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Shear Fatigue in Bolted Connections

I am designing a structure that could be subjected to nearly one million loading cycles per year. The steel members have been designed considering fatigue. We have specified slip-critical connections with ASTM A490 bolts. However, I cannot find a section in the *Specification* from which to determine a reduced bolt stress based on the number of cycles in shear. Is merely specifying slip-critical connections sufficient to ensure the bolts do not fatigue?

Yes. The Commentary to Section 3.4 states: "Bolts installed in joints meeting all the requirements for slip-critical connections survive unharmed when subject to cyclic shear stresses sufficient to fracture the connected parts." Because slip-critical joints transfer shear loads via friction between the faying surfaces, in theory the bolts are not subjected to shear stress. The practice of neglecting secondary bolt shear stresses in slip critical joints, which can occur due to joint deformations, has been verified experimentally. Therefore, the joint fatigue performance of the connected material in simple lap joints can be evaluated using Case 2.1 in 2010 AISC *Specification* Appendix 3 Table A-3.1.

Some connections are more complicated than the lap joint in Case 2.1. The potential for fatigue cracking is higher at copes, transitions, weld terminations and other areas of stress concentration. In these locations, a suitable case from Table A-3.1 should be selected to evaluate the fatigue performance.

Bo Dowswell, P.E., Ph.D.

Fireproofing and Long-Slotted Holes

Is it acceptable to use a connection with long-slotted holes when fireproofing is required? It seems that the degree of movement allowed by the long-slotted holes could damage the fireproofing.

Yes, it is permitted. The use of long-slotted holes is permitted by the AISC *Specification*. Generally long-slotted holes are provided to accommodate tolerances during erection, not to accommodate movement in service. Using slots to accommodate movement is not addressed in the AISC *Specification*, and the engineer must rely on their own judgment when evaluating this condition.

AISC has provided recommendations to avoid the use of bolts moving in long-slotted holes to accommodate expansion. There are concerns that the bolts could bind, preventing the intended movement, or that a sawing effect could occur with repeated motion. The May 2011 SteelWise article "Expansion Joint Considerations for Buildings" (available in the Archives section at **www.modernsteel.com**) provides further information.

Given the possibility that the movement in the slot could damage the bolt, it seems your concerns with the bolt damaging the fire coating could be valid. However, it may not be the only problem with the proposed detail.

Carlo Lini, P.E.

Stiffness Considerations for Fully Restrained Connections

I have several questions related to stiffness requirements for fully restrained moment connections:

1. **Wouldn't the presence of many connection types, even those traditionally used as moment connections, cause a reduction in stiffness below that of the beam (e.g., bolted or welded flange plate moment connections and extended end-plate moment connections)?**
2. **Is it necessary to explicitly account for this decreased stiffness in practice?**
3. **Can a flush end plate moment connection be used as a fully restrained connection?**
4. **Section J6 in the AISC *Specification* states: "Groove-welded splices in plate girders and beams shall develop the nominal strength of the smaller spliced section. Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice." The provision for groove-welded splices would seem to ensure sufficient strength and stiffness. Why is the same requirement not applied to other configurations of splices?**

I have addressed each of your questions below.

1. Yes, it is likely that the stiffness would be lower local to the connection.
2. Generally, no. As long as the connection can be considered fully restrained, then there is no need to account for the reduced stiffness.

   The Commentary to Section B3.6 states: "In many situations, it is not necessary to include the connection elements as part of the analysis of the structural system. For example, simple and FR connections may often be idealized as pinned or fixed, respectively, for the purposes of structural analysis. Once the analysis has been completed, the deformations or forces computed at the joints may be used to proportion the connection elements… For simple and FR connections, the connection proportions are established after the final analysis of the structural design is completed, thereby greatly simplifying the design cycle."

The *Specification* provides general requirements for fully restrained connections in two different sections. Section B3.6b states: "A fully restrained (FR) moment connection transfers moment with a negligible rotation between the connected members. In the analysis of the structure, the connection may be assumed to allow no relative rotation. An FR connection shall have sufficient strength and stiffness to maintain the angle between the connected members at the strength limit states." Section J1.3 states, "End connections of restrained beams, girders and trusses shall be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connections. Response criteria for moment connections are provided in Section B3.6b." Both of these criteria mention stiffness. In practice, stiffness is often not explicitly checked but rather judged by inspection.

