Newsletter for the
Light Gauge Steel Engineers Association
October 2001

Upcoming Events

METALCON Oct 23-25
Las Vegas, NV
Info.: (617) 965-0055

LGSEA Committee Meetings Oct 23-25
Las Vegas, NV
Info: (202) 263-4488

Building Industry Show Nov. 8-9
Long Beach, CA
Info.: (909) 396-9993

SEMINARS: Practical Design of Cold-Formed Steel Structures
Atlanta, GA Dec 12
Nashville, TN Dec 13
Birmingham, AL Dec 14
Info: (202) 263-4488
- A registration form can be downloaded from www.LGSEA.com -

NAHB International Builders Show Feb 8-11 2002
Atlanta, GA
Info.: (800) 368-5242

Association of Wall & Ceiling Contractors International Mar 12-17 2002
San Antonio, TX
Info: (703) 538-1610

CHAPTER MEETINGS

For information, call:
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Atlanta/Southeast Chapter *(615) 394-3112*
Europe Chapter *(202) 263-4488*
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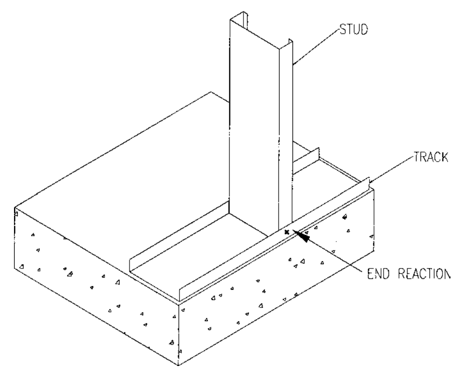
Recent Research on Stud/Track Connections

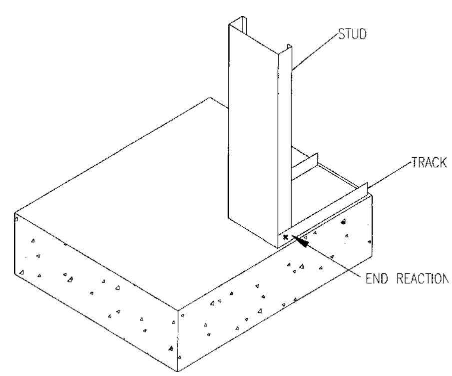
by L. Randy Daudet, P.E., Dietrich Industries, Hammond, IN

The strength of stud-track connections (Fig.1) has traditionally been an issue of debate among engineers designing cold-formed steel. Some engineers argue that web crippling in the stud need not be checked; the argument being that the track leg is not an infinitely stiff bearing surface, and therefore stud shear or track failure will preclude any type of stud web crippling failure. In addition, many argue that track bending on the flange is a limit state that needs to be checked, while others argue that bearing (or punch through) on the track flange should instead be considered.

Stud-Track Condition
Figure 1



Stud Occuring at End of Track
Figure 2



Fortunately, recent research has shed some light on this subject. Steve Fox (General Manager of the CSSBI), and Reinhold Schuster (Professor of Civil Engineering at the University of Waterloo) have published a paper in the year 2000 Proceedings of the 15th International Specialty Conference on Cold-Formed Steel Structures. The paper is entitled "Lateral Strength of Wind Load Bearing Wall Stud-To-Track Connections", and concludes that two basic failure modes need to

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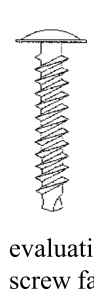
Specification of Screw Fasteners

By Dean Peyton, PE, Anderson-Peyton Structural Engineers, Seattle, WA



When specifying screws, the primary concerns of engineers are that the capacity of the screw meets or exceeds the design load and the screw capacity can be maintained over the service life of the structure. To address these concerns, the engineer usually uses one of three alternative methods for specifying the required screw capacity. These methods include (1) specifying the minimum required loads per screw type **and** material joined, (2) specifying an "approved" manufacturer (based on a specific manufacturer's tested data and the manufacturer's recommended factor of safety), and (3) specifying an approved evaluation service report for a specific screw fastener.

