Suggestions for Avoiding Beam-to-Column Web Connection Failure

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The following discussion draws attention to certain connection details that may not provide the commonly assumed ductility. Such ductility is important in applications where dependence is placed upon redistribution of moments, in plastic design, and in seismic-resistant design. The potential for ductility problems increases as material thickness increases beyond that commonly encountered in building construction.

Although the AISC Specification recommends allowable stresses which provide a larger ratio of limiting stress to service stress for connectors than for the connected members, designers may not be aware of the potential for fracture resulting from the effects of localized high restraint in some connection details.

The paper is also intended to provide a qualitative "feel" for the manner in which stresses are transferred in a beam-to-column web moment connection, so that engineering judgement may be exercised in the design of such connections until the results of research in progress provide information on which quantitative provisions may be based.

American Institute of Steel Construction

As part of a research program sponsored through the Welding Research Council by the American Iron and Steel Institute (AISI) at Lehigh University to evaluate the strength of beam-to-column moment connections, large size specimens were load-tested with a W-shape beam attached to a W-shape column in such a way as to bend the column about its weak axis. Figure 1 shows the connection details. Two of the tests failed due to fracture.

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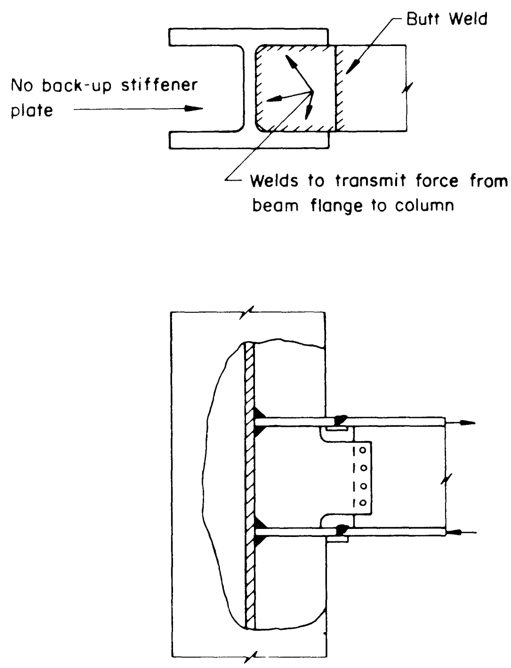


Fig. 1. Beam-to-column web connection

Similar failures occurred in connections tested by Popo and cycled from 30 to 50 times in the inelastic rang However, the authors are not aware of reports of simil failures on the first cycle of loading in other research pro grams.

Although the results of the above-mentioned tests we published,[2] attention is directed to the behavior of th specimens for those who might consider the use of simil connection details. The connections that failed were pro portioned to be "critical" throughout, and were expect to provide inelastic rotations at full plastic load.

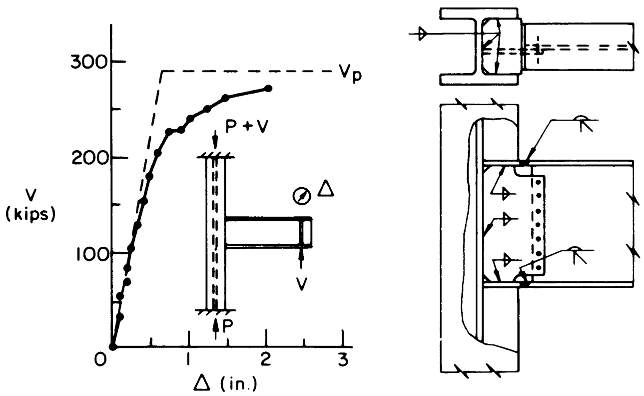
In keeping with the design of test specimens for previous beam-to-column connections, the specimens were designed to combine all the "worst possible" situations:

- The connection plate was designed to yield at the plastic moment of the beam.
- Welds were proportioned according to the specifications—with no excess over requirements.
- The loading conditions and the beam web plate connection to the column web were proportioned for full shear yield on the web plate concurrent with full plastic moment.
- The number and size of high-strength bolts provided was based on criteria for bearing-type connections.

