# Achieving Ductile Behavior Of Moment Connections

## The results of recent tests suggest that weld metal toughness holds a key to achieving better moment connection performance in earthquakes

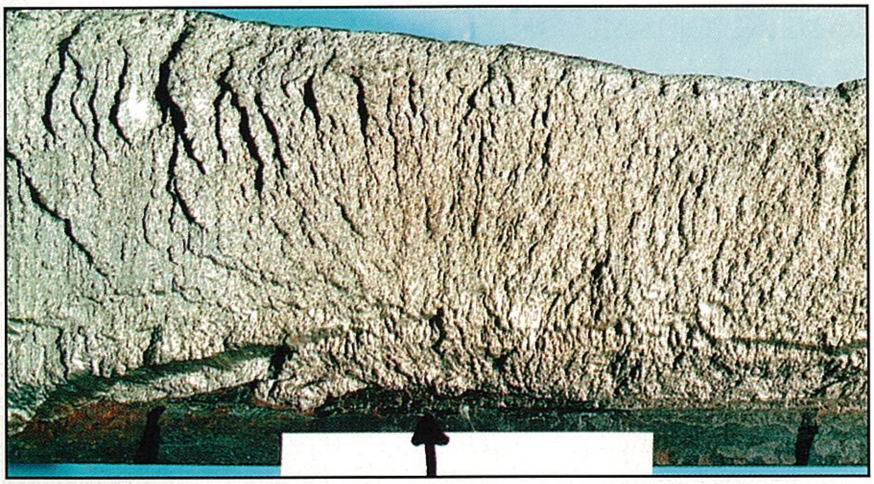


*Figure 1:* *(above) Fracture surface of column divot type fracture. The brittle fracture initiates from a flaw at the weld root caused by inadequate root penetration and the back-up bar lack of fusion.*

*Figure 2:* *(opposite top) Simulated beam flange to column flange test specimen*

*Figure 3:* *(opposite middle) Weld metal charpy V-notch test results*

*Figure 4:* *(opposite bottom) Dynamic load vs. crosshead displacement for simulated beam flange to column tests*

**By Eric J. Kaufmann, Ming Xue, Le-Wu Lu and John W. Fisher**

MUCH HAS BEEN WRITTEN ABOUT THE DAMAGE TO CONNECTIONS IN WELDED STEEL MOMENT FRAMES observed after the 1994 Northridge earthquake. Preliminary investigations of the fractured connections, which were carried out soon after the earthquake, revealed in most cases very low fracture toughness of the weld metal used in making the flange welds as well as defects and crack-like conditions from the back-up bar. The mechanical and chemical properties of the structural steel, however, did meet the ASTM requirements. Several research programs, which are ongoing in the Center for Advanced Technology for Large Structural Systems (ATLSS) at Lehigh University, are examining, among other issues, the effect of weld metal toughness and fabrication defects on the seismic performance of moment connections.

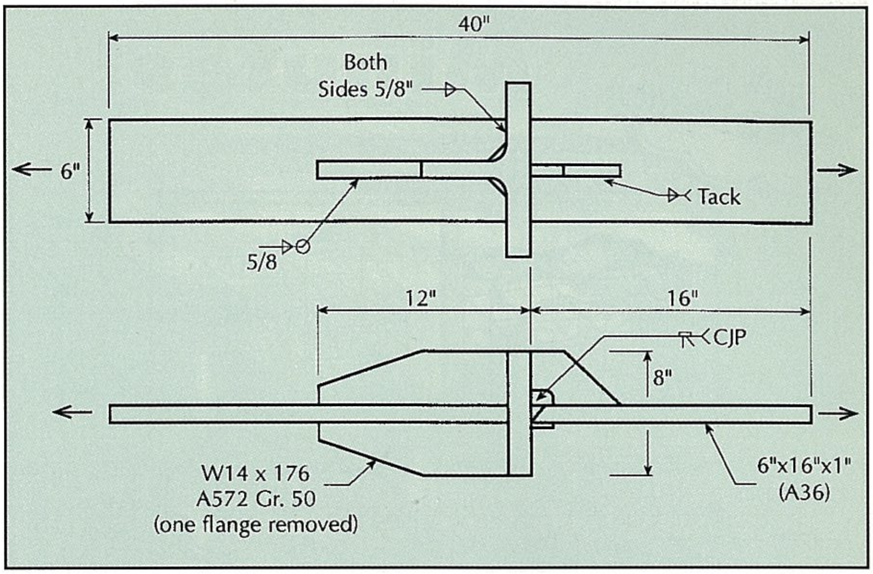
The programs include: (1) examination of the crack surfaces of some damaged connections removed from buildings in the Los Angeles area as well as of laboratory tested connections; (2) dynamic tension tests of simulated beam flange-to-column flange connections; and (3) dynamic cyclic tests of large-size beam-to-column connections. This paper is a brief report of some of the findings of these studies, based on tests of a limited number of weld metals.