Many engineered plans follow method (1)

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Stud/Track Connections

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be checked by the designer. The first failure mode is stud web crippling given by the following equation:

$$P_n = Ct^2F_y(1 - C_R\sqrt{R})(1 + C_N\sqrt{N})(1 - C_H\sqrt{H})$$

R = r/t
N = n/t
H = h/t
t = stud thickness
r = inside radius bend
n = stud seating length
h = flat width of stud web
F_y = yield strength of stud
C = 5.6
C_R = 0.14
C_N = 0.30
C_H = 0.01

The second failure mode is track punch-through given as follows:

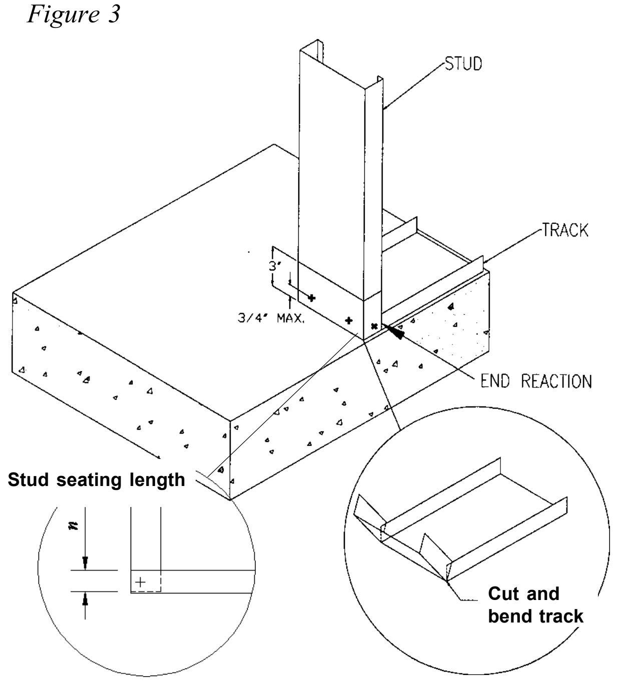
$$P_n = 0.6\, t_t w_b F_{ut}$$

t_t = track thickness
w_b = track shear width = $(0.78t_t + 0.56)$
F_{ut} = tensile strength of track

Fox recommends an ASD factor of safety of 1.69 to be used for each failure mode. In addition, these equations have several specific limitations which should not be overlooked. One limitation is that the stud must be 8" away from the end of the track. Unfortunately, there are many real world conditions where the stud falls directly at the end of the track. One such condition is a jamb stud for a sliding glass door (Fig. 2). Many engineers overlook the stud-track strength at this connection. Pilot tests conducted at Dietrich have suggested that the connection strength for a stud located at the end of the track appears to be governed by web crippling of the stud, and is about one half of the Fox value. Pilot tests were also conducted on a stiffened end condition (Fig. 3), comprised of notching the track flanges and folding the web to simulate a clip angle. The preliminary data for the stiffened detail suggests that the connection strength will exceed the Fox values.

In conclusion, it is important that engineers do not overlook the strength of the stud-track connection. This is especially true where the stud is located directly at the end of the track. In order to strengthen such connections, the detail given in Fig. 3 appears to show promise. ❐

Stiffened End Condition
Figure 3



Data on flat plate buckling needed

Bill Babich of Alpine TrusSteel is currently developing a Technical Note on truss gusset plate design. He has found some limited information from AISC technical documents. His initial draft is based on this information, as well as some testing his company has performed on a similar product. However, he has not been able to find any research data on plate compression members less than ¼" thick. If you know of any research that has been completed or is currently underway on plate buckling or truss gusset plates, please contact the LGSEA at LGSEA@aol.com, or Mr. Babich at bbabich@trussteel.net

Design of Bearing Stiffeners in Cold Formed Steel C-Sections

by Steven R. Fox, P.Eng., General Manager
Canadian Sheet Steel Building Institute

A research project initiated by the *American Iron and Steel Institute* to investigate the capacity of bearing stiffeners used in cold-formed steel joists has resulted in the development of design rules for these stiffener types.

