S_x = 852 mm^3

r_x = 13.9 mm
r_y = 3.69 mm
x_0 = 6.46 mm
J = 57.6 mm^4
C_w = 279 x 10^3 mm^6
M_{rx} = 0.264 kN.m

Check strength:

M_{fx} = 25100 N.mm = 0.0251 kN.m

M_{rx} = 0.264 kN.m >> 0.0251 kN.m ***OK***

Check lateral instability from CAN/CSA-S136-01 Section C3.1.2.1:

E = 203000 MPa
G = 78000 MPa
$K_t = K_y = 1$

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

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

From CAN/CSA-S136-01 Section C3.1.2:

$$\sigma_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L_y}{r_y}\right)^2} = \frac{\pi^2(203000)}{\left[\frac{(1)(400)}{3.69}\right]^2}$$
$$= 170.5 \text{ MPa}$$

$$r_0 = \sqrt{{r_x}^2 + {r_y}^2 + {x_0}^2}$$
$$= \sqrt{13.9^2 + 3.69^2 + 6.46^2}$$
$$= 15.8 \text{ mm}$$

$$\sigma_t = \frac{1}{Ar_0^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right]$$
$$= \frac{1}{(83.6)(15.8)^2}\left[78000(57.6) + \frac{\pi^2(203000)(279)(10^3)}{[(1)(400)]^2}\right]$$
$$= 382.7\,\text{MPa}$$

$$F_e = \frac{C_b r_o A\sqrt{\sigma_{ey}\sigma_t}}{S_f}$$
$$= \frac{(2.20)(15.8)(83.6)\sqrt{(170.5)(382.7)}}{852}$$
$$= 871.2\,\text{MPa}$$

$2.78F_y = 2.78(345) = 959$ MPa
$0.56F_y = 0.56(345) = 193$ MPa

For $2.78F_y > F_e > 0.56F_y$

$$F_c = \frac{10}{9}F_y\left(1 - \frac{10F_y}{36F_e}\right)$$
$$= \frac{10}{9}(345)\left[1 - \frac{10(345)}{36(871.2)}\right]$$
$$= 341.2\,\text{MPa}$$

Since S_x is unreduced for all $f \le F_y$

$M_{rx} = \phi F_c S_x$
$= 0.9(341.2)(852)/10^6$
$= 0.262$ kN.m >> 0.0251 kN.m **OK**

Use 150U50-54 (50) bridging channel with $F_y = 345$ MPa

Note 2-3

For torsional eccentricity to the flange centerline:

e = m + (flange width)/2 = 17.0 + 40.129/2 = 37.1 mm

$P_{Lf} = 1.5(37.1/152.4)(0.985) = 0.360$ *kN*

$M_f = P_{Lf} d = 54800$ *N.mm*

$M_{rx} = 0.262$ *kN.m > 0.0548 kN.m* **OK**

Step 2(b) – Welded Bridging Connection

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

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

M_f = 25100 N.mm

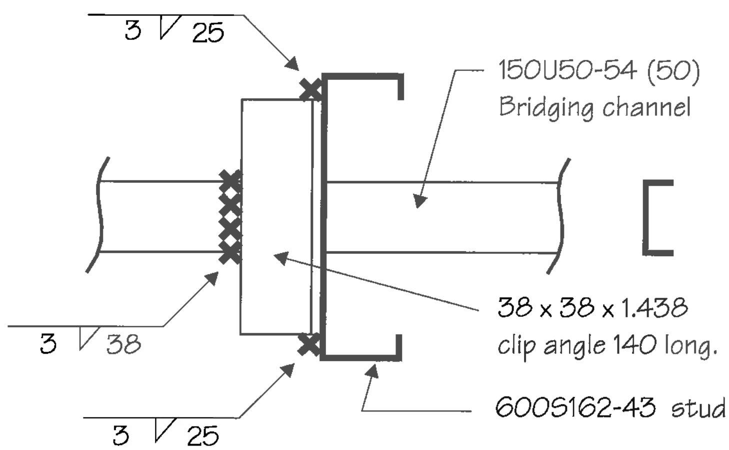


FIGURE 2-10

Clip angle size

See Note 2-5.

Use t = 1.438 mm

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

Clip angle	t = 1.438 mm	F_u = 450 MPa
Bridging channel	t = 1.438 mm	F_u = 450 MPa

Using the linear method, the maximum factored load per mm of weld length is given by:

$$q_f = \frac{M_f}{S_{weld}} = \frac{25100}{(38)^2/6} = 104 \text{ N/mm.}$$

Factored weld resistance per mm of weld length is given by *(See Appendix A.1)*:

$$q_r = \phi P_n/L = 0.75\phi t F_u$$
$$= 0.75(0.40)(1.438)(450)$$

= 194 N/mm > 104 N/mm ***OK***

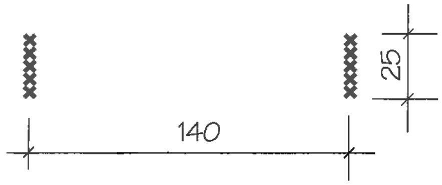


FIGURE 2-11

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

Clip angle	t = 1.438 mm	F_u = 450 MPa
Stud	t = 1.146	F_u = 310 MPa

Weld group allowable moment (stud material governs):

$M_r = 140 \times (0.75)\phi tLF_u$
$= 140(0.75)(0.40)(1.146)(25)(310)$
$= 373000$ N.mm >> 25100 N.mm ***OK***

Note 2-4

For torsional eccentricity to the centerline of the flange:

M_f = 54800 N.mm from Note 2-3.

Bridging channel to clip angle weld:

$$q_f = \frac{M_f}{S_{weld}} = \frac{54800}{(38)^2/6} = 228 \text{ N/mm}$$

and from the Appendix A simplified conservative approach (used above)
q_r = 194 N/mm < 228 N/mm ***UNSATISFACTORY***

A more detailed approach to weld strength is provided in CAN/CSA-S136-01 and is appropriate here. For fillet welds loaded transversely (Section E2.4(b)):

$q_r = \phi P_n/L = \phi tF_u = 0.60(1.438)(450)$
$= 388$ N/mm > 228 N/mm ***OK***

Clip angle to stud weld – OK by inspection.

Step 2(c) – Screwed Bridging Connection

For clip angle size, see Note 2-5. Figure 2-12 illustrates the typical 4 screw connection detail. For member sizes not shown, see Figure 2-10. No. 10-16 self-drilling screws are assumed.

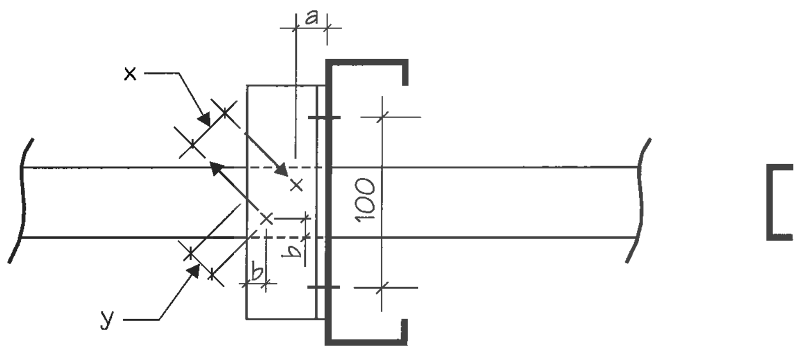


FIGURE 2-12

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 = 15.9 mm for screw gun clearance.
b = minimum edge distance by CAN/CSA-S136-01 Section E4.2.
= 1.5d = 1.5(4.83) = 7.25 mm
x = lever arm for force couple = 27.9 mm
y = edge distance parallel to force = 8.57 mm

Note 2-5

1. *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 1.438 mm.*
   - *Leg lengths to be 38 mm.*
   - *Length to be ≥ stud depth minus 12 mm*
   - *Screws (or welds) in outer portion of the web of the stud.*

Clip angle size

Use t = 1.438 mm.

Factored shear per screw

V_f = 25100/27.9 = 900 N
= 0.900 kN

Screw input values

Clip angle	t = 1.438 mm	F_u = 450 MPa
Bridging channel	t = 1.438 mm	F_u = 450 MPa
Screws	Size = #10-16	d = 4.38 mm *(Table A-2)*

Screw Shear Resistance

Screw shear resistance limited by E4.3.1 tilting and bearing

t_2/t_1 = 1.0 therefore choose the governing P_{ns} from CAN/CSA-S136-01 Equations E4.3.1-1, E4.3.1-2 and E4.3.1-3.

$$P_{ns} = 4.2\,(t_2^3 d)^{1/2}\,F_{u2} = 7160 \text{ N} - \text{governs}$$
$$P_{ns} = 2.7 t_1 d F_{u1} = 8440 \text{ N}$$
$$P_{ns} = 2.7 t_2 d F_{u2} = 8440 \text{ N}$$

Gives:

$V_r = \phi\, P_{ns}$ = (0.40)(7.16)
= 2.87 kN

Screw shear resistance limited by E4.3.2 end distance *(E4.3.2 in turn refers to C2.2. For amendments to C2.2, see the Supplement S136S1-04 (CSA 2004a))*

See Commentary *(CSA 2001b)* Figure B-C2.2-1 for failure path 5,2,3,6 which is the critical path for this geometry.

$L_c = 0.6\, L_{nv}$ = 1.2e - 0.6h *(h is diameter)*
= 1.2(8.57) - 0.6(4.83)
= 7.39 mm

Gives:

$V_r = \phi_u A_n F_u = \phi_u L_c t F_u$
= 0.75(7.39)(1.438)(450)/1000
= 3.59 kN

Screw shear resistance limited by E4.3.3 shear in the screws themselves. Refer to the Supplement S136S1-04 *(CSA 2004a)*

$P_{ns} = P_{ss}$

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

$V_r = \phi P_{ns} = \phi P_{ss} = 0.40(6.23)$
$= 2.49$ kN

The governing V_r is from E4.3.3 and is given by

$V_r = 2.49$ kN > 0.900 kN **OK**

Clip angle to stud screws

Factored tensile force per screw

$T_f = 25100/(120) = 209$ N
$= 0.209$ kN

Factored Tensile Resistance per screw

Screw tensile resistance limited by E4.4.1 pull-out

$P_{not} = 0.85 t_c d F_{u2}$ where $t_c = t_2 = 1.146$
$= 0.85(1.146)(4.83)(310)/1000$
$= 1.46$ kN

Screw tensile resistance limited by E4.4.2 pull-over

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

Does not govern by inspection.

Screw tensile resistance limited by E4.4.3 tension in the screws themselves. Refer to the Supplement S136S1-04 *(CSA 2004a)*

$P_{nt} = P_{ts} = 8.61$ kN

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

The governing nominal tensile resistance is given by $P_{not} = 1.46$ kN and

$T_r = \phi P_{not} = 0.40(1.46)$
$= 0.584$ kN > 0.209 kN **OK**

Note 2-6

For torsional eccentricity to the centerline of the flange:

M_{req} = *54800 from Note 2-3.*

Bridging channel to clip angle screws:

$V_f = 54800/27.9 = 1960\ N = 1.96\ kN$

$V_r = 2.49\ kN > 1.96\ kN$ ***OK***

Clip angle to stud screws:

$T_f = 54800/120 = 457\ N = 0.457\ kN$

$T_r = 0.584\ kN > 0.457\ kN$ ***OK***

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

For both the sill track and the bottom track:
L_u = stud spacing = 400 mm

From the CSSBI load tables *(see Note 2-7)*, the maximum unsupported length for no reduction in moment is given by L_u = 630 mm > 400 mm. Therefore, there is no moment reduction for lateral instability and the fully braced M_{rx} applies. No further design checking is required since the capacity of these members for the fully braced moment was checked in Design Example #1.

Note 2-7

The L_u value in the CSSBI load tables is calculated based on the assumption that $K_t = K_y = C_b = 1$.

Step 4 – Jamb Stud

Step 4(a) – Welded Jamb Stud Interconnection

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

Choose built-up section E. CAN/CSA-S136-01 does not define interconnection requirements for this type of built-up member. Experience indicates that a connection spacing of 400 mm o.c. is adequate. The welds are required to transfer shear between the two stud sections and to generate at least partial closed section torsional behavior. See Figure 2-13.

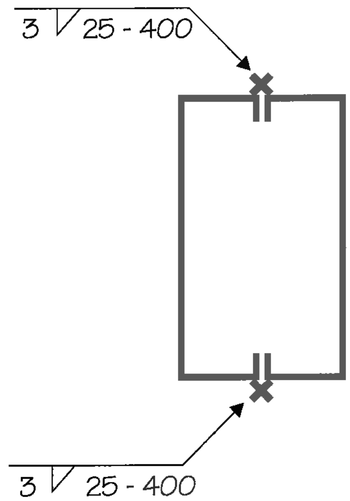


FIGURE 2-13

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. Use instead 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

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.

Factored end reaction for typical stud (with 1.4 load factor):
= 0.5(1.4)wL(spacing)
= 0.5(1.4)(1.32)(4)(0.400)
= 1.48 kN

From Supplement S136S1-04 *(CSA 2004a)* Section D3.2.2

See Figure 2-15.

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

$P_{Lf} = (17.0/152.4)(1.48)$
$= 0.165$ kN

and for 25 mm weld length
$V_r = 0.75\phi tLF_u = 0.75(0.40)(1.146)(25)(310)/1000$
$= 2.66$ kN >> 0.165 kN *OK*

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

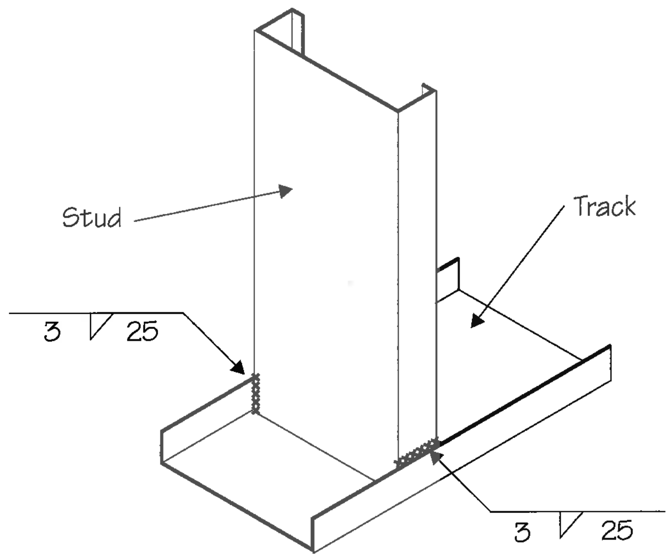


FIGURE 2-15

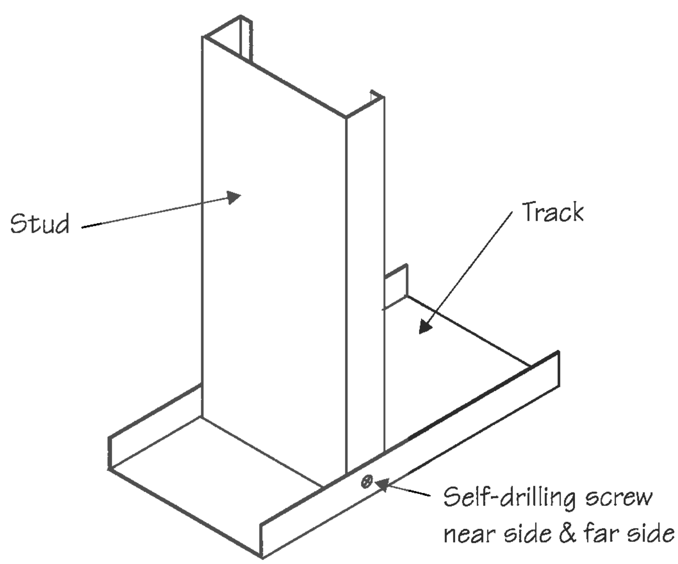


FIGURE 2-16

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

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

P_{Lf} = 0.165 kN/screw

Screw input values

Clip angle	t = 1.146mm	F_u = 310 MPa
Bridging channel	t = 1.146 mm	F_u = 310 MPa
Screws	Size = #10-16	d = 4.38 mm *(Table A-2)*

Screw Shear Resistance

Screw shear resistance limited by E4.3.1 tilting and bearing

t_2/t_1 = 1.0 therefore choose the governing P_{ns} from CAN/CSA-S136-01 Equations E4.3.1-1, E4.3.1-2 and E4.3.1-3.

$$P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2} = 3510\text{ N} - \text{governs}$$
$$P_{ns} = 2.7 t_1 d F_{u1} = 4630\text{ N}$$
$$P_{ns} = 2.7 t_2 d F_{u2} = 4630\text{ N}$$

Gives:

$V_r = \phi\, P_{ns}$ = (0.40)(3.51)
= 1.40 kN

Screw shear resistance limited by E4.3.2 end distance *(E4.3.2 in turn refers to C2.2)*

Not applicable

Screw shear resistance limited by E4.3.3 shear in the screws themselves. Refer to the Supplement S136S1-04 *(CSA 2004a)*

$P_{ns} = P_{ss}$

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

$V_r = \phi P_{ns} = \phi P_{ss}$ = 0.40(6.23)
= 2.49 kN

The governing V_r is from E4.3.1 and is given by

V_r = 1.40 kN > 0.165 kN ***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

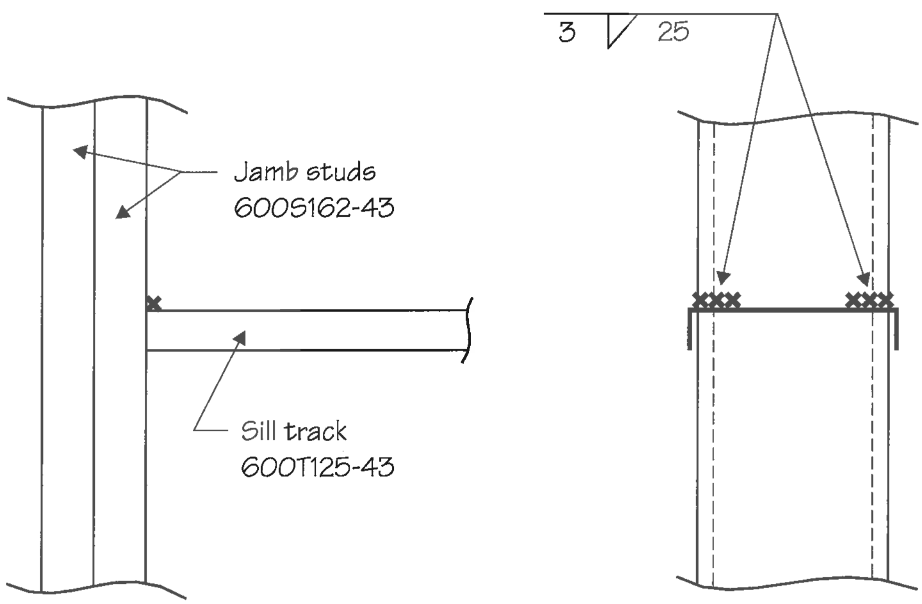


FIGURE 2-17

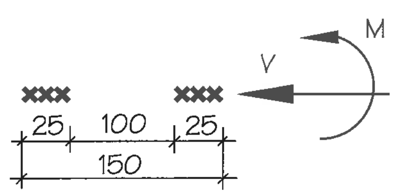


FIGURE 2-18

Stud and sill track:
$t = 1.146$ mm $F_u = 310$ MPa

Factored end reaction with 1.4 load factor *(Design Example #1 Step 5(b))*:
$V_f = 1.4(1.32) = 1.85$ kN

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

$M_f = Rm = V_f\, m$

From CSSBI load tables *(CSSBI 2004)* for track

$m = x_0 - x_{cg} + t/2$
$= 13.0 - 5.08 + 1.146/2$
$= 8.49$ mm

$M_f = 1850(8.49) = 15700$ N.mm

See Figure 2-18. Using the linear method, the maximum factored load per mm of weld length is given by:

$$q_f = \frac{M_f}{S_{weld}} + \frac{V_f}{A_{weld}}$$

$S_{weld} = I_{weld}/c$
$= 2\,[1/12(25)^3 + 25(75)^2]\,/\,75$
$= 3780\ \text{mm}^2$

$A_{weld} = L = 2(25) = 50$ mm

$$q_f = \frac{15700}{3780} + \frac{1850}{50} = 4 + 37$$

$= 41$ N/mm

$q_r = \phi P_n/L = \phi 0.75tF_u$
$= 0.40(0.75)(1.146)(310)$
$= 107\ \text{N/mm} > 41\ \text{N/mm}$ ***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 = 1320 N (specified)
M = Vm = 1320(8.49) = 11200 N.mm (specified)

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

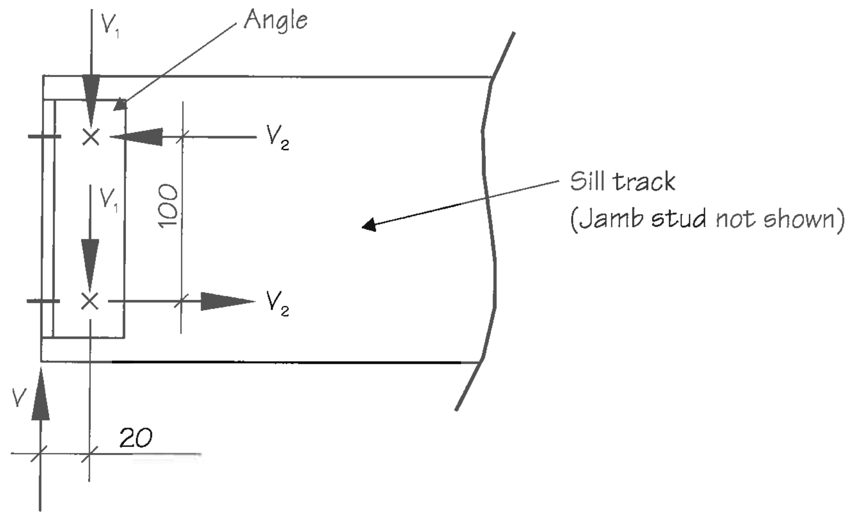


FIGURE 2-19

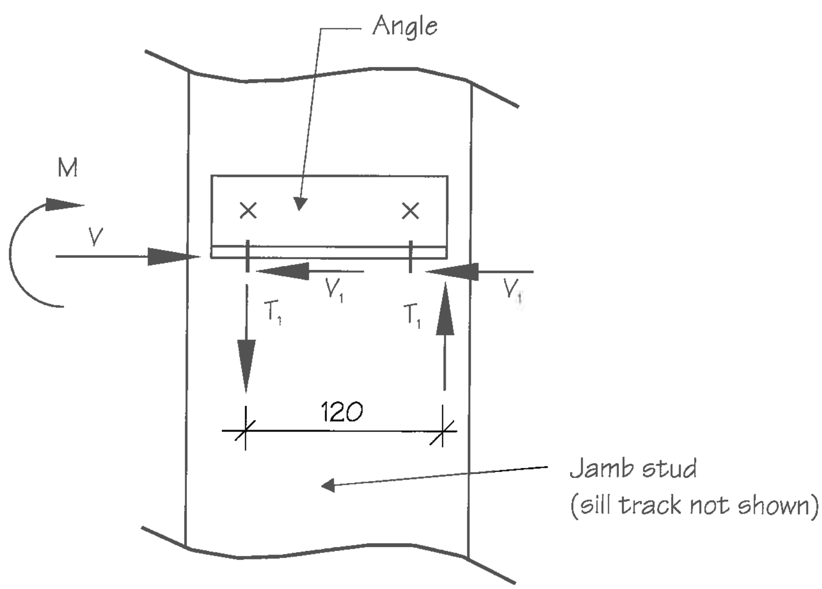


FIGURE 2-20

V_1 = V/2 = 1.32/2 = 0.66 kN (specified)
V_2 = Ve/100 = 20V/100 = 20(1.32)/100 = 0.264 kN (specified)

Factored shear resultant (1.4 load factor)

$V_f = 1.4\sqrt{V_1^2 + V_2^2} = 0.995$ kN

T_1 = M/120 = 11200/120/1000
= 0.0933 kN (specified)

$T_f = 1.4(0.0933) = 0.131$ kN

Screw shear resistance from Design Example #1 Step 8(f)

$V_r = 1.40$ kN per screw

Screw Tensile Resistance

Screw tensile resistance limited by E4.4.1 pull-out

$P_{not} = 0.85 t_c d F_{u2}$ where $t_c = t_2 = 1.146$
$= 0.85(1.146)(4.83)(310)/1000$
$= 1.46$ kN

Screw tensile resistance limited by E4.4.2 pull-over

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

Does not govern by inspection.

Screw tensile resistance limited by E4.4.3 tension in the screws themselves. Refer to the Supplement S136S1-04 *(CSA 2004a)*

$P_{nt} = P_{ts} = 8.61$ kN

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

The governing nominal tensile resistance is given by $P_{not} = 1.46$ kN and

$T_r = \phi P_{not} = 0.40(1.46)$
$= 0.584$ kN

Assuming a linear interaction between tension and shear:

$$\frac{V_f}{V_r} + \frac{T_f}{T_r} \leq 1.0$$

$$\frac{0.995}{1.40} + \frac{0.131}{0.584} = 0.94 < 1.00$$ ***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***

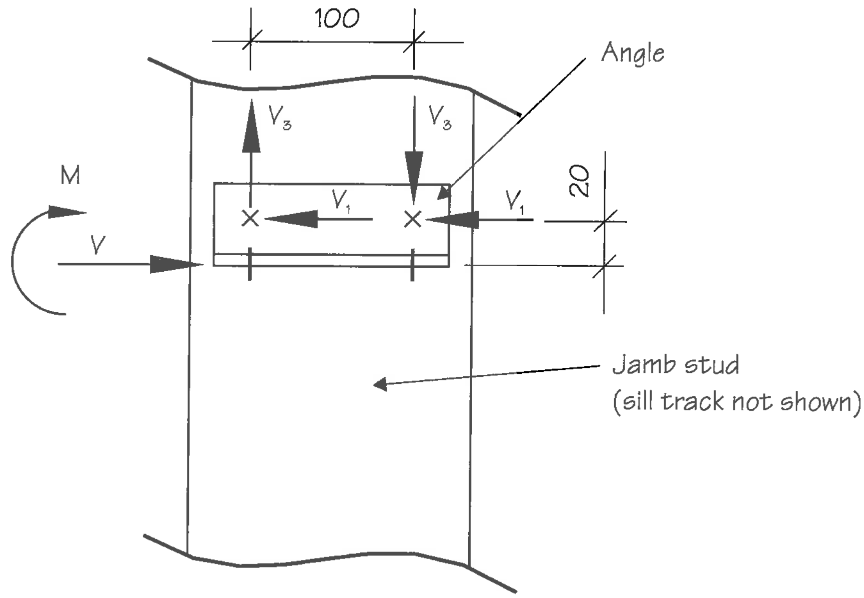


FIGURE 2-21

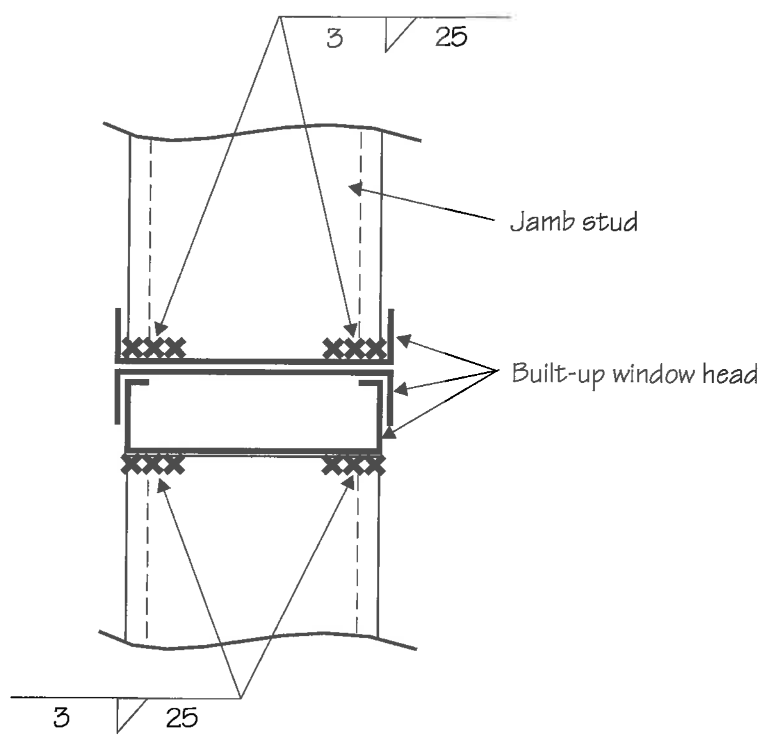


FIGURE 2-22

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 2.54 mm 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 3 mm accompanied by a note "For material less than 2.54 mm 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. A common recommended minimum thickness is 1.146 mm. There is no minimum weld length requirement in CAN/CSA-S136-01 - this Guide uses a 25 mm minimum.

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 2002b, 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) may have less labour content than screwed connections. This is particularly the case in thicker material (say ≥ 1.1438 mm) and in shop conditions.

Disadvantages of Welded Connections

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

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

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

Step 7 – Details at Shearwalls

See Figure 2-23 and Note 2-8.

