Are Tier Building Column Splices Designed?

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THE DESIGN of splices for multistory building columns sometimes presents an apparent conflict between theory and practice. Examination of several steel design textbooks in current use shows a wide divergence of solutions. Most disconcerting is one statement that the AISC Specification does not spell out requirements for splicing joints of compression members and, therefore, to design the splice, reference must be made to a specification for designing railroad bridges. Even if the AISC Specification did not adequately cover the problem (and it does), the employment of a specification not applicable to buildings can hardly be considered a satisfactory substitute. Misleading information such as the foregoing may be one reason for inefficient, uneconomic details that often appear on design drawings.

What are the facts regarding AISC Specification requirements for splicing multistory columns and how are they commonly designed? For nearly 35 years the AISC Specification has contained all the necessary requirements for properly designing splices in compression members, including tier building columns. In effect the provisions,* unchanged over the years, are:

(a) Where tier building columns are finished to bear, there shall be sufficient rivets, bolts, or welds to hold all parts securely in place.

(b) The joint shall be proportioned to resist any tension that would be developed by specified lateral forces acting in conjunction with 75 percent of the calculated dead load stress and no live load.

AXIAL LOADS

Requirement (a) recognizes the complete transfer of direct load through bearing on the finished surfaces. A physical connection between the upper and lower column shafts would seldom be necessary were it not for the obvious considerations of safety during erection and structural integrity of the finished structure. There evolved nominal splice plates, standardized years ago for general application. Their use is not mandatory; simply a recommended practice covered in the AISC publication Structural Steel Detailing. Tabulations and data covering the complete range of common column sections and fasteners is given in the Appendix to that publication.

When the column joint in the typical building is finally secured, fasteners tightened and welds in place, the joint is usually only partially loaded. Naturally, the splice material and its fasteners share some of the remaining dead load and the subsequent live loads. Admittedly, as several authors have stated, the distribution of forces are complex. They resolve the problem by assigning various factors or percentages to imaginary variations of contact bearing, then design the splice plates for the assumed share of the axial loads. However, in light of the long standing satisfactory experience with nominal sized splices, it becomes evident that some slippage of fasteners and minute deformations result in substantially all axial loads being transferred through bearing on the finished contact surfaces.

The current (April 17, 1963) AISC Specification permits a unit bearing stress F_p of $0.90F_y$ on finished surfaces. Thus, for A36 steel, F_p is equal to 33,000 psi—a very much greater stress than that permitted for axial or bending forces on the column shaft. Filler plates, when used in conjunction with splice plates, are sometimes needed to provide additional bearing area, in which case the finished edge of the filler is developed on the basis of F_p. Typical examples are Cases II and V in the Appendix to Chapter 11 of Structural Steel Detailing.

LATERAL FORCES

Most tier building columns are subjected to lateral forces due to wind. In the usual analysis a point of contraflexure is assumed at mid-height of the column between floors. Since the typical splice is located within about 2 ft of the floorbeam connection, the bending moment is near the maximum; therefore the splice must be capable of resisting a moment that may produce tension in the splice plates. Obviously, moment that produces compression is not a factor.

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* *See Sect. 1.15.8 of the April, 1963, AISC Specification.*

Requirement (b) permits using 75 percent of the permanent dead load compression stresses to offset the tensile bending stresses. On this point four of the five textbooks examined ignored the presence of dead load stresses; naturally, a splice designed for 100 percent of the tensile moment will be ultra-conservative and often wasteful. In one textbook the author suggested as a "practical" compromise that the size of splice plates be made equivalent to the column flange. This could be baffling to an engineer detailing a column splice for a 14WF "jumbo" column whose flange is $4\frac{1}{2}$ in. thick.

The design procedure conforming to AISC requirements is relatively simple for the case of axial loads and wind moment. The effective compressive stress due to dead load is $f_c = 0.75\ P_D/A_c$, where A_c is the gross area of the upper shaft. The effective compressive load on the area of one flange of the upper column shaft is

$$C = A_f \times f_c$$

(Note that the small contribution of the column web is neglected for simplicity.)

Maximum bending stress $f_b = M/S$, where M is the wind moment and S is the section modulus. The effective tension load on one flange is

$$T = A_f \times f_b(\text{avg.})$$

where f_b (avg.) is the average bending stress across the flange.

When $C \geq T$, use nominal splice plates as given in Structural Steel Detailing. When $C < T$, check the tensile capacity inherent in the nominal splice plates that are tabulated for the particular columns. Their capacity will be the lesser of (a) $A_n \times 1.33F_t$, where A_n is the net area of the plate, or (b) the strength of the fastener group. When insufficient to develop load $T - C$, increase size of plate and add fasteners to suit.

More often than not, nominal sized splices, including the partially butt-welded type, will be adequate for resisting the net wind moment stresses in the typical building.

WIND SHEAR

Most textbooks show splice plates on the column webs as if they are a necessary part of specification requirements. Horizontal forces are resisted by the friction obtained on the finished bearing surfaces and, in addition, by the bending resistance of the flange splice plates or in the case of a butt-welded joint, the shearing resistance of the welds. Since this provides ample resistance to horizontal shear in the typical building, the AISC detailing textbook does not suggest the use of web splice plates. Of course, there may be an unusual case of wind shear where rational analysis of the selected column splice shows a need for additional splice material on the column webs. Such information should appear on the design drawings; otherwise, in accordance with normal practice, column webs will not be spliced.

JUMBO SECTIONS

Until recently the heaviest rolled steel column section was the 14WF426. Large American steel mills now roll 14-in. nominal size sections in weight per foot up to 730 lbs. The question of what size nominal splice plates has arisen because this information is not in the current AISC detailing textbook. It may be possible to use the same splice details as now given for 14WF426, or to beef-up their fasteners for greater moment resistance, at least for the lighter sections in the jumbo series. On the other hand, slightly larger plates and increased moment capacity may be desirable for the heaviest sections. The problem is under study by the Institute and the information is expected to be included in a future edition of Structural Steel Detailing.