Floor joists used in cold-formed steel construction are often C-sections, and depending on the thickness, can be susceptible to web crippling when subjected to concentrated loads. Bearing stiffeners are normally added to avoid the capacity reductions associated with this type of failure.

The current design provisions for transverse web stiffeners in the AISI *Specification for the Design of Cold-Formed Steel Structural Members* and the Canadian Standards Association CSA-S136 Standard *Cold Formed Steel Structural Members* do not apply to the common types of bearing stiffeners being used today in light gauge steel framing.

The AISI and CSA design documents require a bearing stiffener when *h/t* of the web of a flexural member exceeds 200, and design equations are provided. However, there are some practical problems with the current requirements. The most significant requirement is that the flat width of any element in the bearing stiffener shall not exceed the limit for local buckling. This means that no element in the stiffener can be subject to effective width reductions up to the design stress level. This condition is not met by most of the bearing stiffeners in common use today. A stud or track section as a bearing stiffener will be subject to effective width reductions at modest stress levels and fall outside the provisions of the specification.

A total of 263 end and interior two-flange-loading tests were carried out on different stiffened C-section assemblies. Figures 1, 2 and 3 show some of the tested assemblies and typical failure modes. The following conclusions have been reached:

1. The current design provisions in the AISI and CSA specifications can be unconservative if incorrectly applied to the types of bearing stiffeners commonly used in cold-formed steel framing.

2. For the stud and track stiffener types, the failure mode is local buckling of the stiffener acting as a short beam-column member. Overall column buckling can be a failure mode for deeper joists with stiffeners made from smaller sections such as bridging channels.

3. The capacity of the assembly is influenced by the following parameters:
   - stiffener type and material properties,
   - bearing width,
   - joist size and material properties,
   - number and pattern of fasteners connecting the stiffener to the joist,
   - location of the stiffener on the joist (i.e. end or intermediate, inside or outside),
   - gap between the stiffener and the joist flanges.

4. The capacity of the stiffened joist assembly can be calculated as a combination of the web crippling capacity of the joist plus the axial capacity of the stiffener, times a reduction factor.

5. The web crippling capacity of the joist is increased as a result of the connections between the stiffener and the joist web. Web crippling of the joist should be considered as a serviceability limit sate for the assembly.

6. A more detailed design of the stiffener would also take into account the eccentric axial loads and lateral loads transferred from the fasteners.

This project has only considered the stiffener and joist assembly. No recog-

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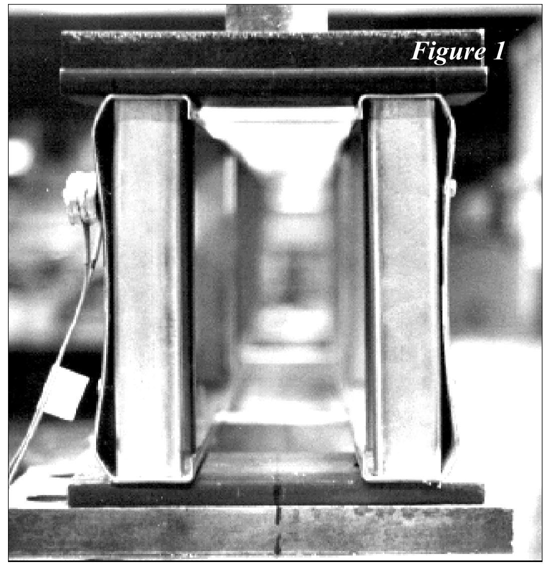
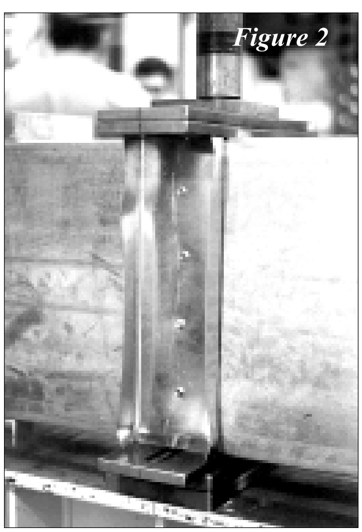
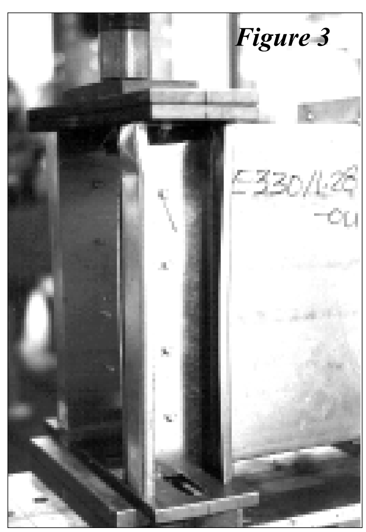