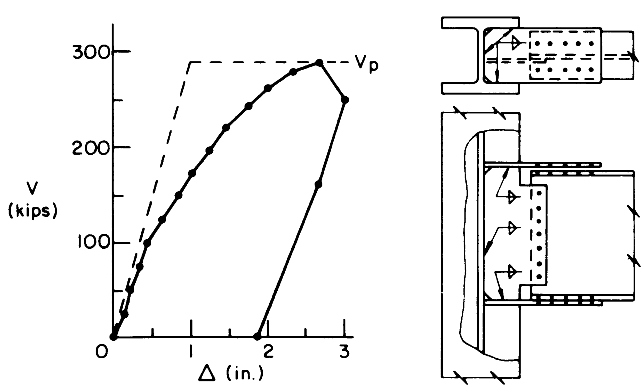
The two specimens that failed gave marginally acceptable results, but fractures occurred prior to significant inelastic rotation. Both specimens supported the service load with no distress, but the lack of deformation capacity at the loading corresponding to the plastic moment is considered unsatisfactory.

The details of the two specimens and their behavior in the load tests are shown in Fig. 2. The beam-to-column connections are such that the beam is framed into a column web using moment connection plates welded to the inside surfaces of the column flanges and to the surface of the column web. There is also a shear plate connecting the web of the beam to the column web. The moment connection plates were of the same thickness as the tension flanges of the beams. As is shown in the sketch, no back-up stiffener was provided on the side of the column web opposite to the connection plate.

At essentially the load corresponding to the plastic moment, fractures occurred in the tension moment connection plates of both specimens as shown in Fig. 3. Fracture initiated in the connection plate at the termination of the weld

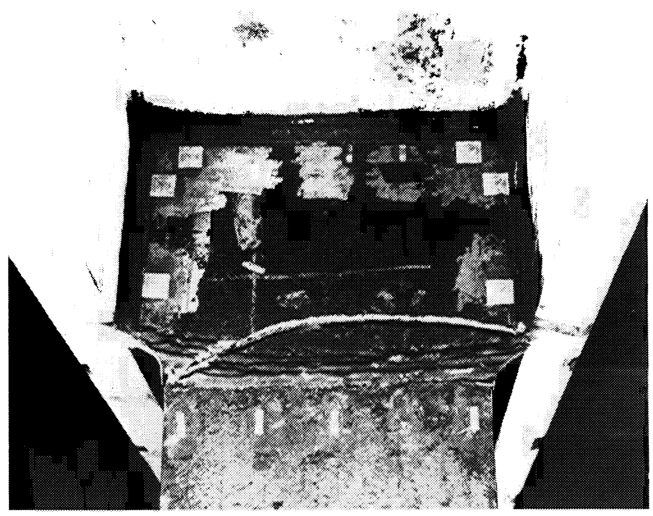


(a) Test 14-1

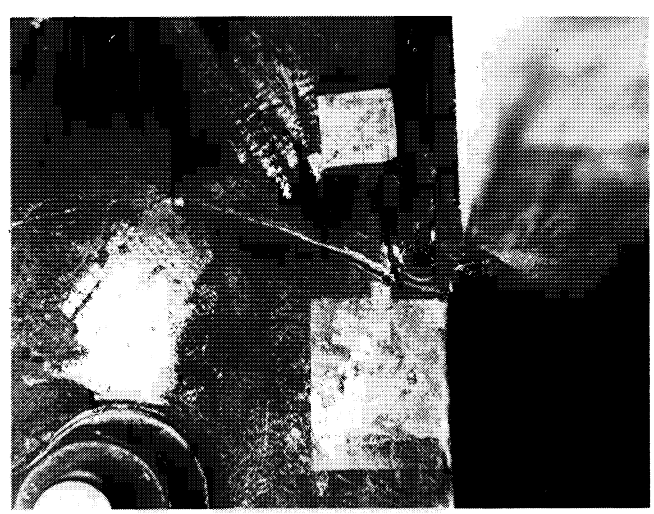


(b) Test 14-3

Fig. 2. Load-deflection curves of beam-to-column web connections



(*a*) *Test 14-1*



(*b*) *Test 14-3*

Fig. 3. Fractures in beam-to-column connections

at the tip of the column flange in Test 14-3, and at the end of the butt weld connecting the beam flange to the connection plate in Test 14-1. In both specimens the fracture propagated in a brittle manner. Samples of the connection plates were subjected to chemical analysis, Rockwell hardness tests, Charpy impact tests, and metallographic examinations.[3] The plate from Test 14-1 met the chemical requirements of ASTM A572 Grade 50 steel, had a Rockwell hardness HRB-90, and had a Charpy impact value of 15 ft-lb at 29°F (specimen oriented parallel to the tensile strain).