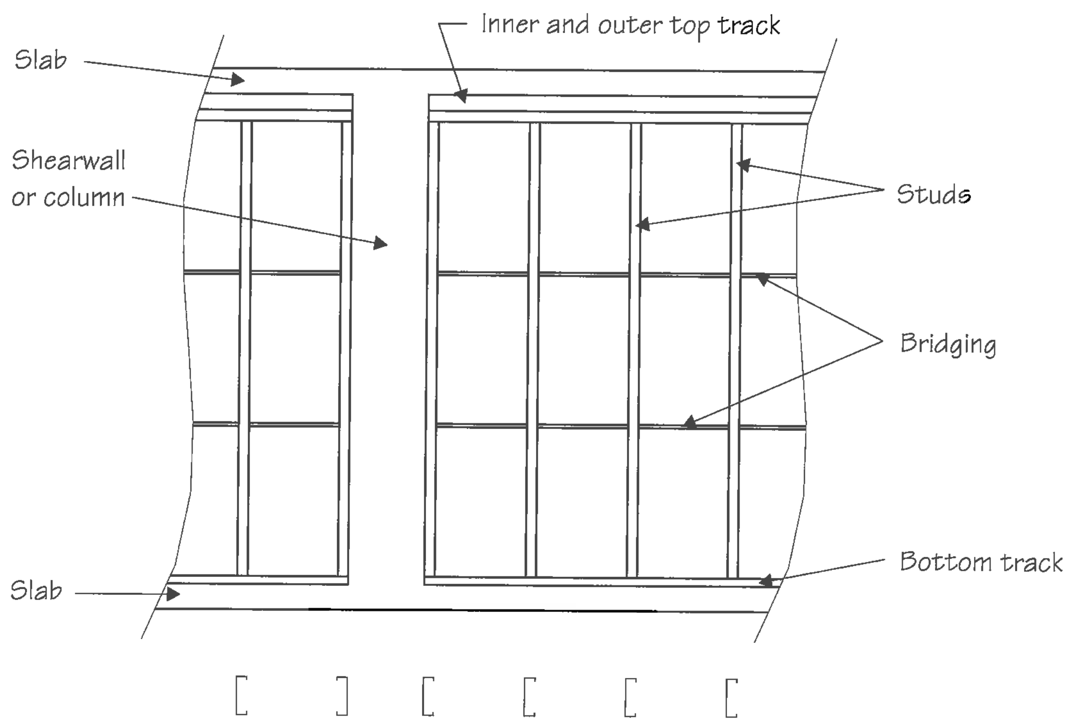


FIGURE 2-23

> *Note 2-8*
>
> *Anchor the last stud to the shearwall or column with concrete anchors at say 800 mm 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.*

Step 8 – Parapets

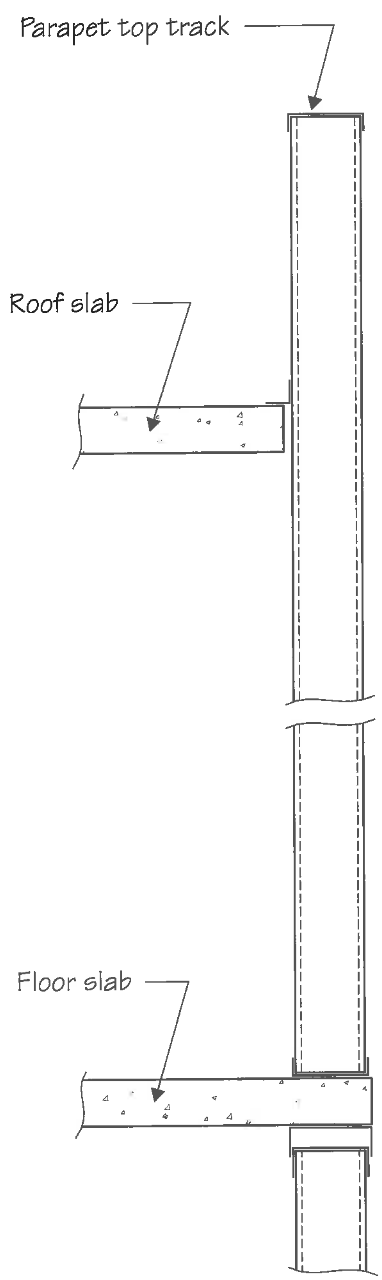


FIGURE 2-24

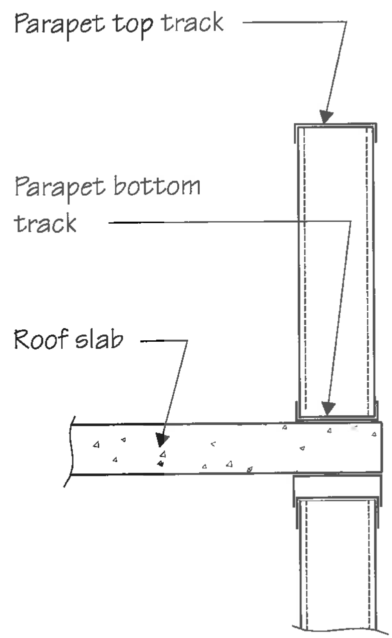


FIGURE 2-25

Note 2-9

1. *Some parapets are designed to accept substantial vertical and lateral loads from window washing equipment, swing stages etc. 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 sections welded to the anchor plates and cantilevered 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 hot-rolled cantilevers and the bottom track is designed to span between fasteners which will be controlled by the 800 mm o.c. maximum recommended spacing in Drysdale 1991.

3. *The design calculations for Figure 2-24 follow.*

   In order to accommodate roof slab deflections, a proprietary slide clip/continuous angle connection detail has been shown. The continuous angle is required to correct for any out-of-straightness in the edge of the roof slab (in plan). Note that this particular slide clip provides little torsional restraint at the roof slab support and a line of bridging is therefore required. (Some slide clips do provide torsional restraint.)

   For this purpose, flat strap face bridging is provided 300 mm below the roof support angle. The 300 mm distance between the reaction point and the line of bridging will induce warping torsional stresses in the studs which is checked in Step 8(b) using the model proposed in Appendix C. The bridging itself is checked in Step 8(f) (welded) and 8(g) (screwed).

   For projects where the parapet is low, the line of bridging at the roof slab level may not be required because the parapet top track is close enough to provide adequate torsional restraint. The warping torsional stresses, for this case could also be checked using the model proposed in Appendix C.

Step 8(a) – Cantilevering Typical Stud Selection for Parapet

The parapet dimensions are illustrated in Figure 2-26 & 2-30. The partial load provisions in the NBCC 2005 Clause 4.1.7.3 are assumed not to apply.

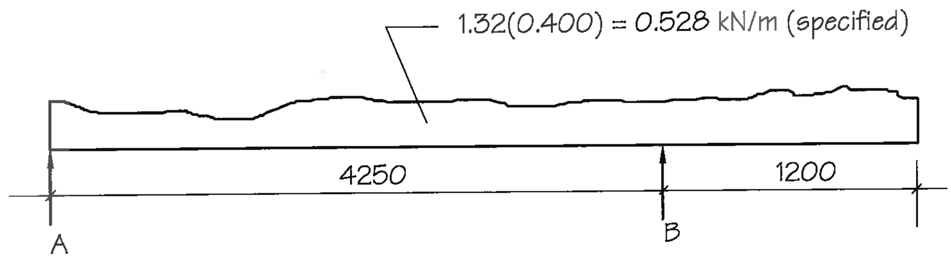


FIGURE 2-26

Reactions

Moments about A
$4.25R_B - 0.528(5.45)^2/2 = 0$

$R_B = 1.845$ kN (specified)

Factored Moment, Reaction and Shear (1.4 load factor)

$M_f = 1.4M_B = 1.4(0.528)(1.2)^2/2 = 0.532$ kN.m

$P_f = 1.4\ R_B = 1.4(1.845) = 2.583$ kN

V_f (to the right of B) = 1.4(0.528)(1.2) = 0.887 kN

V_f (to the left of B) = $P_f - V_f$ = 2.583 - 0.887 = 1.696 kN

Try 600S162-43 stud F_y = 230 MPa

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

Strength	H_{MAX} = 4.88 m > 4.25 m	***OK***
Deflection	H_{MAX} = 4.42 m > 4.25 m	***OK***
Web crippling	– not flagged	***OK***

For cantilever

M_r = 2.59 kN.m > 0.532 kN.m ***OK***
V_r = 7.03 kN > 1.696 kN ***OK***

Check web crippling at B. See Note 2-10

Note 2-10

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 CAN/CSA-S136-01 Section C3.4.2 Eq. C3.4.2-2 by setting $R_c = 1$ and solving. Gives:*

$$x = 1.89h + 0.887d_0$$
$$= 1.89(146.5) + 0.887(38.1)$$
$$= 311 \text{ mm}$$

Therefore, the required distance between the edge of bearing and the edge of the punchout must be ≥ 311 mm for no reduction in web crippling.

Derive the web crippling factored resistance at B for interior one flange condition, unfastened and 75 mm bearing length. From CAN/CSA-S136-01 Section C3.4 and Table C3.4.1-2

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

where:

R	= 1.808 mm
t	= 1.146 mm
Depth	= 152.4 mm
h	= Depth - 2t - 2R = 146.5 mm
N	= 75 mm
F_y	= 230 MPa
θ	= 90 degrees
C	= 13
C_R	= 0.23
C_N	= 0.14
C_h	= 0.01
ϕ	= 0.80

substituting

P_r = 4.23 kN > 2.58 kN ***OK***

Check combined bending and web crippling at B *(Supplement S136S1-04 (CSA 2004a) Section C3.5.2)*

$$0.91\left(\frac{\overline{P}}{P_n}\right)+\left(\frac{\overline{M}}{M_{nxo}}\right)\leq 1.33\,\phi$$

where:
$\overline{P}$ = P_f = 2.58 kN
$\overline{M}$ = M_f = 0.532 kN.m
P_n = P_r/ϕ = 4.23/0.80 = 5.29 kN
M_{nx0} = M_r/ϕ = 2.59/0.90 = 2.88 kN.m
ϕ = 0.75

Substituting:

$$0.91\left(\frac{2.58}{5.29}\right)+\left(\frac{0.532}{2.88}\right)\leq 1.33(0.75)$$

$0.63 \leq 1.00$ ***OK***

Check combined bending and shear at B *(Supplement S136S1-04 (CSA 2004a) Section C3.3.2)*

$$\sqrt{\left(\frac{\overline{M}}{\phi_b M_{nxo}}\right)^2+\left(\frac{\overline{V}}{\phi_v V_n}\right)^2}\leq 1.00$$

where:
$\overline{M}$ = M_f = 0.532 kN.m
$\overline{V}$ = V_f = 1.696 kN
$\phi_b\, M_{nxo}$ = M_r = 2.59 kN.m
$\phi_v\, V_n$ = V_r = 7.03 kN

Substituting:

$$\sqrt{\left(\frac{0.532}{2.59}\right)^2+\left(\frac{1.696}{7.03}\right)^2}\leq 1.00$$

$0.32 \leq 1.00$ ***OK***

Check Cantilever Deflection. *(See Note 2-11.)*

Note 2 - 11

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 2-27 and 2-28. . This approach is based on the assumption that a cantilever is analogous to a portion of a simply supported beam.

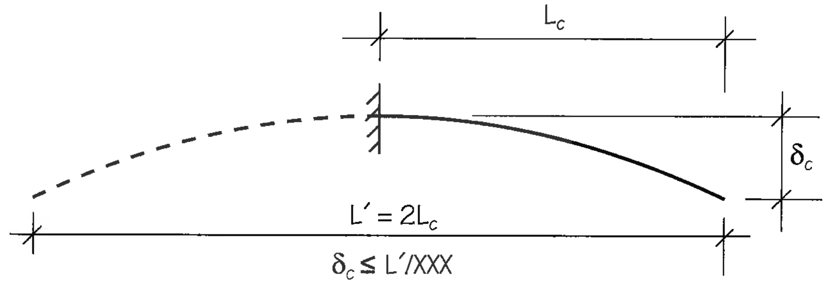


FIGURE 2-27
(from LGSEA 2001)

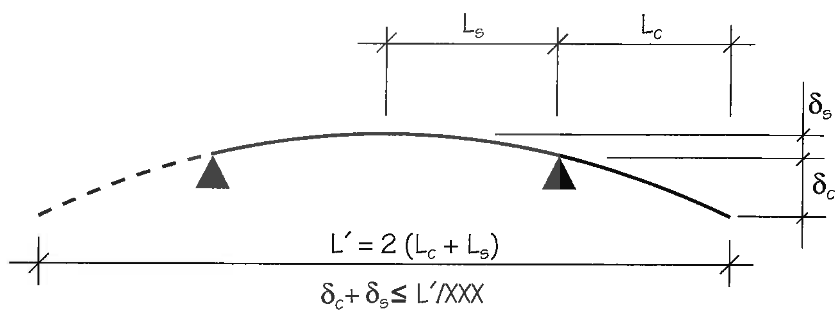


FIGURE 2-28
(from LGSEA 2001a)

By computer analysis with all spans loaded as in Figure 2-26 except:
$w = I_w\ (0.528) = 0.75(0.528)$
$= 0.396$ kN/m

$\delta_c = -4.76$ mm
$\delta_s = +6.94$ mm *(midspan)*

and the deflected shape (not shown here) is approximated by Figure 2-28. *(L_s and δ_s are taken to the middle of the back-span. A more rigorous procedure would locate the point of zero slope for both L_s and δ_s.)*

L_s = 2125 mm
L_c = 1200 mm
L' = 2(L_s + L_c) = 2(2125 + 1200) = 6650 mm

L'/360 = 6650/360 = 18.5 mm

$|\delta_c + \delta_s| = |4.76 + 6.94| = 11.7\,\text{mm} < 18.5\,\text{mm}$ ***OK***

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

As discussed in Note 2-9 Item 3, the proprietary slide clip is assumed to provide no torsional restraint and a line of bridging is required. As shown in Figure 2-30, there is a distance of 300 mm 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.

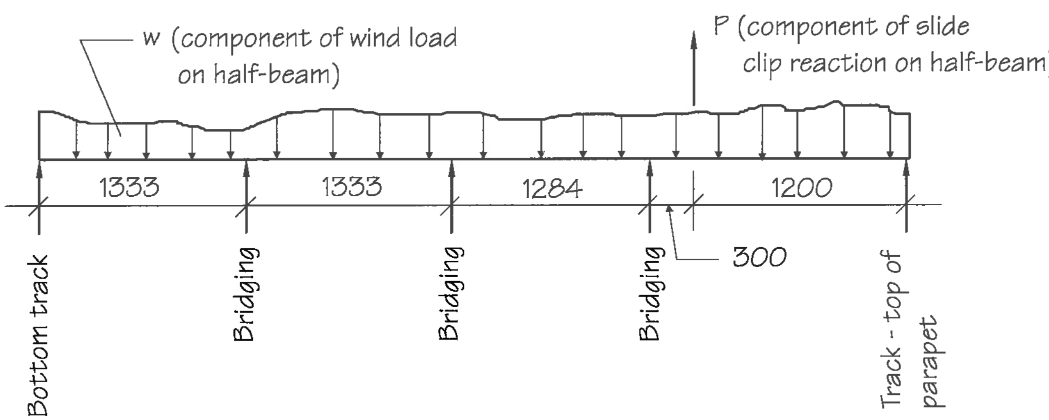


FIGURE 2-29

The torsional loads on the "half-beam", w and P, are illustrated in Figure 2-29 and are derived as follows:

e = m = 17.0 mm (distance from shear center to web centerline)
h = depth – t = 152.4 - 1.146 = 151 mm

w_f = 1.4(1.32)(0.400)(e/h) = 1.4(1.32)(0.400)(17.0/151)
= 0.0832 kN/m

P_f = (Factored Reaction at B)(e/h) = 2.583(17.0/151)
= 0.291 kN

The maximum factored moment on the half-beam is found by computer analysis of the continuous beam illustrated in Figure 2-29. 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 (i.e. the cantilever reaction point).

M_f = 40700 N.mm

Warping torsional stress:

$\sigma_w = M_f/S_y$ *(S_y from Step 1(a))*
= 40700/980
= 41.5 MPa (factored)

Major axis bending stress:

$S_{x(eff)}$ = 12500 mm^3 (Step 1(a))

M_{fx} = 0.532 kN (at the cantilever reaction point)

Then
$\sigma_{bend} = M_{fx}/S_{x(eff)} = 0.532(10^6)/12500$
= 42.6 MPa (factored)

R factor:

$$R = \frac{\sigma_{bend}}{\sigma_{bend} + \sigma_w} = \frac{42.6}{42.6 + 41.5}$$
$$= 0.507$$

Reduced factored moment resistance
= $R.M_{rx}$ = 0.507(2.59)
= 1.31 kN.m > 0.532 kN.m **OK**

Note 2-12

For torsional eccentricity to the centerline of the flange:
e = m + (flange width)/2 = 37.1 mm

For the half-beam
w_f = 0.182 kN/m
P_f = 0.635 kN.
M_f = 88900 N.mm
σ_w = 90.7 MPa

$R.M_{rx}$ = (0.320)(2.59) = 0.829 kN.m > 0.532 kN.m ***OK***

Note that the $R.M_{rx}$ should not be used in the interaction equations for moment and shear and moment and web crippling in Step 8(a).

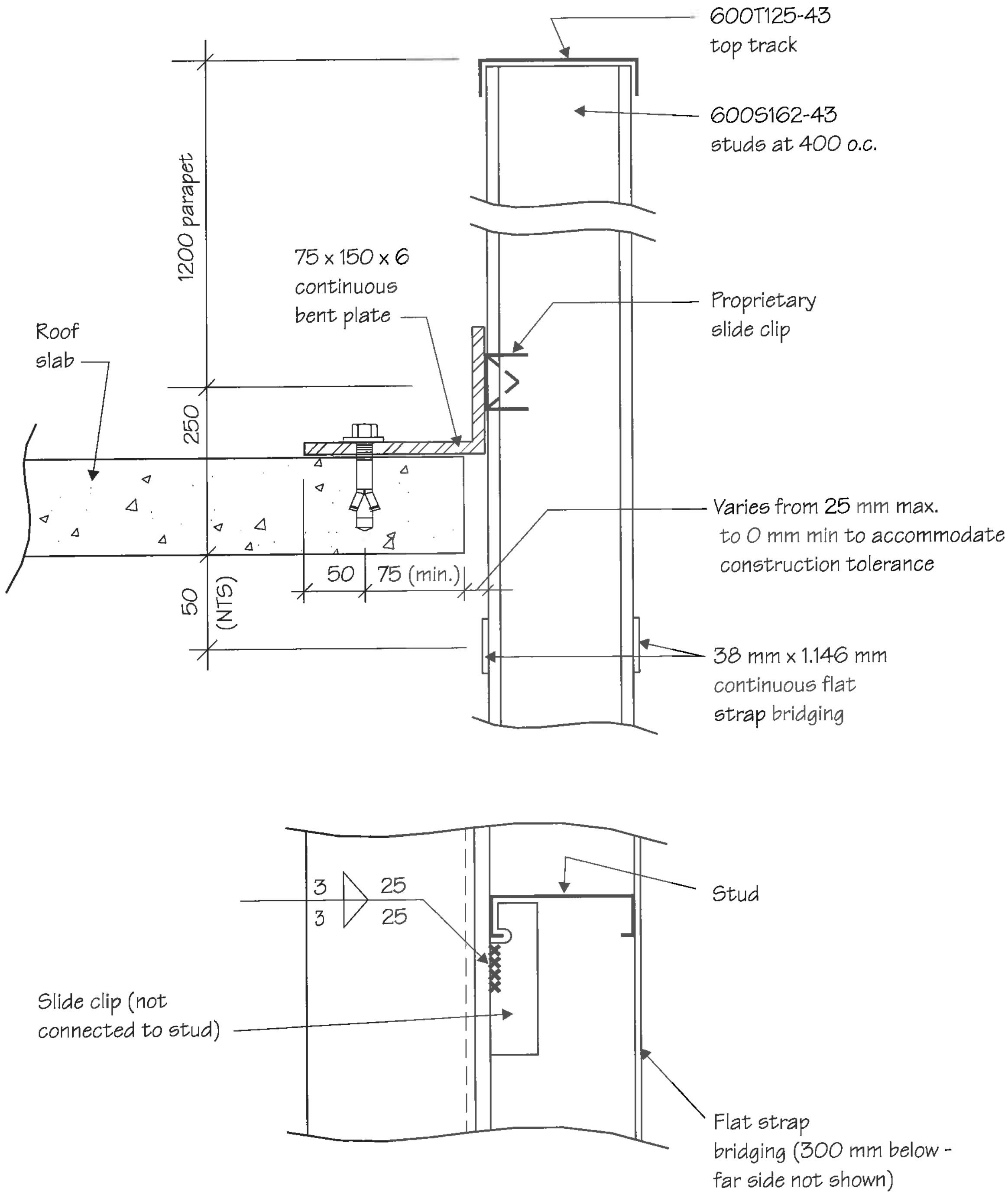

600T125-43
top track
600S162-43
studs at 400 o.c.
1200 parapet
75 x 150 x 6
continuous
bent plate
Roof
slab
Proprietary
slide clip
250
Varies from 25 mm max.
to 0 mm min to accommodate
construction tolerance
50
75 (min.)
50
(NTS)
38 mm x 1.146 mm
continuous flat
strap bridging
3
3
25
25
Stud
Slide clip (not
connected to stud)
Flat strap
bridging (300 mm below -
far side not shown)

FIGURE 2-30

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

Continuous Bent Plate

Use 75 mm x 150 mm bent plate with long leg horizontal. By inspection, t = 6 mm is adequate.

Proprietary Slide Clip

Check capacity with manufacturer's test reports.

Concrete Anchor for Bent Plate to Roof Slab Connection

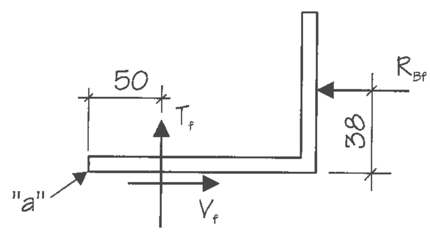


FIGURE 2-31

Factored shear
$V_f = R_{Bf} = 2.583$ kN/stud *(from Step 8(a))*
$= 2.583/0.4 = 6.46$ kN/m

Factored tension from moments about "a" Figure 2-31
$T_f = 38R_f/50 = 38(6.46)/50$
$= 4.91$ kN/m

Try 9.5 mm diameter wedge type expansion anchor with:

$f_c' = 25$ MPa
Edge distance = 75 mm
Embedment = 50 mm

Factored shear resistance *(from Design Example #1 – Step 8(k))*

$R_r = 6.53$ kN/fastener
$f_{RV1} = 0.610$ (edge distance reduction factor)

$V_r = f_{RV1}R_r = 0.610(6.53)$
$= 3.98$ kN/fastener

Factored tensile resistance *(from Design Example #1 – Step 8(k))*

R_r = 6.53 kN/fastener
F_{RN} = 0.933 (edge distance reduction factor)

$T_r = f_{RN}R_r = 0.933(6.53)$
$= 6.09$ kN/fastener

Check interaction of shear and moment using the equation from Appendix B.1 Note #3 with S = fastener spacing in metres.

$$\left(\frac{ST_f}{T_r}\right)^{5/3} + \left(\frac{SV_f}{V_r}\right)^{5/3} \leq 1.0$$

$$\left(\frac{4.91S}{6.09}\right)^{5/3} + \left(\frac{6.46\,S}{3.98}\right)^{5/3} \leq 1.0$$

Solving by trial and error

S = 524 mm o.c.

Use 500 mm o.c.

For jamb locations (by inspection) use 2 anchors spaced at $2.25h_{act} = 2.25(50) = 112.5$ mm – use 125 mm.

Bent Plate Bending Strength Between Concrete Anchors

OK by inspection.

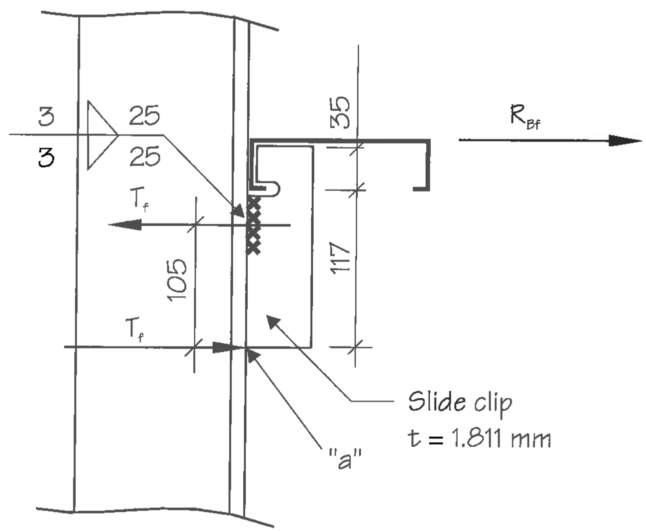


FIGURE 2-32

Slide Clip to Bent Plate Welded Connection

See Figure 2-32.

R_{Bf} = 2.583 kN/stud *(from Step 8(a))*

Assuming a uniform stress along the length of the weld the load per weld is given by taking moments about "a"

T_f = $(152R_{Bf}/105)/2$ *(welds top and bottom)*
= [152(2.583)/105]/2
= 1.87 kN

For both the slide clip and the bent plate F_u = 450 MPa

Factored weld resistance per mm of weld length is given by:
$q_r = \phi P_n/L = \phi 0.75tF_u$ *(Appendix A.1)*
= 0.40(0.75)(1.811)(450)
= 244 N/mm of weld length

Required weld length = T_f/q_r
= 1870/244
= 7.7 mm

Use 25 mm weld length top and bottom as a minimum practical weld length.

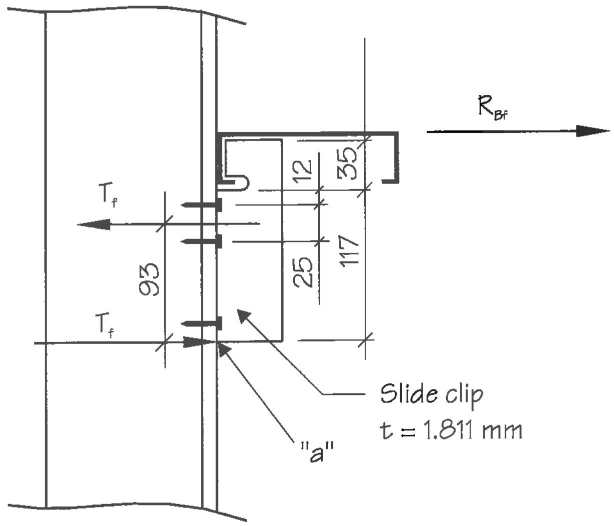


FIGURE 2-33

Slide Clip to Bent Plate Powder Actuated Fastener (PAF) Connection

See Figure 2-33 and Note 2-13

R_{Bf} = 2.583 kN/stud *(from Step 8(a))*

Assuming equal tensile force in each PAF the force in each PAF is given by taking moments about "a"

$T_f = (152R_{Bf}/93)/2$ (2 PAF's)
$= [152(2.583)/93]/2$
$= 2.11$ kN/PAF

Factored tensile resistance per PAF *(Appendix B.4)*

For the bent plate F_u = 450 MPa and t = 6.35 mm. PAF diameter = 3.68 mm

$T_r = \phi_e \overline{X}/1.33 = 0.32(15.0)/1.33$
$= 3.61$ kN > 2.11 kN **_OK_**

Check pullover assuming CAN/CSA-S136-01 Section E4.4.2 applies to PAF's

For head diameter = 8.18 mm *(Appendix B.4 Table B.4-1)*

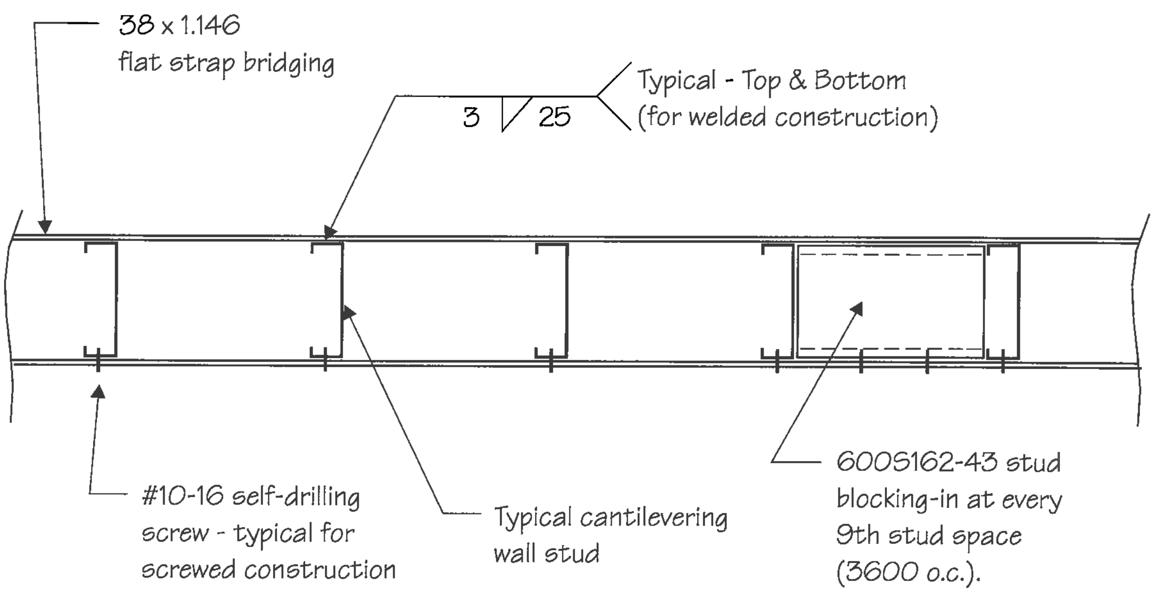
$P_r = \phi P_{nov} = \phi 1.5 t_1 d_w F_{u1}/1000$
$= 0.40(1.5)(1.811)(8.18)(450)/1000$
$= 2.61$ kN > 2.11 kN **_OK_**

Note 2-13

Where powder 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 powder actuated fasteners in tension - see NBC 2005 Clause 4.1.8.17 (8) (d)

Step 8(d) – Welded Flat Strap Bridging Design

As discussed in Note 2-9 Item 3, the proprietary slide clip is assumed to provide no torsional restraint and a line of bridging is required. The line of bridging is shown in Figure 2-30, a distance of 300 mm from the roof reaction point (R_B). For the bridging flat strap details see Figure 2-34. For blocking-in details see Figure 2-35.



Note: Figure 2-34 shows flat strap connection details.
For blocking-in connection details see Figures 2-35 & 2-36.

FIGURE 2-34

Check bridging force required to restrain the stud according to Supplement S136S1-04 *(CSA 2004a)* Section D3.2.2. 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(a)
R_{Bf} = 2.583 kN

and
m = 17.0 mm
d = 151 mm (centreline stud depth)

The force R_{Bf} is within 0.3 a of the bridging line, therefore:

$P_{Lf} = (m/d)R_{Bf}$
$= (17.0/151)(2.583)$
$= 0.291$ kN

(The applied wind load 0.5a either side of the bridging line offsets P_{Lf} and has been conservatively neglected here.)

Assume blocking-in every 9 stud spaces = 3.6 m o.c.

The accumulated factored bridging force
$T_f = 9(0.291) = 2.62$ kN

Try 38 mm x 1.146 mm flat strap bridging *(Figure 2-34)*

Check tensile capacity of flat strap – gross cross section check only – CAN/CSA-S136-01 Section C2.1.