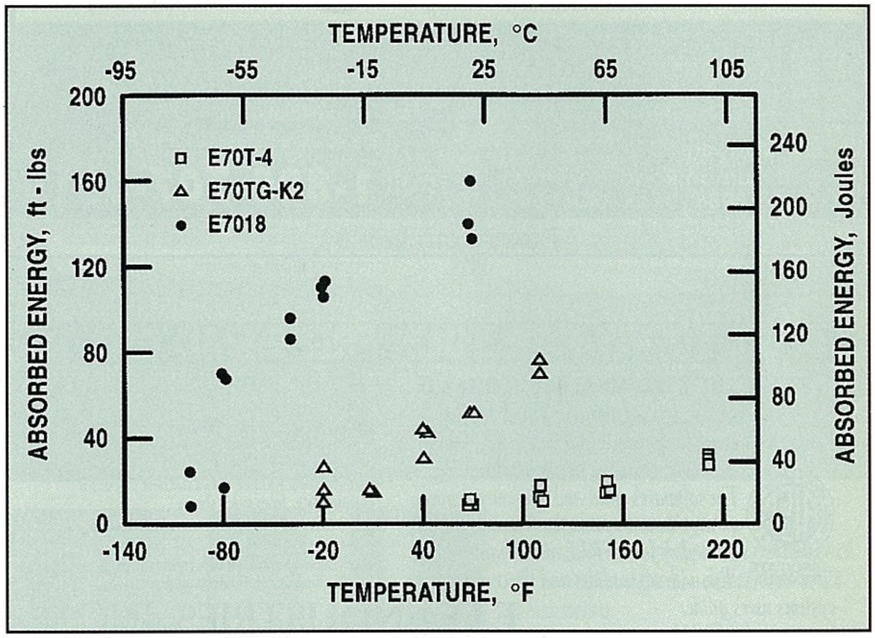
### Fracture Analyses

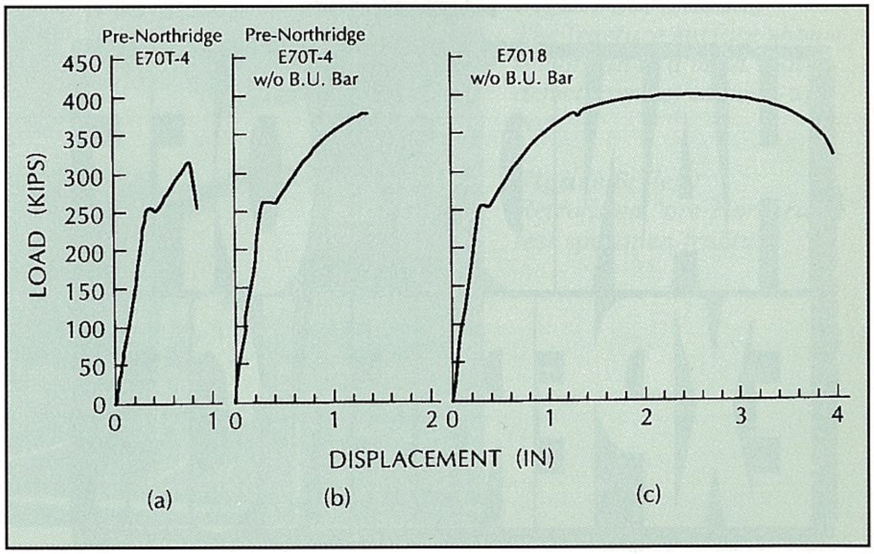
Detailed failure analyses of nearly 20 fractures removed from damaged buildings have been performed. These have included the most common types of weld and column fractures reported in building surveys. In all fractures of these types that were examined by the authors, the origin of the fracture has been identified with areas of inadequate weld root penetration arising from entrapped slag or porosity at the back-up bar. Invariably, these have been observed near the mid-length of the weld where the weld passes through the web cope in bottom flange connections and where weld discontinuities are more prevalent. Figure 1 shows the crack surface of a fracture which resulted in a column divot type of fracture. The inadequate root penetration flaw from which the brittle fracture initiates is clearly seen. Similar root flaws were also observed to be the origin of fractures which propagated close to the fusion line of the weld or propagated across the column flange. A common factor in all of these fractures, regardless of the path of crack propagation, has been the initiation of cleavage fracture from the weld root adjacent to the back-up bar. The other common factor has been that the welds were deposited with E70T-4 electrodes in common use prior to the earthquake. Charpy V-notch tests of weld metal from fractured connections have shown this weld metal to have very low fracture resistance (less than 10 ft.-lbs. at 70 degrees F). This combination of factors is consistent with fractures that have occurred in other structures in the past where similar geometric conditions and low toughness weld metal were both present.

### Simulated Beam Flange to Column Flange Tests

To study the effects of material property and fabrication variables on connection performance without the complexity and high cost of large-scale testing, the small-scale tension specimen shown in Figure 2 was developed. The intent of this specimen was to study aspects of connection behavior and the relative effects of individual variables, and identify conditions which improved or deteriorated weld joint performance. A pilot study has utilized this specimen to study the effects of beam and column material properties, filler metals, and weld procedures on moment connection performance. In addition to material and fabrication variables, the effect of strain rate was also studied by conducting tests both statically



