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:

Steel Interchange
Modern Steel Construction
One East Wacker Dr., Suite 3100
Chicago, IL 60601-2001

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).

The opinions expressed in *Steel Interchange* do not necessarily represent an official position of the American Institute of Steel Construction, Inc. and have not been reviewed. It is recognized that the design of structures is within the scope and expertise of a competent licensed structural engineer, architect *or other licensed professional for the application of principals to* a particular structure.

Information on ordering AISC publications mentioned in this article can be obtained by calling AISC at 312/670-2400 ext. 433.

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

When asked to design a temporary bracing system for steel beams and columns during the erection phase of construction, what loads are used and what factors of safety are employed for the bracing and its connections?

The primary source for the loading requirements for temporary bracing is the AISC Code of Standard Practice Section 7.9.1 which states that the temporary supports are to be designed for "...loads comparable in intensity to those for which the structure was designed, resulting from wind, seismic forces and erection operations, but not loads resulting from the performance of work by or the acts of others, nor such unpredictable loads as those due to tornado, explosion or collision." It should be noted that steel frames are open structures and may have wind loads greater than predicted by the projected area of the completed building. Also partially completed cladding may add to loads in the supports for non-self-supporting structures. The topic of erection bracing and the loads applied to it is discussed in detail in a paper by Fisher and West published in the Proceedings of the 1993 SSRC Conference held in Milwaukee.

(Information on obtaining a copy of "Is Your Structure Suitably Braced" can be obtained from the Structural Stability Research Council by calling 215/758-3522.)

Michael A. West, P.E.
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Milwaukee, WI

When is it appropriate to use clips instead of hook bolts to secure rails for top running crane runways? What are service versus cost considerations?

In general, hook bolts can be used for attaching rails for light duty, slow moving cranes and clips are used for heavy duty, faster cranes. the dividing point is arbitrary. However, hook bolts should never be used for heavy duty cranes as they are not strong enough to resist the lateral forces likely to be present. Clip plates, on the other hand, can be used for the full range of crane sizes but, for smaller cranes, it may be impossible to place the required gauges in the crane beam and cap channel. This is the situation where hook bolts can be used. There is a third option for attaching rails to crane beams—proprietary adjustable clip devices which are welded to the tops of the crane beams. Crane rails should never be welded directly to the crane beams, regardless of crane size.

When considering the economics of hook bolts vs. clips there is nothing to consider. One must do what must be done. If one owns a Ferrari, one should not worry about the cost of high octane gasoline. A crane runway is a dynamically loaded structure placed inside a statically loaded structure and must be treated as such. Hook bolts are usually a purchased item, the rails must be punched or drilled to receive them, and they must be monitored frequently to keep them tight and to assure proper alignment of the rails. Rail clips are easily fabricated by most fabricators, the crane beams must be punched or drilled, alignment is more positive, lock nuts or fully torqued high strength bolts are required.

For the sake of this discussion light duty could be interpreted as CMAA classes A2 and B; heavy duty as CMAA classes A1, C, D, E, and F. Each crane capacity has its relative speeds, for instance, 150 ft per minute would be slow for a 50 ton crane, medium for a 100 ton crane, and fast for a 150 ton crane. For further information see, Tips for Avoiding Crane Runway Problems, AISC Engineering Journal, Vol. 19, No. 4 (1982), pages 181-205.

David T. Ricker, P.E.
Payson, AZ

When welding to AWS D1.1 requirements what is a "seal" weld and what are the applicable inspection criteria for the same?

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A seal weld is designed primarily to provide a specific degree of tightness against leakage, or melt-through, until the final weld meets the requirements for throat size in assembly gaps of 5/16-in. or less. The seal weld is not included in the integrity inspection of the weld itself.
David L Simpson
U.S. Army Corps of Engineers
Muscat, Oman

AWS A3.0 defines a seal weld as "any weld designed primarily to provide a specific degree of tightness against leakage." This does not define the type of weld but rather its intended function.

AWS D1.1 does not address the specifics of a seal weld, however for a weld to conform to the requirements of AWS D1.1 it must meet the prequalified criteria of Section 2 (unless qualified through testing). Consequently any type of prequalified weld which is applicable to the given joint may be used as a seal weld. The inspection criteria for the weld would be the same as if it were used in a structural application.

This may seem excessive to some readers but consider one recent example:

A construction elevator derailed because a seal welded cap plate on the tube support breaks away allowing the support to fill with water. The weep holes were clogged and during the winter months the water froze deforming the support.
Neal White, P.E., CWI
Special Testing Laboratories
Hartford, CT

New Questions

Listed below are questions that we would like the readers to answer or discuss.

If you have an answer or suggestion (or an additional question) please send it to the Steel Interchange Editor, Modern Steel Construction, One East Wacker Dr., Suite 3100, Chicago, IL 60601-2001.

Questions and responses will be printed in future editions of Steel Interchange.

In designing composite steel girders in accordance with the LRFD Method, it has been well established that significant reductions in beam sizes can be achieved. However, in my experience, I have found, in some cases, the more economical girder sizes derived may be unsafe during unshored wet concrete construction. This can occur when the metal deck runs parallel to the girder and, in my judgement, does not afford significant lateral restraint to the top flange of the girder in compression. For this condition, the unbraced length is the spacing between the beams supported by the girder. Significant reduction in the non-composite moment capacity can occur due to lateral torsional bucking which may not be adequate for the unshored wet concrete construction.

No criteria for this serviceability problem or guidance appears to be given in the LRFD specification. I would like to know whether there has been any testing or research to demonstrate that metal deck, parallel to the girder does indeed provide adequate restraint or that checking the beam size for the temporary construction condition, should be carried out as outlined above.
Peter J. Maranian, S.E.
Brandow & Johnson Associates
Los Angeles, CA

When designing using the ASD Manual, what is the allowable weak axis bending stress on channel? The manual does not seem to specify this.
Adam Samuel
Riley Stoker Corp.
Worcester, MA

When welding a steel that has dual certification (A36 and A572 Gr50) is there a low hydrogen electrode requirement?

A36 is a group I base metal and ASTM A572 is a group II base metal. Is Aws D1.1 Table 4.1, note 1 applicable to this condition?
Neal White, P.E., CWI
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Hartford CT

A box at the top of each Steel Interchange contains a sentence that has caused a lengthy discussion in the office. The sentence is as follows: "It is recognized that the design of structures is within the scope and expertise of a competent licensed engineer, architect or other licensed professional for the application of principals to a particular structure."

Fellow office members did not understand the assertation made by the sentence. We think it means architects and other design professionals can practice the art of sizing structural members. If we have the correct meaning, then this presents an open conflict between two design professions. We could only guess at the meaning of "other licensed professional." Please tell me we do not understand the sentence. We fear that the meaning is an example of further eroding of the position professional engineers active in the design of structures enjoy. Persons not trained nor licensed as professional engineers competent in structures, use competitive methods which the average buyer may not understand. Perhaps a better comment would be explaining the many benefits of hiring the trained professional with the legal authority to assume responsibility.
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