$T_r = \phi_t T_n = \phi_t A_g F_y$
$= 0.9(38)(1.146)(230)/1000$
$= 9.01 \text{ kN} > 2.62 \text{ kN}$ ***OK***

Blocking-in connection - see Figure 2-35.

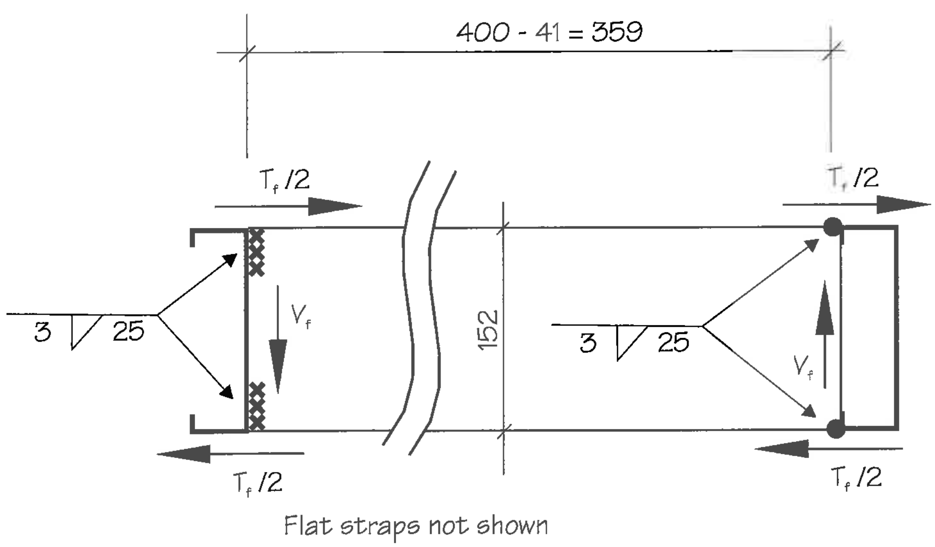


FIGURE 2-35

The blocking-in is essentially a shear panel with equal and opposite applied loads, T_f, and with the internal forces, V_f, required for rotational equilibrium. The forces, V_f, are in turn resisted by the major axis bending strength of the connecting studs. Also see Note 2-14.

> *Note 2-14*
>
> *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 recommended here such that any unbalanced loads when they do occur are relatively small.*

The details in Figure 2-35 assume that there is no direct weld between the flat strap and the blocking-in. *(It is difficult to weld a 38 mm wide flat strap to a 41 mm 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 2-34 *(not shown in Figure 2-35)*.

Check weld capacity - strap load is transferred through 2 adjacent studs with 2 welds per stud for a total weld length of 4 x 25 = 100 mm.

$$V_r = \phi 0.75tLF_u = 0.40(0.75)(1.146)(100)(310)/1000$$
$$= 10.66 \text{ kN} > 2.62 \text{ kN} \qquad \textbf{\textit{OK}}$$

Then the loads are transferred to the blocking-in via the welds shown in Figure 2-35. But these welds are also simultaneously resisting the internal shear force V_f. Each of the 4 welds in Figure 2-35 is therefore subject to $T_f/2$ and $V_f/2$.

$$V_f = 152T_f/359 = 152(2.62)/359$$
$$= 1.109 \text{ kN}$$

For each 25 mm weld:

Factored resultant load = $\sqrt{(V_f/2)^2 + (T_f/2)^2} = \sqrt{(1.109/2)^2 + (2.62/2)^2}$
= 1.42 kN/25 mm weld

$$V_r = \phi 0.75tLF_u = 0.40(0.75)(1.146)(25)(310)/1000$$
$$= 2.66 \text{ kN} > 1.42 \text{ kN} \qquad \textbf{\textit{OK}}$$

For torsional eccentricity to the centreline of the flange see Note 2-15.

Step 8(e) – Screwed Flat Strap Bridging Design

Refer to Step 8(d) for the derivation of the factored design loads T_f and V_f. Refer also to Note 2-14. For the bridging flat strap details see Figure 2-34. For blocking-in details see Figure 2-36.

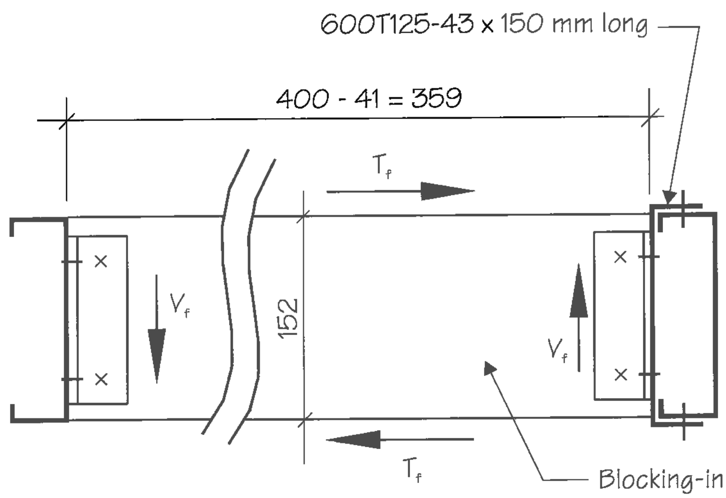


FIGURE 2-36

From Step 8(f) assuming again blocking-in every 9 stud spaces:
T_f = 2.62 kN
V_f = 1.109 kN

Try 38 mm x 1.146 mm flat strap bridging *(Figure 2-34)*

Check tensile capacity of flat strap – net cross section check because of the screw hole – CAN/CSA-S136-01 Section C2.2.

#10-16 self-drilling screw diameter = 4.83 mm *(Table A-2)*

$T_r = \phi_u T_n = \phi_u A_g F_u$
$= 0.75(38 - 4.83)(1.146)(310)/1000$
$= 8.84 \text{ kN} > 2.62 \text{ kN}$ ***OK***

Gross area check – see Step 8(f) ***OK***

From Step 5(d) the factored shear resistance of the #10-16 self-drilling screw is given by:
V_r = 1.40 kN/screw

Unlike the welded detail, the flat strap is screw connected directly to the blocking-in. This connection is shown in Figure 2-34.

Number of screws require for the flat strap to blocking-in connection is given by:

No. of screws = T_f / 1.40 = 2.62/1.40 = 1.9 screws

Use 2 screws ***OK***

Blocking-in connection - see Figure 2-36

V_f = 1.109 kN *(from Step 8(f))*

The factored resistance of the angle connection in shear was previously investigated in Design Example #1 Step 8(f). The same detailing as illustrated in Figure 1-18 is assumed.

From that example:

V_f = 1.4(1.32) = 1.848 *(factored shear applied to the connection)*

resulting in:
V_f /screw = 0.995 kN

But:
V_r /screw = 1.40 kN

Therefore, the maximum factored resistance of the connection is given by:

V_r = 1.848(1.40/0.995)
= 2.60 kN > 1.109 kN ***OK***

Note 2-15

For torsional eccentricity to the centerline of the flange:
e = m + (flange width)/2 = 37.1 mm

Gives:
T_f *= (37.1/17.0)(2.62) = 5.72 kN*
V_f *= (37.1/17.0)(1.109) = 2.42 kN*

For welded blocking-in use same details but reduce spacing to 7 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(f) – 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. The connection to the roof slab will require reinforcement.

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

Introduction

This design example reviews three alternative methods for framing strip windows with lightweight 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 centerline 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 = 600 mm o.c.
- Stud depth = 152 mm
- 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 = 0.8 kPa
- Geometry *(in elevation)* is illustrated in Figure 3-2.

Step 2 – Design Wind Loads

From the NBCC 2005:

Specified (unfactored) wind load for strength	= 1.20 kPa
Specified (unfactored) wind load for deflection	= I_w (1.20) = 0.75(1.20)
	= 0.900 kPa

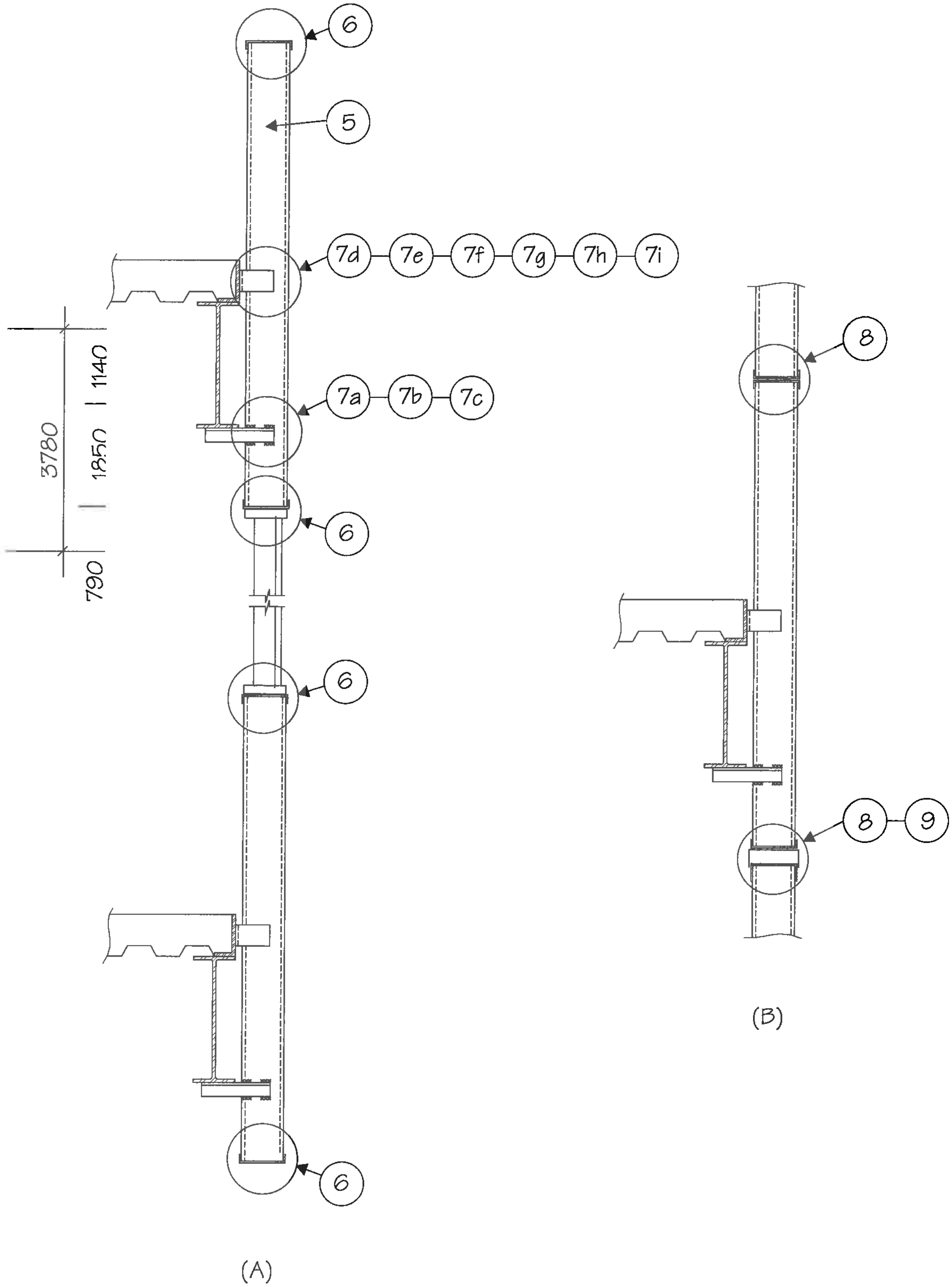


FIGURE 3-1

Step 3 – Design Earthquake Loads

From NBCC 2005 Clause 4.1.8.17:

Seismic force acting in any horizontal direction:

For cantilevering exterior walls: $V_p = 0.3W_p$

For the body of ductile connections: $V_p = 0.3W_p$

For fasteners in the connections: $V_p = 0.75W_p$

where W_p = the weight of the wall

Step 4 – Alternative Design Approaches

Step 4(a) – Large 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 5600 mm.

Check the ability of the window sill to span 5600 mm.

Wind load tributary width:

$= (1.85 + 0.79)/2 = 1.32$ m

Factored moment

$$M_f = 1.4(1.20)(1.32)(5.6)^2/8$$
$$= 8.69 \text{ kN.m}$$

Required inertia

$$\delta = \frac{5wL^4}{384EI} = \frac{5(0.75)(1.20)(1.32)(5600)^4}{(384)(203000)I}$$
$$= \frac{74.9(10)^6}{I}$$

Substituting $\delta = L/360 = 5600/360$ and solving for I gives:

$I_{req} = 4.82(10)^6 \text{ mm}^4$

For built-up window sill illustrated in Figure 3-3, try 600S162-68 stud and 2 - 600T125-68 track all with F_y = 345 MPa. *(Track with F_y = 345 MPa may require a special order – check with local manufacturers.)*

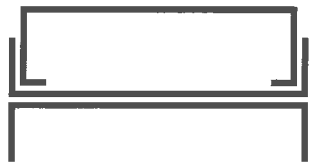


FIGURE 3-3

From manufacturer's literature *(see Note 1-5 for an alternative approach to built-up member analysis)*:

M_r = 5.98 + 2(4.36)
= 14.70 kN.m > 8.69 kN.m **OK**

I_{req} = [1.47 + 2(1.22)](10)^6
= 3.91(10)^6 mm^4 < 4.82(10)^6 mm^4 **UNSATISFACTORY**

Sill deflections are excessive even for a built-up section made with thick members *(design thickness = 1.811 mm was checked here)*. Therefore, the one large punched window approach is not practical.

Step 4(b) – LSF Mullions (Design Alternative #2)

The span length of window head and sill members can be reduced with the introduction of LSF mullions as shown in Figure 3-4.

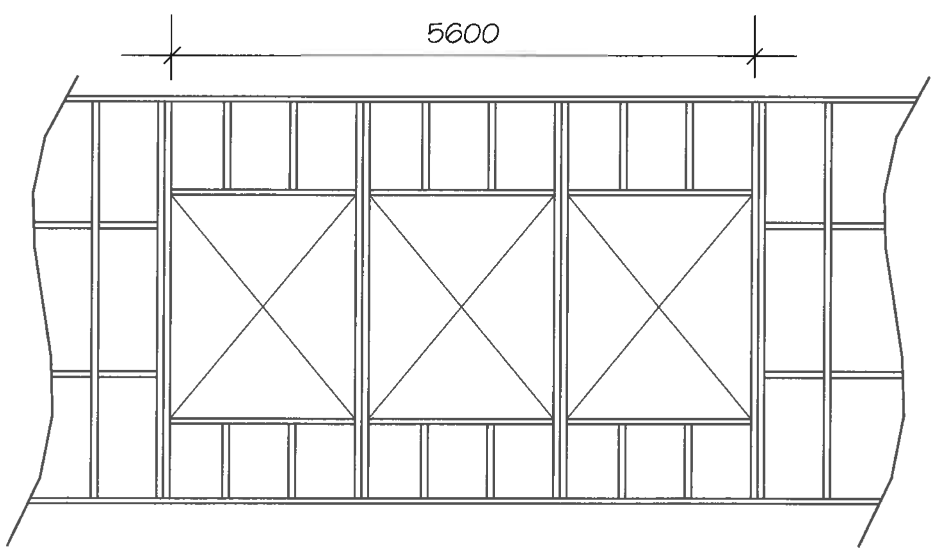


FIGURE 3-4

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 2-9 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 LSF members are designed as infill around the hot-rolled cantilevers.

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.

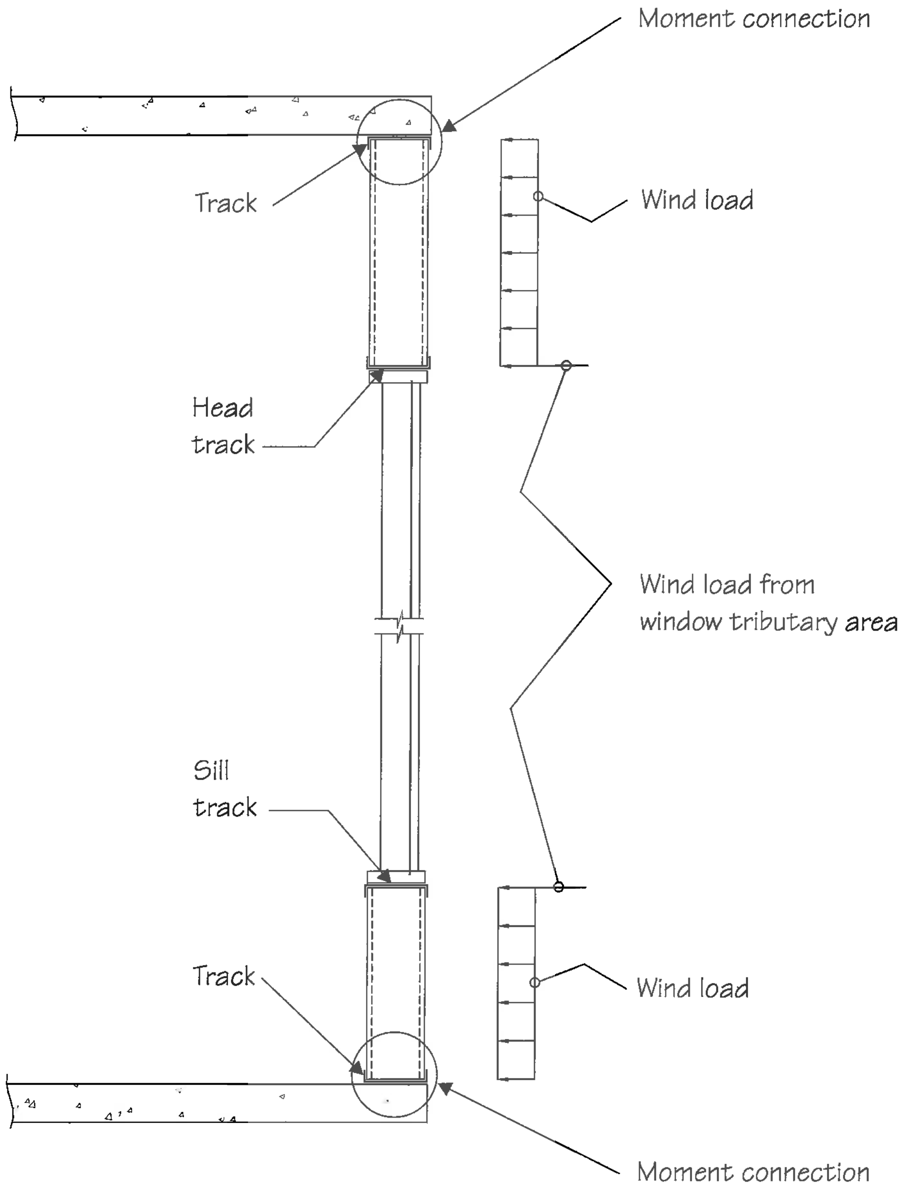


FIGURE 3-5

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 concrete slab floors.

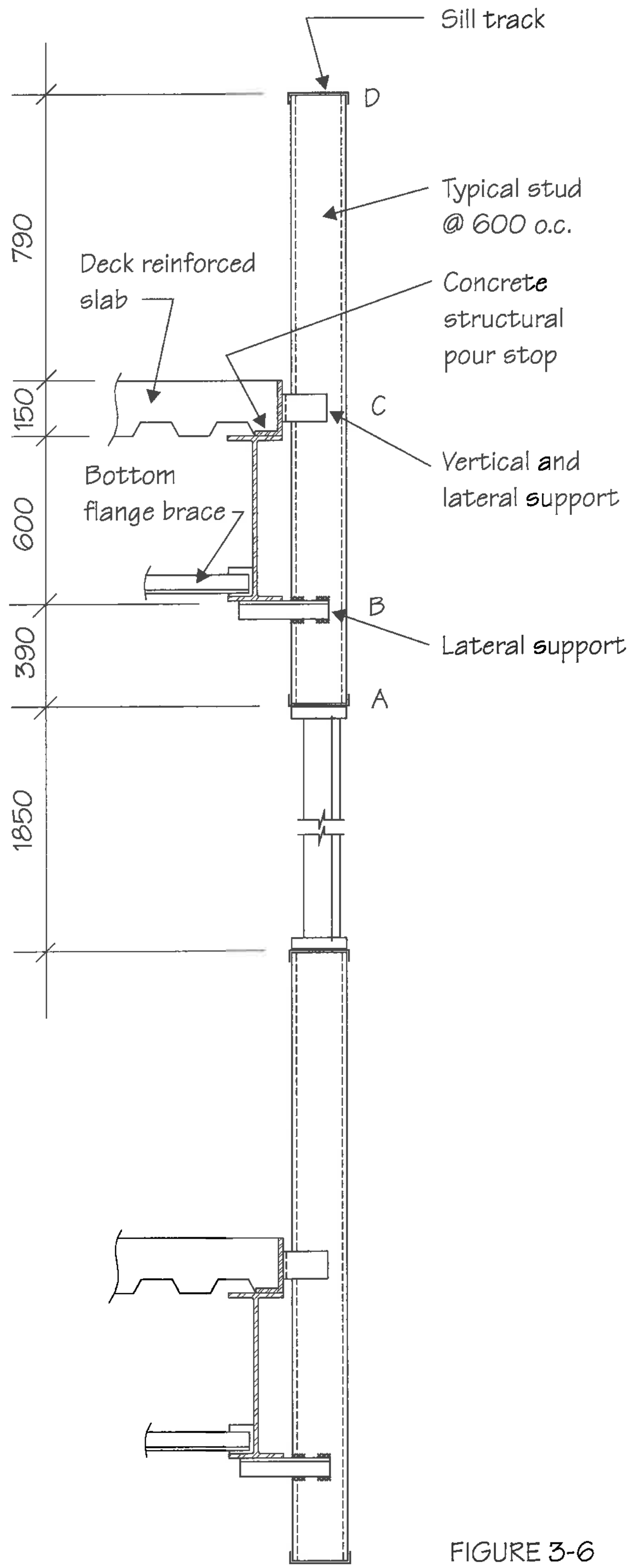


FIGURE 3-6

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 & C assuming a 50 x 50 angle at B. The NBCC 2005 partial wind load provisions *(4.1.7.3)* are assumed not to apply.

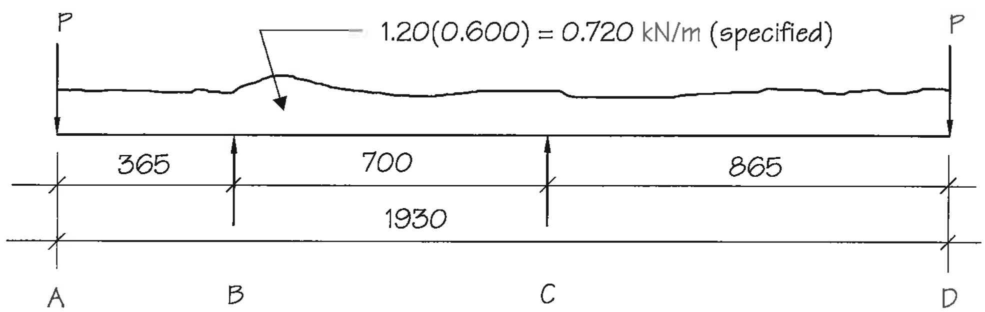


FIGURE 3-7

> *Note 3-1*
>
> *Co-ordination with the hot-rolled steel design is required to insure that adequate bottom flange braces are provided to pick up the stud reaction at B.*

P = (1.85/2)(1.20)(0.600)
= 0.666 kN per stud (specified)

From moments about B

$0.700R_c + 0.365(0.666) + 0.720(0.365)^2/2 - 1.565(0.666) - 0.720(1.565)^2/2 = 0$

R_C = 2.333 kN (specified)

R_B = 0.720(1.93) + 2(0.666) - 2.333
= 0.389 kN (specified)

Maximum factored moment at C *(1.4 load factor)*

$M_f = 1.4[(0.720)(0.865)^2/2 + 0.865(0.666)]$
=1.184 kN.m

Maximum factored shear to the right of C *(1.4 load factor)*

$V_{req} = 1.4[(0.720)(0.865) + 0.666]$
= 1.804 kN

Try 600 S162-43 with F_y = 230 MPa

Check moment and shear at C

M_r = 2.59 kN.m > 1.18 kN.m **_OK_**

V_r = 7.03 kN > 1.80 kN **_OK_**

Check combined moment and shear at B *(Supplement S136S1-04 (CSA 2004a) Section C3.3.2)*

$$\sqrt{\left(\frac{\overline{M}}{\phi_b M_{nxo}}\right)^2 + \left(\frac{\overline{V}}{\phi_v V_n}\right)^2} \le 1.00$$

where:
$\overline{M} = M_f = 1.18$ kN.m
$\overline{V} = V_f = 1.80$ kN
$\phi_b M_{nxo} = M_r = 2.59$ kN.m
$\phi_v V_n = V_r = 7.03$ kN

Substituting:

$$\sqrt{\left(\frac{1.18}{2.59}\right)^2 + \left(\frac{1.80}{7.03}\right)^2} = 0.52 \le 1.00$$ **_OK_**

> *Note 3-2*
>
> *No web crippling check is required here because the connection transfers shear directly from the web of the stud. This contrasts with the flange clip connection in Design Example #2 Step 8(a) where both web crippling and combined web crippling and bending must be checked.*

Check lateral instability

_OK_ by inspection. See Design Example #2, Step 1(b) for procedure.

Check deflection

By computer analysis with all spans loaded as in Figure 3-7 except:
$w = I_w (0.720) = 0.75(0.720)$
$= 0.540$kN/m

δ_c = 1.47 mm *(end of cantilever)*
δ_s = -0.12 mm *(midspan)*

and the deflected shape (not shown here) is approximated by Figure 2-28. See also Note 2-11. *(L_s and δ_s are taken to the middle of the back-span. A more rigorous procedure would locate the point of zero slope for both L_s and δ_s.)*

L_s = 700/2 = 350 mm
L_c = 865 mm
$L' = 2(L_s + L_c)$ = 2(350 + 865) = 2430 mm

L'/360 = 2430/360 = 6.75 mm

$|\delta_c + \delta_s| = |1.47 + 0.12| = 1.59\,\text{mm} < 6.75\,\text{mm}$ ***OK***

> *Note 3-3*
>
> *Based on the above design checks, a 600S162-33 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.

Wind load = 1.2 kPa

Earthquake load acting on exterior walls = 0.3 W_p where W_p = 0.8 kPa

Then earthquake load = 0.3(0.8) = 0.24 kPa in any horizontal direction

With earthquake in the plane of the wall, the applied weak axis factored moment is given by:

M_f = (0.24/1.20)(1.184)/1.4 *(Earthquake load factor is 1.0)*
= 0.169 kN.m

From CSSBI load tables *(CSSBI 2004)* choose the lesser of the weak axis moment with the lips or the web in compression.

M_{ry} = 0.363 kN.m > 0.169 kN.m **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, use 600S162-43 studs at 600 mm o.c. with F_y = 230 MPa.

Step 6 – Typical Track

Arbitrarily select 600T125-54 track at window head and window sill for resistance to construction abuse.

Step 7 – Typical Stud Connections

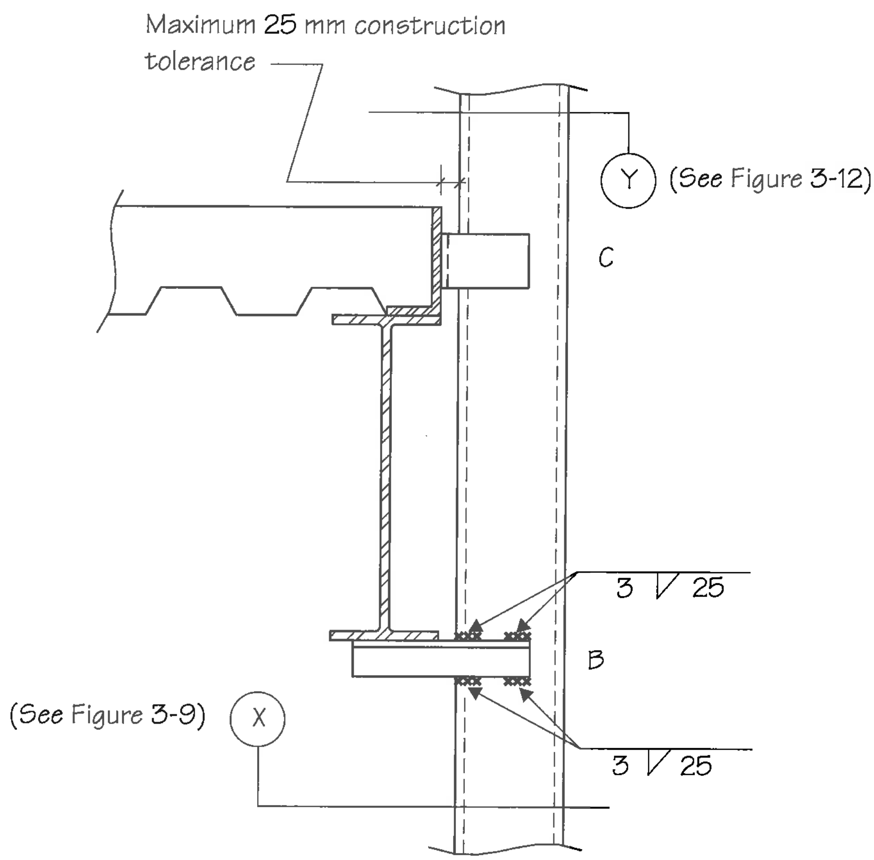


FIGURE 3-8

Step 7(a) – Lateral Support Connection at B Under Wind Load – Body of the Connector

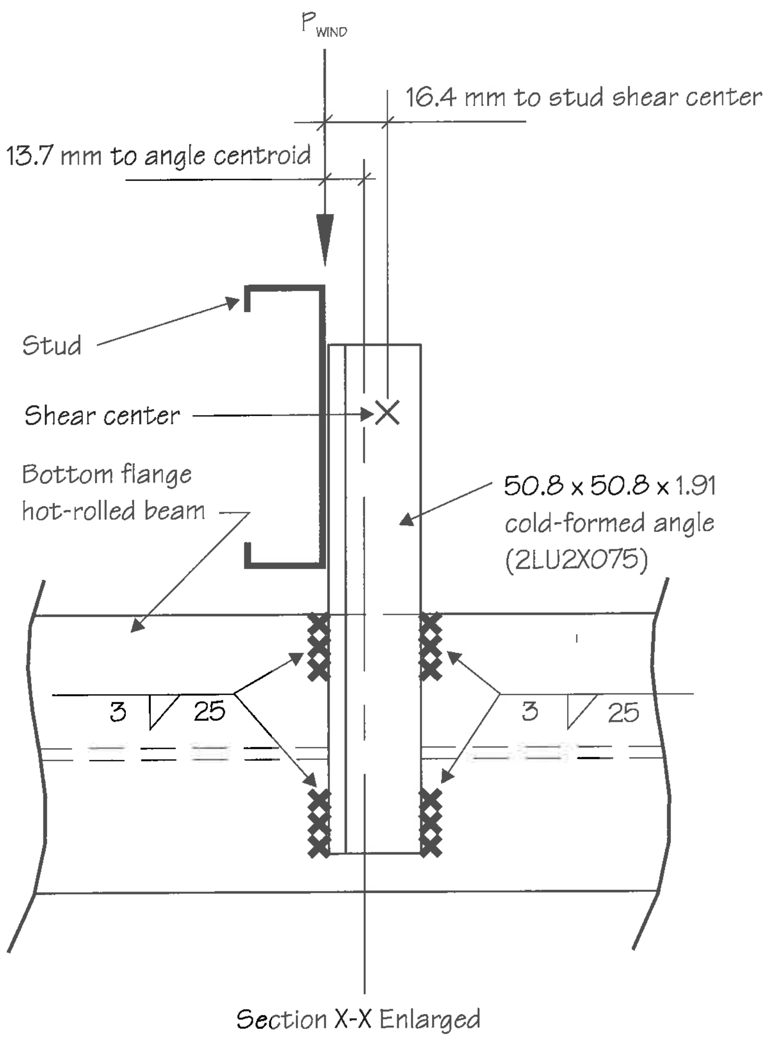


FIGURE 3-9

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 simplifying and conservative assumptions.*

2. *The 1.91 mm angle thickness has been chosen to match angle section properties available in AISI 2002b Part 1, Table 1-7. Check availability before specifying – a design thickness of 1.811 mm is more common in the LSF industry.*

Try 50.8 x 50.8 x 1.91 mm cold-formed angle

For properties see AISI 2002b, Part 1, Table 1-7.