(a strain rate on the order of $10^{-3}$-$10^{-4}$ $sec^{-1}$)and at dynamic strain rates (a strain rate on the order of $10^{-2}$ $sec^{-1}$) similar to earthquake conditions. A total of 15 tests were conducted in the pilot study. Several of the tests were designed to parallel the weld joints tested in full-size connection tests in order to compare their behavior.

The test specimen, shown in Figure 2, consists of a section of W14x176 (A572 Gr. 50) with one flange removed. A pull plate was welded to the web of the section for gripping in a universal test machine. A 6-in. x 16-in. x 1-in. (A36 or A572) plate was then groove welded to the column flange face to simulate the beam flange connection. A coped web plate was also added to introduce welding access restrictions similar to welding the bottom flange of a connection. This plate was removed before testing to permit installation of a strain gauge at the beam mid-width close to the weld.

Test specimens were fabricated using three different filler metals (E70T-4, E70TG-K2 and E7018). A 0.120-in.-diameter E70T-4 electrode was used similar to the electrode commonly used in moment connections. Welds were deposited with this electrode using both manufacturers recommended welding parameters and parameters that produced high heat input deposits similar to those observed in numerous fractured connections in buildings. The E70TG-K2 electrode ($^5/_{64}$-in. diameter), a candidate substitute for E70T-4, has a minimum CVN toughness requirement of 20 ft.-lbs. at -20 degrees F, which is similar to E71T-8 filler metal now used in many connection repairs. Test specimens were also prepared using the shielded-metal-arc (SMAW) process with an E7018 electrode. Figure 3 shows the wide range of Charpy V-notch toughness obtained for the three filler metals. The low notch toughness measured for the E70T-4 weld metal (approximately 10 ft.-lbs. at 70 degrees F) is consistent with CVN toughness of weld metal measured in fractured connections in buildings. Little difference in toughness was measured in welds deposited using manufacturers recommended parameters and parameters which produced high heat inputs. There are no AWS notch tough requirements for E70T-4 electrode. The E70TG-K2 electrode provided improved toughness although substantially less than E7018. In addition to varying filler metal in the test specimen, tests were also performed with and without the back-up bar, using end dams instead of proper end tabs, and using beam flange plates with yield strengths ranging from 40 ksi to 58 ksi. A complete report describing the results of all tests will be issued in the near future.

