A Cautionary Note on Beam Copes

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Routinely in steel construction it is necessary for many beams to be coped, at either the top or bottom flange, in order to provide clearance around the flanges of the beams which provide support. In by far the majority of cases, hot-rolled laterally supported beams with span-depth ratios in the range of 20 to 24 are involved. Generally shear capacity of the beam web is not a critical factor, the size of the copes are of reasonable proportions, and no particular problems are involved in providing adequate strength for the beam end reaction.

There are instances, however, where special consideration must be given to the effect of copes; in recognition of such situations the following warning appears on page 4-113 of the Seventh Edition AISC *Manual of Steel Construction* and on page 4-167 of the Eighth Edition *Manual:* "Unusually long and deep copes and blocks, or blocks in beams with thin webs, may materially affect the capacity of the beam. Such beams must be investigated for both shear and moment at lines A and B and, when necessary, adequate reinforcement provided."

The infinite variety of situations that are encountered in practice make it impossible to provide all-inclusive definitive rules. However, in order to provide some guidance and examples, the AISC book *Structural Steel Detailing,* Chapter 9, provides examples of special beam connections, several of which are for coped conditions. The design recommendations in Chapter 9 direct attention to the consideration of the adequacy of the reduced web to resist shear and bending and suggest means that may be used to provide reinforcement when required. It should be noted that the examples are directed to the more usual case of hot-rolled beams with webs of stocky proportions, that is, with depth-to-thickness ratios generally in the range of 60 or less.

Practical cases may be hypothesized for which these recommendations are inadequate and involve considerations not touched upon in the examples. A case in point is coped thin web plate girders.

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The AISC Specification provisions for design of plate girders are based primarily upon the work of Basler and Thurliman, reported in the ASCE *Journal of the Structural Division,* August, 1961. It is apparent in these papers that a basic assumption of the theory for predicting the strength of a plate girder web panel is that the panels are bounded on all four sides either by girder flanges or by web stiffeners. This is emphasized in the *Commentary* to the AISC Specification, Sect. 1.10.5, which says: "Unlike columns, which actually are on the verge of collapse as their buckling stage is approached, panels of the plate girder web, *bounded on all sides* by the girder flanges and transverse stiffeners, are capable of carrying loads far in excess of their web buckling load." The obvious corollary is that for panels which are not bounded on all four sides, the allowable stress formulas (1.10-1) and (1.10-2) provided in the Specification are not appropriate and do not assure the usual factor of safety. If a cope is introduced into the end panel of a plate girder, then certainly initial assumption of the theories upon which specification provisions are based is violated to a greater or lesser degree, depending upon the size of the cope. Logically, reinforcement of the web or restoration of continuous simple support around the panel is required.

Chapter 9 of *Structural Steel Detailing* is silent on recommendations for handling such cases. Until the problem is addressed, it is recommended that coping of plate girders be avoided or, if copes are unavoidable, at least simple support be provided to the web along the coped edge. The support elements need not be of the same proportions as the girder flange, but should be adequate to restrain the coped edge of the web plate against out-of-plane buckling.

A second case in point is that the theoretical formulas for laterally unbraced beams, upon which the provisions of specification Sect. 1.5.1.4.5 are based, assume that the beam is restained in an upright position at the point of support. A laterally unsupported beam with long or deep copes, especially with both flanges coped, would be elastically restrained rather than rigidly restrained. The resistance to overturning would be dependent upon the torsional strength and stiffness of the coped section. Two potential problems would need to be considered. In the case of top flange loading which does not provide lateral support, localized distortion and twisting of the cross section within coped region could occur. In other loading cases, the lateral strength of the beam as a whole could be less than assumed by the specification provisions.

Two situations have been cited as examples in which long or deep copes may render invalid the initial assumptions for the design of the main member. There may be other cases, but these two indicate proper design of connections must extend beyond simple consideration of adequate strength to support the end reaction. Because such situations are infrequently encountered, special attention should be given to all long or deep copes to assure that problem areas are not overlooked and that all aspects of the details are rationally considered.