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DEVELOPING M_p

Does an *R*=3 directly welded flange moment connection do it?

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CHANGE TO AN AUTHORITATIVE work on a seemingly unchangeable practice or engineering "norm" can understandably be met with confusion.

Here's an example: A common *R*=3 moment connection is the directly welded flange connection shown in Figure 1. These connections are designed based on the assumption that the web connection carries the entire shear force and the moment is resolved into a couple with a lever arm equal to the distance between the flange centroids. This assumption was clearly stated in the 9th Ed. ASD and the 1st Ed. LRFD *Manuals of Steel Construction*.

With the format change in the 2nd Ed. LRFD *Manual*, further explanation of the behavior of these connections, with references to the research, was included. However, while no change had occurred in the underlying philosophy for designing these connections, all reference to the plastic moment of the beam had been removed from the discussion in the 3rd Ed. LRFD *Manual*, and only an allowance for some inelastic deformation and a reference to some of the research remained.

This change has led to some confusion regarding these connections. What was once a commonly held truth—that these connections could develop the design strength of the beam through the flanges alone—is now frequently questioned and disputed. Our hope here is to reintroduce some age-old wisdom to today's engineers.

As stated previously, it is assumed that the flexural stresses over the entire cross section can be safely carried by the flanges, as shown in Figure 2. If the beam is loaded to its plastic moment capacity, the axial stress in the flange is greater than its yield strength, due to the bending stress in the beam web. However, tests have shown that these connections can carry moments greater than the plastic capacity of the beam, even when combined with shear loads approaching the shear yield strength of the beam.

There have been many test programs with directly welded moment connections loaded to failure under monotonic and cyclic loading (see sidebar on p. 19). The specimens generally had a final failure mode

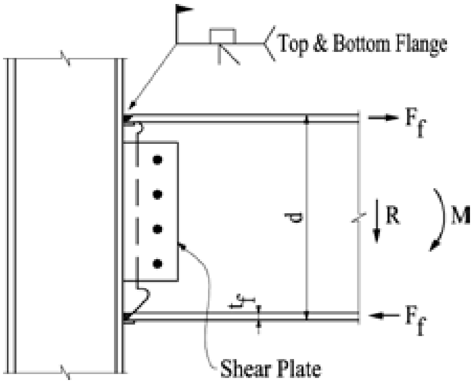
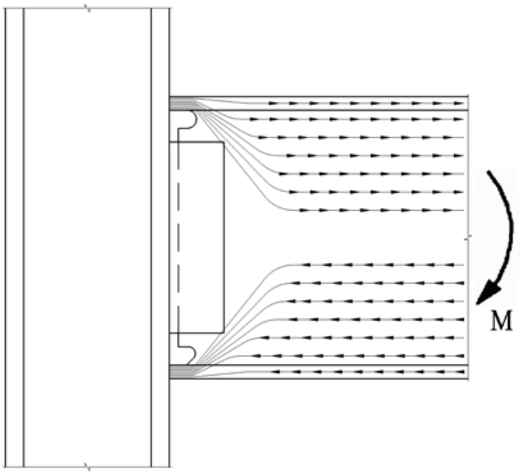


Fig. 1: Directly welded moment connection.

Fig. 2: Idealized stress flow.



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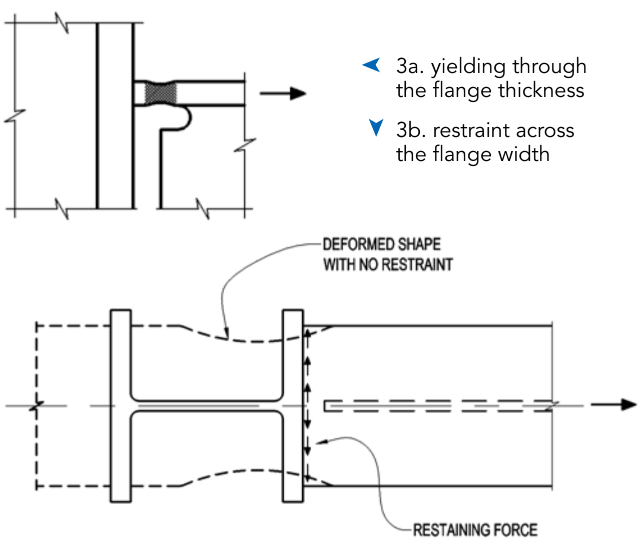
of tension flange rupture. The applied moment consistently exceeded the plastic moment capacity of the beam calculated with the yield strength from tensile coupon tests. Strain hardening is the reason provided by most researchers to explain the ability of the flanges to carry loads exceeding their yield strength; however, several specimens were loaded well in excess of the measured tensile strength.

While it is clear that strain hardening of the beam flanges plays a significant role in the performance of directly welded moment connections, another important factor is the transverse restraint of the flange at the column face. Generally, the flange is free to deform through the thickness as shown in Figure 3a. However, deformation across the width of the flange is restrained as shown in Figure 3b.



3a. yielding through the flange thickness

3b. restraint across the flange width

Fig. 3. Restraint at the beam flange.

The triaxiality increases with the level of restraint, which results in increased strength and decreased ductility. Figure 4 shows the stress-strain curves for tension members with various levels of restraint. If the member is restrained in one direction, the yield and tensile strength is higher than that of a uniaxially loaded member, but there is a decrease in ductility. Members restrained in two directions have a much higher strength, but very limited ductility.