Metallographic examination showed the rolling direction of the plate from Test 14-1 to be perpendicular to the direction of tensile strain, a conclusion supported by a Charpy impact value of 31 ft-lb at 40°F when the specimen was transverse to the tensile strain. The plate from Test 14-3 met the chemical requirements of ASTM A588 Grade B steel, had a Rockwell hardness HRB-84, and a Charpy impact value of 15 ft-lb at minus 50°F with the specimen parallel to the tensile strain. It was not possible to determine the rolling direction of the connection plate from Test 14-3, but it was believed that it was perpendicular to the direction of tensile strain. However, Charpy impact values of 12 ft-lb at minus 30°F with the specimen transverse to the direction of tensile strain, by their lower value, suggest that the rolling direction was parallel to the direction of tensile strain.

These results indicate that the connection plates had normal strength and ductility, and metallographic examination of welds did not reveal any flaws. According to calculations, adequate weld metal was applied at the column flange and web to transmit the needed force.

WHY THE FAILURES OCCURRED

Calculations indicate that the failures occurred due to high strain concentrations. These concentrations are: (1) at the junction of the connection plate and the column flange tip and (2) at the edge of the butt weld joining the beam flange and the connection plate.

Figure 4 illustrates the distribution of longitudinal stress across the width of the connection plate and the concentrations of stress in the plate at the column flange tips. It also illustrates the uniform longitudinal stress distribution in the connection plate at some distance away from the connection. The stress distribution shown represents schematically the values measured during the load tests and those obtained from finite element analyses. (σ_o is a nominal stress in the elastic range.) The results of the analyses are valid up to the loading that causes the combined stress to equal the yield point. Furthermore, the analyses indicate that localized yielding could begin when the applied uniform stress is less than one-third of the yield point. Another contribution to the non-uniformity is the fact that there is no back-up stiffener. This means that the welds to the web near its center are not fully effective.

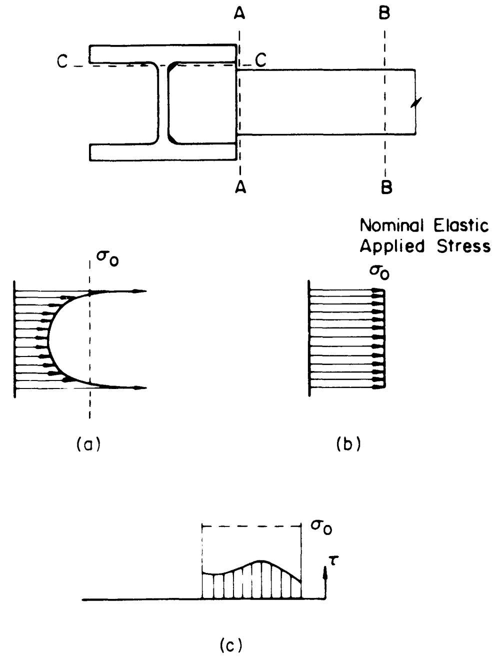


Fig. 4. Stress distribution: (a) longitudinal stresses on Sect. A-A; (b) longitudinal stresses on Sect. B-B; (c) shear stresses on Sect. C-C

The longitudinal stresses in the moment connection plate introduce strains in the transverse and the through-thickness directions (the Poisson effect). Because of the attachment of the connection plate to the column flanges, restraint is introduced and this causes tensile stresses in the transverse and the through-thickness directions. Thus, referring to Fig. 4, triaxial tension stresses are present along Section A-A, and they are at their maximum values at the intersections of Sections A-A and C-C. In such a situation, and when the magnitudes of the stresses are sufficiently high, materials that are otherwise ductile may fail by premature fracture.

HOW TO IMPROVE DUCTILE ACTION

Until the results of research in progress provide more definitive information, the following suggestions are intended to help the designer improve the connection performance, especially the rotation capacity:

1. Use connection plates that are somewhat oversized (Fig. 5a). The connection plates used in the tests were of the same thickness and approximately the same width as the tension flange of the beam. Thus, while the nominal tension stress (P/A) was essentially the same as in the beam flange, the relatively short edge attachment for transferring the force by shear to the column flanges resulted in quite significant non-uniformity in the tensile stresses across the plate at

