Paper No. 174. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

TESTING OF DAMAGED STEEL MOMENT CONNECTIONS

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ABSTRACT

Four welded, beam to column, moment connections were taken from a two story building which was severely damaged during the Northridge earthquake (1994). All connections contained cracks which started in the weld at the bottom beam flange and propagated across the column flange into the column panel zone. The test specimens were tested in the "as received" condition to evaluate their residual strength and deformation capacity. They were then repaired in the laboratory and retested to evaluate the effectiveness of the repair procedure. A new "benchmark" specimen was fabricated and tested to serve as a basis of comparison.

Results indicate that the badly damaged specimens still have significant residual strength and deformation capacity. All damaged specimens were tested to 3% rotation. Repair techniques being used in the field proved to be effective by increasing both the moment capacity and the ductility capacity, however, the displacement ductility is still less than four.

KEYWORDS

welded connections, steel frame, earthquake damage, cyclic testing

INTRODUCTION

The most significant engineering feature of the Northridge Earthquake (1994) was the discovery of severe cracking in the welded connections of moment resistant steel frames (Bertero et al., 1994). Damage to connections occurred in buildings ranging in height from two stories to more than twenty stories. This paper describes the experimental studies conducted on four moment connections taken from a building that suffered significant damage during the earthquake and was later demolished.

The building, shown in Fig. 1, is a two-story office building which was constructed in 1991 and is located about thirteen miles north of the epicenter. Lateral resistance was provided by single bay, ductile steel moment frames on each of the four sides of the approximately square structure. Following the earthquake, it was immediately "red tagged" as being unsafe for occupancy. Field observations indicated that it was out of plumb by four inches to the south and west with three inches of this displacement occurring in the first story. In order to inspect the moment connections, it was necessary to remove the architectural finish on the exterior of the building. Once this was done it was found that nearly all of these connections at the second floor level had major cracks which appeared to start in the weld at the lower beam flange and extend through the adjacent column flange and in many cases into the column panel zone as shown in Fig. 2.

A detailed cost analysis indicated that the estimated cost for the repair of the existing building would be nearly 72% of the replacement cost. Therefore, the owner elected to build a new structure having the same plan as the existing building but incorporating a new facade and a strengthened lateral force system. The decision to replace the existing building presented a unique opportunity to obtain cracked steel connection specimens from a damaged building and take them to the laboratory for additional testing under controlled conditions. Four tee shaped, exterior connection specimens were obtained from two of the moment frames.

Initial tests were conducted on the specimens in the "as received" condition in order to evaluate the performance of the damaged connections under additional cyclic loading. The primary goal of these tests was to evaluate the residual strength and rotation capacity remaining in these connections to resist seismic aftershocks. These tests were conducted in a careful manner in order not to damage the specimens beyond repair.

Following the four initial tests, the specimens were repaired in the laboratory and retested. Since all four specimens had cracks in the column flange which extended into the panel zone, they presented a good opportunity to evaluate two of the repair techniques which were being used in the field for this type of failure. Three "benchmark" specimens which were similar to one of the building specimens in its original condition were obtained from local fabricators and tested. These provided a reference by which the efficiency of the repair techniques could be evaluated.

The experimental studies were supported by extensive analytical studies using linear and nonlinear finite element models. These mathematical models included both global idealizations of the overall structural system and macro models of the beam and column connections. Detailed discussions of these studies are beyond the scope of the present paper and are reported elsewhere (Anderson, et al., 1996).

FINITE ELEMENT STUDIES

Finite element analyses can be used to gain better insight into the behavior of welded beam to column connections and to give direction to the testing program by estimating the force requirements needed to reach given displacement limits. Detailed finite element analyses were conducted prior to testing in the laboratory with analytical models having dimensions identical to those of the test specimens. Since the thicknesses of these beam and column sections are all less than one inch, thick shell elements are used to represent all of the connection and member components. Analyses consider both material and geometric nonlinearity under monotonically increasing load at the end of the beam. Yielding is determined using the Von Mises' yield criteria.