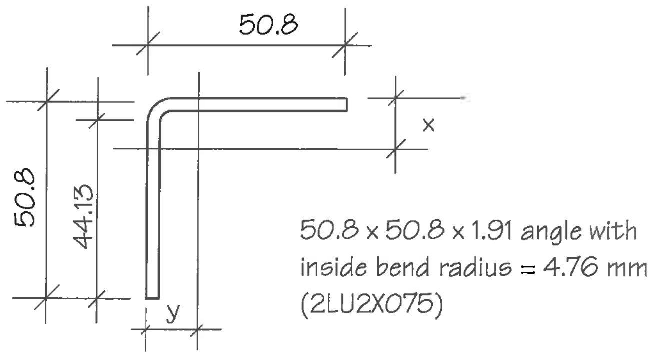


FIGURE 3-10

A = Area = 185 mm^2
x = y = distance to centroid = 13.7 mm
$S_x = S_y$ = section modulus about horizontal and vertical axis = 1304 mm^3

and the shear centre to the back of the stud web is given by:
m - t/2 = 17.0 - 1.146/2 = 16.4 mm

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 (similar to the torsional end restraint 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.

By Appendix H compressive stresses in the angle are limited to:

$$f = \frac{35700}{(w/t)^2}$$

$$= \frac{35700}{(44.13/1.91)^2} = 66.9\,\text{MPa}$$

By inspection, overall stability effects can be neglected. Check interaction for strength only by CAN/CSA-S136-01 Section C5.2.2:

$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \leq 1.0$$

Where:

$\overline{P}$ = factored wind load at B *(1.4 load factor)*
= $R_B(1.4)$ = 389(1.4)
= 545 N

$\overline{M}_x = \overline{M}_y = \overline{P}e$ = (545)(13.7) = 7470 N.mm

$\phi_c P_{no}$ = 0.80fA = 0.80(66.9)(185) = 9900 N

$\phi_b M_{nx} = \phi_b M_{ny} = \phi_b$ fS = 0.90(66.9)(1304) = 78500 N.mm

Substituting:

$$\frac{545}{9900} + \frac{7470}{78500} + \frac{7470}{78500} = 0.25 < 1.00 \qquad \textit{OK}$$

Step 7(b) – Lateral Support Connection at B Under Earthquake Load – Body of the Connector

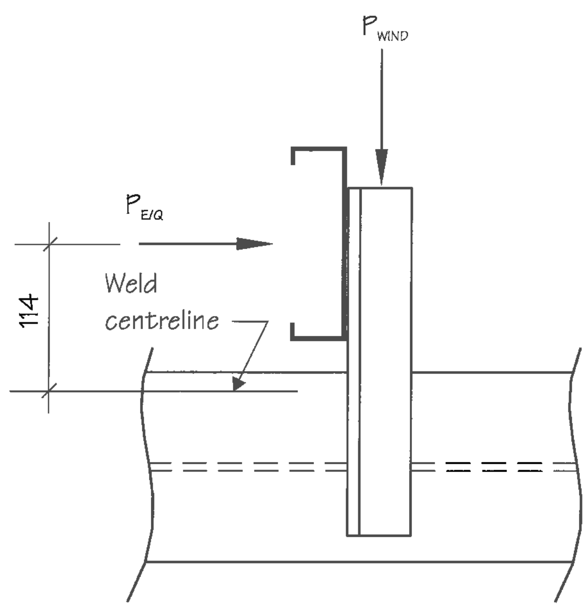


FIGURE 3-11

Assume the earthquake factored reaction can be found by proportioning the wind load effect.

$$E/Q\ \text{load} = 0.3(W_p) = 0.3(0.8) = 0.24\ \text{kPa}$$

Wind Load = 1.2 kPa (specified)

$P_{WIND} = R_B = 0.389$ kN *(Step 5(a))*

Then the factored earthquake reaction at B (with an earthquake load factor = 1.0)

$$P_f = P_{E/Q} = 0.389(0.24/1.2)(1.0) = 0.0778\ \text{kN}$$

$P_{E/Q}$ induces a moment in the angle with a lever arm of say 114 mm to the centerline of the 25 mm weld *(Figure 3-11)*.

$$M_f = 114P_{E/Q} = 114(77.8) = 8870\ \text{N.mm}$$

Using the limiting stress from Step 7(a) for fully effective behavior

$$M_r = \phi f S_x = (0.90)(66.9)(1304) = 78500\ \text{N.mm} > 8870\ \text{N.mm}$$ ***OK***

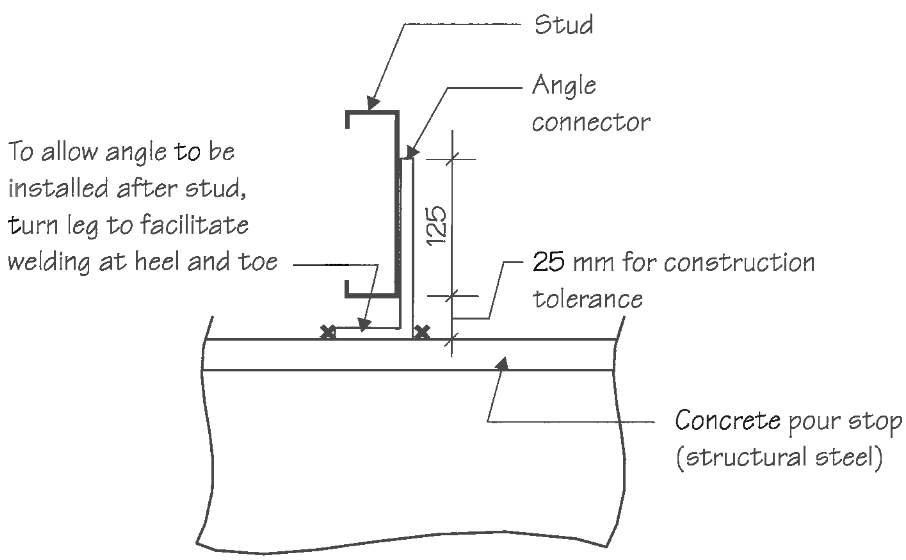
Therefore, a 50.8 x 50.8 x 1.91 mm angle *(2LU2X075)* is adequate for both wind and earthquake.

> *Note 3-6*
>
> *The 50.8 x 50.8 x 1.91 mm angle has a large strength reserve. This reserve is useful for miscellaneous parts since they can be engineered with a minimum of effort without too much concern for precise eccentricities and loads. Also, the welding position for these parts is often awkward and the thicker 1.91 mm material is less susceptible to welding damage.*

Step 7(c) – Lateral Support Connection at B Under Wind or Earthquake Load – Welds at Either End

The welds have a 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



Section Y-Y

FIGURE 3-12

By NBCC 2005 possible load combinations are:
1.4D
1.25D + 1.4W

At reaction C from Step 5(a)
$P_{WIND} = R_c = 2.333$ kN (specified

P_{DL} = (stud spacing)(W_D)($H_{FLR/FLR}$)
= (0.400)(0.8)(3.78)
= 1.210 kN (specified)

See Figure 3-13.

$P_1 = P_{DL} = 1.210$ kN (specified)

$P_2 = P_{WIND} = 2.333$ kN (specified)

Similar to the torsional end restraint discussed in Appendix I, the angle transfers the torsional restraint component Rm - Kawm. The term Kawm is conservatively neglected here.

$P_3 = (m/d)P_2$

where:

d = 100 mm for center to center spacing of 25 mm long welds when studs are 25 mm from face of structure

m = centerline of web to shear center of stud = 17.0 mm

P_3 = (17.0/100)(2.333)
= 0.397 kN (specified)

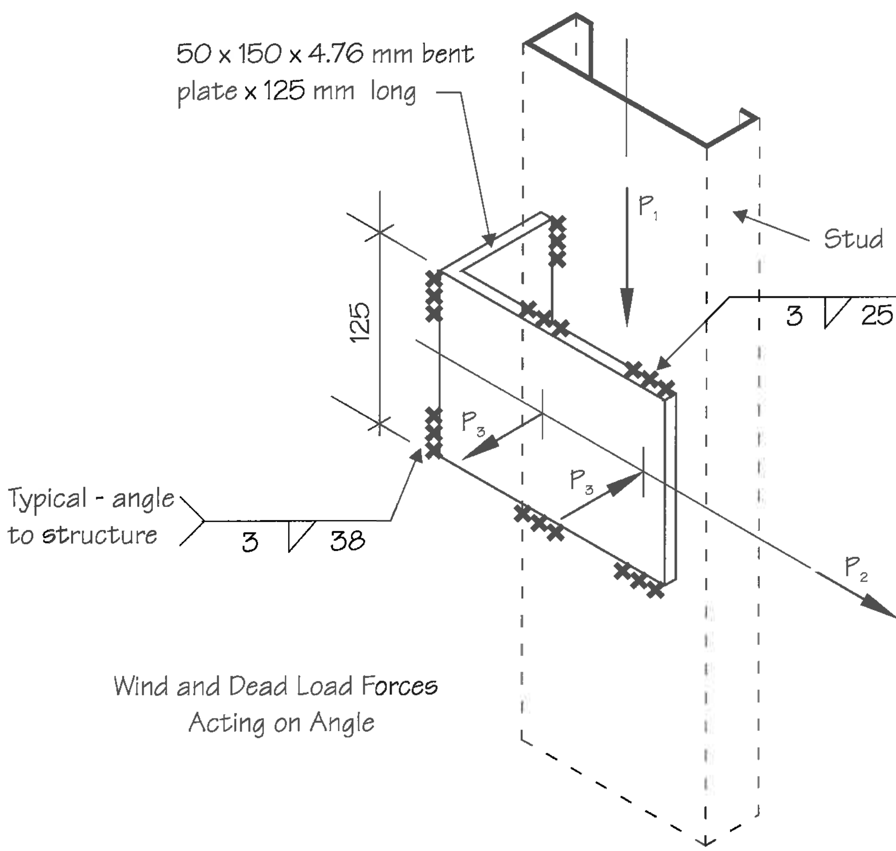


FIGURE 3-13

Calculate thickness of angle, t.

A = 125t
$S_x = (1/6)t(125)^2 = 2604t$
$S_y = (1/6)(125)t^2 = 20.8t^2$

For 1.25D + 1.4W at heel of angle

M_{fx} = 1.25P_1(100) = 1.25(1210)(100)
= 151300 N.mm

M_{fy} = 1.4P_3(138-38) = 1.4(397)(100)
= 55600 N.mm

T_f = 1.4P_2 = 1.4(2330)
= 3260 kN

Combined stresses – stability effects are negligible *(The following stress check is equivalent to checking CAN/CSA-S136-01 Section C5.1.2 for strength only.)*

$$\frac{M_{fx}}{S_x} + \frac{M_{fy}}{S_y} + \frac{T_f}{A} \leq \phi F_y$$

For F_y = 345 MPa

$$\frac{151300}{2604t} + \frac{55600}{20.8t^2} + \frac{3260}{125t} = 0.9(345)$$

Gives:

$311t^2 - 84.2t - 2670 = 0$

Solving the quadratic gives:

t = 3.07 mm

For 1.4D at heel of angle

Does not control by inspection

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. Assume, therefore, the earthquake reaction can be found by proportioning the wind load reaction. See Figure 3-14.

Possible load combinations:
1.4D
1.0D + 1.0E

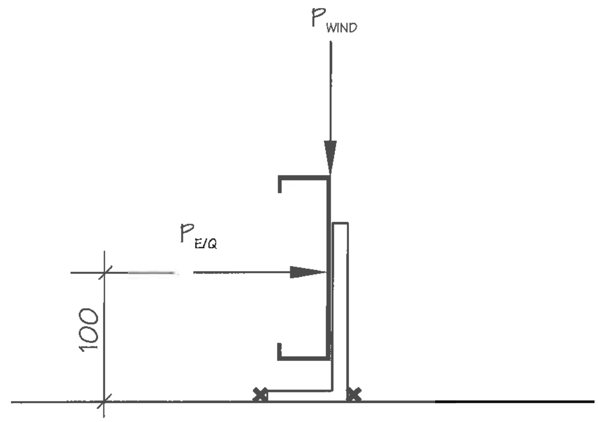


FIGURE 3-14

E/Q load = 0.3(0.8) = 0.24 kPa (specified)

Wind load = 1.2 kPa (specified)

$P_{WIND} = R_C$ = 2.333 kN (specified – Step 5a)

$P_{E/Q} = (0.24/1.2)\ R_c$
= 0.467kN

Due to dead load *(1.00 load factor and P_1 from Step 7(d))*
$M_{fx} = 1.00P_1(100) = 1.00(1210)(100)$
= 121000 N.mm

Due to earthquake *(1.00 load factor)*
$M_{fy} = 1.00P_{E/Q}(100) = (1.00)(467)(100)$
= 46700 N.mm

Compared with the wind and dead load case, the earthquake moments are less severe. Therefore t = 3.07 mm is ***OK*** by inspection.

Conservatively use t = 4.76 mm. Bent plate then is 50 x 150 x 4.76 mm x 125 mm long with F_y = 345 MPa.

Step 7(f) – Vertical and Lateral Support at C Under Dead and Wind Load – Angle to Stud Welds

See Figure 3-13.

Possible load combinations:
1.4D
1.25D + 1.4W

Each weld is loaded by 3 orthogonal forces. The specified forces from Step 7(d) are given by:

$P_1 = 1210$ N
$P_2 = 2333$ N
$P_3 = 397$ N

For 1.25D + 1.4W

$P_{1f} = 1.25(1210) = 1513$ N
$P_{2f} = 1.4(2333) = 3270$ N
$P_{3f} = 1.4(397) = 556$ N

Assuming uniform stress along the length of each weld, the factored shear resultant per weld is given by:

$$V_f = \sqrt{(P_{1f}/4)^2 + (P_{2f}/4)^2 + (P_{3f}/2)^2}$$
$$= \sqrt{(1513/4)^2 + (3270/4)^2 + (556/2)^2}$$
$$= 943 \text{ N/weld}$$

For 1.4W

Does not control by inspection.

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.

Possible load combinations:
1.4D
1.0D + 1.0E

> *Note 3-7*
>
> *The earthquake load for weld design is higher than the earthquake load for the design of the body of the connector. See Step 3.*

Assume the earthquake reaction can be found by proportioning the wind load reaction.

Earthquake load = $0.75W_p = 0.75(0.8)$
$= 0.600$ kPa

Wind load = 1.2 kPa (specified)

$P_{WIND} = R_C = 2.333$ kN (specified – Step 5(a))

$P_{E/Q} = (0.600/1.2)\ R_c = (0.600/1.2)(2333)$
$= 1167$ N (specified)

$P_{DL} = P_1 = 1210$ N (specified -Step 7(d))

For 1.0D + 1.0E

Assuming $P_{E/Q}$ is distributed equally to all 4 welds, factored resultant load per weld is given by *(1.0 load factors)*:

$$V_f = \sqrt{(P_1/4)^2 + (P_{E/Q}/4)^2}$$
$$= \sqrt{(1210/4)^2 + (1167/4)^2}$$
$$= 420 \text{ N/weld}$$

For 1.4D

$V_f = 1.4(1210)/4$
$= 424$ N/weld

Therefore, dead + wind controls from Step 7(f)

$V_f = 943$ N/weld

The weld strength is governed by the steel properties for the stud

$V_r = 0.75\phi tLF_u$ *(See Appendix A.1)*
$= 0.75(0.40)(1.146)(25)(310)$
$= 2660$ N/weld > 943 N/weld ***OK***

Therefore, use the angle to stud welds illustrated in Figure 3-13.

Note 3-8

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 factored load per mm of weld length $q_f = M_f / S_{weld}$ (where $M_f = 100P_3$ this example). Convert the other components to factored loads per mm of weld length and find the resultant maximum q_f using the square root function.*
- *Compare the resultant q_f with the factored weld resistance per mm of length as given by $q_r = \phi P_n / L = 0.75\phi\, tF_u$*

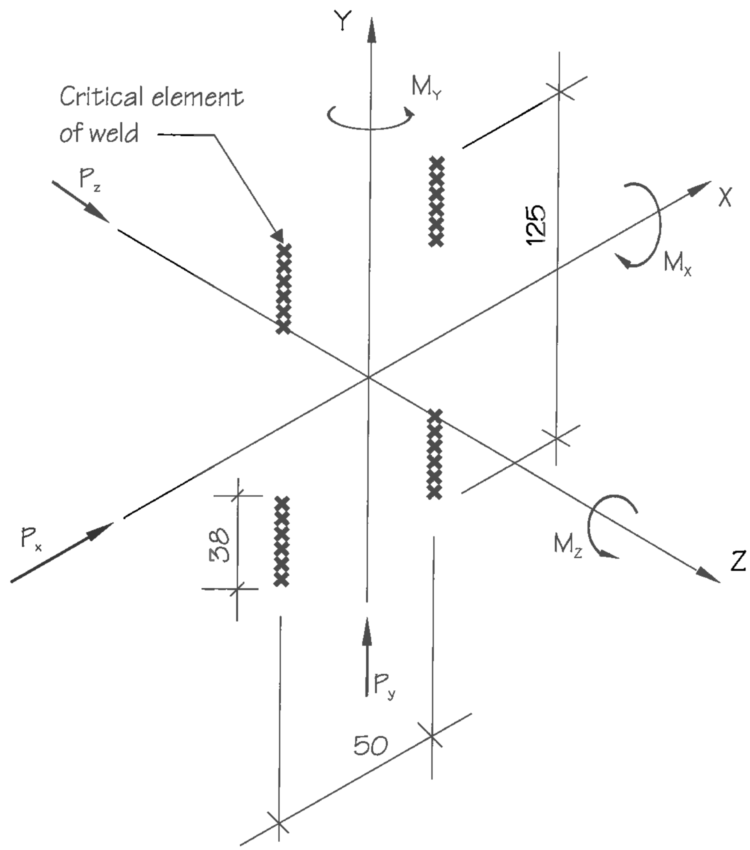
The "exact" procedure has been demonstrated in Design Example #2 Step 5e and Design Example #3, Steps 7(h) and 7(i). Given the strength reserve in this weld group the extra design time for the "exact" procedure is not justified.

Step 7(h) – Vertical and Lateral Support at C Under Dead and Wind Load – Angle to Concrete Pour Stop (Structural Steel) 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

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. Co-ordinates 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.



Weld Configuration with Axes Through
the Centroid of the Weld Group

FIGURE 3-15

See Figures 3-13 and 3-15. For P_1, P_2 and P_3 see Step 7(d).

Load Combinations
1.4D
1.25D + 1.4W

The specified forces from Step 7(d) are given by:

P_1 = 1210 N
P_2 = 2333 N
P_3 = 397 N

For 1.25D + 1.4W

P_{1f} = 1.25(1210) = 1513 N
P_{2f} = 1.4(2333) = 3270 N
P_{3f} = 1.4(397) = 556 N

Try 38 mm long welds.

$P_x = 0$

$P_Y = -P_{1f} = -1513$ N

$P_Z = P_{2f} = 3270$ N

$M_X = 100P_{1f} = 100(1513)$
$= 151300$ N.mm

$M_Y = 100P_{3f} + 25P_{2f} = 100(556) + 25(3270)$
$= 137400$ N.mm

$M_z = 25P_{1f} = 25(1513)$
$= 37800$ N.mm

Linear properties with t=1

A = 4(38) = 152 mm

$I_X = 4[(1/12)(38)^3 + 38(43.5)^2]$
$= 306000\ mm^3$

$I_Y = 4(38)(25)^2$
$= 95000\ mm^3$

$I_Z = I_X + I_Y = 401000\ mm^3$

For the critical element of weld in Figure 3-15 *(x = -25 mm and y = 62.5 mm)* with due regard to signs *(see Note 3-9)*:

$q_x' = P_X/A = 0$

$q_y' = P_Y/A = -1513/152 = -9.95$ N/mm

$q_z' = P_Z/A = 3270/152 = 21.51$ N/mm

$q_x'' = M_Zy/I_Z = 37800(62.5)/401000 = 5.89$ N/mm

$q_y'' = M_Zx/I_Z = 37800(-25)/401000 = -2.36$ N/mm

$q_z'' = M_Xy/I_X - M_Yx/I_Y = 151300(62.5)/306000 - 137400(-25)/95000$
$= 30.90 + 36.16 = 67.06$ N/mm

$q_x = q_x' - q_x'' = 0 - 5.89 = -5.89$ N/mm

$q_y = q_y' + q_y'' = -9.95 - 2.36 = -12.31$ N/mm

$q_z = q_z' + q_z'' = 21.51 + 67.06 = 88.57$ N/mm

Resultant maximum factored load per mm of weld length given by:

$$\begin{aligned} q_f &= \sqrt{q_x^2 + q_y^2 + q_z^2} \\ &= \sqrt{(5.89)^2 + (12.31)^2 + (88.57)^2} \\ &= 89.6 \text{ N/mm} \end{aligned}$$

For 1.4D *(calculations not shown here)*

$$\begin{aligned} q_f &= \sqrt{q_x^2 + q_y^2 + q_z^2} \\ &= 37.8 \text{ N/mm} \end{aligned}$$

Step 7(i) – Vertical and Lateral Support at C Under Dead and Earthquake Load – Angle to Concrete Pour Stop (Structural Steel) Welds

See Figures 3-13, 3-14 and 3-15.

Load Combinations
1.4D
1.0D + 1.0E

The specified forces are given by:

$P_1 = 1210$ N *(from Step 7(d))*
$P_{E/Q} = 1167$ N *(from Step 7(g))*

For 1.0D + 1.0E

$P_x = 1167$ N (assuming $P_{E/Q}$ acts in the positive X direction)

$P_Y = -P_{1f} = -1210$ N

$P_Z = 0$

$M_X = 100P_{1f} = 100(1210)$
$= 121000$ N.mm

$M_Y = 100P_{E/Q} = 100(1167)$
$= 116700$ N.mm

$M_z = 25P_{1f} = 25(1210)$
$= 30300$ N.mm

Linear properties from Step 7(h)
$A = 152$ mm
$I_X = 306000$ mm^3
$I_Y = 95000$ mm^3
$I_Z = 401000$ mm^3

For the critical element of weld in Figure 3-16 with due regard to signs *(see Note 3-9)*:

$q_x' = P_X/A = 1167/152 = 7.68$ N/mm

$q_y' = P_Y/A = -1210/152 = -7.96$ N/mm

$q_z' = 0$

$q_x'' = M_Z y/I_Z = 30300(62.5)/401000 = 4.72$ N/mm

$q_y'' = M_Z x/I_Z = 30300(-25)/306000 = -2.48$ N/mm

$q_z'' = M_X y/I_X - M_Y x/I_Y = 121000(62.5)/306000 - 116700(-25)/95000$
$= 55.42$ N/mm

$q_x = q_x' - q_x'' = 7.68 - 4.72 = 2.96$ N/mm

$q_y = q_y' + q_y'' = -7.96 - 2.48 = -10.44$ N/mm

$q_z = q_z' + q_z'' = 0 + 55.42 = 55.42$ N/mm

Resultant maximum factored load per mm of weld length given by:

$$q_f = \sqrt{q_x^2 + q_y^2 + q_z^2}$$
$$= \sqrt{(2.96)^2 + (10.44)^2 + (55.42)^2}$$
$$= 56.5 \text{ N/mm}$$

Therefore, 1.25D + 1.4W controls with:
$q_f = 89.6$ N/mm

Factored weld resistance

The welding of the 4.76 mm thick angle to the structural steel pour stop is still within the scope of CAN/CSA-S136-01 *(See Supplement S136S1-04 (CSA 2004a) Section E2a for thickness limit)*

For Grade 350 steel and using the approach in Appendix A.1
$q_r = \phi P_n/L = 0.75\phi t F_u = 0.75(0.40)(4.76)(450)$
$= 643$ N/mm

But since t > 2.54 mm weld factored resistance is also limited by the stresses in the throat of the weld itself – see CAN/CSA-S136-01 Section E2.4 and E2.5.

Assuming the outside bend radius, R, on the 50 x 150 bent plate = 2t = 2(4.76) = 9.52 mm:

t_w = lesser of 5R/16 or 0.707w
= 5(9.52)/16 or 0.707(3)
= 2.98 mm or 2.12 mm

t_w = 2.12 mm governs

For Grade 350 steel and E49xx electrode,
$q_r = \phi P_n/L = 0.75\phi t_w F_{xx} = 0.75(0.50)(2.12)(490)$
= 390 N/mm

q_r = 390 N/mm governs > 89.6 N/mm ***OK***

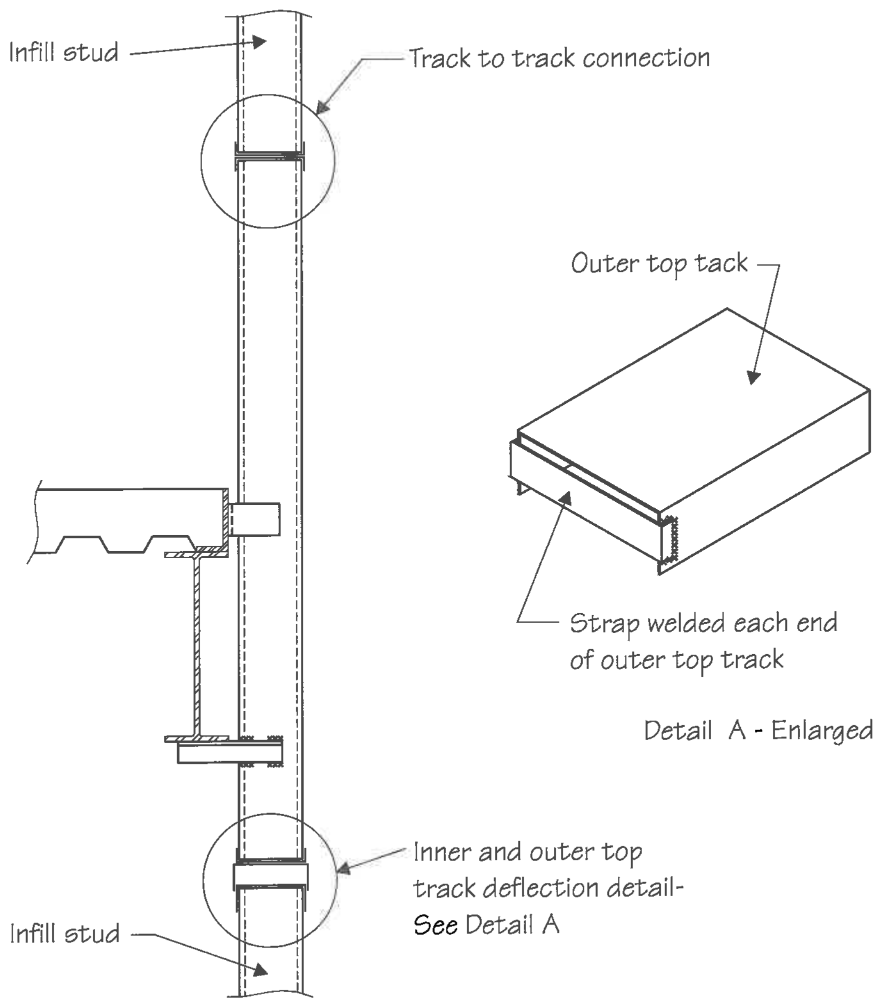
Step 8 – Stud Infill

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 would 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.

Where the deflection gap detail is used, add welded straps each end of the outer top track to provide racking resistance for the infill studs.



Section Between Strip Windows

FIGURE 3-16

Step 9 – Alternative Detail for Shop Applied Finishes

Care is required with the detailing of the welded connections because access is limited by the presence of the shop applied exterior finish.

An alternative deflection gap detail for the stud infill with exterior insulation and finish system is shown in Figure 3-17.