Of course, the preceding discussion is over-simplified to illustrate the effect of restraint on the performance of moment connections. One test (Shafer et al; see sidebar) showed that the level of triaxiality is actually non-uniform across the width and through the thickness of the beam flange. It also showed that the experimental rupture load increased with the level of triaxiality in the beam flange, and determined that the flanges ruptured at stresses between 120% and 170% of the tensile strength.

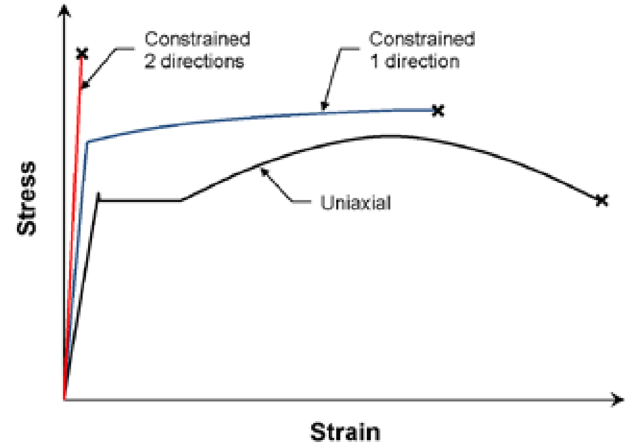


Fig 4. Stress-strain curves for steel under various levels of restraint.

Web Connection

A common misconception is that slip-critical joints are necessary at the web connection to limit the vertical movement of the beam after the flanges have been welded. This would presumably prevent secondary bending and shear stresses in the beam flange in the area between the column flange and the weld access hole. However, the tests showed no decrease in strength when bearing joints were used. Furthermore, most of the tests with slip-critical joints had slip occur at some point in the testing, effectively rendering the web connection a bearing joint anyway.

An additional advantage of using bearing joints is the potential for reduced cost of installing the bolts and preparing the faying surfaces. In most bearing joints, the bolts are only required to be snug tight, which takes less time to install and inspect than the pretensioned bolts that are required in slip-critical joints. Bearing joints will also eliminate the cost of blocking paint at the faying surfaces or wire brushing at galvanized faying surfaces that may be required for slip-critical joints.

Testing has shown that web connections perform well with either standard holes or horizontal slots. An advantage of using short slots is the ability to facilitate shop and erection tolerances. A further practical consideration is weld shrinkage. Typical complete-joint-penetration groove welds in a directly welded flange connection can be expected to shrink about $\frac{1}{16}$ in. when the weld cools and contracts. For beams with thicker flanges, shrinkage could be around $\frac{3}{16}$ in. For this reason, it is usually advisable to use short slotted holes in the shear connection and leave the bolts snug tightened to better accommodate the weld shrinkage.

Physical tests have shown that the plastic moment of the beam can be developed with sufficient inelastic rotation and deformation capacity through the beam-flange-to-column connection. Therefore, in *R*=3 applications, the moment can be resolved into an effective tension-compression couple acting as axial forces at the beam flanges. This apparent increase in strength over the prediction of elastic theory is due to strain hardening and transverse restraint of the beam flange at the column face. MSC

Moment Connections, Tested

Several tests have been performed, and subsequent papers/reports written, on directly welded moment connections. Here are ten:

Blackman, B. and Popov, E.P. (1995), "Studies in Steel Moment Resisting Beam-to-Column Connections for Seismic-Resistant Design," Report No. UCB/EERC-95/11, Earthquake Engineering Research Center, October.

Chen, W.F. and Patel, K.V. (1981), "Static Behavior of Beam-to-Column Moment Connections," *Journal of the Structural Division*, ASCE, Vol. 107, No. ST9, September.

Engelhardt, M.D. and Husain, A.S. (1992), "Cyclic Tests on Large Scale Steel Moment Connections," Report No. PMFSEL 92-1, Phil M. Ferguson Structural Engineering Laboratory, The University of Texas at Austin, June.

Huang, J.S., Chen, W.F. and Beedle, L.S. (1973), "Behavior and Design of Steel Beam-to-Column Connections," WRC Bulletin 188, Welding Research Council, October.

Krawinkler, H. and Popov, E.P. (1982), "Seismic Behavior and Design of Moment Connections and Joints," *Journal of the Structural Division*, ASCE, Vol. 108, No. ST2, February.

Popov, E.P. and Tsai, K.C. (1989), "Performance of Large Seismic Steel Moment Connections Under Cyclic Loads," *Engineering Journal*, AISC, Second Quarter.

Popov, E.P., Amin, N.R., Louie, J.C. and Stephen, R.M. (1986), "Cyclic Behavior of Large Beam-Column Assemblies," *Engineering Journal*, AISC, First Quarter.

Popov, E.P. and Stephen, R.N. (1970), "Cyclic Loading of Full-Size Steel Connections," Report No. EERC 70-3, Earthquake Engineering Research Center, July.

Shafer, B.W., Ojdrovic, R.P., and Zarghamee, M.S. (2000), "Triaxiality and Fracture of Steel Moment Connections," *Journal of Structural Engineering*, ASCE, Vol. 126, No. 10, October.

Stojadinoviv, B., Goel, S.C., Lee, K.H. and Choi, J.Y. (2000), "Parametric Tests on Unreinforced Steel Moment Connections," *Journal of Structural Engineering*, ASCE, Vol. 126, No. 1, January.