End-Inside Stud Stiffener and Web Crippling of the Joist



Failure of End-Outside Stud Stiffener



Failure of Interior-Outside Track Stiffener

TECHNICAL EXCHANGE

The Light Gauge Steel Engineers Association needs you and your experience. Please mail or fax your opinions, questions, and design details that are relevant to the cold-formed steel industry (fax to Dean Peyton at (253) 941-9939). Upon editorial review, your submission may be printed in the Technical Exchange Section of this Newsletter.

Bearing Top Plate to Avoid In-line Framing

By Don Allen, P. E., Starzer Brady Fagan Associates, Atlanta, GA

Using the Prescriptive Method or the building codes for design of cold-formed steel requires that the floor, roof and wall framing members all are aligned within ¾" of one another. Part of the reason for this requirement is the relatively low capacity of the top wall tracks in its weak axis. The unstiffened flanges of these members buckle under very low loads when placed in flexural compression, unless fairly thick steel is used. Being constrained by this in-line framing requirement can be costly and reduce the available options for design and construction. To alleviate this requirement, designers and framers have come up with several options for strengthening the top plate to allow joist, truss, and rafter loads to fall between the stud locations. These solutions fall primarily into three categories:

Adding members to the top track

Not only does this solution increase the capacity of the top track in its weak axis bending, but it can perform better as a drag strut carrying lateral load into a braced wall or shear panel, and can act as part of the chord for a floor or roof diaphragm. One of the most common configurations with an added member is using a double wood top plate above the top track (Fig. 1). With this, the designer usually neglects the track bending capacity altogether, and designs based on the capacity of the wood alone. As in wood construction, the splice location of the wood members must be offset. Because of this variable splice location, designers must consider the capacity when a splice point falls directly beneath a load point. In high wind and seismic areas, the connection between the top track and the top plate becomes an issue due to increased lateral and uplift loads. Frequently, the easiest connection is a series of screws installed from the track into the wood members.

For an all steel design, a segment of stud may be added either on top of the track, and welded down (Fig.s 2 and 3), or inside a deep leg track (Fig. 4). With both options, the designer needs to investigate flange buckling if high concentrated loads bear on the top member near a support. However, with the configuration of the stud inside the deep leg track, the double thickness of the material helps reduce this problem.

Another added member often used in light commercial construction is a structural tube on top of the track (Fig. 5). The tube, when the same depth as the wall, provides a flat welding surface for x-bracing and other connectors that

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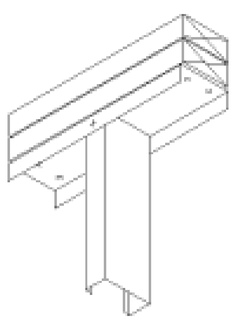
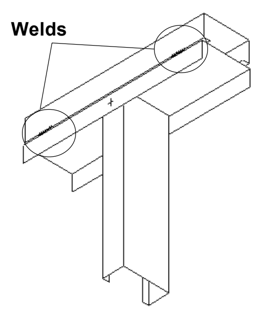
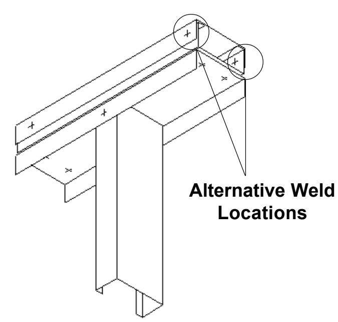