Figure 4 shows load vs. crosshead displacement traces for three specimens fabricated using (1) "pre-Northridge" fabrication practice (E70T-4 filler metal with back-up bar retained), (2) pre-Northridge practice retrofitted by removing the back-up bar and adding fillet reinforcements to the weld root and weld toe with a notch tough electrode (E71T-8), and (3) using E7018 electrode also with the back-up bar removed. All three test specimens were tested under dynamic strain rate conditions.

Brittle fracture developed in the "pre-Northridge" specimen (Figure 4a) shortly after the beam plate yielded. The A36 beam plate in these tests had a yield strength of 40 ksi. The fracture origin was identified at the weld root adjacent to the notch introduced by the back-up bar at a location with inadequate root penetration (see Figure 5). Figure 4b shows the result of an identically fabricated specimen with the back-up bar removed. Brittle fracture again occurred although after greater plastic deformation of the beam plate (see Figure 6). In this case the fracture initiated within the weld metal and not close to the

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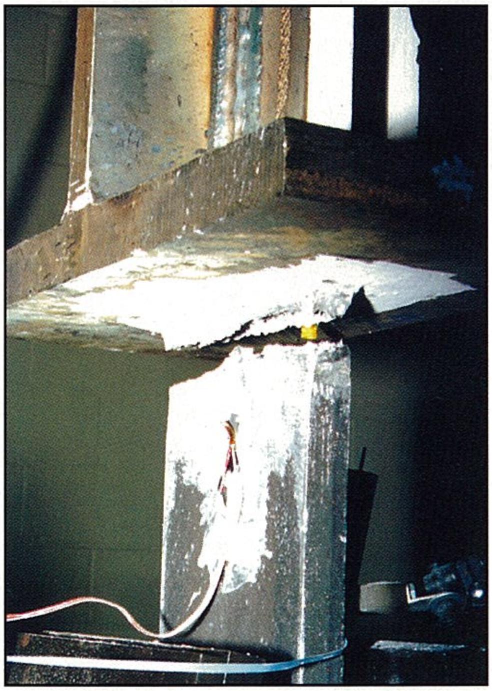


***Figure 5a:*** *(top) "Pre-Northridge" simulated beam flange to column test fracture*

***Figure 5b:*** *(middle) The fracture surface shows the origin is a weld root defect (arrow) at back-up bar*

***Figure 6:*** *(left) Retrofitted "pre-Northridge" test specimen fracture*

| Specimen | Weld Metal | Back-up Bar & Tabs | Beam Web Connection | Continuity Plate |
|---|---|---|---|---|
| A1 | E70T-4 | Not Removed | Bolted | No |
| A-2 | E70T-4 | Removed | Bolted | No |
| A-3 | E7018 | Removed | Welded | yes |

**Table 1: Connection Details of Beam and Column Test Assemblies**

column fusion line as occurred in the previous test. In contrast, when an E7018 filler metal was used to fabricate the joint, no weld cracking occurred and the ultimate strength of the beam plate was developed (see Figure 4c and Figure 7).

These tests indicate that the notch toughness of the weld metal used in the fabrication has a strong effect on the performance of the connections.

## Large Size Beam & Column Assembly Tests

Parallel to the simulated tension connection studies, full-scale beam-and-column assemblies have been tested dynamically, utilizing the facilities in the ATLSS Center. Figure 8 shows a schematic test assembly laid on the laboratory floor and the test setup used. The column was a W14x311, 12.75-ft. long, and made of A572 Gr. 50 steel. It was supported by a pin at its bottom and a roller at its top. The A36 steel W36x150 beam (flange yield strength equal to 38 ksi), approximately 10-ft. long, was connected to the column at the mid-height. A dynamic cyclic load was applied at the free end of the beam by two servo-controlled actuators, as shown.

