STEEL INTERCHANGE

Steel Interchange is an open forum for *Modern Steel Construction* readers to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine. If you have a question or problem that your fellow readers might help you to solve, please forward it to *Modern Steel Construction*. At the same time, feel free to respond to any of the questions that you have read here. Please send them to:

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Answers and/or questions should be typewritten and double-spaced. Submittals that have been prepared by word-processing are appreciated on computer diskette (either as a Wordperfect file or in ASCII format).

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Information on ordering AISC publications mentioned in this article can be obtained by calling AISC at 800/644-2400.

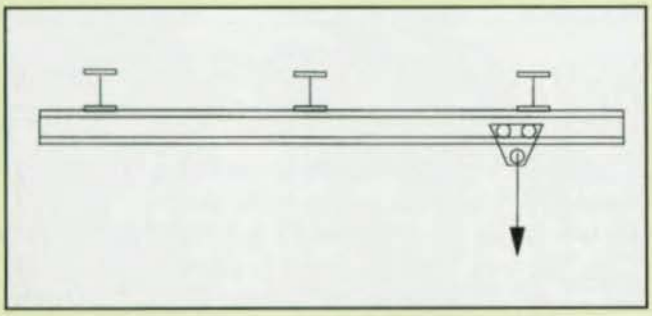
The following responses from previous Steel Interchange columns have been received:

Are there any limitations on the span to depth ratio of beams required by AISC Specification for Structural Steel Buildings?

In the Commentary on the AISC Specification (9 edition) chapter L, section L3 the following rule was suggested: "The depth of fully stressed beams and girders in floors should, if practicable, be not less than (F_y/800) times the span. If members of less depth are used, the unit stress in bending should be decreased from than recommended above."

Alex Krasilovsky, P.E.
Ridgefield Park, NJ

For a continuous trolley beam with multiple spans and cantilevered ends what is the lateral unbraced length for the bottom flange?



The lateral unbraced length of a cantilever trolley beam is approximately half[1] of the cantilever span, provided the support is braced against twist. The AISC code[2] states that L shall conservatively be taken as a unity, if the support is braced against twist. ANSI[3] Monorail Specifications specify L to be 2 times the length of cantilever, since the cantilever is not fully stayed at its outer end. The ANSI method is the most logical, L unbraced equals 2 times the actual length. If the cantilever is braced, the brace should connect to the top tension flange at the end, to offset twist[4].

The interior unbraced length, should be the distance between supports, per ANSI specifications on Monorail Systems[3].

Often the size of the trolley beam is controlled by the flange width, to be wide enough for the bolted gage plus 2 proper edge distances. Also the beam depth must be deep enough for the trolley wheel diameter. Flange bending strength under the wheels is important also.

[1] Timoshenko, S., "Theory of Elastic Stability", p. 260 & 269, 1961.
[2] AISC, "Manual of Steel Construction, Allowable Stress Design", p. 5-47, Ninth Edition.
[3] ANSI, "American National Standard Specifications for Underhung Cranes and Monorail Systems", p. 7, MH 27.1-1981.
[4] AISC, "Steel Design Current Practice, Bending Members - Buckling and Bracing", p.27.

Claude R. Krout, P.E.
Birmingham, AL

Can an existing steel beam and concrete slab be made to work together in composite action by adding studs to the steel through cored holes?

This question was previously responded to and it was the author's opinion that the existing loads presently on the beam should act on the bare steel. However, utilizing the Load & Resistance Factor Design code (LRFD) and assuming plastic stress distribution for positive moment, the composite section can carry all the load if the following requirements are met:

1. $\phi M_n \geq M_u$ for the composite section.
2. Yielding of the beam does not occur at the

maximum possible service load.

3. Composite action between the grouted holes and existing concrete occurs.
4. The deflection of the member is acceptable.

Using load factor design, the nominal moment of the section is developed when the entire section is fully plastic. When this occurs, the relationship between stress and strain is non-linear. Therefore, the existing dead load can be assumed to be carried by the composite section. Reference should be made to Chapter I of the LRFD code under Sections I1. and I3. By utilizing this type of analysis, significant load increase may be permitted under the provisions of the LRFD code.

Kurt Seidler, P.E.
Canfield, OH

Can you provide some information on eccentric effects on single angle bending members?

The specifications for LRFD design of single angle members is valid for angles loaded eccentric to the neutral axis and to the shear center; however, the effects of these eccentric loadings must be considered in combination with stresses form all other load effects if the provision of sect. 5.2.1a are not met. That is, if the brick wall does not have the stiffness or connectivity to the angle to prevent lateral torsional buckling or if the angle is not independently restrained then the effects of this eccentricity must be considered. Furthermore, the location of the restraint with respect to the vertical leg of the angle (i.e. the extreme fiber of the compression portion of the angle) should be considered when determining restraint.

The location of the load with respect to the shear center will cause a torsional eccentricity. This eccentricity will increase the shear and normal stress in the angle requiring an increase in the strength of the angle. The effect of the load eccentricity to the shear center will induce torsional stresses in the member. These stresses can be categorized into two types, pure torsion (St. Venant's torsion) and warping torsion. Pure torsion causes pure shear stress only $\tau_t = Gt(d\phi/dz)$, warping torsion causes warping shear stress $\tau_w = ES_w(d^3\phi/dz3)$ and warping longitudinal stress $\sigma_w = E\omega_n(d^2\phi/dz^2)$. Under the loading conditions shown all three of these stresses can be present. Their magnitude will depend on the magnitude and location of the loads and the boundary conditions of the angle. For design purposes these stresses should be considered as acting in combination with the normal and shear stress due to bending.

In addition, deflection and rotation due to these torsional stresses should be added to the deflection due to bending when considering the serviceability requirements of this angle.

A very good source which outlines the calculation of the torsional stresses indicated above is found in "Cold Formed Steel Design 2nd edition appendix B", Wei-Wen Yu. Additional information on torsional stresses and their effects can be found in "Steel Structures design and behavior", Salmon & Johnson and in the AISC publication "Torsional Analysis of steel members".

The commentary to the specification for single angle members section C6 states "... the applied moments should be resolved about the principal axes for the interaction check." If Mn is determined about the principal axis then Mu must be converted from geometric axis loading to equivalent principal axis loading. Hence, Mnx = Mn(sin) and Muy = Mn (cos).

The resolution of loading and stresses about the principal axes may be neglected if the provision of sect. 5.2.1a is met. That is, as stated in the commentary to this section, if the angle is restrained against lateral torsional buckling along its length then bending occurs without any torsional rotation or lateral deflection. Therefore, only bending about the geometric x axis and shear in the geometric y axis need be considered.

Thomas M. Vossmeyer, P.E.
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New Question

In field bolted connections for galvanized members, are bolt holes enlarged to account for the layer of zinc which will be deposited? If so, by how much and is it possible to treat the resulting hole as a standard hole in design?

How do you calculate the moment capacity of a double angle shear connection?