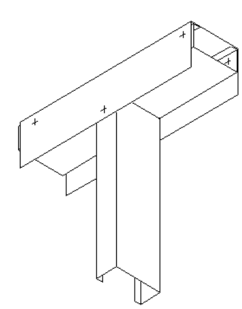
Double Wood Top Plate
Figure 1



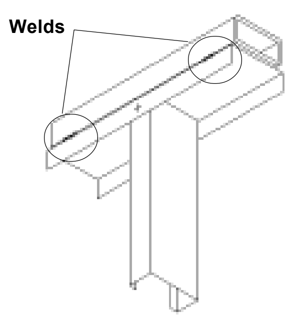
Single Stud Welded to Top of Track
Figure 2



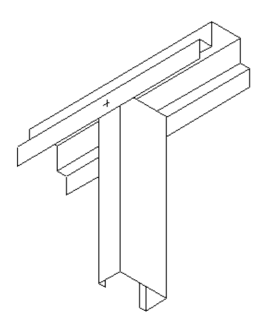
Built Up Stud in Track
Figure 3



Stud Inside Deep Leg Track
Figure 4



Structural Tube on Top Track
Figure 5



Hat Top Track
Figure 6

Slip Track Testing Begins at MSOE

A capstone project / thesis testing program designed to shed some light on the behavior of commonly used slip track connections has begun at the Milwaukee School of Engineering's Architectural Engineering Lab. Graduate student James Gerloff is performing the testing under the supervision of Dr. Peter Huttelmaier. LGSEA board member Patrick Ford is advising. The required cold-formed members are being supplied by Dietrich Industries, with additional funds provided by Matsen-Ford Design Associates, Inc.

Slip Track Assembly in Test Rack

Even with a fairly limited scope, 72 trials are currently scheduled to account for the variables of stud spacing, slip gap, stud and track flange, and track thickness. The tests will address 16 and 24 inch stud spacings, slip gaps of 1/2" to 1" stud flange widths of 1-5/8" and 2-1/2", and slip track thicknesses of 43 mils (18 gage, .0451") to 68 mils (14 gauge, .0713"), also with flange depths varied to check the effect on load capacity. Both 2" deep leg tracks and 3" leg tracks are to be tested. At the time of this report, about ten of the trials had been run. Some preliminary observations indicate that:

- the slip track flanges distribute the stud end reactions fairly widely across their length and have significant comparative load capacity after initial yielding;
- alternate fastener spacings of the slip track to the supporting structure have little effect;
- stud flange rotation, or the possible initiation of web crippling in the stud may be a serious design concern;
- with track flanges of at least 3 times the slip gap, there is typically not a dramatic failure mode (ie; stud popping out of the track).

Some of the questions that will hopefully be answered at least in part by this testing are what the effects of the variables have on the overall performance of the slip connection itself; how well is the stud end reaction distributed along the slip track flange (ie: what is the "b effective" given various conditions); what are the appropriate serviceability limits and safety factors to the connection capacity. ❒

Bearing Top Plate

Continued from page 4

must align with the wall face.

Using more steel in the top track

Even though the flange buckling problem still exists in thicker members, by adding steel and lengthening the flanges, top tracks themselves can have greatly increased capacity for weak axis bending.

Using a different configuration member for the top bearing plate

Although not common in North America, the top "Hat" track is used frequently in Australia and parts of New Zealand (Fig. 6). With the raised center section, framers can use this area as a chase to run wiring and small piping. Also, some manufacturers have pre-punched holes in the top for bolting down trusses and rafters.