A typical finite element model for a standard connection with continuity plates has 1604 nodes and 967 plate elements. Calculations were done on a Hewlett Packard LM 5/60 computer with a 60 MHz pentium processor using the COSMOS/M program. The load at the end of the beam is plotted versus the calculated beam tip displacement in Fig. 3. For this model, a beam tip force of 130 kips is required to reach a two inch deflection which is representative of 3% rotation. Contours of Von Mises' stress at maximum tip displacement are shown in Fig. 4. The high stress region extends to the beam centerline with the peak stress reaching 55.9 ksi, indicative of a plastic hinge. It can also be seen that the panel zone is a region of high stress suggesting that deformation of the panel zone may be significant for these specimens.

EXPERIMENTAL STUDIES

Demolition of the building began in December, 1994. The four moment connections at the second floor level were obtained from the moment frames on the west and south sides of the building, Fig. 5. The columns were flame cut five feet above and below the beam flanges and the beam was flame cut eight feet from the column face. These "rough cut" specimens were transported to a local fabricator for cleanup and addition of end plates. The dimensions of the finished test specimen are given in Fig. 6. All columns are W14x132. Two specimens (#3 and #4) use W24x76 beams and two specimens (#1 and #2) use W27x96 beams. The steel was specified as A36, however, tests on coupon samples taken from the specimens indicate the yield stress is approximately 47.5 ksi.

The specimens were tested in a self reacting load frame with the column in the vertical position. The column is pinned at both ends of a nine foot height. Reactions from the applied beam moment are transmitted by pin ended "A" frames to a reaction frame. A constant axial load of 40 kips is applied to the top of the column, representative of the gravity load.

Specimen #4, "As Received"

This specimen was taken from the north side of the west frame. When taken from the building, it had a crack through the column flange adjacent to the lower beam flange. The crack appeared to have started in the weld connecting the bottom beam flange to the column flange, propagated across the column flange and then into the panel zone of the column. About eight inches into the panel zone, it turned horizontal and ran to the middle of the panel zone where it stopped at the intersection with a vertical plate used for a simple beam connection perpendicular to the plane of the frame. With additional cyclic loading, the crack forked into two branches and did not propagate any further. The test was stopped when the weld connecting the top beam flange to the column flange failed.

A picture of the panel zone is shown in Fig. 7. The hysteretic behavior is shown in Fig. 8, where it can be seen that the behavior remains quite stable up the crack in the top flange weld. With the application of positive load, the cracks are opening and with negative load they are closing. This action accounts for the lack of symmetry in the hysteresis loops generated by all specimens tested in the "as received" condition. The moment versus rotation curve indicates that the specimen was able to reach a positive moment of 4000 inch-kips and a negative moment of 8000 inch-kips compared to the plastic moment capacity of 9440 inch-kips based on the actual yield stress.

Benchmark Specimen

This was a new test specimen having the same dimensions as specimens #3 and #4. Prior to testing this specimen, the test fixture was modified for future tests. Two tie down bolts were placed along the centerline of the six inch diameter half-round at the base of the column. The holes were slightly oversize to allow fit-up and permit rotation. However, the bolts which were tapped into the base of the test fixture permit the upward cyclic load at the beam tip to induce a tension force into the lower half of the column while the upper half of the column experiences a compression load. This loads the connection at the lower beam flange in a state of biaxial tension on the up stroke. It is felt that this stress state may have contributed to the numerous failures observed in the bottom beam flange welds.

The test proceeded in a normal manner to a displacement amplitude of two inches at which point the specimen failed abruptly on the up stroke by a pull out of the bottom beam flange from the column flange. Rather than stop the test, it was continued under displacement control for six additional cycles. The hysteresis curve, shown in Fig. 9, indicates that at the time of pullout, the load drops from over 100 kips to a new equilibrium state just under 50 kips and the tip displacement continues increasing out to the controlled two inch displacement. With reversal of load, the specimen begins following a new hysteresis curve which is very similar to that obtained for the specimens in the "as received" condition. The test was stopped due to degradation of the bolted shear tab, Fig. 10.

<u>Specimen #4, "Repaired"</u>

Due to the crack across the column face at the level of the bottom beam flange, it was necessary to remove a section of the column flange and replace it with a new plate, a technique known as "flange replacement". The crack in the top beam flange which occurred during the initial tests also had to be removed. The crack in the panel zone was ground out with a stopper hole placed at the tip of the crack. Rectangular flange plates having the same thickness as the beam flange, were added to both the top and bottom beam flanges with fillet welds on three sides and a full penetration groove weld to the column flange. The shear tab, loosened during the initial testing, was welded on three sides to the beam web using fillet welds. The repaired specimen is shown in Fig. 11.