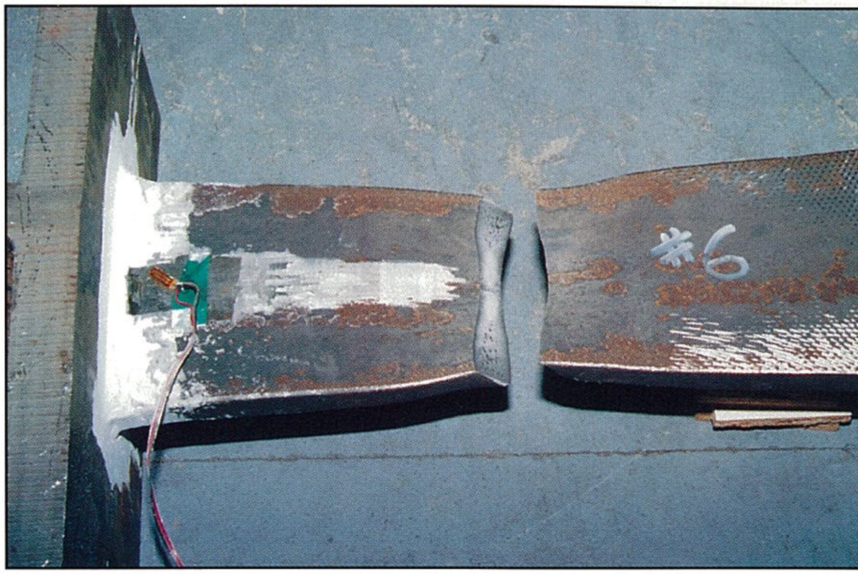
The specimens tested have the same overall dimensions, but different details to connect the beam to the column. Specimen A-1 was detailed to represent typical pre-Northridge practice. Specimen A-2 was the same as A-1, but with the back-up bar and weld tabs removed and a $^{3}/_{8}$-in. reinforcing fillet weld (E71T-8) added to its weld root. E70T-4 was used as the filler metal in both specimens. The groove welds were ultrasonically tested. Specimen A-3 was a fully welded connection, using E7018 electrode. The beam web was welded to the column flange through a groove weld. The differences in detailing of the three specimens are summarized in Table 1.

Fully reversed dynamic displacement cycles were applied at predetermined rates. Two issues were considered in determining the rates. One was the response frequency of a typical multi-story steel structure and the other was the appropriate kinetic deformation rate of steel in the plastic range, which can be represented as strain rate. At small displacements a rate of 2 cycles per second was used. Detailed descriptions of the rates adopted are not present here but will be included in a forthcoming report.

Specimen A-1 fractured at the bottom flange connection in a brittle manner. The maximum bending moment achieved during test was only 87% of the yield moment of the beam. The fracture occurred when both the beam and column were still in the elastic range, because no visible sign of beam yielding was observed, except some flaking of the whitewash in small areas around the access holes. The crack started at the root of the weld, extended into the column flange (causing divot type fracture) and eventually caused the separation of the bottom beam flange from the column. These characteristics are similar to those observed in the simulated connections and in some of the damaged connections after the Northridge earthquake.

The removal of the back-up bars in Specimen A-2 resulted in an improvement of its performance, but the brittle fracture of the flange welds again led to failure. Other than the limited yielding around the access holes, the connection behaved elastically when the welds of the top and bottom flanges fractured almost simultaneously during a reversed loading cycle. The fracture surface of the welds showed the same characteristics as the fracture surfaces of the simulated tests. The maximum bending moment resisted by the connection was approximately the yield moment of the beam. Figure 9 shows the fractured bottom flange and Figure 10 is a plot of the load vs. displacement (of the load point) relationship of the specimen.