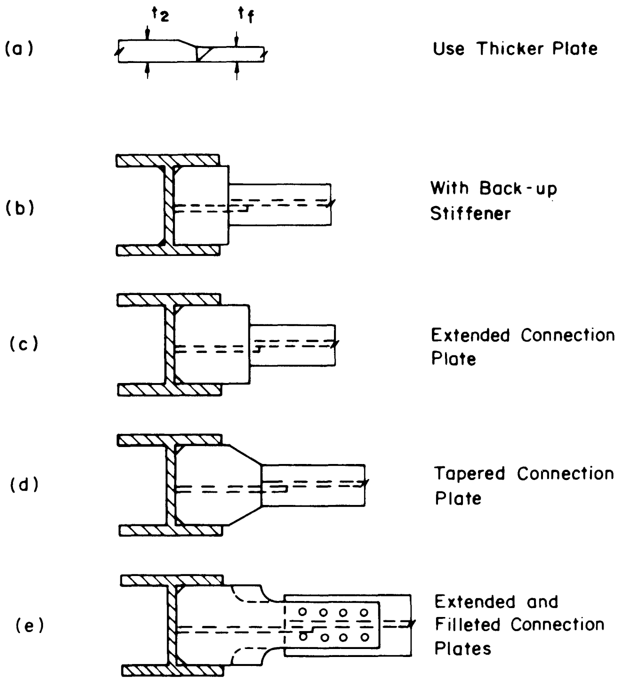


Fig. 5. Possible approaches, for use individually or in combination, for improving performance of tension flange connections to column webs

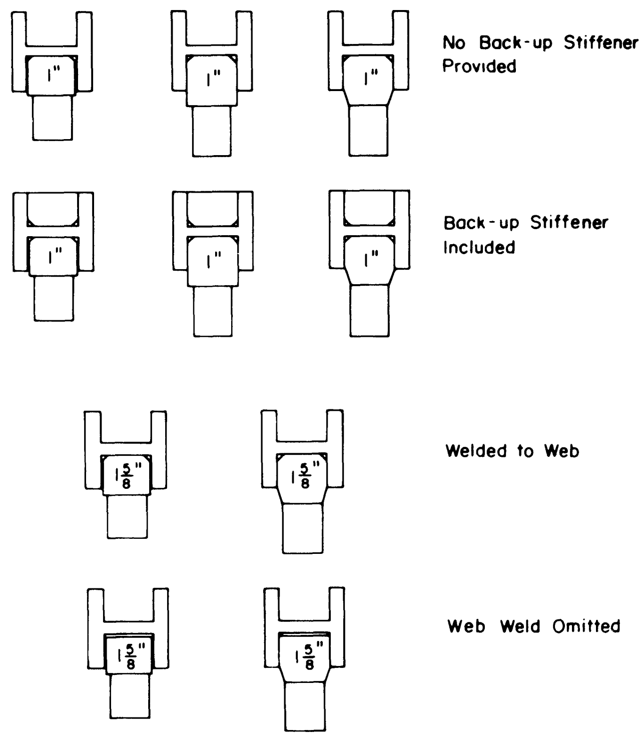


Fig. 6. Planned test specimens

the region of the column flange tips (the shear lag effect).

2. Use a back-up stiffener (Fig. 5b). Limited finite element analyses indicate that the stress concentration at the flange tip is reduced by at least one-third when a back-up stiffener plate is used.
3. Extend connection plate so as to move beam flange butt welds away from the welds to the column flanges to avoid intersecting welds and associated residual stresses (Figs. 5c and 5d).
4. Use a tapered plate (Fig. 5d) to decrease the stress concentration at the critical section.
5. Provide a reduced width in the connection plate between the connection to the beam flange and the connection to the column flanges at some distance away from both connections (Fig. 5e). This is the old "top plate" idea that has provided consistently satisfactory designs, recognizing that the moment imposed on the connection will be somewhat less than the plastic moment of the beam.
6. Insofar as practicable, insure that the beam web force is delivered directly to the column, and not partially through the flange plate. Use of "bearing connection values" and thus higher bolt loads for web connections increases the possibility of slip at high loads and transfers shear force into the tension flange, again increasing the connection plate force.

Current research sponsored by the National Science Foundation and the AISI is examining details of the types shown in Fig. 6. They are tested in tension to simulate beam flange-to-column connections. These specimens have been designed after theoretical analysis, including elastic-plastic finite element studies. The specimens were designed in an effort to develop connection details having adequate rotation capacity at the plastic moment and possibly eliminate what many have felt is unnecessary welding of back-up plates.

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