With all of these options, the designer must ensure that gravity, lateral, and uplift loads have a path to follow between the bearing roof or floor members and the supporting wall studs. Although this may be complicated by using a non-standard configuration, eliminating the constraints of in-line framing may be helpful to the builder and designer. ❒

Specifying Screws

Continued from page 1

and provide a table of required design *allowable shear* (bearing and tilting) and tension (pullout) loads. An example of such a table is shown in Table 1. The values in this table are based on AISI Specification screw provisions that are adopted in current building codes and are similar to what may be found in steel stud manufacturers' catalogs. The allowable stress design (ASD) values in Table 1 *are specified* for different screw diameters and material thickness joined. As such, these values are based on the overall performance of the assembly, specifically, the behavior of the connected elements/parts. The performance of the fastener itself is not directly addressed in the Table 1 values. Since the *ASD* loads are computed for an assembly of the screw and the two parts joined, the factor of safety involved is an assembly factor of safety that is given as 3.0 in the AISI Specification. In addition to the required factor of safety for the assembly, the AISI Specification imposes, and code compliance requires, that the screw fastener itself meet specific requirements. Thus, where method (1) is used, it is incumbent on the engineer to verify that the ASD values (per Table 1) are in compliance with the requirements for the screw fastener. The following paragraphs elaborate further on this issue of assembly and fastener capacities and provides a framework for engineers to properly consider the requirements of the AISI Specification.

When the strength of a screw-fastened connection is determined by calculation, the AISI Specification (Section E4.3.2) requires screws to have a minimum capacity of 1.25 P_n, where P_n is the nominal calculated capacity of a single screw in the connection assembly. The intent of this requirement is to ensure that the screw itself will not control the capacity of the connection.

At thinner thicknesses (gauges), screw manufacturer tests indicate that the capacity of the connection may be controlled by the steel sheet. However, for thicker sheet connections, the engineer should specify screw capacities that are higher than indicated in Table 1.

Based on these provisions, the required nominal screw strengths for the assemblies in Table 1 can be computed as shown in Table 2. Thus, for compliance with the current building codes, if the engineer is planning on using the AISI calculated design allowable values per Table 1 then she/he should specify that the screws meet or exceed the Table 2 values. The Table 2 values would generally be provided by the screw manufacturer based on testing similar to the recently adopted AISI test protocol for screws. In situations where a manufacturer's data indicates that the Table 1-Table 2 relationship cannot be met, the engineer should consider working backward from the screw strength (per the manufacturer) to determine the screw strength per the assembly. To use Table 1 values without regard for the values in Table 2, the engineer must understand and be capable of rationalizing the difference in behavior of a screw in an assembly and a screw test (per the AISI test protocol).

Example: Assume an engineering analysis indicates that a shear connection requires 2560 lb (ASD) design load between two sections of 68 mil (14 gauge) material. The engineer anticipates a No. 10 screw and uses the AISI connection equations in Section E4.3 to calculate a nominal strength, P_n, of 2265 lb (per the assembly). For ASD P_n is divided by a factor of safety of 3 to get an allowable capacity of 755 lb/screw. The engineer then determines the number of screws as 2560 lb divided by 755 lb/screw equals 3.39 or (4)-#10 screws are required. To comply with the building code, the engineer must qualify that the screw has a capacity of 1.25 times P_n (= 1.25 x 2265 lb = 2831 lb).

It is interesting to note that a review of industry screw manufacturers data suggests that the maximum shear capacity of a #10 screw is on the order of 1500 lb. For the example under consideration here, this would imply an available screw shear strength of 0.66 P_n as opposed to the 1.25 P_n required by the code. To be in compliance with the code, the engineer can compute P_n based on the manufacturer's data. Following this approach, P_n will be 1500/

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Table 1. AISI Calculated Allowable Loads for Screw Connections

Material Thickness MILS	Design Thickness INCHES	Ultimate Strength F_u, (KSI)	#8 Screw DIA. = 0.164 SHEAR (LBS)	#8 Screw PULLOUT (LBS)	#10 Screw DIA. = 0.190 SHEAR (LBS)	#10 Screw PULLOUT (LBS)	#12 Screw DIA. = 0.219 SHEAR (LBS)	#12 Screw PULLOUT (LBS)
33	0.0346	45	164	72	177	84	190	97
43	0.0451	45	244	94	263	109	282	126
54	0.0566	65	496	171	534	198	573	228
68	0.0713	65			755	249	811	288
97	0.1017	65			1130	356	1303	410