Much improved performance in terms of both strength and ductility was observed in Specimen A-3. A typical "moment gradient" plastic hinge



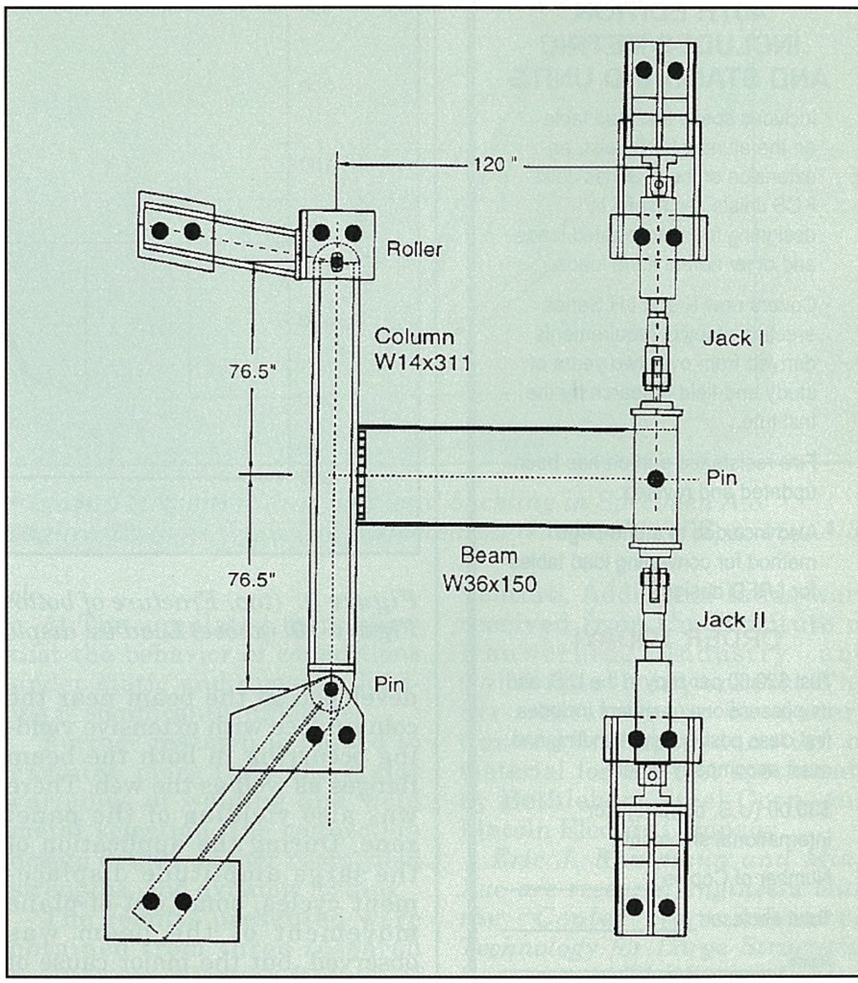
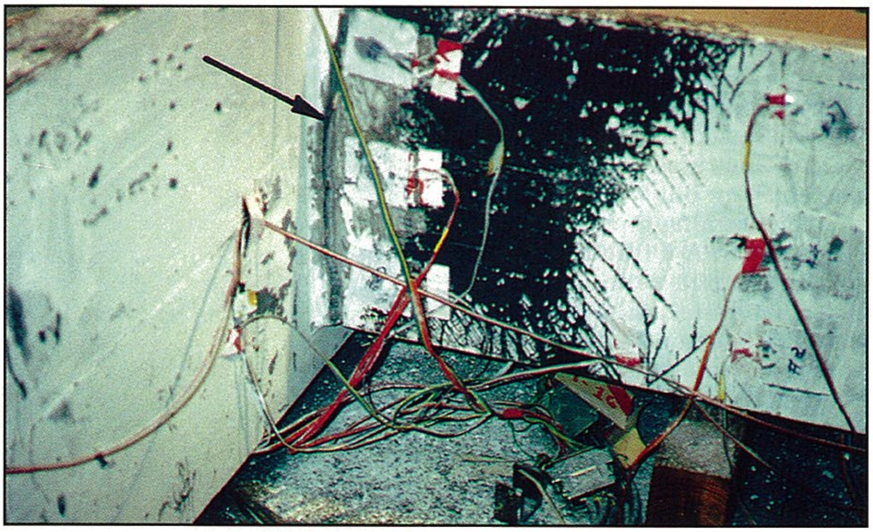