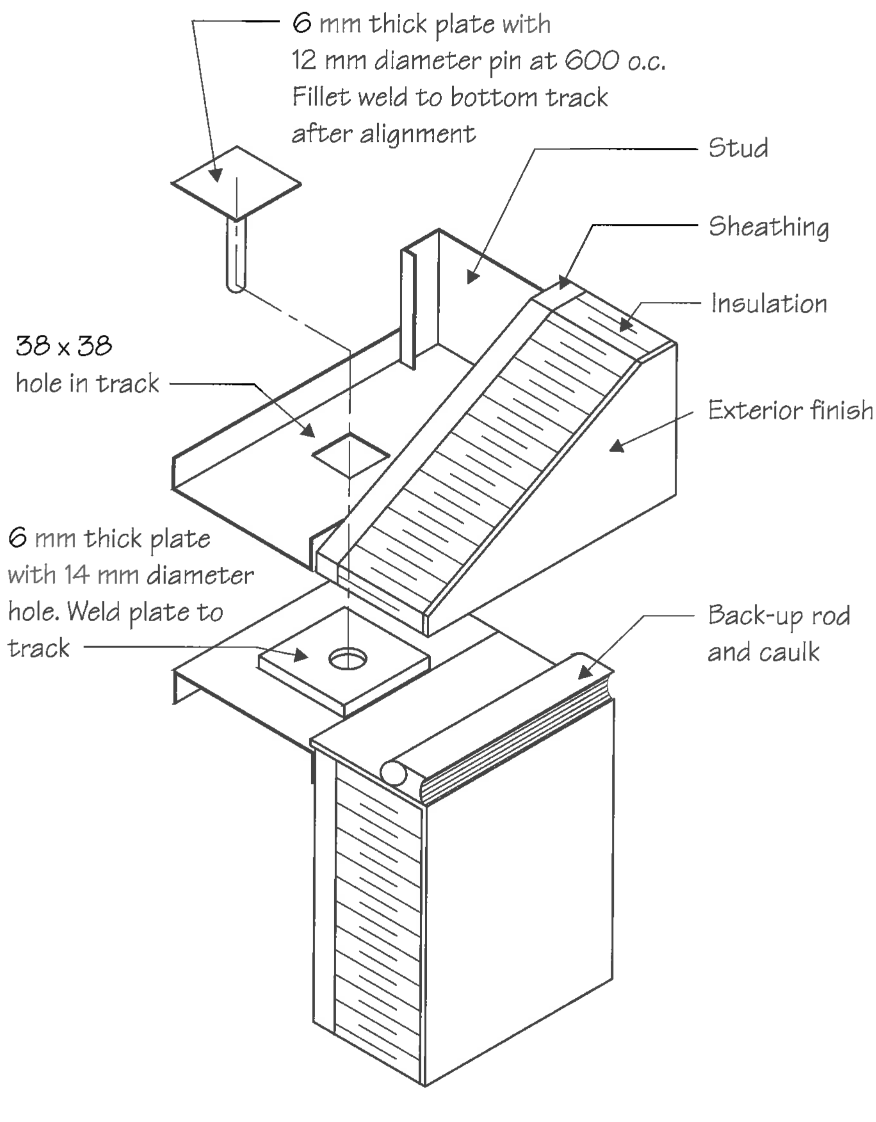


FIGURE 3-17

Design Example #4
LSF Floor and Axial Load Bearing Stud Wall

Introduction

This example covers the design of a lightweight 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.

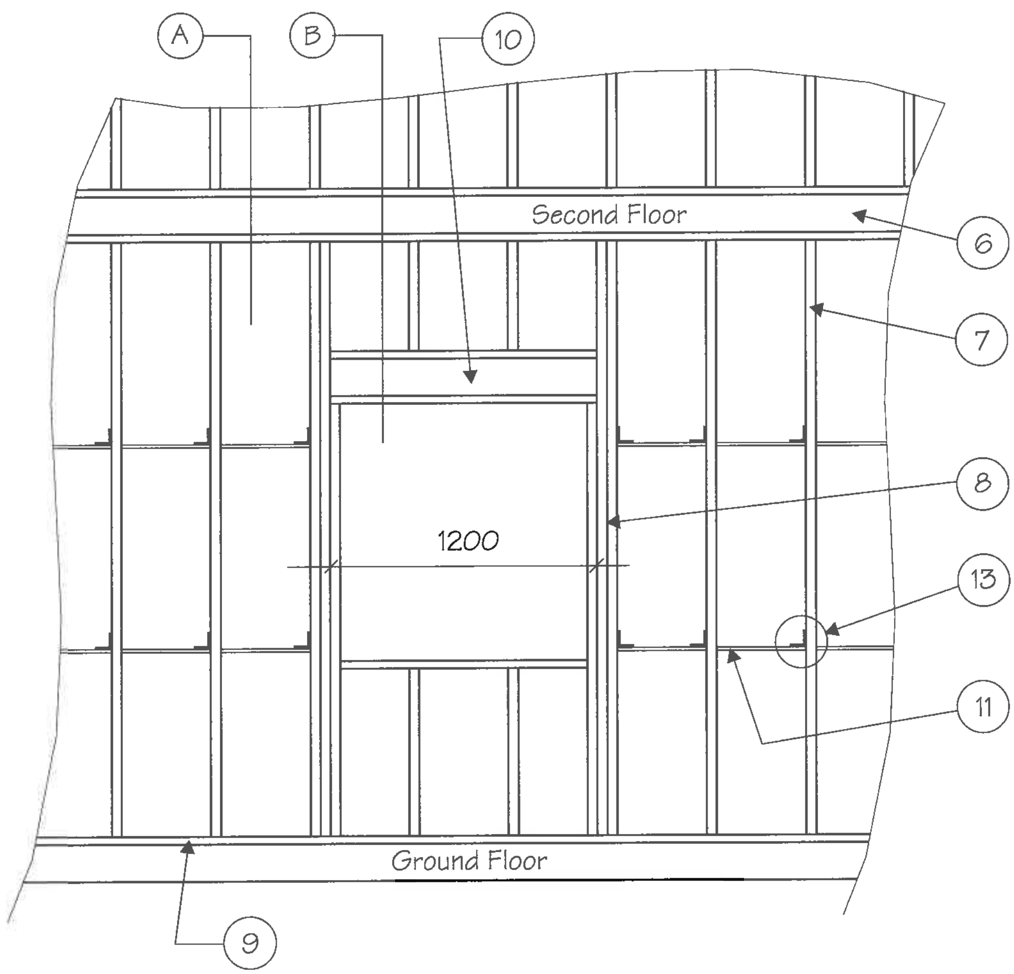
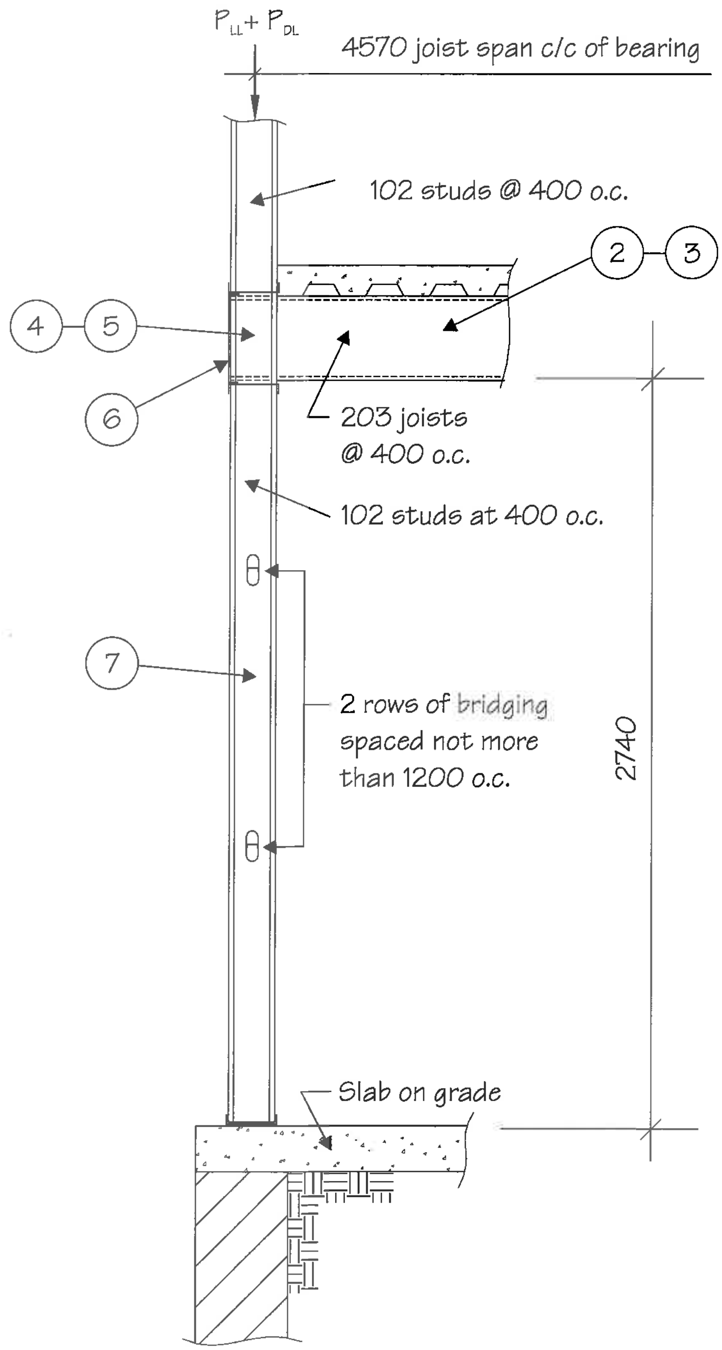


FIGURE 4-1

Step 1 – Given

- Design wind load = 1.2 kPa (specified)
- Floor design live load = 1.9 kPa (specified)
- Floor partition allowance = 0 kPa
- Wall loads from above (no snow load in this example):
  P_{LL} = 5.93 kN/stud (specified)
  P_{DL} = 2.97 kN/stud (specified)
- Wall deflection limit = L/360
- Floor deflection limit = L/360 for live 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 = 102 mm



Section A

FIGURE 4-2

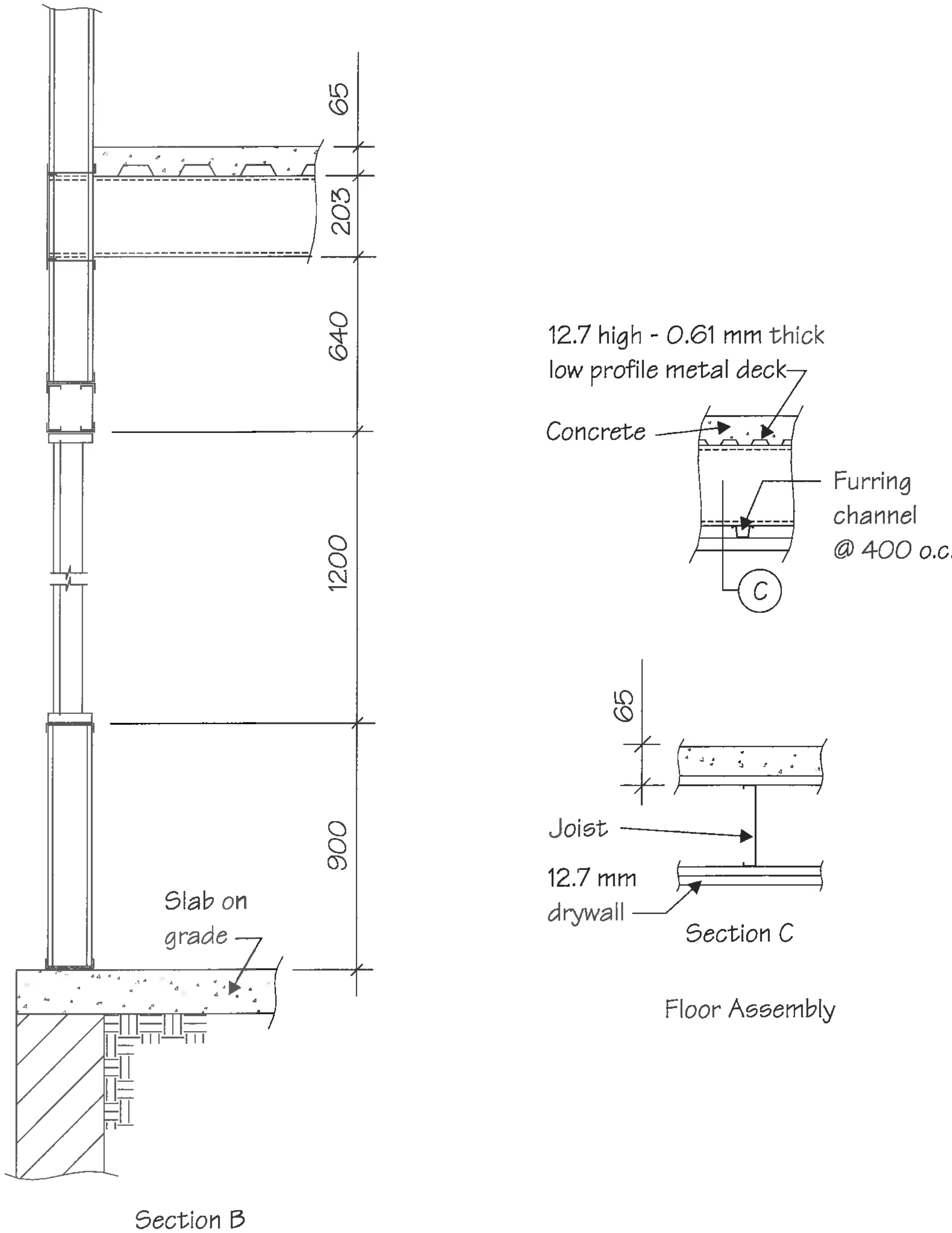

65
203
640
1200
900
Slab on grade
Section B
12.7 high - 0.61 mm thick low profile metal deck
Concrete
Furring channel @ 400 o.c.
C
65
Joist
12.7 mm drywall
Section C
Floor Assembly

FIGURE 4-3

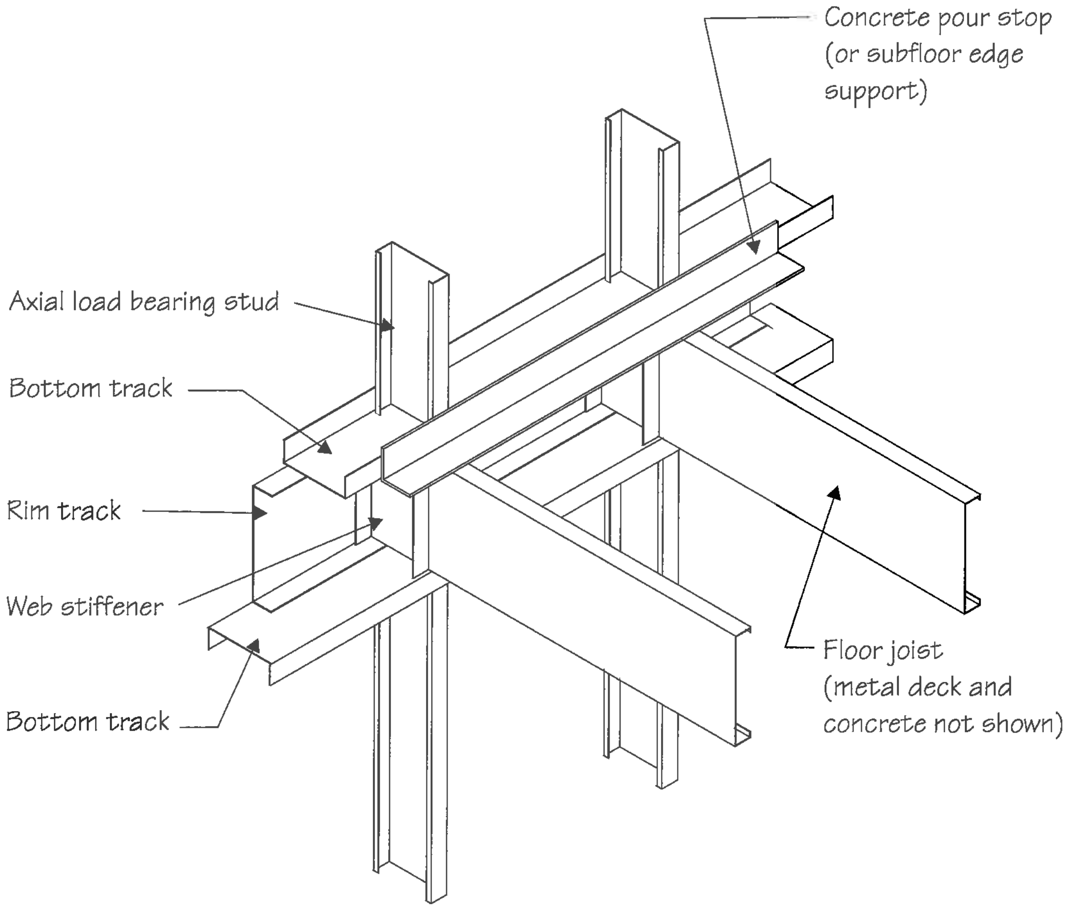


FIGURE 4-4

Step 2 – Floor Joist Selection

Live load		1.90 kPa
Dead load		
65 mm concrete	1.48 kPa	
Metal deck	0.06	
800162-54 joists @ 400	0.10	
Furring + drywall	0.10	
Floor finish & misc.	0.24	1.98 kPa

Span length L = 4570 mm single span c/c of bearing *(Figure 4-2)*

Deflection limit = L/360

Vibration criteria = none

Try 800S162-54 (50) joist @ 400 mm o.c. with F_y = 345 kPa.

Governing load combination for strength check
1.25D + 1.5L

Factored load = 1.25(1.98) + 1.5(1.90) = 5.33 kPa

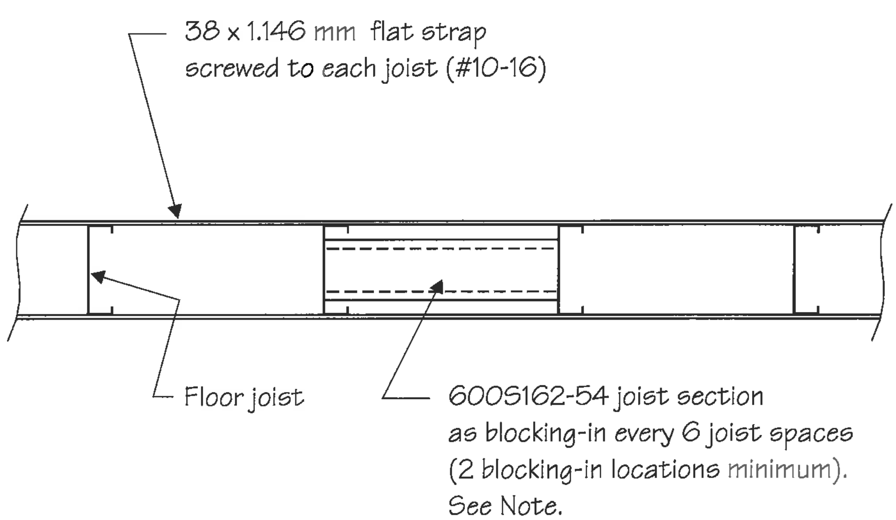
Specified live load for deflection check = 1.9 kPa

From CSSBI tables *(CSSBI 2004)* for 800S162-54 (50) and 4570 mm span length:

Factored load = 5.89 kPa > 5.33 kPa ***OK***

Specified load @ L/360 = 2.59 kPa > 1.9 kPa ***OK***

Step 3 – Floor Joist Bridging



Note:

Connect blocking-in with 38 x 38 x 1.438 angle x 140 mm long each end similar to Figure 1-18. Alternatively, provide a 800S162-54 joist section as blocking-in with connection details similar to Figure 2-36.

FIGURE 4-5

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. Appropriate details are shown in Figure 4-5.

A maximum bridging spacing of 2400 mm o.c. is commonly used in this situation. With one line of midspan bracing, spacing = 4570/2 = 2285 mm < 2400 mm ***OK***

Step 4 – Floor Joist Web Stiffener

Floor joists typically require web stiffeners to resist both 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 the Supplement S136S1-04 *(CSA 2004a)* Section C3.6.2. A two flange loading case is conservatively assumed 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 LSF industry considers framing aligned when the centerlines 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 COFS 2004c, Section C1.

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 COFS 2004c do. Tighter tolerances are required for the case of stiffeners outside.

Stiffeners outside the joist can be full height whereas stiffeners inside must be cut short to fit. The Supplement S136S1-04 *(CSA 2004a)* Section C3.6.2 specifies that the length of stiffeners shall not be less than the outside depth of the joist minus 9 mm. Other requirements also apply – see C3.6.2.

For stiffener details on this project see Figures 4-4 and 4-6.

Governing load combination

1.25D + 1.5L

P_{LL} = stud load above + floor joist reaction
= 5.93 + (4.57/2)(0.400)(1.9)
= 7.67 kN/stud (specified)

P_{DL} = stud load above + floor joist reaction
= 2.97 + (4.57/2)(0.400)(1.98)
= 4.78 kN/stud (specified)

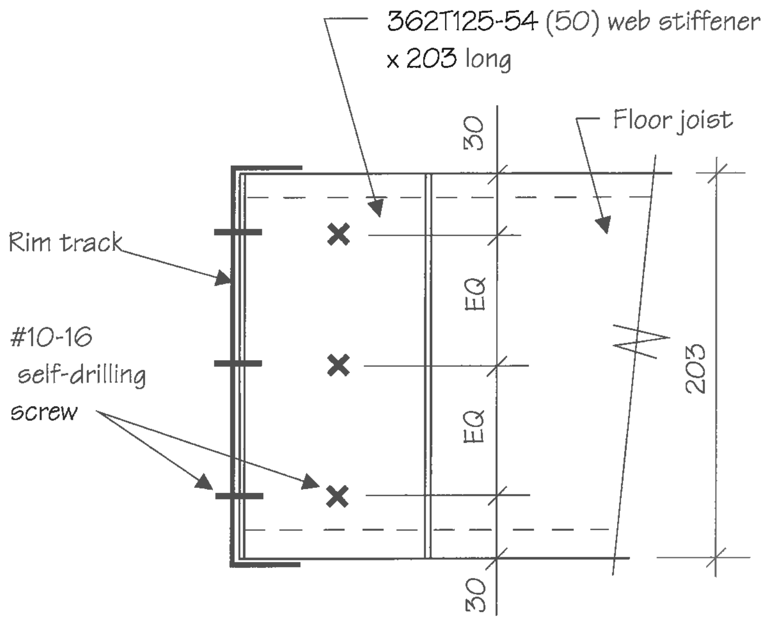


FIGURE 4-6

$P_f = 1.25P_{DL} + 1.5P_{LL} = 1.25(4.78) + 1.5(7.67)$
$= 17.48$ kN/stud

Check web crippling capacity of stiffened joist

By Supplement S136S1-04 *(CSA 2004a)* Section C3.6.2

$P_n = 0.7(P_{wc} + A_e F_y) \geq P_{wc}$

where:

$P_{wc} = P_r/\phi$ = nominal web crippling for unstiffened joist by CAN/CSA-S136-01 Eq. 3.4.1-1 with bearing length = stud depth = 102 mm. Use web crippling coefficients for fastened to support two-flange loading *(Table C3.4.1-2)*

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

where:

R	= 2.157 mm
t	= 1.438 mm
Depth	= 203.2 mm
h	= Depth - 2t - 2R = 196.0 mm
N	= 102 mm
F_y	= 345 MPa

θ = 90 degrees
C = 7.5
C_R = 0.08
C_N = 0.12
C_h = 0.048
ϕ = 0.75

Substituting:

$P_{wc} = P_r/\phi = 4.27$ kN

A_eF_y = stub column strength of stiffener

For 362T125-54 (50) stiffener with F_y = 345 MPa the term A_eF_y is available in AISI 2002b Table III-3
A_eF_y = 49.8 kN

$P_n = 0.7(4.27 + 49.8)$
$= 37.8$ kN

$P_r = \phi P_n = 0.80(37.8)$
$= 30.2 \text{ kN} > 17.48 \text{ kN}$ ***OK***

Note 4-1

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 insure 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 S136S1-04 Section C3.6.2. 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 = 203/8 = 25.4 mm
Use 30 mm – see Figure 4-6.

Note that C3.6.2 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 – Closure Channel

Step 6(a) – Section Size

Use 800T125-54 (50) rim track *(thickness to match thickness of floor joists)*. See Note 4-2.

> *Note 4-2*
>
> *It is common practice to supply rim track with narrow flanges (32 mm 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 LSF 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 3.2.2 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 *(including the CSSBI tables – CSSBI 2004)*.

3. Published load tables *(including the CSSBI tables – CSSBI 2004)* usually assume concentric axial loads and it is common practice to use this assumption in design.

Try 400S162-43 stud with F_y = 230 MPa and bridging spaced at 1200 mm maximum.

For strength limit states the following load combinations apply (snow load is zero in this example):

1.4D
1.25D + 1.5L + 0.4W
1.25D + 0.5L + 1.4W

Step 7(a) – Check Web Crippling

Check web crippling from the CSSBI wind bearing tables *(CSSBI 2004)*. See Notes 1-1 and 1-2.

For a load factor = 1.4
Factored wind w_f = 1.4(1.20) = 1.68 kPa

From CSSBI tables, choose next highest factored wind load = 1.80 kPa *(although not used here interpolation between factored wind loads is acceptable when required)*

No asterisk appears on the strength allowable height, therefore, web crippling does not control.

Step 7(b) – Check Deflection

Check deflection at L/360 from CSSBI wind bearing tables *(CSSBI 2004)*.

Specified wind load $w_s = I_w$ (1.20) = 0.75(1.20) = 0.900 kPa

From CSSBI tables, choose next highest specified wind load = 0.96 kPa *(although not used here interpolation between specified wind loads is acceptable when required)*.

H_{MAX} = 3.44 m > 2.74 m ***OK***

Step 7(c) – Axial Load Capacity

Loads from the stud above plus the floor joist reaction *(from Step 4)*

P_{LL} = 7.67 kN/stud (specified)
P_{DL} = 4.78 kN/stud (specified)

1.4D load case

W_f = 0
C_f = 1.4(4.78) = 6.69 kN/stud

From unsheathed tables for 400S162-43 stud and 0 kPa factored wind

C_r = 19.26 kN > 6.69 kN ***OK***

1.25D + 1.5L + 0.4W load case

W_f = 0.4(1.20) = 0.48 kPa

C_f = 1.25(4.78) + 1.5(7.67) = 17.48 kN/stud

From unsheathed tables for 400S162-43 stud and 0.479 kPa factored wind (no interpolation required for wind):

C_r = 16.15 kN < 17.48 kN ***UNSATISFACTORY***

Try instead 400S162-54 (50) with F_y = 345 MPa

From unsheathed tables for 400S162-54 (50) stud and 0.479 kPa factored wind (no interpolation required for wind):

C_r = 26.6 kN > 17.48 kN ***OK***

1.25D + 0.5L + 1.4W load case

W_f = 1.4(1.20) = 1.68 kPa

C_f = 1.25(4.78) + 0.5(7.67) = 9.81 kN/stud

From unsheathed tables for 400S162-54 (50) stud:

C_r = 21.2 kN @ 1.44 kPa factored wind
= 18.6 kN @ 1.92 kPa factored wind
= 19.9 kN @ 1.68 kPa factored wind by interpolation > 9.81 kN ***OK***

Use 400S162-54 (50) with F_y = 345 MPa

Step 8 – Jamb Studs

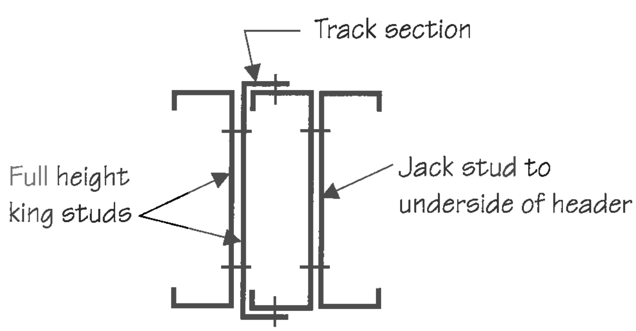


FIGURE 4-7

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 2 studs (1 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 built-up behavior.

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 enclose the LSF floor members over each load bearing wall. See also the discussion on in-line framing in Step 4.
- 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 3.2 mm is permitted by COFS 2004c Section C3.4.4.
- Track may also by 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.

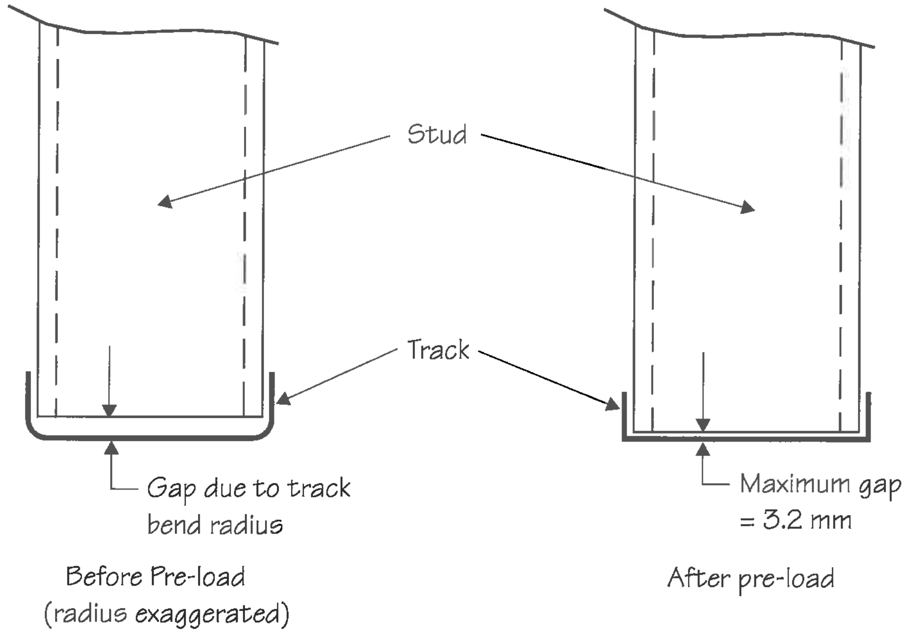


FIGURE 4-8

Check concrete bearing under the bottom track using the approximate method in Appendix F.

Try 400T125-54 (50) track *(Track with F_y = 345 MPa may require a special order.)*

From Step 4
P_{LL} = 7.67 kN/stud (specified)
P_{DL} = 4.78 kN/stud (specified)

For 1.25D + 1.5L
$C_f = 1.25(P_{DL}) + 1.5(P_{LL}) = 1.25(4.78) + 1.5(7.67)$
$= 17.48$ kN/stud

From Appendix F assuming concrete f_c' = 25 MPa

$$x = 0.902 t_t \sqrt{\frac{F_y}{f_c'}} = 0.902(1.438)\sqrt{\frac{345}{25}}$$
$$= 4.82 \text{ mm}$$

$$A_{brg} = (B + 2x)(C + x)(2) + [A - 2(C + x)][t_s + 2x]$$

where:
A = 101.6 mm

$B = 41.3$ mm
$C = 12.7$ mm
$t_s = 1.438$ mm
$x = 4.82$ mm

Substituting
$A_{brg} = 2522\ mm^2$

$B_r = A_{brg} 0.85 \phi_c f_c'$
$= 2522(0.85)(0.65)(25)/1000$
$= 34.8$ kN > 17.48 kN and 400T125-54 (50) track ***OK***

Step 10 – Header

A box header detail is proposed – see Figure 4-10. The Standard for Header Design (*COFS 2004b)* includes special provisions for the design of this member but the design expressions have not been calibrated to Canadian requirements. The design expressions in CAN/CSA-S136-01 have, therefore, been used instead.