Table 2. Minimum Required Ultimate Load for the Screw

(1.25 * 3.0 * Table 1 values = 3.75 * Table 1 values)

Material Thickness MILS	Design Thickness INCHES	Ultimate Strength F_u, (KSI)	#8 Screw DIA. = 0.164 SHEAR (LBS)	#8 Screw TENSION (LBS)	#10 Screw DIA. = 0.190 SHEAR (LBS)	#10 Screw TENSION (LBS)	#12 Screw DIA. = 0.219 SHEAR (LBS)	#12 Screw TENSION (LBS)
33	0.0346	45	615	270	664	315	713	364
43	0.0451	45	915	353	986	409	1058	472
54	0.0566	65	1860	641	2003	743	2148	855
68	0.0713	65			2831	934	3041	1080
97	0.1017	65			4238	1335	4886	1538

Bearing Stiffeners

Continued from page 3

nition was made of the other components commonly present in a floor (e.g. rim joist, sub-floor) that add to the strength of the assembly.

Simplified Design Expression (strength)
Based on the work described in the research report, the following simplified design expressions are proposed for computing the strength of a C-section member with a bearing stiffener subjected to two-flange loading:

$$P_a = P_n / \Omega \quad \text{(ASD)}$$
or
$$P_a = \Phi P_n \quad \text{(LRFD or LSD)}$$

The nominal strength is determined as follows:

$$P_n = 0.7(P_{wc} + A_e F_y)$$

United States and Mexico		Canada
ASD, Ω	LRFD, Φ	LSD, Φ
1.68	0.912	0.833

Where,

P_{wc} = Web crippling strength for the C-section joist calculated in accordance with the current AISI Specification and Supplement provisions for single web members, end or interior locations

A_e = Effective area of the bearing stiffener subjected to uniform compressive stress, calculated at the yield stress

F_y = Yield strength of the stiffener steel

This expression applies within the following limits:

(a) Stiffeners can be stud or track members (nominal 3-5/8" wide)
(b) The stiffener is attached to the joist web with at least three fasteners
(c) The length of the stiffener shall not be less then the depth of the joist minus 3/8"
(d) If the width of bearing is less than the width of the stiffener, the capacity must be reduced by 50 percent.

Web Crippling (serviceability)
The serviceability limit state is based on the web crippling of the joist member for those assemblies with the bearing stiffener installed between the joist flanges (see Figure 1). If the gap between the end of the stiffener and the joist flange is excessive, this web crippling deformation may cause serviceability problems. The AISI and CSSBI installation guidelines specify a maximum of 3/8 in. difference in length between the stiffener and the joist depth to limit the effects. The web crippling capacity of the joist should be calculated based on the AISI design equations available for two-flange loading of built-up sections. ❒

Specifying Screws

Continued from page 4

1.25 lb (= 1200 lb), the allowable screw strength in the assembly will be 1200/3.0 lb (= 400 lb.) and the required number of screws will be 2560 lb divided by 400 lb/screw for 6.4 or (7)-#10 screws - (3) more than first thought.

In summary, engineers using the AISI calculation approach (Table 1 values) should be sure that they first confirm screw manufacturers can meet the 3.75 factor of safety for their screw design. It is very possible that engineers will need to use lower design values than those indicated in Table 1 in order to meet these requirements with industry available screws. If an engineer chooses to specify approved screws (assembly tested) by methods 2 or 3 listed above, then she/he should keep in mind that the listed allowable loads will be assembly tests only and no additional factor has been applied to the fastener itself. ❒

WANTED

Engineering firm in Houston, TX has an opening for a structural engineer experienced in cold-formed steel design. Fax resume to Structuneering at (713) 779-1708.

Light Gauge Steel Engineers Association

LGSEA

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