*Figure 7:* *(top) Ductile beam plate fracture in E7018 welded test specimen*

*Figure 8:* *(bottom) Full-size assembly test specimen and setup*



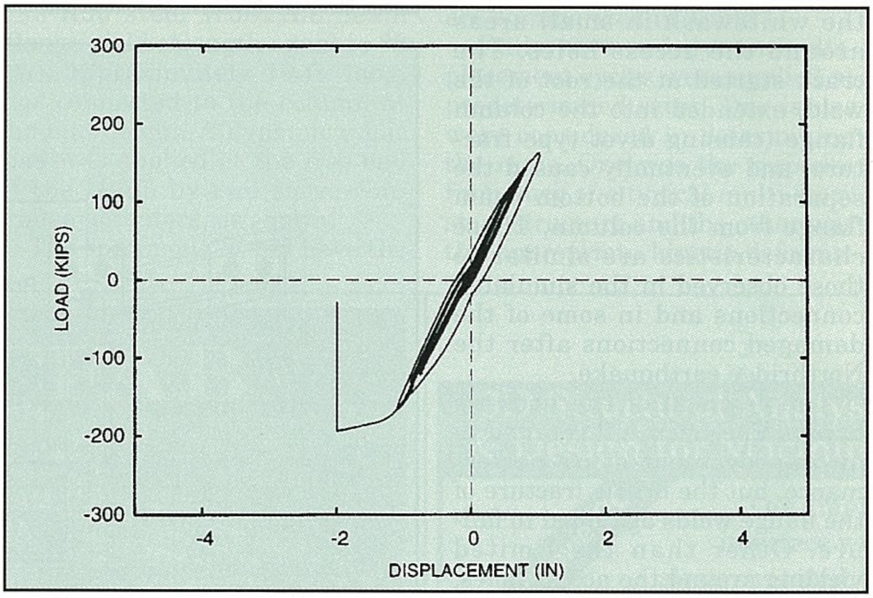



***Figure 9:*** *(top) Fracture of bottom flange in Specimen A-2 (see arrow)*
***Figure 10:*** *(above) Load vs. displacement relationship of Specimen A-2*

developed in the beam near the column face with extensive yielding occurring in both the beam flanges as well as the web. There was also yielding of the panel zone. During the application of the large amplitude displacement cycles, some out-of-plane movement of the beam was observed, but the major cause of failure was inelastic local buckling of the beam flanges and web. The test stopped when the out-of-plane movement continued to increase and local buckling deformation became excessive.

Figure 11 shows yielding and local buckling in the critical region after the specimen was completely unloaded. The load vs. displacement relationship recorded during the test is presented in Figure 12. The maximum displacement applied was plus/minus 3.84 inches and the corresponding maximum bending moment at the face of the column was 1.22 times of the plastic moment of the beam. The plastic hinge rotation reached was about 0.027 radian (calculated by assuming that the hinge is located at a distance d/2 from the column surface, where d is the depth of the beam).