The header load condition is shown in Figure 4-9.

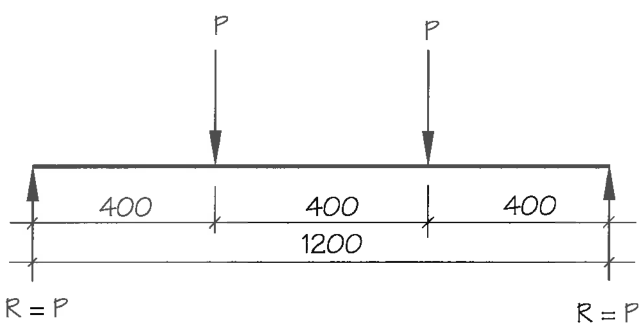


FIGURE 4-9

From Step 4
$P_{LL} = 7.67$ kN/stud (specified)
$P_{DL} = 4.78$ kN/stud (specified)

For the 1.25D + 1.5L load case
$P_f = 1.25PD_L + 1.5P_{LL} = 1.25(4.78) + 1.5(7.67)$
$= 17.48$ kN

Try 2 - 800S162-68 (50) unperforated joist sections *(Fy = 345 MPa)* with 2 - 400T125-54 (50) track sections *(Fy = 345 ksi)*. 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.

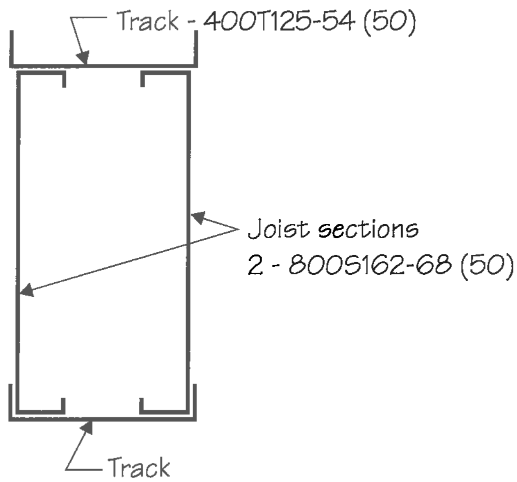


FIGURE 4-10

From the CSSBI tables *(CSSBI 2004)* for 2 - 800S162-68 (50) with F_y = 345 MPa:

M_r = 2(8.45) = 16.90 kN.m
V_r = 2(24.0) = 48.0 kN
P_r = 2(5.16) = 10.32 kN *(@ 25 mm end one flange bearing)*
$I_{x(def)} = 2(2.93 \times 10^6) = 5.86 \times 10^6\ mm^4$
L_u = 810 mm
m = 14.9 mm

Step 10(a) – Moment Capacity (Gravity Loads)

$M_f = 0.400P_f = 0.400(17.48)$
= 6.99 kN.m < 16.90 kN.m ***OK***

Step 10(b) – Interior Web Crippling (Gravity Loads)

Derive the web crippling factored resistance 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 = 41.3 mm. From CAN/CSA-S136-01 Section C3.4 and Table C3.4.1-2.

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

where:

R = 2.717 mm
t = 1.811 mm
Depth = 203.2 mm
h = Depth - 2t - 2R = 194.1 mm
N = 41.3 mm
F_y = 345 MPa
θ = 90 degrees
C = 13
C_R = 0.23
C_N = 0.14
C_h = 0.01
ϕ = 0.80

substituting for 2 sections

P_r = 2(12.64)
= 25.28 kN > 17.48 ***OK***

For this factored web crippling resistance 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 *(Supplement S136S1-04 (CSA 2004a) Section C3.5.2)*

$$0.91\left(\frac{\overline{P}}{P_n}\right)+\left(\frac{\overline{M}}{M_{nxo}}\right)\leq 1.33\,\phi$$

where:
$\overline{P} = P_f = 17.48$ kN
$\overline{M} = M_f = 6.99$ kN.m
$P_n = P_r/\phi = 25.28/0.80 = 31.6$ kN
$M_{nx0} = M_r/\phi = 16.90/0.90 = 18.78$ kN.m
$\phi = 0.75$

Substituting:

$$0.91\left(\frac{17.48}{31.6}\right)+\left(\frac{6.99}{18.78}\right)\leq 1.33(0.75)$$

$0.88 \leq 1.00$ ***OK***

Step 10(d) – Combined Bending and Shear (Gravity Loads)

Check combined bending and shear at the location of load P *(Supplement S136S1-04 (CSA 2004a) Section C3.3.2)*

$$\sqrt{\left(\frac{\overline{M}}{\phi_b M_{nxo}}\right)^2 + \left(\frac{\overline{V}}{\phi_v V_n}\right)^2} \leq 1.00$$

where:
$\overline{M} = M_f = 6.99$ kN.m
$\overline{V} = P_f = 17.48$ kN
$\phi_b M_{nxo} = M_r = 16.90$ kN.m
$\phi_v V_n = V_r = 48.0$ kN

Substituting:

$$\sqrt{\left(\frac{6.99}{16.90}\right)^2 + \left(\frac{17.48}{48.0}\right)^2} \leq 1.00$$

$0.55 \leq 1.00$ ***OK***

Step 10(e) – Deflection (Gravity Loads)

See Figure 4-9.

$$\delta_{LL} = \frac{Pa}{24EI}\left(3L^2 - 4a^2\right)$$

where:
$P = P_{LL} = 7670$ N (specified)
$E = 203000$ MPa
$L = 1200$ mm
$a = 400$ mm

$$\delta_{LL} = \frac{7670(400)}{24(203000)I}\left[3(1200)^2 - 4(400)^2\right]$$

$$= \frac{2.32(10)^6}{I} \text{ mm}$$

for $\delta_{LL} = L/360 = 1200/360 = 3.33$ mm

$I_{req} = 2.32(10)^6/3.33$
$= 0.697 \times 10^6 \text{ mm}^4$

For 2 - 800S162-68 (50)

$I_{x(def)} = 5.86 \times 10^6\ mm^4 > 0.697 \times 10^6\ mm^4$ ***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 the Supplement S136S1-04 *(CSA 2004a)* Section D3.2.2. See also Figure 2-8.

$P_{Lf} = (m/d)P_f$

where:

$P_f = 17.48/2 = 8.74$ kN/joist section

d = 203 mm

m = 14.9 mm

$P_{Lf} = (14.9/203)(8.74)$
$= 0.642$ kN

For #10-16 self-drilling screw in shear use $V_r = 2.49$ kN from Example #2 Step 2(c). *(This factored resistance is based on 2 sheets at t = 1.438 mm and Fu = 450 MPa - shear in the screw itself governs.)*

$V_r = 2.49\ kN > 0.642\ kN$ ***OK***

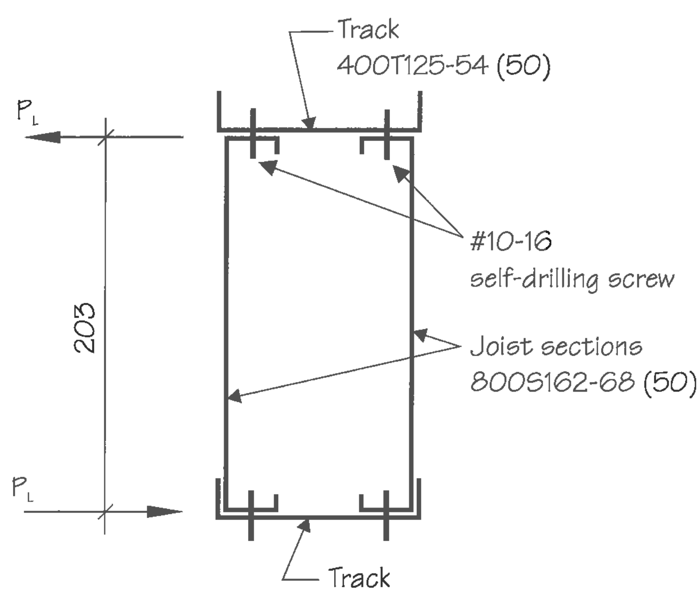


FIGURE 4-11

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 web crippling resistance (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 web crippling resistance 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 load bearing stud above = 41.3 mm. From CAN/CSA-S136-01 Section C3.4 and Table C3.4.1-2.

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

where:

R	= 2.717 mm
t	= 1.811 mm
Depth	= 203.2 mm
h	= Depth - 2t - 2R = 194.1 mm
N	= 41.3 mm
F_y	= 345 MPa
θ	= 90 degrees
C	= 4
C_R	= 0.14
C_N	=0.35
C_h	= 0.02
ϕ	=0.70 (unfastened - conservative)

substituting for 2 - 800S162-68 (50) sections

P_r = 2(5.56)
= 11.12 kN

(For this factored web crippling resistance to be valid, web punchouts are not permitted in the vicinity of the point loads. The header member has therefore been specified as unperforated.)

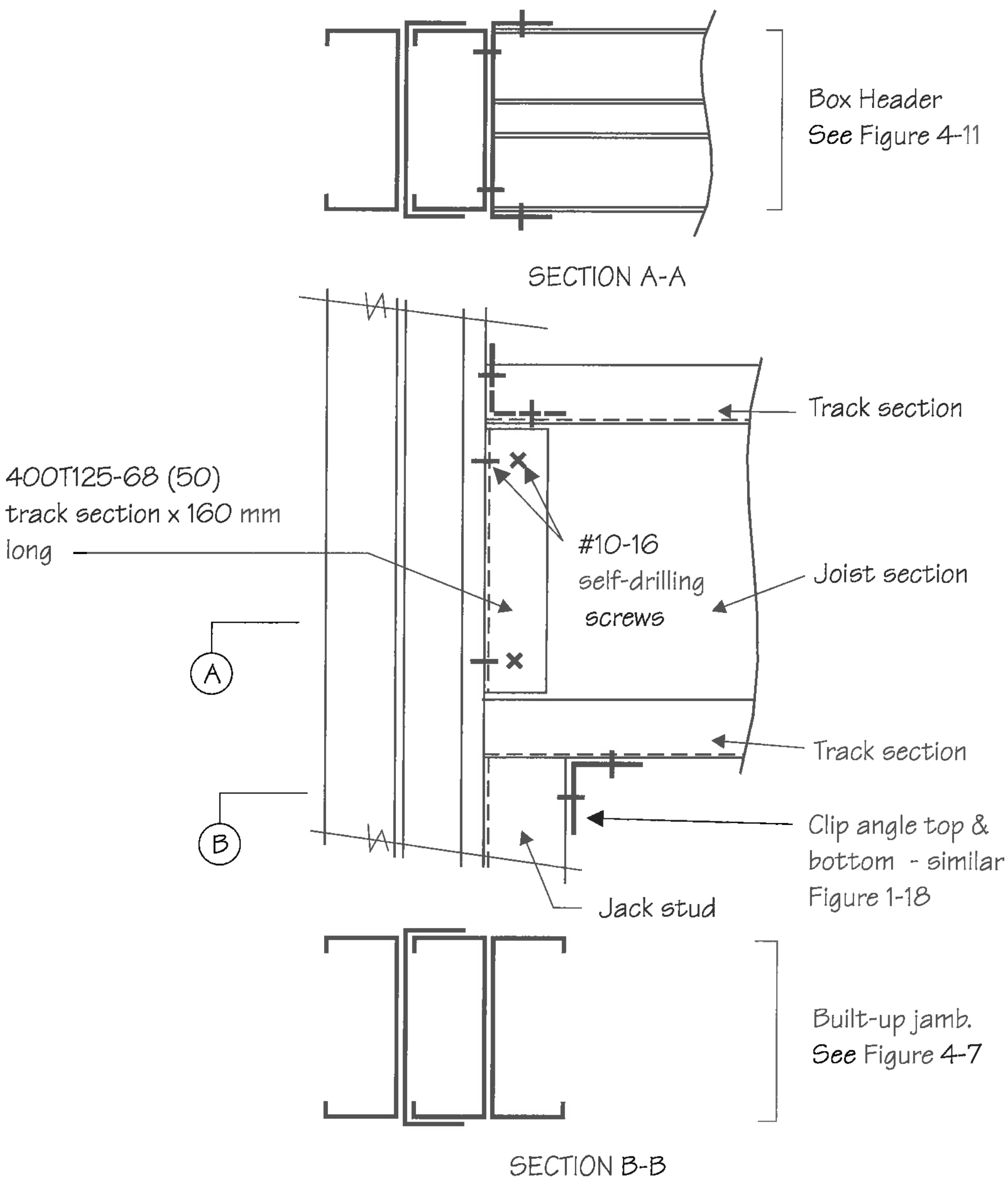


FIGURE 4-12

The web crippling load P_r = 11.12 kN is carried by the jack stud. The capacity of the jack stud is OK by inspection.

The balance of the factored reaction is carried by a shear connection to the king stud. This factored force is given by:

$V_f = 17.48 - 11.12 = 6.36$ kN

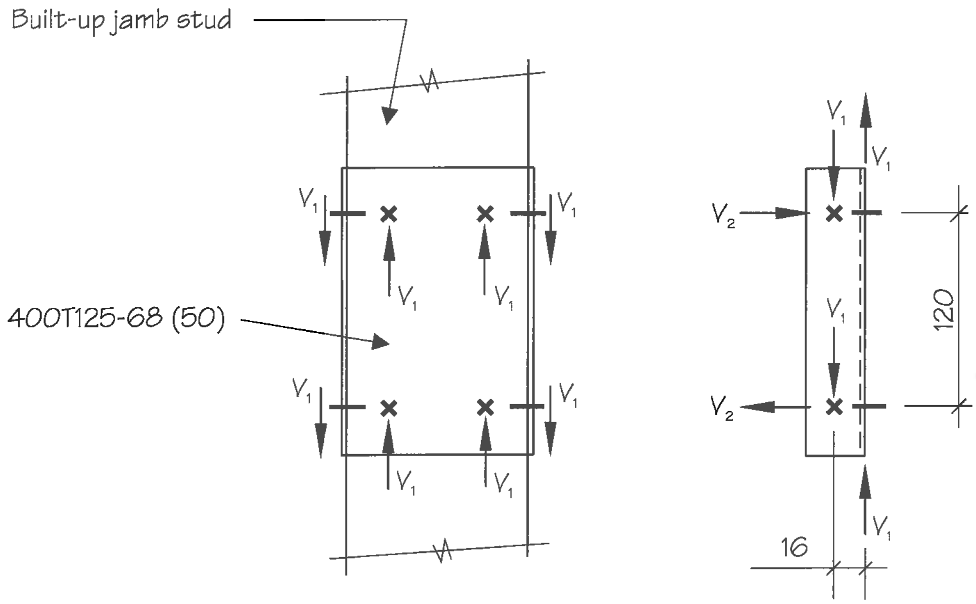


FIGURE 4-13

Provide a short piece of track to act as a shear connector. Use t = 1.811 mm to match header joist section.

Calculate screw forces assuming #10-16 self-drilling screws:

From Figure 4-13:

V_{1f} = 6.36/4 = 1.59 kN

V_{2f} = [6.36(16)/120]/2 = 0.42 kN

Resultant $V_f = \sqrt{P_{1f}^{\,2} + P_{2f}^{\,2}} = \sqrt{1.59^2 + 0.42^2}$
= 1.64 kN/screw

For #10-16 self-drilling screw in shear use V_r = 2.49 kN from Example #2 Step 2(c). *(This factored resistance is based on 2 sheets at t = 1.438 mm and Fu = 450 MPa - shear in the screw itself governs.)*

Gives V_r = 2.49 kN/screw > 1.64 kN **OK**

Note the clip angles connection details at the top and bottom of the box header. 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 – Bridging

Design bridging to resist torsion induced in the studs by wind load (*Supplement S136S1-04 D3.2.2*) and weak axis buckling of the studs.

The torsional effect was previously reviewed in Design Example #2, Step 2.

CAN/CSA-S136-01 is silent on the subject of bracing for singly symmetric columns subject to weak axis and/or torsional-flexural buckling. For purposes of this design example, assume a bracing axial force equal to 2% of the stud axial load. A bracing stiffness requirement is assumed not to apply.

> *Note 4-3*
>
> *This 2% approach to bracing design is based on historical practice and is included in CAN/CSA-S136-01 for the bracing of symmetric columns. More sophisticated approaches including both strength and stiffness requirements are available. See Galambos 1998 and Green 2004b.*

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 CAN/CSA-S136-01 C5.2.2.

Step 11(a) – Applied Loads

i) Bridging axial load

Required bridging axial load = 0.02 x 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 the Supplement S136S1-04 *(CSA 2004a)* Section D3.2.2.

$P_L = 1.5(m/d)W$

where:
a = bridging spacing = 1.2 m
w_s = specified wind load/m
= 0.4(1.2) = 0.48 kN/m
W = aw_s = 1.2(0.48) = 0.576 kN (specified)
m = stud web center line to shear center = 19.2 mm

$d = 101.6$ mm

Substituting:
$P_L = 1.5(19.2/101.6)(0.576)$
$= 0.1633$ kN (specified)

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.

$M = P_L\, d = 163.3(101.6) = 16590$ N.mm (specified) and the resulting moment values in the outside span are illustrated in Figure 4-14.

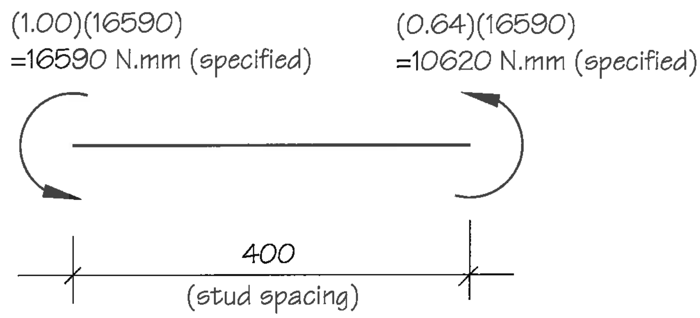


FIGURE 4-14

iii) Bridging Minor Axis Moment, M_y

Bridging minor axis moment is illustrated in Figure 4-15. See Note 4-4.

$M_y = (X_{cg})$(Bridging axial load)
$= (3.21)$(Bridging axial load)

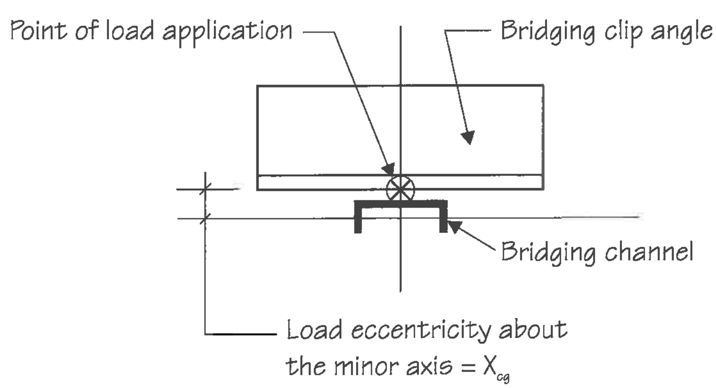


FIGURE 4-15

Note 4-4

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) – Factored Resistances

Use 150U50-54 (50) bridging channel with F_y = 345 MPa. *(Note: Check the availability of Fy=345 MPa before specifying.)* See Figure 4-16.

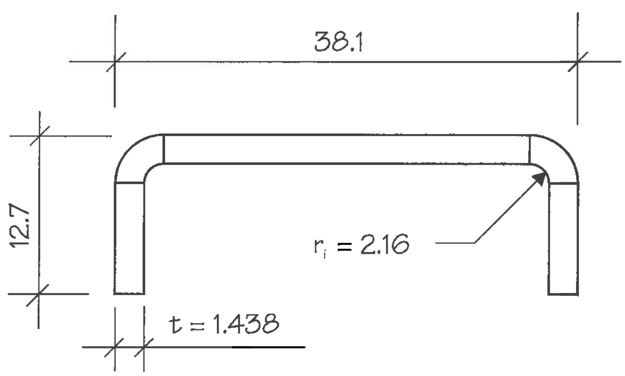


FIGURE 4-16

Design strengths will be checked using the combined compressive axial load and bending provisions in CAN/CSA-S136-01 C5.2.2.

i) The following bridging channel section properties are not available in the CSSBI load tables *(CSSBI 2004)* but have been derived using the formulae in AISI 2002b Part I. *(Note that the section is fully effective at a uniform stress of Fy = 345 MPa - i.e. $\lambda \le 0.673$ for all elements at f = Fy CAN/CSA-S136-01 B2.1 -calculations not shown here.)*

t = 1.438 mm
r_i = 2.16 mm
A = fully effective (unreduced) area = 83.6 mm^2
r_x = 13.9 mm
r_y = 3.69 mm
x_0 = 6.46 mm
$r_0 = \sqrt{r_x^2 + r_y^2 + x_0^2}$ = 15.8 mm
I_x = fully effective (unreduced) inertia = 16200 mm^4
I_y = fully effective (unreduced) inertia = 1140 mm^4

X_{cg} = location of fully effective (unreduced) centroid = 3.21 mm
C_w = 279000 mm^6
J = 57.6 mm^4
j = 20.0 mm

S_{fx} = fully effective (unreduced) major axis section modulus = 852 mm^3

S_{fy} = fully effective (unreduced) major axis section modulus
= I_y /(12.7 - X_{cg})
= 120 mm^3

ii) Nominal axial strength, P_n (*CAN/CSA-S136-01 C4 & C4.2*)

Assume $K_xL_x = K_yL_y = K_tL_t$ = 400 mm

Determine the controlling critical elastic buckling stress, F_e:

$$\sigma_{ey} = \frac{\pi^2 E}{\left(\dfrac{K_y L_y}{r_y}\right)^2} = \frac{\pi^2(203000)}{\left[\dfrac{400}{3.69}\right]^2} = 170.5 \text{ MPa}$$

$$\sigma_{ex} = \frac{\pi^2 E}{\left(\dfrac{K_x L_x}{r_x}\right)^2} = \frac{\pi^2(203000)}{\left[\dfrac{400}{13.9}\right]^2} = 2419 \text{ MPa}$$

$$\sigma_t = \frac{1}{Ar_0^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right]$$
$$= \frac{1}{(83.6)(15.8)^2}\left[78000\,(57.6) + \frac{\pi^2(203000)(279000)}{400^2}\right]$$
$$= 382.7 \text{ MPa}$$

$$\beta = 1-(x_0/r_0)^2$$
$$= 1-(6.46/15.8)^2 = 0.8328$$

From CAN/CSA-S136-01 C4.1, the flexural critical elastic buckling stress is given by:

F_e = the lesser of σ_{ex} or σ_{ey}
= 170.5 MPa

From CAN/CSA-S136-01 C4.2, 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:

$F_e = 371.4$ MPa

$F_e = 170.5$ MPa governs.

From CAN/CSA-S136-01 C4:

$P_n = A_eF_n$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{345}{170.5}} = 1.422$$

For $\lambda_c \leq 1.5$

$$F_n = (0.658)^{\lambda_c^2} F_y = (0.658)^{1.422^2}(345)$$
$$= 148.0 \text{ MPa}$$

$P_n = A_eF_n = (83.6)(148)/1000$ *(No local buckling)*
$= 12.37$ kN

iii) Nominal flexural strength M_{nx}

Check lateral buckling by CAN/CSA-S136-01 C3.1.2.1

From Design Example #2 Step 2(a)

$M_{nx} = F_cS_x = (341.2)(852)/10^6$
$= 0.291$ kN.m

iv) Nominal flexural strength, M_{ny}

Lateral buckling associated with bending about the y-axis can be checked using CAN/CSA-S136-01 C3.1.2.1 with the critical elastic stress defined by Equation C3.1.2.1-6.

This expression applies to bending about the centroidal axis perpendicular to the symmetry axis. For typical LSF members, it is the weaker axis by a significant margin, lateral buckling does not occur and $F_c = F_y$.

That is,
$M_{ny} = S_{fy}F_y = 120(345)/10^6$ *(No local buckling)*
$= 0.0414$ kN.m

v) Nominal axial strength, P_{no}

$P_{no} = A_eF_y = 83.6(345)/1000$ *(No local buckling)*
$= 28.84$ kN

vi) P_{Ex} and P_{Ey}

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} = \frac{\pi^2(203000)(16200)}{(400)^2(1000)}$$

$= 202.9$ kN

$$P_{Ex} = \frac{\pi^2 EI_y}{(K_y L_y)^2} = \frac{\pi^2(203000)(1140)}{(400)^2(1000)}$$

$= 14.28$ kN

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 one end and an eccentric axial load the other with $e_y = X_{cg}$. This gives:

$C_{my} = 0.6$

Summarizing:

$P_n = 12.37$ kN
$M_{nx} = 0.291$ kN.m
$M_{ny} = 0.0414$ kN.m
$P_{no} = 28.8$ kN
$P_{Ex} = 203$ kN
$P_{Ey} = 14.3$ kN
$C_{mx} = 0.344$
$C_{my} = 0.6$

$\phi_c = 0.80$
$\phi_b = 0.90$

Step 11(c) – Interaction Checks

By CAN/CSA-S136-01 C5.2.2

Interaction Equation #1 (Eq. C5.2.2-1)

$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\left(1-\dfrac{\overline{P}}{P_{Ex}}\right)} + \frac{C_{my}\overline{M}_y}{\phi_b M_{ny}\left(1-\dfrac{\overline{P}}{P_{Ey}}\right)} \le 1.00$$

Substituting design strengths from Step 11(b)

$$\frac{C_f}{0.80(12.37)} + \frac{(0.344)M_{fx}}{0.90(0.291)\left(1-\dfrac{C_f}{203}\right)} + \frac{(0.600)M_{fy}}{0.90(0.0414)\left(1-\dfrac{C_f}{14.3}\right)} \le 1.00$$

Interaction Equation #2 (Eq. C5.2.2-2)

$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \le 1.00$$

Substituting design strengths from Step 11(b)

$$\frac{C_f}{0.80(28.8)} + \frac{M_{fx}}{0.90(0.291)} + \frac{M_{fy}}{0.90(0.0414)} \le 1.00$$

Try anchoring bridging every 16 studs.

Factored bridging loads are derived as follows:

C_f = (0.02 x Factored Axial Load per Stud x n) kN

where:

Factored Axial Load per Stud is taken from the appropriate load case in Step 7(c)

n = number of studs to be braced

0.02 = 2% bracing force factor from Step 11(a)i

M_{fx} = (0.01659 x Load Factor for Wind) kN.m

where:

0.01659 is taken from Step 11(a)ii

$M_{fy} = 0.00321C_f$ kN.m

where:

0.00321 it taken from Step 11(a)iii

<u>Load Case I</u> 1.25D + 1.5L + 0.4W

C_f/stud = 17.48 kN (from Step 7(c))

C_f for bridging channel with n = 16
= 17.48(0.02)(16)
= 5.59 kN

M_{fx} = 0.01659(0.4)
= 0.00664 kN.m

M_{fy} = 0.00321(5.59)
= 0.0179 kN.m

Substituting in Interaction Equation #1

0.565 + 0.009 + 0.473 = 1.047 ≈ 1.00 ***OK***
(The 4.7% overstress is considered acceptable here considering the conservative approximations in the design procedure – primarily the 2% bracing force factor.)

Substituting in Interaction Equation #2

0.243 + 0.025 + 0.480 = 0.748 < 1.00 ***OK***

<u>Load Case II</u> 1.25D + 0.5L + 1.4W

C_f/stud = 9.81 kN (from Step 7(c))

C_f for bridging channel with n = 16
= (9.81)(0.02)(16)
= 3.14 kN

M_{fx} = 0.01659(1.4)
= 0.0232 kN.m

M_{fy} = 0.00321(3.14)
= 0.0101 kN.m

Substituting in Interaction Equation #1

0.317 + 0.031 + 0.208 = 0.556 < 1.00 ***OK***

Substituting in Interaction Equation #2

0.136 + 0.089 + 0.271 = 0.496 < 1.00 ***OK***

Therefore, from Load Cases #1 and #2 interaction checks, anchoring bridging every 16 studs is ***OK***. See Note 4-5.

Note 4-5

1. *Flat strap tension bridging (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 8 stud spaces away from an anchorage location.*

Step 12 – Bridging Anchorage

From Step 11(c), the bridging must be anchored every 16 studs. See Figure 4-17 for a suggested anchorage detail using flat strap X-bracing.

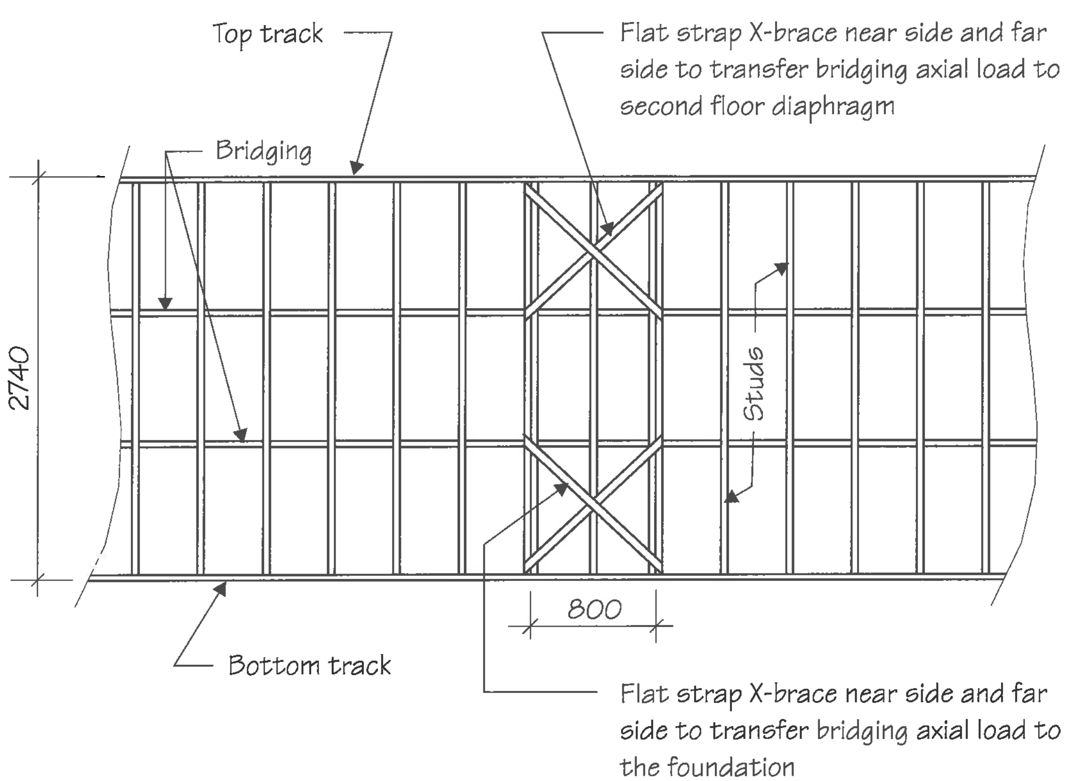


FIGURE 4-17

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.

