One Engineer's Opinion

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Q. *Is it permissible, under the AISC Specification, and in accordance with good engineering practice, to provide a splice for shear only (web connection) at the point of contraflexure in a continuous beam, whether designed elastically or plastically?*

A. There is certainly no prohibition, explicit or implicit, in the Specification on the type of splice detail in question. It is fairly common practice to use such a splice, especially in plastically designed roof systems. This type of design and construction technique is also the essence of the cantilever relief method of design and construction which equalizes maximum positive and negative moments.

In order to conform to good engineering practice, one must examine the service conditions, joint detail and the relationship between the mathematical design model and the actual service requirements.

The type of shear splice under discussion must fulfill the following structural requirements:

1. Resist, adequately, all moments, shears and thrusts that occur at the splice section due to any load imposed on the structure, up to ultimate load, while at the same time being capable of undergoing the resulting deformations.
2. Allow the redistribution of bending moments, and consequent migration of inflection point, as each subsequent plastic hinge forms, up to ultimate load.

The latter requirement, while seemingly applicable only to plastic design, is also really the implied rationale behind elastic design in steel.

The pertinent portions of the AISC Specification in this regard are Sect. 1.10.8 and Sect. 2.7. Section 2.7 states: *All connections, the rigidity of which is essential to the continuity assumed as the basis of the design analysis, shall be capable of resisting moments, shears and axial loads to which they may be subjected by ultimate loading.* Further, the Commentary on Sect. 2.7 says: *Connections located outside of regions where hinges would have formed can be treated in the same manner that similar connections designed in accordance with the provisions of Part 1 would be treated.*

If the several span lengths in a plastically designed system are such that mechanisms could form in the spans considered for splice locations, at ultimate loading, and "all spans fully loaded" is assumed to be the design condition, splices located at ultimate-load contraflexure points would not, at any time, be subject to any applied moment, provided that the connection (splice) is itself sufficiently flexible to accommodate the required rotation at service loading. The same is true of cantilever-and-suspended-span systems with connections located so as to require the same bending strength at interior support points as at midspan.

In both of these design concepts the necessary rotation capacity is generally assumed to be adequately provided by connections consisting of bolted web splice plates (Fig. 1). Such connections utilize common bolts, in holes with nominal $\frac{1}{16}$-in. clearance, subject only to shear. Another type of connection used consists of pairs of web framing angles installed on both sides of the splice point with their outstanding legs field bolted (Fig. 2). The former type of connection was used in the design example on page 27 of the AISC Plastic Design Manual.[1]

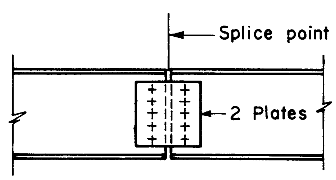

Figure 1

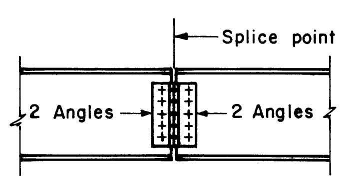

Figure 2

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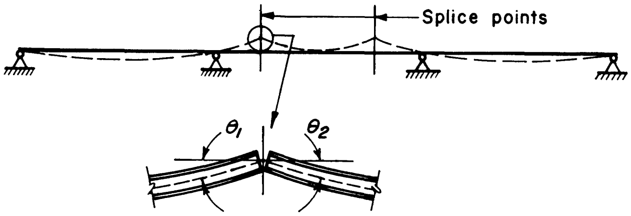

Figure 3

If the type of connection used, located at ultimate-load contraflexure points, lacks rotation capacity it must be able to resist the moment imposed upon it at service loading due to the continuity developed because of its rigidity. For construction utilizing fully rigid connections and designed for uniform live load, this moment can be calculated, by rational analysis, to be equal to one-third of the elastic bending strength of the connected beams.

This is an extremely important consideration. There have been reports of distress in "so-called" shear splices due to inadequate rotation capacity. In one such case the suspended spans were fitted with standard framing angle connections. The outstanding legs of the angles were field bolted to end-plates which were welded to the extremities of the cantilever span. A compositely connected concrete slab was cast on the steel framing. It is likely that temporary shores were utilized. It was observed that the bottom bolt of the suspended beam web-connection was sheared off with the application of dead load. It is apparent that the end-plate detail was so rigid as to induce bending moment at the splice point that could not be accommodated by the framed connection on the web. It should be noted that at the splice point (Fig. 3) the connection is called on to accommodate a two-fold rotation. One portion is due to the simple-beam rotation of the suspended span. The second is the cantilever end rotation, opposite in sense to the simple beam rotation. The need for flexibility, or conversely for adequate moment capacity, in this connection becomes obvious. It is of interest to note that adequate rotation capacity of a standard framing angle connection is achieved principally by distortion in the outstanding angle legs. Such distortion occurs in the tension area of the induced bending moment. In the case noted above, the concrete slab would have inhibited such distortion at the top of the connection. The failure described previously was probably the result of insufficient ability to distort above the center of rotation (which would have shifted upward significantly), so that to satisfy compatibility the movement (or distortion) was accomplished by shearing the bottom bolt. Based on the considerations presented above, the following can be concluded:

1. For those situations where sufficiently flexible splices are located at ultimate-strength contraflexure points and checkerboard live loading is not a practical consideration, as in most roof construction, beam splices should be designed for shear alone, as illustrated in the AISC Plastic Design Manual.
2. Rigid-type connections, similarly located when checkerboard loading is not involved, should be capable of developing one-third the bending strength of the connected beam. If this requirement necessitated splicing the flanges at all, then the present Specification Sect. 1.10.8 would require a splice to develop a minimum of 50 percent of the capacity of web and flanges. In this regard, it should be noted that the intent of Sect. 1.10.8 is *not* to require that all splices be proportioned for 50 percent of the capacity of the entire section. Rather, the requirement relates to the "material spliced." That is, if a web splice is required, the splice should be capable of developing one-half the capacity of the web, as a minimum. Likewise if the function of the joint is to unite both web and flanges, a minimum of one-half the capacity of web and flanges must be developed by the splice.
3. For situations where checkerboard loading *is* a practical consideration, as in warehouse floors, and flexible connections are used to splice beams at points between supports, the required bending strength of these beams should be determined by elastic design methods (AISC Specification, Part 1) based upon the selected splice location. For a further discussion refer to a recent paper by Hart and Milek.[2]
4. If rigid-type connections are used and checkerboard loading is a practical consideration, these splices should be proportioned for the largest moment to which they may be subject.

REFERENCES

1. Plastic Design in Steel, *American Institute of Steel Construction, New York, N. Y., 1959.*
2. *Hart, W. H. and Milek, W. A., Jr.* Splices in Plastically Designed Continuous Structures, *AISC Engineering Journal, April, 1965.*