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 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
1 East Wacker Dr.
Suite 3100
Chicago, IL 60601

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. 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 principles 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 to questions from previous Steel Interchange columns have been received:

An oddity exists when comparing $0.4F_y$ versus $0.3 F_u$ shear stress values. The ratio of F_y to F_u for A36 and A572 Gr. 50 steel is not proportional. For applications based on $F_v = 0.4F_y$, the allowable shear stress for A572 Gr. 50 is 39% greater than A36 steel; however, for applications based on $F_v = 0.3F_u$, the allowable stress is only 12% greater for the A572 Gr. 50 Steel. For A36 steel there is an increase in going from $0.4F_y$ to $0.3F_u$. For A572 Gr. 50 there is a decrease.

a) When a single round hole penetration is required in a beam web, is it proper to use $F_v = 0.4F_y$ or $F_v = 0.3F_u$ when calculating shear capacity?

b) Would a row of bolt holes behave differently than one large round hole which resulted in the same net area?

c) Does the presence or absence of bolts in holes affect the shear capacity of the member?

Mr. Ricker has identified one of the oddities that occurs in the AISC Specifications when more than one limit state (yielding and fracture) must be satisfied. In addition, Ricker also seeks clarification between bolt holes and beams with web openings. Because the inquiry is discussed in allowable stress terms, the response will be based on the June 1, 1989 AISC ASD Specification. The same condition exists in the September 1, 1986 LRFD Specification.

Equation F4-1 ($0.4F_y$) is the limiting stress allowed on the beams gross section to prevent yielding whereas Equation J4-1 ($0.3F_u$) is the limiting stress along the net section that will prevent fracture from occurring through the bolt holes. As Ricker indicates, as the yield stresses increases or the ratio of tensile stress to yield stress decreases (F_u/F_y) the controlling limit state will change from yielding to fracture. Whether this is considered on oddity or a design consideration based on differing limit states is subjective based on the individual engineer's perception. With the preceding information identified the answers to Ricker's three questions are as follow:

a) If one assumes that the hole is of the size intended for a bolted connection (d_b 1½ in.) then both Sections F4 and J4 must be checked and the one giving the lower answer governs. Larger diameter holes suggest a web penetration for an electrical/mechanical duct which should be checked using the provisions of the AISC publication *Steel and Composite Beams with Web Openings*. In my opinion if the diameter of the hole is less than ⅓ of the beam depth, then web opening provisions are unlikely to govern, and the two limit states identified earlier must be checked. As the hole increases in size the web opening provisions and Equation F4-1 must be checked to determine the governing condition.

b) Because bolt spacing is usually three times the bolt diameter (minimum three inches) the maximum material removed by a row of holes is less than ⅓ the beam depth. Under these conditions it is unlikely that there will be any behavioral differences whether there is several holes or one hole whose diameter equals the sum of the individual bolt hole diameters.

c) There have been bolted connection tests indicating that initially there is some variations in connection behavior due to the clamping effect of high strength bolts. However, as the applied load increases and the tightened bolts become loose the difference in behavior is largely undetectable. This suggests that the presence or absence of bolts will not affect the shear capacity of a beam web.

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The AISC design procedure for end-plate moment connections is for static loading only. Can this connection be used for a utility bridge that has wind loading? Can it be used on a frame that supports a crane runway?

In AISC Design Guide Series No. 4, Extended End-Plate Moment Connections, Chapter 2 - Recommended Design Procedures, only static loading is permitted. However, static loading as it states, includes wind, temperature, and snow. Therefore, the utility bridge subjected to wind loading would qualify as a statically loaded structure and could utilize extended end-plate moment connections. The frame supporting the crane runway may experience many more loading cycles than could be considered as static and should be designed with fatigue in mind.