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 at the maximum permissible 1200 mm.

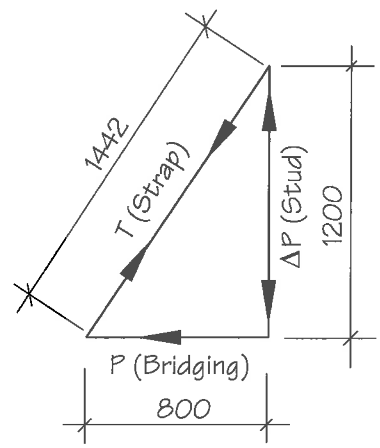


FIGURE 4-18

From Step 11(c) the maximum axial load in the bridging channel is given by Load Case I (1.25D + 1.5L + 0.4W)

C_f = 5.59 kN (n = 16)

For X-bracing straps on both sides of the studs

T_f /strap = 5.59(1442/800)/2
= 5.04 kN

Try 38 x 1.146 mm flat strap with F_y = 230 MPa.

i) X-bracing vertical reaction

The vertical component of force in the flat straps increases the stud axial load *(for the studs with strapping attached)*:

The vertical component from two levels of straps is given by:

ΔC_f = 5.59(2)(1200/800)
= 16.77 kN

The back-to back studs illustrated in Figure 4-19 are adequate by inspection.

ii) Number of screws required to connect flat straps

For #10-16 screws:
d = 4.83 mm *(Appendix A Tables A-2)*

For detailing screw locations assume the following distances:

- end distance = 3d = 3(4.83) = 14.50 mm
- minimum centre to centre spacing = 3d = 3(4.83) = 14.50 mm
- minimum edge distance = 1.5d = 1.5(4.83) = 7.25 mm

Screw design input values:

Strap	t_1 = 1.146 mm	F_{u1} = 310 MPa
Stud	t_2 = 1.438 mm	F_{u2} = 450 MPa
Screw	Size = #10-16	d = 4.83 mm *(Appendix A Table A-2)*

Screw Shear Resistance

Screw shear resistance limited by E4.3.1 tilting and bearing

t_2/t_1 = 1.438/1.146 = 1.255

From CAN/CSA-S136-01 Equations E4.3.1-1, E4.3.1-2 and E4.3.1-3.

$$P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2} = 7163 \text{ N} - \text{governs}$$
$$P_{ns} = 2.7 t_1 d F_{u1} = 4633 \text{ N}$$
$$P_{ns} = 2.7 t_2 d F_{u2} = 8439 \text{ N}$$

$P_{ns} = 2.7t_1dF_{u1}$ governs for $t_2 / t_1 \leq 1.0$ and also for $t_2 / t_1 \geq 2.5$. Therefore, no interpolation for t_2 / t_1 = 1.255 is required.

Gives
$V_r = \phi\, P_{ns}$ = (0.40)(4.63)
= 1.85 kN

Screw shear resistance limited by E4.3.3 shear in the screws themselves. Refer to the Supplement S136S1-04 *(CSA 2004a)*

$P_{ns} = P_{ss}$

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

$V_r = \phi P_{ns} = \phi P_{ss}$ = 0.40(6.23)
= 2.49 kN

The governing V_r is from E4.3.1 and is given by

$V_r = 1.85$ kN

Number of screws required each end of each strap = 5.04/1.85 = 2.7

Use 4 screws = 2 for each stud. See Figure 4-19.

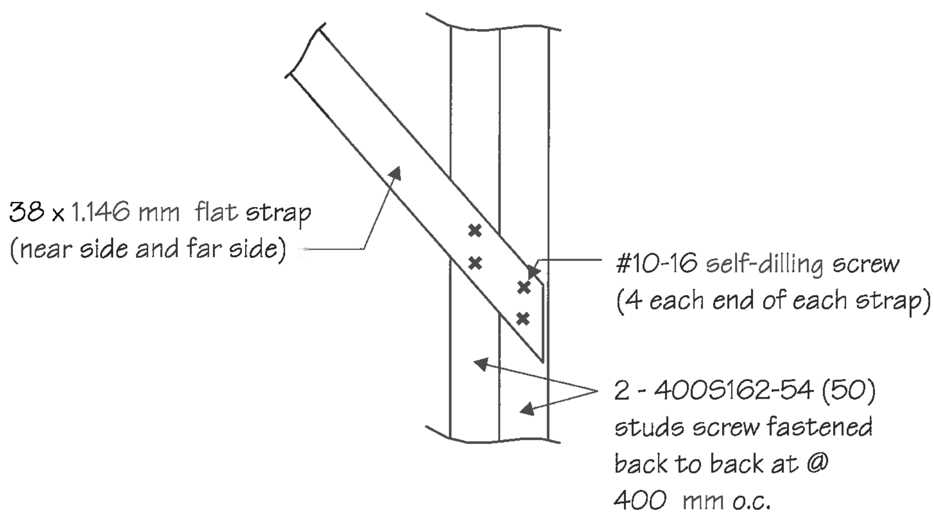


FIGURE 4-19

iii) Flat strap size

Try 38 x 1.146 mm flat strap with $F_y = 230$ MPa.

T_f /strap = 5.04 kN

Check yielding of the gross section CAN/CSA-S136-01 Section C2.1

$$T_r = \phi_t A_g F_y = 0.9(38)(1.146)(230)$$
$$= 9010 \text{ N}$$

Check fracture of the net section CAN/CSA-S136-01 Section C2.2 assuming two screws are aligned across the width of the strap.

$$T_r = \phi_u A_n F_u$$

where:
$A_n = L_c t = L_t t$

$T_r = 0.75[38 - 2(4.83)](1.146)(310)$
$= 7550$ N

Check block tear out of all 4 screws – ***OK*** by inspection

iv) X-bracing horizontal reaction

The top and bottom tracks and their connections to the floor diaphragms resist the horizontal component of force in the flat straps – not checked here.

Step 12(b) – Bridging Clip Angle at Double Stud Anchorage Point

In axial load bearing construction to insure a stiff connection detail, size clip angles as per Note 2-5 except that it is recommended that the thickness of the bridging clip angle be the greater of 1.438 mm or one thickness heavier than the thickness of the stud.

For clip angle with 400S162-54 stud use t = 1.811 mm.

i) Connection of bridging channel to bridging clip angles at anchorage point.

See Figure 4-20.

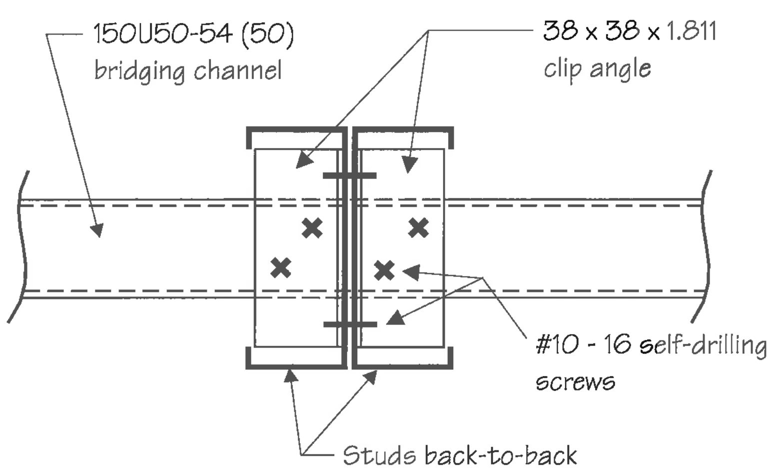


FIGURE 4-20

From Step 11(c) the maximum axial load in the bridging channel is given by Load Case I (1.25D + 1.5L + 0.4W)

$C_f = 5.59$ kN (n = 16)

From Design Example #2 Step 2(c), use:

V_r/screw = 2.49 kN *(This factored resistance is based on 2 sheets at t = 1.438 mm and Fu = 450 MPa - shear in the screw itself governs.)*

Required number of screws
= 5.59/2.49 = 2.2

Use 4 screws total – 2 per clip angle.

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 angles, then to the back-to-back studs and finally to the flat strap X-bracing.

The load transfer between the clip angles and the back-to-back studs is as follows:

- On one side the clip angle pushes against the back-to back studs and the load transfer is via bearing.
- On the other side the clip angle pulls away from the back-to back studs and the load transfer is via the self-drilling screws in tension.

The force transferred in bearing is assumed to be limited by the shear capacity of the screws connecting the bridging channel to the angle:
= 2(V_r/screw) = 2(2.49)
= 4.98 kN

The remaining factored load to be transferred through the other clip angle and the screws in tension is:
= 5.59 - 4.98
= 0.61 kN

and the factored load per screw
= 0.61/2 = 0.305 kN which is ***OK*** by inspection.

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.

Connection detail is similar to Figure 2-12.

For the 400S162-54 typical stud from Step 11(a)ii, the twisting moment applied at each line of bridging due the accumulated wind load torsion in the stud is given as:
= 16590 N.mm (specified)

Step 13(a) – Bridging Channel to Bridging Clip Angle Screws

From Design Example #2 Step 2(c) and Figure 2-12 the spacing between the screws is given by:
x = 27.9 mm

and the shear per screw
V = 16590/27.9
= 595 N/screw (specified - due to wind)

Load Case I   1.25D + 1.5L + 0.4W

C_f/stud = 17.48 kN (from Step 7(c))

Portion to bridging channel = 0.02(17.48)(1000) = 350 N

V_f/screw due to axial effects
= 350/2 = 175 N

V_f/screw due to wind effects (0.4 load factor)
= 595(0.4) = 238 N

V_f/screw total = 175 + 238 = 413 N
= 0.413 kN

(The V_f/screw total conservatively assumes the two contributing components are directly additive even though they act in different directions.)

Load Case II   1.25D + 0.5L + 1.4W

C_f/stud = 9.81 kN (from Step 7(c))

Portion to bridging channel = 0.02(9.81)(1000) = 196 N

V_f/screw due to axial effects
= 196/2 = 98 N

V_f/screw due to wind effects (1.4 load factor)
= 595(1.4) = 833 N

V_f/screw total = 98 + 833 = 931 N
= 0.931 kN

V_f/screw = 0.931 kN governs

From Design Example #2 Step 2(c), use:

V_r/screw = 2.49 kN *(This factored resistance is based on 2 sheets at t = 1.438 mm and Fu = 450 MPa - shear in the screw itself governs.)*

V_r = 2.49 kN > 0.931 kN ***OK***

Step 13(b) – Bridging Clip Angle to Stud Screws

Similar to Design Example #2 Step 2(c) and Figure 2-12 the spacing between the screws is 50 mm *(reduced from 100 mm in Figure 2-12).*

and the tension force per screw
T = 16590/50
= 332 N/screw (specified - due to wind)

Load Case I 1.25D + 1.5L + 0.4W

C_f/stud = 17.48 kN (from Step 7(c))

Portion to bridging channel = 0.02(17.48)(1000) = 350 N

T_f/screw due to axial effects
= 350/2 = 175 N

T_f/screw due to wind effects (0.4 load factor)
= 332(0.4) = 133 N

T_f/screw total = 175 + 133 = 308 N
= 0.308 kN

Load Case II 1.25D + 0.5L + 1.4W

C_f/stud = 9.81 kN (from Step 7(c))

Portion to bridging channel = 0.02(9.81)(1000) = 196 N

T_f/screw due to axial effects
= 196/2 = 98 N

T_f/screw due to wind effects (1.4 load factor)
= 332(1.4) = 465 N

V_f/screw total = 98 + 465 = 563 N
= 0.563 kN

T_f/screw = 0.563 kN governs

From Design Example #2 Step 2(c), conservatively use:

T_r/screw = 0.584 kN *(0.584 kN/screw is based on t = 1.438 mm angle and t= 1.146 mm stud)*

T_r = 0.584 kN > 0.563 kN ***OK***

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Appendix A

Design Values for Self-Drilling Screws and Welds

There are a variety of acceptable fasteners for connecting LSF 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 *CAN/CSA-S136-01* Sections E2.4 and E2.5. 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 CAN/CSA-S136-01):*

- The factored resistance of all fillet and flare-bevel groove welds irrespective of the length to thickness ratio or direction of loading is set equal to $0.75\phi tLF_u$ with $\phi = 0.40$. 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 $t > 2.54$ mm, the factored resistance determined above shall not exceed $0.75\phi t_w LF_{xx}$ with $\phi = 0.50$.

- For welded connections in which the thickness of the thinnest connected part is greater than 4.76 mm, reference is made to CSA Standard W59-03, " Welded Steel Construction (Metal Arc Welding)." See *CAN/CSA-S136-01* Section E2a for additional limitations.

In the design examples, the drawings show a nominal weld size of 3 mm. Where this approach is used on engineering drawings, it should be accompanied by a note: "For material less than or equal to 2.54 mm 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 are calculated in accordance with the requirements of *CAN/CSA-S136-01* E4. The relevant sub-sections are as follows:

- Section E4.3.1 – tilting and bearing failure modes
- Section E4.3.2 – connection shear limited by end distance
- Section E4.3.3 – 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 C2 – tension members

It is assumed that pull-over, Section E4.4.2, does not govern for typical CFSF 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 (kN)	Ultimate Screw Shear Strength (kN)
8 - 18	6.87	4.45
10 - 16	8.61	6.23
10 - 24	12.02	6.67
12 - 14	12.36	8.90
12 - 24	13.43	9.34
1/4 - 14	18.06	11.57

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 appropropriate 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. *CAN/CSA-S136-01allows the use of test values in lieu of the design expressions in Section E4.*

In addition, the design of screwed connections require the nominal hole diameter. Appropriate values for design are provided in Table A-2 *(taken from reference CSA 2001b).*

Table A – 2
Nominal Diameters for Screws

Number Designation for Screw	Nominal Diameter (mm)
6	3.51
8	4.17
10	4.83
12	5.49
1/4	6.35

Appendix B
Anchor Design Values

There are a variety of acceptable fasteners for connecting LSF members to either concrete or steel structures. This appendix provides design data for three types of anchors: wedge type expansion anchors (concrete), self-tapping concrete screw anchors (concrete) and low velocity pins (concrete & steel).

B.1 Wedge Type Expansion Anchors

The design values in this appendix have been taken from Hilti Kwik Bolt 3 – 2005 Product Technical Guide Supplement. The data is specific to carbon steel Kwik Bolt 3 fasteners by Hilti Inc. and will not be appropriate for other anchor types or anchors of similar type by other manufacturers. Other anchor 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 2005 Product Technical Guide and the Kwik Bolt 3 Supplement for additional information such as detail on the fasteners themselves, installation, other concrete types, other fastener diameters, minimum base material thickness and limits of applicability for the design values.

Table B.1-1
Carbon Steel Kwik Bolt-II Ultimate Tension and Shear Values in Normal Weight Concrete

Anchor Diameter (mm)	Embedment Depth (mm)	f_c' = 20.7 MPa		f_c' = 27.6 MPa		f_c' = 41.4 MPa	
		Tension (kN)	Shear (kN)	Tension (kN)	Shear (kN)	Tension (kN)	Shear (kN)
6.4	29	6.1	8.9	7.2	8.9	9.2	8.9
	51	12.0	8.9	13.3	8.9	14.1	8.9
	76	13.3	8.9	14.0	8.9	14.1	8.9
9.5	41	15.2	21.3	18.2	21.9*	18.2	21.9
	64	25.9	21.9	30.8	21.9	34.4	21.9
	89	29.6	21.9	32.8	21.9	35.8	21.9
12.7	57	23.8	36.8*	27.0	36.8*	33.0	36.8
	89	36.5	36.8	40.7	36.8	54.0	36.8
	121	39.3	36.8	42.9	36.8	60.4	36.8

Table B.1-2
Reduction Factors

Description	$h_{min} \le h_{act} \le h_{nom}$	$h_{act} > h_{nom}$
Spacing Tension	$f_{AN} = \dfrac{\frac{s}{h_{act}} + 0.88}{3.13}$ $s_{crit} = 2.25 h_{act}$ $s_{min} = h_{act}$	$f_{AN} = \dfrac{\frac{s}{h_{nom}} + 0.88}{3.13}$ $s_{crit} = 2.25 h_{nom}$ $s_{min} = h_{nom}$
Spacing Shear	$f_{AV} = \dfrac{\frac{s}{h_{act}} + 10.25}{12.5}$ $s_{crit} = 2.25 h_{act}$ $s_{min} = h_{act}$	$f_{AV} = \dfrac{\frac{s}{h_{nom}} + 10.25}{12.5}$ $s_{crit} = 2.25 h_{nom}$ $s_{min} = h_{nom}$
Edge Distance Tension	$f_{RN} = \dfrac{\frac{c}{h_{act}} + 2.00}{3.75}$ $c_{crit} = 1.75 h_{act}$ $c_{min} = h_{act}$	$f_{RN} = \dfrac{\frac{c}{h_{nom}} + 2.00}{3.75}$ $c_{crit} = 1.75 h_{nom}$ $c_{min} = h_{nom}$
Edge Distance Shear	**Direction of Force**	**$h_{act} \ge h_{min}$**
	Perpendicular toward edge	$f_{RV1} = \dfrac{c}{3h_{min}}$ $c_{crit} = 3 h_{min}$ $c_{min} = 1.5 h_{min}$
	Parallel to edge	$f_{RV2} = \dfrac{\frac{c}{h_{min}} + 0.75}{3.75}$ $c_{crit} = 3 h_{min}$ $c_{min} = 1.5 h_{min}$
	Perpendicular away from edge	$f_{RV3} = \dfrac{\frac{c}{h_{min}} + 5.82}{8.82}$ $c_{crit} = 3 h_{min}$ $c_{min} = 1.5 h_{min}$

Table B.1-3
Standard Anchor Embedments

Anchor Diameter (mm)	h_{min} (mm)	h_{nom} (mm)	h_{deep} (mm)
6.4	29	51	76
9.5	41	64	89
12.7	57	89	121

Notes B.1

These note apply to Tables B.1-1, B.1-2 and B.1-3

1. Terminology:

f_{AN} = anchor spacing reduction factor for tension loading
f_{AV} = anchor spacing reduction factor for shear loading
f_{RN} = edge distance reduction factor for tension loading
f_{RV1} = edge distance reduction factor for shear loading towards edge
f_{RV2} = edge distance reduction factor for shear loading parallel to edge
f_{RV3} = edge distance reduction factor for shear loading away from edge
s = actual anchor spacing distance (mm)
c = actual edge distance (mm)
h_{min} = minimum embedment depth (mm)
h_{nom} = standard (nominal) embedment depth (mm)
h_{act} = actual embedment depth (mm)
s_{crit} = critical anchor spacing (no reduction if $s \geq s_{crit}$)
s_{min} = minimum anchor spacing
c_{crit} = critical edge distance (no reduction if $c \geq c_{crit}$)
c_{min} = minimum edge distance

2. From CSA Standard A23.3-94, Design of Concrete Structures, Clause D7.1, the factored resistance is given by:

$$R_r = \phi_e \bar{x} / 1.33$$

where :

ϕ_e = resistance factor for expansion anchors = 0.4

$\bar{x}$ = mean ultimate strength from tests

3. For combined tension and shear (*modified to accommodate Limit States Design terminology)*:

$$\left(\frac{T_f}{T_r}\right)^{5/3} + \left(\frac{V_f}{V_r}\right)^{5/3} \leq 1.0$$

where:
T_f = factored tension load
V_f = factored shear load
T_r = factored tension resistance
V_r = factored shear resistance

4. Shear values are reduced, where required, for the effect of fastener threads in the shear plane. No further reduction is therefore required. Capacities in Table B.1-1 marked with an asterisk have been reduced to be consistent with neighboring values.

5. Intermediate values for other concrete strengths and embedments can be calculated by linear interpolation.

6. Reduction factors are multiplied when considering simultaneous reductions due to anchor spacing and edge distance.

7. Uncracked concrete is assumed.

B.2 Self-Tapping Concrete Screw Anchors

The design values in this appendix have been taken from the ITW Ramset/Red Head website. These values, in turn, were derived from the ICBO Evaluation Report No. ER-3370 (Re-issued July 1, 2001) by multiplying the allowable values by a factor of safety of 4. The design values are for Tapcon concrete screw anchors and will not be appropriate for other anchor types or anchors of similar type by other manufacturers. Other screw anchor 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 manufacturer for additional information such as detail on the fasteners themselves, installation, inspection, other embedment depths, other concrete types, other fastener types, identification requirements and limits of applicability for the design values.

Table B.2-1
Tapcon Concrete Anchors Ultimate Tension and Shear Values
Normal Weight Concrete

Anchor Diameter (mm)	Embedment Depth (mm)	f_c' = 20.7MPa		f_c' = 27.6 MPa		f_c' = 34.5 MPa	
		Tension (kN)	Shear (kN)	Tension (kN)	Shear (kN)	Tension (kN)	Shear (kN)
4.8	25	1.4	3.7	1.8	3.7	2.0	3.8
	32	2.5	3.7	2.7	3.7	2.8	3.8
	38	3.7	3.8	3.9	3.9	4.1	4.0
	44	4.6	4.5	5.0	4.5	5.5	4.5
6.4	25	3.2	6.5	3.6	6.9	4.1	7.4
	32	5.0	7.1	5.5	7.4	6.2	7.7
	38	6.8	7.2	7.1	7.5	7.5	7.8
	44	8.2	9.5	9.1	9.6	10.0	9.8

Notes B.2

These notes apply to Table B.2-1

1. From CSA Standard A23.3-94, Design of Concrete Structures, Clause D7.1, the factored resistance is given by:

$$R_r = \phi_e \bar{x} / 1.33$$

where:

ϕ_e = resistance factor for expansion anchors = 0.4

$\bar{x}$ = mean ultimate strength from tests

The expression for factored resistance is intended for expansion anchors and is assumed to apply to concrete screw anchors. This is justified by the historic use of similar factors of safety for these two anchor types. *(Source: ICC-ES Acceptance Criteria for screw anchors, AC106, and expansion anchors, AC01)*

2. For 100% anchor efficiency, space anchors at a minimum of 12 diameters on center with a minimum edge distance of 10 diameters.

3. Spacing and edge distance may be reduced to 6 diameter spacing and 5 diameter edge distance providing values (shear and tension) are reduced 50%. Linear interpolation may be used for intermediate spacings and edge distances. *(Source: ICBO Evaluation Report No. ER-3370)*

4. Linear interpolation for concrete strengths is assumed to be acceptable.

5. For combined tension and shear *(Source: ICBO Evaluation Report No. ER-3370 modified to accommodate Limit States Design terminology)*

$$\left(\frac{T_f}{T_r}\right)+\left(\frac{V_f}{V_r}\right)\leq 1.0$$

where:
T_f = factored tension load
V_f = factored shear load
T_r = factored tension resistance
V_r = factored shear resistance

6. Uncracked concrete is assumed.

B.3 Powder Actuated Fasteners into Concrete

The design values in this appendix have been taken from the Hilti North America Product Technical Guide, 2005 Edition. The published allowable strengths were translated to ultimate strengths by multiplying by a factor of safety of 5. The design values are for Hilti X-DNI Universal Nail powder actuated fasteners (PAF) and will not be appropriate for other PAF types or PAF's 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 manufacturer 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
Powder Actuated Fasteners Ultimate Tension and Shear Values Normal Weight Concrete

Anchor Diameter (mm)	Embedment Depth (mm)	f_c' = 13.8 MPa		f_c' = 27.6 MPa		f_c' = 41.4 MPa	
		Tension (kN)	Shear (kN)	Tension (kN)	Shear (kN)	Tension (kN)	Shear (kN)
3.68 (shank) 8.18 (head)	19	1.3	2.3	2.4	2.7	–	–
	25	3.2	4.1	3.6	5.3	2.2	2.8
	32	3.6	6.4	5.1	7.3	4.4	5.6
	38	4.9	7.3	7.1	9.5	–	–

Notes B.3

These notes apply to Table B.3-1

1. The expression for factored resistance has been taken from CSA Standard A23.3-94, Design of Concrete Structures, Clause D7.1, with a modified factored resistance.

$$R_r = \phi_e \bar{x} / 1.33$$

where:

ϕ_e = resistance factor modified for powder actuated fasteners = 0.32

$\bar{x}$ = mean ultimate strength from tests

The expression for factored resistance is intended for expansion anchors and is assumed to apply to powder actuated fasteners with the phi factor modified. The modified phi factor is given by 0.4 x 4/5 = 0.32 where:

$0.4 = \phi_e$ for expansion anchors
4/5 = (safety factor for expansion anchors)/(safety factor for powder actuated fasteners) *(Source: ICC-ES Acceptance Criteria for expansion anchors, AC01, and power actuated fasteners , AC70)*

2. Minimum edge distance and centre to centre spacing for 3.68 mm diameter fasteners = 60 mm.

3. For combined tension and shear *(Source: ICBO Acceptance Criteria for Power-Driven Fasteners in Concrete, Steel and Masonry Elements, A70 – modified to accommodate Limit States Design terminology)*:

$$\left(\frac{T_f}{T_r}\right)+\left(\frac{V_f}{V_r}\right)\leq 1.0$$

where:
T_f = factored tension load
V_f = factored shear load
T_r = factored tension resistance
V_r = factored shear resistance

4. Uncracked concrete is assumed

B.4 Powder Actuated Fasteners into Steel

The design values in this appendix have been taken from the Hilti North America Product Technical Guide, 2005 Edition. The published allowable strengths were translated to ultimate strengths by multiplying by a factor of safety of 5. The design values are for Hilti X-EDNI Universal Knurled Shank powder actuated fasteners (PAF) and will not be appropriate for other PAF types or PAF's 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 manufacturer 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.4-1
Powder Actuated Fasteners Ultimate Tension and Shear Values
Steel ($F_y \geq 248$ MPa $F_u \geq 400$ MPa)

Anchor Diameter (mm)	Steel Thickness (mm)	Tension (kN)	Shear (kN)
3.68 (shank) 8.18 (head)	3.18	2.1	5.2
	4.76	5.0	12.3
	6.35	15.0	13.8
	9.53	16.7	14.7
	12.70	14.1	13.5
	> 19.05	5.8	8.6

Notes B.4

These notes apply to Table B.4-1

1. The expression for factored resistance has been taken from Note 1 Table B.3-1.

$$R_r = \phi_e \bar{x} / 1.33$$

where:

ϕ_e = resistance factor modified for powder actuated fasteners = 0.32

$\bar{x}$ = mean ultimate strength from tests

The expression for factored resistance is intended for powder actuated fasteners into concrete and is assumed to apply to powder actuated fasteners into steel. This is justified by the historic use of similar factors of safety for these fasteners into concrete and steel. *(Source: ICC-ES Acceptance Criteria for Power-Driven Fasteners in Concrete Steel and Masonry Elements, AC70)*

2. For 3.68 mm diameter fasteners, minimum edge distance = 12 mm and minimum centre to centre spacing = 25 mm. Minimum base steel thickness = 3.2 mm.

3. X-EDNI fasteners installed into 19.1 mm or thicker steel require minimum 11.1 mm penetration. For thinner base steel, the fasteners shall be driven to where the point of the fastener penetrates in accordance with Section 3.1.3.3 of the Hilti North America Product technical Guide, 2005 Edition.