The issue of strength and

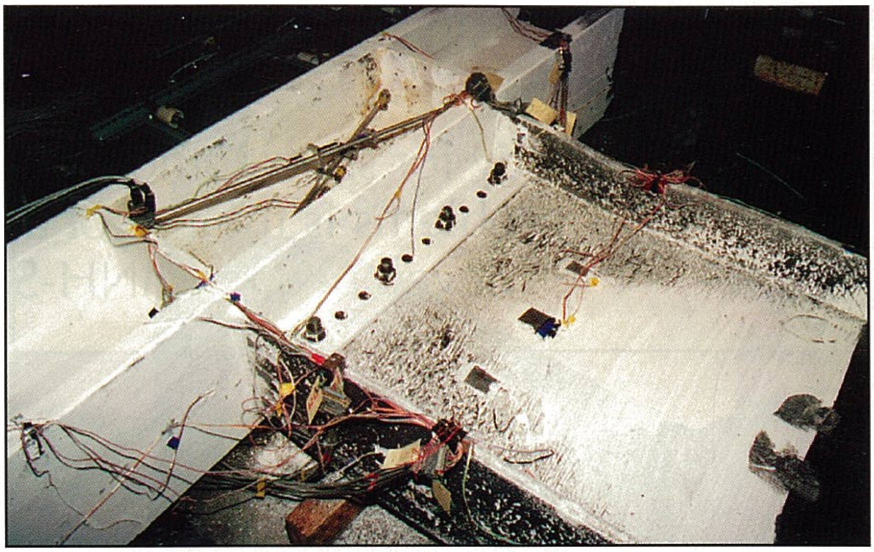
rotation capacity of plastic hinges in WF beams was studied in detail at Lehigh University as part of the research program on plastic design of the steel frame structures. It is known that the behavior is affected by such factors as: moment gradient, strain-hardening properties of materials width-to-thickness ratios of flange and web, and the presence (or absence) of stiffeners. The lateral bracing system was later modified and an attempt was made to perform additional testing with larger displacement cycles. But ductile fracture of base metal of the top flange soon developed due to accumulated local strains. The crack, initiated at a flange tip at the weld toe and extended in a stable fashion in the base metal until fracture, caused the flange to separate from the column.

## Conclusions

1) Both small-scale simulated beam flange-to-column flange tests and full-size dynamic cyclic tests of pre-Northridge connections show failure modes similar to those observed in the field. Brittle fracture developed in the elastic range of response from flaws in the low toughness E70T-4 weld metal and geometric conditions.

2) Only marginal improvement in connection performance can be obtained by retrofitting pre- Northridge connections by removing back-up bars and adding higher toughness weld reinforcement. Fracture instability is still likely to develop at internal weld flaws within the low toughness weld.

3) Acceptable connection performance appears to be obtainable by using a higher toughness weld metal in conjunction with removing back-up bars and a fully welded web. The simulated beam flange test with E71T-8 and E70TG-K2 weld metal provided the same performance as the E7018 test. A connection assembly with E70TG-K2 welds was tested in December, after the press deadline for this article.



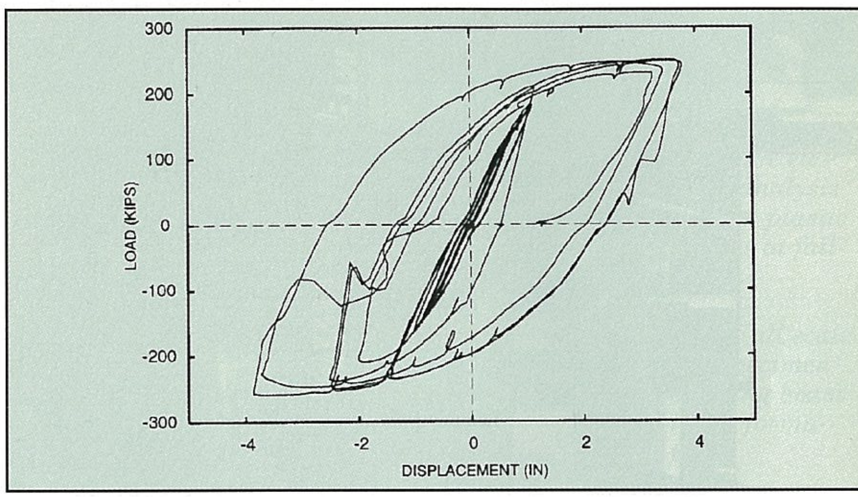



***Figure 11:*** *(top) Yielding and local buckling in Specimen A-3*
***Figure 12:*** *(above) Load vs. displacement relationship of Specimen A-3*

4) The simulated tests show that the behavior of connections under static and dynamic loading is significantly different.

Further research is needed to examine more fully the major variables of welding and weld metal selection, the connection design details, higher beam yield strengths, and dynamic loading.

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