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Steel Interchange

I have read the July 1992 issue of MSC and find an extremely disturbing anomaly. In response to a query in *Steel Interchange*, "what is a good wind connection for the top of a column," two engineers responded with suggested details. In the same issue, Mr. William McGuire has an article cautioning engineers of the dangers of using prepared material without adequate understanding of the behavior of the structure.

I must agree with Mr. McGuire. The only satisfactory answer to the question asked is: "one that satisfies all of the requirements and costs as little as possible." Another response may lead to inappropriate use of a standard which can easily be done when proper informed thought has not been given to a design.

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The AISC design procedure for end-plate moment connections is for static loading only. (See LRFD Manual, 1st Edition, p. 5-143 and ASD Manual, 9th Edition, p. 4-116) Why is this restriction made?

In response to Barry K. Shriver's question regarding the AISC design procedure for end-plate moment connections, I offer the following:

While a designer may be tempted to use the AISC design procedure for end-plate moment connections for any loading combination, a closer examination of the procedure indicates that no provisions for fatigue loading are included. Indeed, repeated loading and unloading even if the yield point is never reached may result in the eventual failure of this connection as a result of fatigue. The procedure apparently considers this possibility by stipulating that the design procedure is only valid for static loading conditions.

The main factors governing fatigue strength are the applied stress, the number of expected loading cycles, the type of detail, and the load path redundancy of the overall structural system. Fatigue need not be considered for a number of cycles less than 20,000 which corresponds to two applications every day for 25 years. Obviously wind loading is not fatigue loading and can be considered a static loading. This leaves high cycle fatigue such as in crane girders, and alternating plasticity as in high seismic regions as areas of concern.

When the beam web and flanges are connected to the end plate by fillet welds, the stresses in the welds are considered to be shear stresses so that the detail is classified as Category F. The allowable stress range in shear is 12 ksi for 500,000 loading cycles (50 applications every day for 25 years). If the beam is connected by full penetration groove welds the detail is Category C for which the allowable stress range in the base material is 21 ksi for 500,00 cycles for the thickness of the flanges and web not greater than 1½ in.

These factors are crucial to the proper design of this type of connection. In the case of Example 39 (9th Edition page 4-120), the designed flange to end-plate connection results in an allowable fatigue stress range of 5.88 ksi for a non-redundant system expecting 500,000 load cycles. Since this example does not differentiate between dead load (static load) and live load (dynamic load), it is impossible to determine if the dynamic stress range will exceed this limit. However as this example clearly illustrates, dynamic loading considerations may well govern the design.

A great deal of judgement is required in determining whether or not dynamic loading will govern the design. It can generally be anticipated that under normal wind loading, fatigue will not be a governing factor. On the other hand, the design of a crane support system may indeed be governed by fatigue. While general assumptions can be made about the type of structure, these assumptions should not replace good sound engineering principles. The AISC design detail outlined in the ASD Manual on pg. 4-116 is useful in performing preliminary designs for structures expecting relatively small stress ranges and a low number of load cycles (less than 20,000). Where this is not the case, an estimation of fatigue strength should be included in the preliminary design. In either case, the consideration of dynamic loading (in conjunction with load path redundancy) should be included in the final design of any structure. Where a high number of cycles and/or a large stress range is expected—as with a vehicular bridge—serious consideration should be given to using bolted connections in lieu of welding.

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New Questions

Listed below are some questions that we would like the readers to answer or discuss. If you have an answer or suggestion please send it to the Steel Interchange Editor. Questions and responses will be printed in future editions of Steel Interchange. Also, if you have a question or problem that readers might help solve, send these to the Steel Interchange Editor.

I am interested in reference sources that address the design of "curved" beams (supporting members rolled to a radius) frequently encountered at the perimeter of buildings, canapies, etc. as well as highway bridges and overpasses. Sources dealing with hot rolled (WF, C, L, etc.) sections are preferred, although any information regarding built-up members would also be appreciated.

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Correction: The limit in the last paragraph of Don Sherman's response in the September, 1992 Steel Interchange should read $253/\sqrt{F_y}$.