4. For combined tension and shear *(Source: ICBO Acceptance Criteria for Power-Driven Fasteners in Concrete, Steel and Masonry Elements, A70 – modified to accommodate Limit States Design terminology)*:

$$\left(\frac{T_f}{T_r}\right)+\left(\frac{V_f}{V_r}\right)\leq 1.0$$

where:
T_f = factored tension load
V_f = factored shear load
T_r = factored tension resistance
V_r = factored shear resistance

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 midspan *(of a single span beam)* and considering the displacement of the midspan 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 βh/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.)*

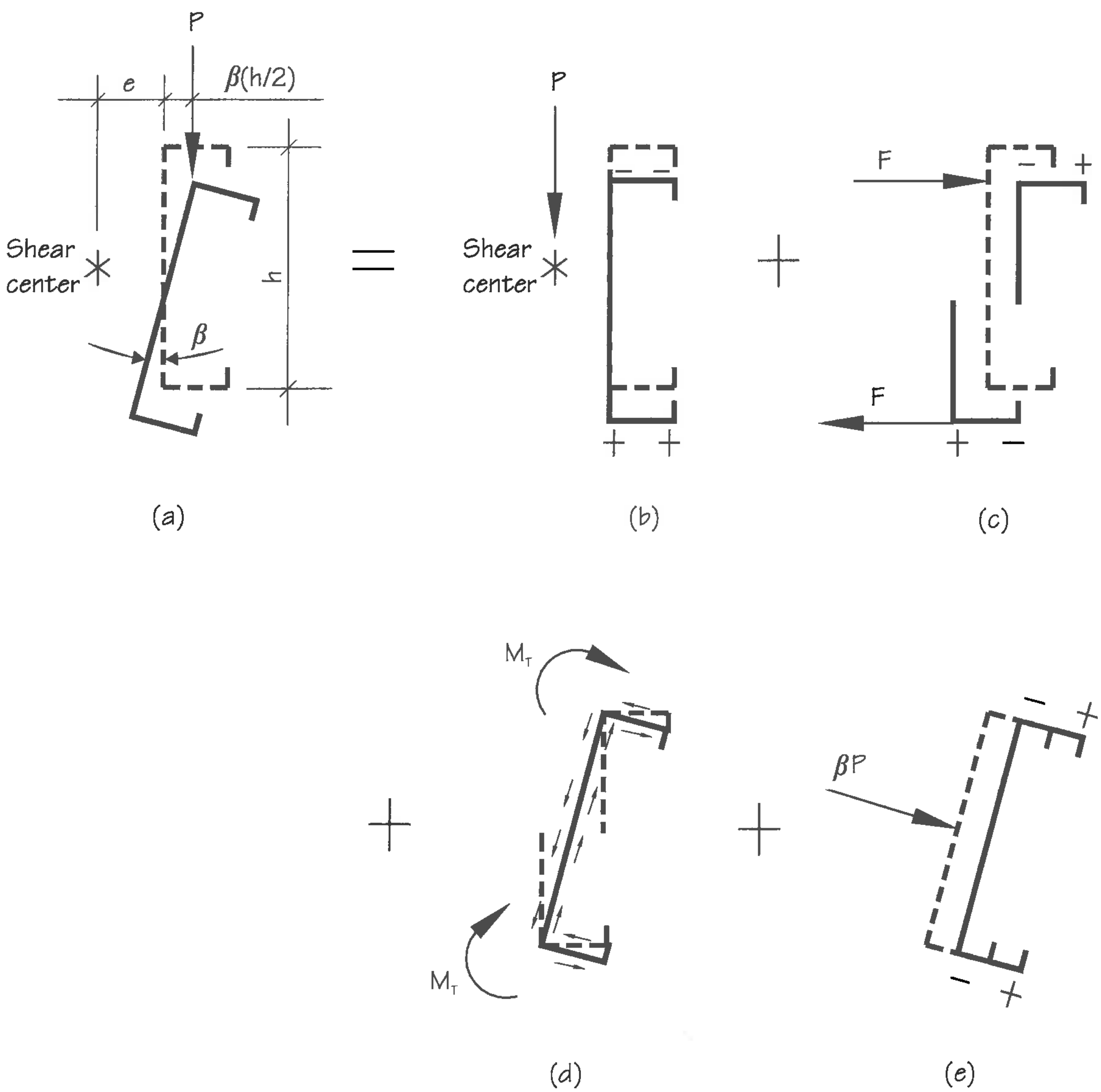
... For design purposes, it is now possible to take one of two positions. Conservatively, one can stipulate that the maximum corner stress shall not exceed the yield point ... Here the use of a single channel with discrete bracing will always be less economical than one with continuous bracing since 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 test 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 permitted 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. *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.*



$Fh + 2M_T = P(e + \beta h/2)$

- is compression
+ is tension

FIGURE C-1

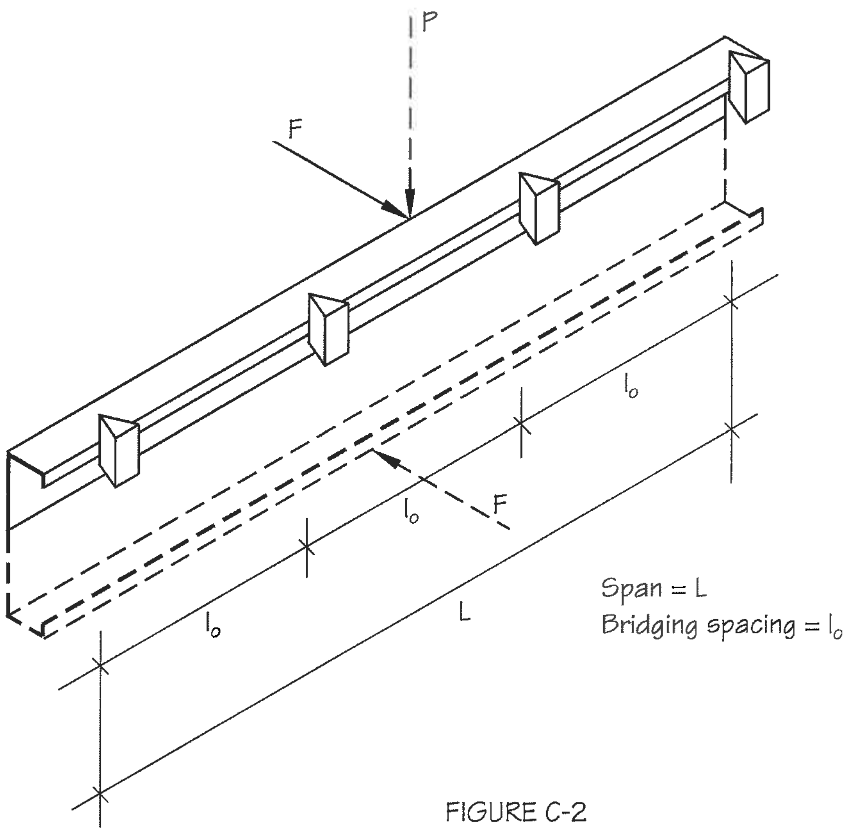


FIGURE C-2

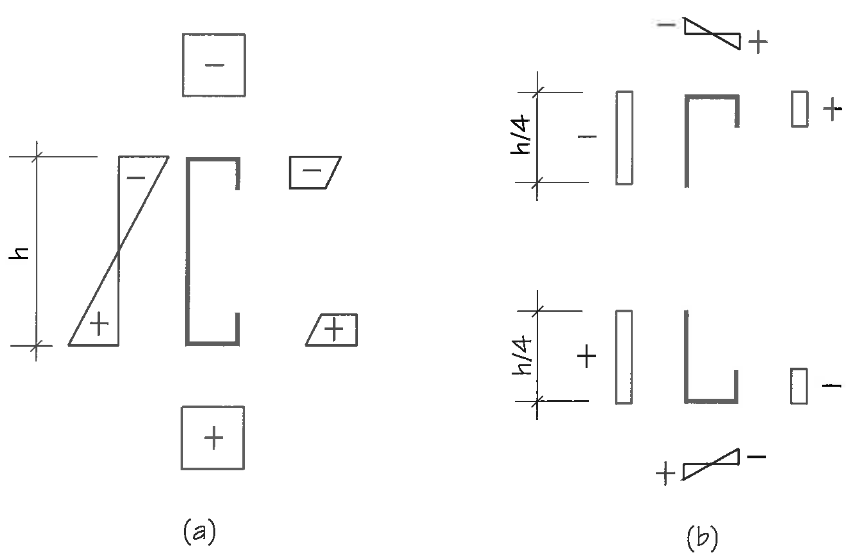


FIGURE C-3

Appendix D
Outer Top Track Flexibility Formulae

To connect wind bearing studs to the structure, inner and outer top track details are useful for accommodating floor deflections and construction tolerances.

The detail is, however, inherently flexible. Some horizontal movement occurs in the track whenever the studs are loaded by wind.

The following approximate formulae 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.

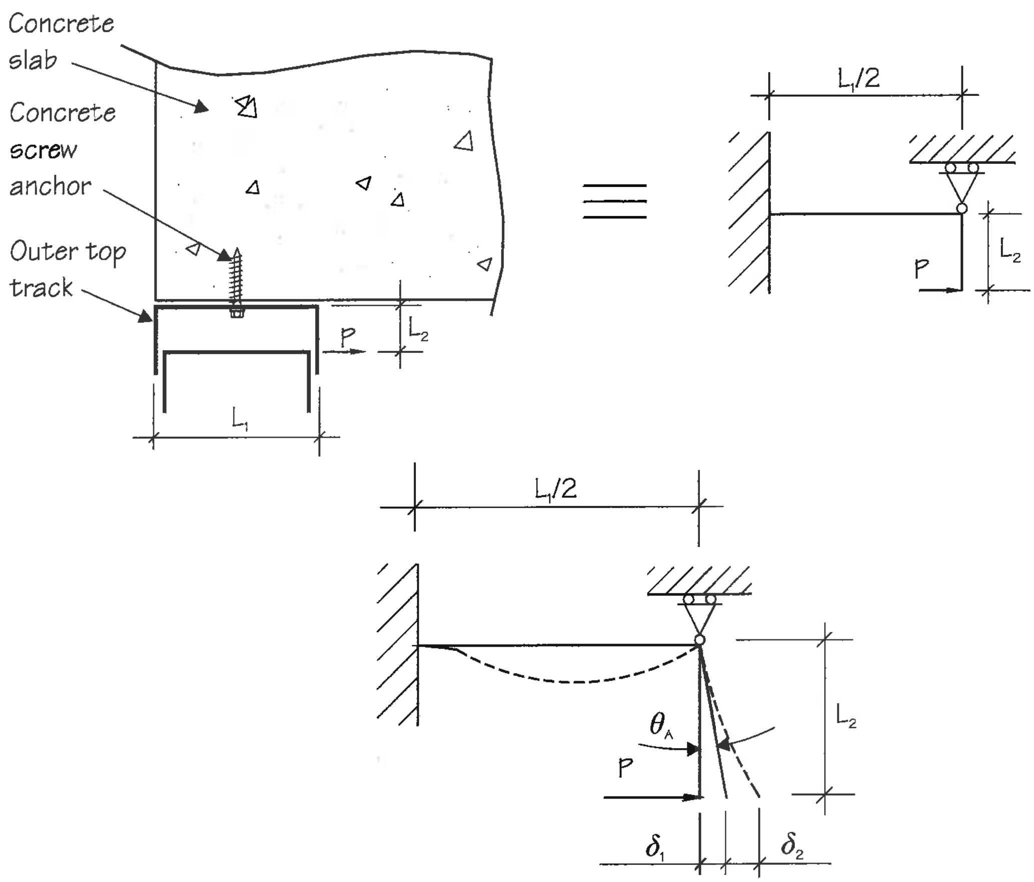


FIGURE D-1

From Figure D-1:

$$
\begin{aligned}
\delta_{total} &= \delta_1 + \delta_2 \\
&= \theta_A L_2 + \delta_2 \\
&= \frac{PL_2^2 L_1}{8EI} + \frac{PL_2^3}{3EI} \\
&= \frac{P}{EI}\left(\frac{L_2^2 L_1}{8} + \frac{L_2^3}{3}\right)
\end{aligned}
$$

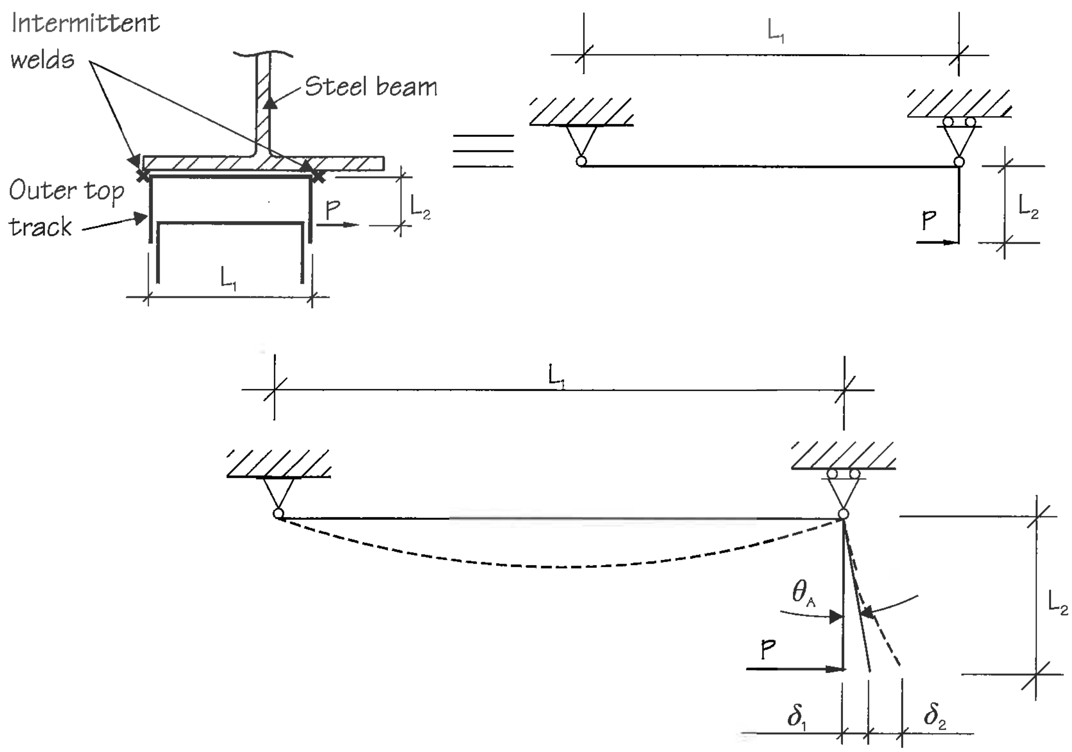


FIGURE D-2

From Figure D-2:

$$
\begin{aligned}
\delta_{total} &= \delta_1 + \delta_2 \\
&= \theta_A L_2 + \delta_2 \\
&= \frac{PL_2^2 L_1}{3\,EI} + \frac{PL_2^3}{3EI} \\
&= \frac{P}{3EI}\left(L_2^2 L_1 + L_2^3\right)
\end{aligned}
$$

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 400 mm or 600 mm 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 Reference 1.

$\beta = (k/4EI)^{1/4}$
$C_2 = \cosh \beta L \sin \beta L + \sinh \beta L \cos \beta L$
$C_3 = \sinh \beta L \sin \beta L$
$C_4 = \cosh \beta L \sin \beta L - \sinh \beta L \cos \beta L$
$C_{11} = \sinh^2 \beta L - \sin^2 \beta L$
$C_{A1} = \cosh \beta(L - a) \cos \beta(L - a)$
$C_{A2} = \cosh \beta(L - a) \sin \beta(L - a) + \sinh \beta(L - a) \cos \beta(L - a)$
$F_2 = \cosh \beta x \sin \beta x + \sinh \beta x \cos \beta x$
$F_1 = \cosh \beta x \cos \beta x$
$F_{A4} = \cosh \beta(x - a) \sin \beta(x - a) - \sinh \beta(x - a) \cos \beta(x - a)$

If $x \leq a$ then $F_{A4} = 0$

$$\theta_A = \frac{W}{2EI\beta^2}\frac{C_2C_{A2} - 2C_3C_{A1}}{C_{11}}$$

$$y_A = \frac{W}{2EI\beta^3}\frac{C_4C_{A1} - C_3C_{A2}}{C_{11}}$$

y = local horizontal deflection in outer top track leg

$$= y_AF_1 + \frac{\theta_AF_2}{2\beta} - \frac{WF_{A4}}{4EI\beta^3}$$

where:
L = Beam length, mm
a = Distance from left end to point load, mm
x = Distance from left end to deflection location, mm
k = Spring constant for outer top track, N/mm per mm of deflection
I = Inner top track major axis beam inertia, mm^4
W = Point load, N

Example E1

Check the inner and outer top track design from Design Example 1 as shown in Figure E-1.

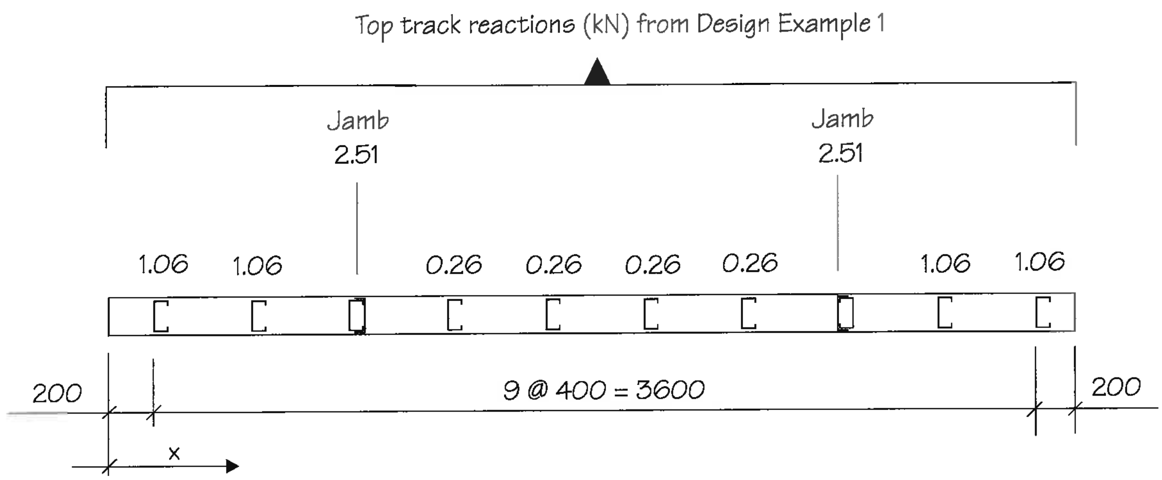


FIGURE E-1

Assume inner top track length = 4000 mm

Approximate inertia of long legged inner top track by using the deflection inertia for 600T200-43 track section with F_y = 230 MPa.

$I_{x(def)} = 0.857 \times 10^6$ mm^4

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 mm is given by:

$$k = \frac{EI}{\dfrac{L_2^2 L_1}{8} + \dfrac{L_2^3}{3}}$$

$$= \frac{24EI}{3L_2^2 L_1 + 8L_2^3}$$

For t = 1.811 mm outer top track with a maximum deflection gap of 41 mm

L_1 = 152 mm
L_2 = 41 mm
$I = (1/12)bt^3$
$= (1/12)(1)(1.811)^3$
$= 0.495\ mm^4/mm$
E = 203000 MPa

Substituting and solving for k:

k = 1.830 N/mm per mm of deflection

See Table E-1 for calculations of outer top track horizontal local deflections at x = 0 mm and at x = 1000 mm due to stud reactions.

Check βL

$\beta L = 0.00127(4000) = 5.08 < 6.0$ ***OK***

Note E-1

1. *Reference 1 restricts the beam on an elastic foundation to $\beta L \leq 6$ 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 $\beta L \leq 6$ limit somewhat. See Roark 1975 for alternative equations when $\beta L > 6$.*
3. *Theory has not been confirmed by test.*

Table E-1

Stud Reaction W		$\delta_{(horizontal)}$ (mm)	
Location a (mm)	Magnitude (N)	@ x = 0 mm	@ x = 1000 mm (at jamb stud)
200	1060	-1.107	-0.191
600	1060	-0.496	-0.321
1000	2510	-0.286	-0.916
1400	260	+0.013	-0.077
1800	260	+0.025	-0.047
2200	260	+0.021	-0.021
2600	260	+0.014	-0.005
3000	2510	+0.061	+0.022
3400	1060	+0.003	+0.021
3800	1060	-0.015	+0.025
	Σ	-1.767	-1.510

δ = 1.77 mm at x = 0 mm 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:
$w = 2(1.32) = 2.64$ N/mm
$\delta = w/k = 2.64/1.83 = 1.44$

Then:

$$\frac{\delta_{(\text{outer top track as an elastic foundation})}}{\delta_{(\text{outer top track as a uniformly loaded cantilever})}} = \frac{1.77}{1.44} = 1.23$$

Note that stresses in the cantilevering outer top track leg also increase (locally) by a factor of 1.23.

Example E2

Repeat Example E1 except with the inner top track cut-off 25 mm to the left of the jamb. See Figure E-2.

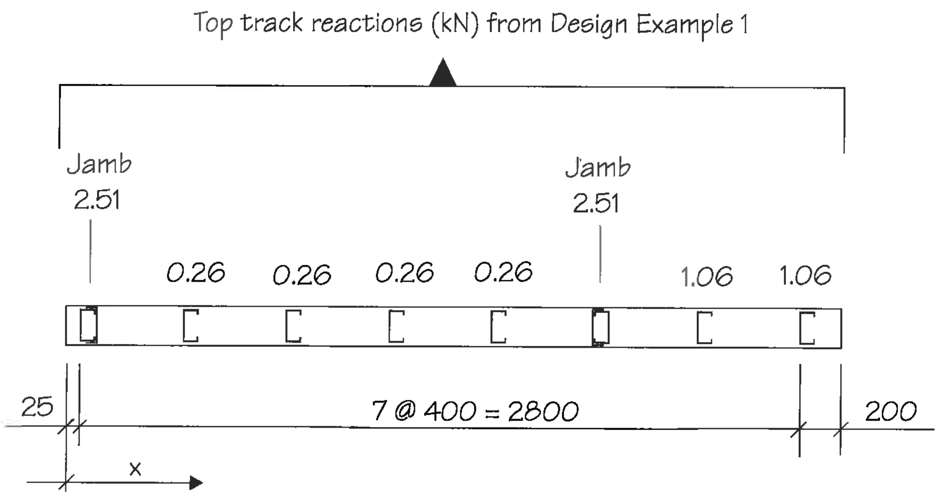


FIGURE E-2

Table E-2

Stud Reaction W		$\delta_{(horizontal)}$ (mm)	
Location a (mm)	Magnitude (N)	@ x = 0 mm	@ x = 400 mm
25	2510	-3.385	-1.807
425	260	-0.181	-0.142
825	260	-0.063	-0.086
1225	260	-0.000	-0.040
1625	260	+0.023	-0.010
2025	2510	+0.234	+0.039
2425	1060	+0.068	+0.039
2825	1060	+0.027	+0.046
	Σ	-3.278	-1.961

For this case at x = mm

$$\frac{\delta_{(\text{outer top track as an elastic foundation})}}{\delta_{(\text{outer top track as a uniformly loaded cantilever})}} = \frac{3.28}{1.44} = 2.28$$

and at x = 16"

$$\frac{\delta_{(\text{outer top track as an elastic foundation})}}{\delta_{(\text{outer top track as a uniformly loaded cantilever})}} = \frac{1.96}{1.44} = 1.36$$

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 2.28 times those from the simple uniformly loaded assumption. Note, however, that the stresses will be localized and that at a distance of 400 mm 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 2.28 ratio will be alleviated somewhat by plastic redistribution.

See also Note 1-8 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 factored bearing resistance stress on concrete is assumed to be given by $0.85\phi_c f_c'$ *(A23.3-04 CSA 2004b Clause 10.8.1)*

> *Note F-1*
>
> *Where the ratio of bearing area to the area of the concrete support is greater than 1, a higher allowable bearing stress may be permitted. Refer to A23.3-04 Clause 10.8.1*

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:

$$M_f = \frac{0.85\phi_c f_c' x^2}{2}$$

$$M_r = \phi Z F_y$$

where:

Z = plastic section modulus

$= (1/4) b t_t^2$ with $b = 1$ mm

Set $M_f = M_r$, $\phi_c = 0.65$ & $\phi = 0.9$

Solving for x gives:

$$x = 0.902 t_t \sqrt{\frac{F_y}{f_c'}}$$

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.

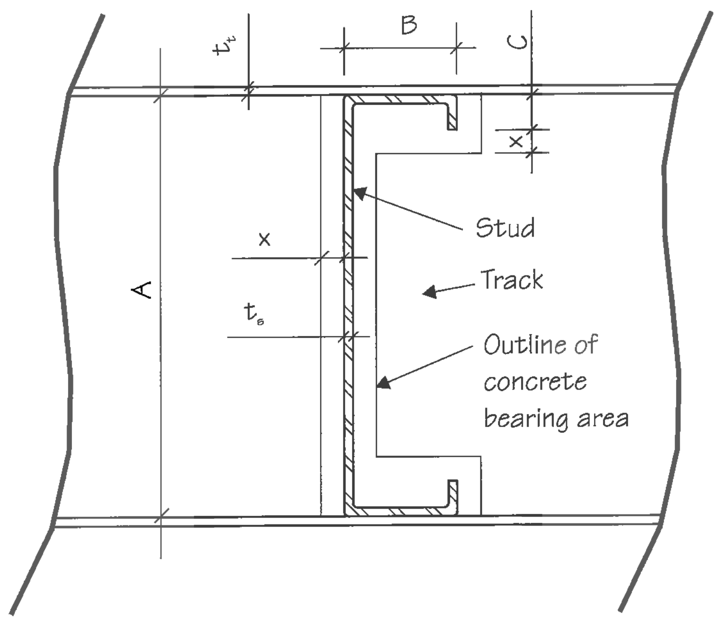
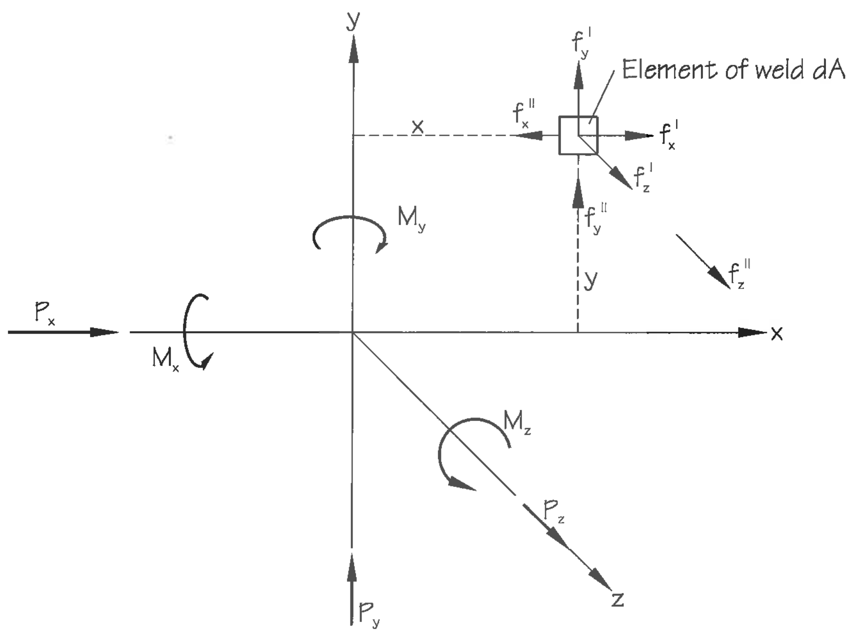


FIGURE F-1

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 formulae (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:

$$f_x^I = \frac{P_x}{A}, \qquad f_y^I = \frac{P_y}{A}, \qquad f_z^I = \frac{P_z}{A} \tag{1}$$

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_r, the maximum resultant shear force per unit length of weld, and the latter is arbitrarily considered a "shear" force acting on the throat section as follows:

$$q = tf = t\sqrt{f_x^2 + f_y^2 + f_z^2}$$

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_r 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 examples 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 particular 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.

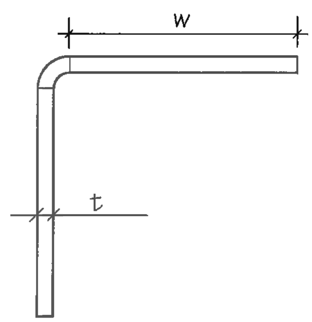


FIGURE H-1

From CAN/CSA-S136-01 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:

$$F_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2$$

$\mu = 0.3$
$E = 203000$ MPa
$k = 0.43$

and reworking gives:

$$f = \frac{35700}{(w/t)^2} \text{ MPa}$$

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.

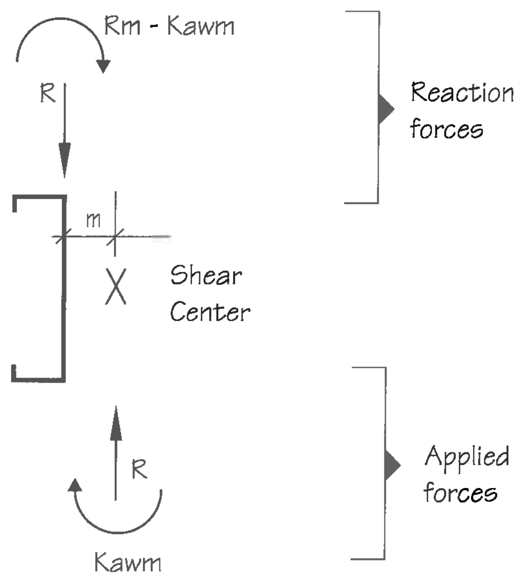


FIGURE I-1

<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 centerline of the stud web to the shear center
  K = some unknown constant. (Where the accumulated torsion $Kawm$ relieves the design loads, it is conservative to underestimate the value for the constant K. CAN/CSA-S136-01 uses K = 1.5 for interior 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 the first line of bridging. Alternatively, where the accumulated torsion adds to the design load K = 1.5(0.50) = 0.75 would be appropriate.)

The reaction forces (and the forces applied to the end connection) consist of:

- 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.

Appendix J
Product Identification

The 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

Member depth in 1/100ths inches. Thus 600 means 600/100 = 6" [152.4 mm]

Flange width in 1/100ths inches. Thus 162 means 162/100 = 1.62" or 1-5/8" [41.28 mm]

600 S 162 - 54

Style:
S = Stud or joist sections
T = Track sections
U = Channel sections
F = Furring channel sections

Material thickness in 1/1000ths inches. Thus 54 means 54/1000 = 0.054" [1.367 mm]

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 CAN/CSA-S136-01 Section A2.4. (The reference to the Canadian Appendix was removed in the Supplement - CSA 2004.)*

3. *For those sections with a yield strength other than 33 ksi (230 MPa), 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 (345 MPa) yield material. "600S162-54 (50) is also acceptable.]*

4. *For track, "T", sections, depth is a nominal inside to inside dimension. Other dimensions are out to out.*

5. *For "S" sections (studs and joist) CSSBI lip lengths are standardized as follows:*

Flange Width Designation	*Flange Width (mm)*	*Stiffening Lip (mm)*
S125	*31.75*	*4.775*
S162	*41.28*	*12.70*
S200	*50.80*	*15.88*
S250	*63.50*	*19.05*
S300	*76.20*	*19.05*

6. *Section styles are defined in Figure J-1*

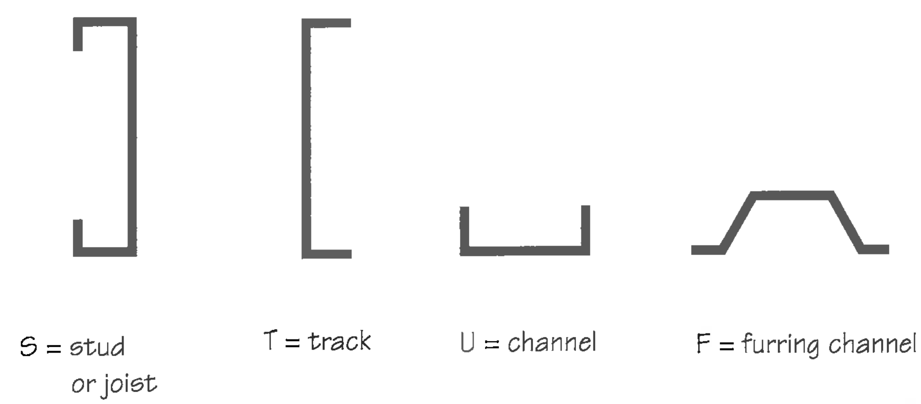


FIGURE J-1

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