The definition of a fully restrained connection (or Type 1 connection, as it was once called) has varied some with time, as has the guidance related to this topic. The Commentary to the 2010 *Specification* provides a definition based on stiffness: "If $K_SL/EI \geq 20$, it is acceptable to consider the connection to be fully restrained (in other words, able to maintain the angles between members)." K_S is the secant stiffness of the connection at service loads. Relative to strength, it states: "The strength of a connection can be determined on the basis of an ultimate limit-state model of the connection, or from a physical test. If the moment-rotation response does not exhibit a peak load then the strength can be taken as the moment at a rotation of 0.02 rad (Hsieh and Deierlein, 1991; Leon et al., 1996)."

AISC Design Guide 16: *Flush and Extended Multiple-Row Moment End-Plate Connections* (a free download for AISC members at **www.aisc.org/dg**) refers to criteria related to both strength and stiffness: "For beams, guidelines have been suggested by Salmon and Johnson (1980), and Bjorhovde, et al. (1987, 1990), to correlate M-θ connection behavior and AISC construction type. Traditionally, Type 1 or FR connections are required to carry an end moment greater than or equal to 90% of the full fixity end moment of the beam and not rotate more than 10% of the simple span rotation (Salmon and Johnson 1980)." It also suggests a change might have been in the works when the design guide was being written: "More recently, Bjorhovde, et al. (1987, 1990) has suggested rotation criteria as a function of the connected beam span." Such criteria are now given in the *Specification*, as indicated above.

There is generally some correlation, though not direct, between strength and stiffness. Often, when designing for a specified strength, you get the stiffness for free, so to speak. This is not always the case, so some care must be exercised. Moment connections should look like they have significant rotational stiffness. If there is doubt, then more rigorous analysis should be conducted.

3. Yes, though this is the one connection type for which AISC provides a formal adjustment to account for the inherent flexibility of the connection. Relative to flush end-plates AISC Design Guide 16 states: "For FR rigid frame construction, the required factored moment, M_u, must be increased 25% to limit the connection rotation at ultimate moment to 10% of the simple span beam rotation. Therefore, the factor $\gamma_r = 1.25$ is used in the procedure for the flush connection plate design."

   For the conditions addressed in Design Guide 16, the authors use a 25% increase in the demand to "limit the connection rotation at ultimate moment to 10% of the simple span beam rotation," which was taken to be the criterion to consider full fixity in the model. (See Answer 2 for references.) The 25% increase seems to come from the early work of Thomas Murray at the University of Oklahoma; in that work it was applied as a factor of 1/0.8.

   The following reports seem to be pertinent:
   - "Analytical and Experimental Investigation of Stiffened Flush End-Plate Connections with Four Bolts at the Tension Flange" Report No. FSEL/MBMA 84-02, September 1984 by Hendrick, Kukreti, and Murray seems to suggest that the strength of the end plate be reduced by a factor of 0.75 to ensure Type I (fixed) behavior.
   - "Unification of Flush End-Plate Design Procedures" Report No. FSEL/MBMA 85-01, March 1985 by Hendrick, Kukreti, and Murray uses the 1/0.8 factor, which is equal to the 1.25 used in the Design Guide. Section 4.2 provides some further discussion related to the stiffness of these connections.

   Both of the reports and others related to end plate moment connections can be downloaded at **tinyurl.com/OUreports**.

   Thomas Murray continued working on end-plate moment connections at Virginia Tech, so you can find further information there as well.

4. The J6 provision reflects a practical, not a theoretical, consideration. It is intended to address concerns that during a retrofit or evaluation, a groove-welded butt splice might be overlooked if it were covered by fireproofing or even paint. Other splices would involve additional plates and/or bolts, which would be more easily seen during a survey of the structure. Requiring the groove-welded splices, to develop the full strength of the member ensures that even if they are missed, they will perform sufficiently. The requirement is not related to stiffness considerations.

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