The repaired specimen performed very well on retesting. The specimen sustained nine cycles of loading with the last three cycles at a displacement amplitude of 3 inches (4.5% total rotation). The hysteresis loops of load versus displacement are shown in Fig. 12 along with the similar curve for the specimen in the "as received" condition. It can be seen that the repair has been very effective in restoring the original stiffness, increasing the moment capacity and increasing the rotation capacity.

SUMMARY AND CONCLUSIONS

The two story steel building, severely damaged during the Northridge earthquake, presented a unique opportunity to remove damaged connection specimens from welded moment frames and to subject them to further cyclic loading under controlled conditions in the laboratory. The tests conducted as part of this study addressed the following issues: (1) Do structures having similar damage pose a threat to life safety in the event of strong aftershocks? (2) Can severely damaged connections be returned to their original strength and stiffness using prescribed repair procedures? (3) What is the rotational capacity that can be supplied by these connections before and after repair? The test results from the three tests described above are representative of the results obtained from the thirteen tests conducted as part of this study.

The following summary of results and conclusions address these issues:

(1). Cyclic tests on the damaged connection specimens indicate that these specimens can develop a total rotation capacity of at least 3% without complete failure. The specimens were also able to develop more than 29% of the plastic moment capacity in the "crack opening" condition and more than 75% in the "crack closing" condition based on the actual yield strength of the material. Hence, these members still have some residual strength and rotation capacity for resisting aftershocks of lesser magnitude. However, it does not imply that they would be capable of resisting another strong earthquake.

(2). In general, recommended repair procedures were able to restore the original stiffness, increase the moment capacity and increase the rotation capacity of the connection.

(3). Cracks in the column flanges were repaired using either the "flange buildup" method or the "flange replacement" method. Each method was used on two of the original specimens and tested in three different configurations. No problems were incountered, indicating that both of these methods of repair are adequate if correctly applied.

(4). Cracks in the beam flanges were repaired using rectangular flange plates that were eighteen inches in length, the same thickness as the beam flange and one inch wider than the beam flange. They were welded to the beam flanges using fillet welds on three sides and a full penetration groove weld to the column flange. They were shown to be very effective in improving connection behavior.

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ACKNOWLEDGMENT

Funding for this research was provided by the Federal Emergency Management Agency through the SAC Joint Venture. SAC is a partnership of the Structural Engineers Association of California, the Applied Technology Council and California Universities for Research in Earthquake Engineering.

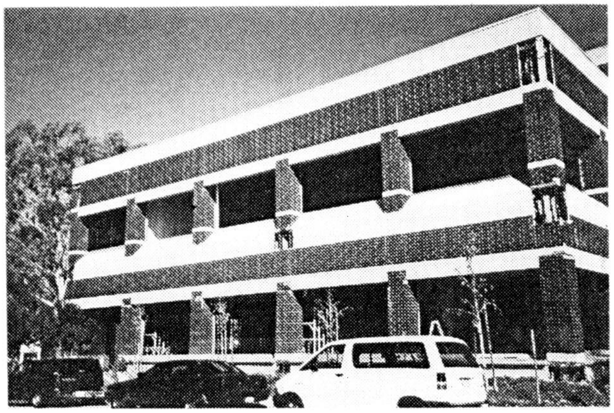

Fig. 1 Two Story Building

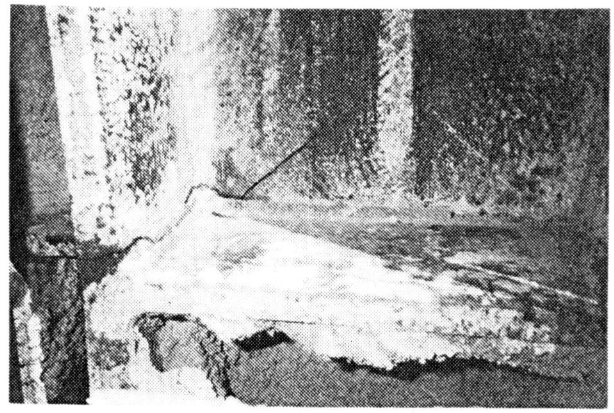

Fig. 2 Cracked Connection

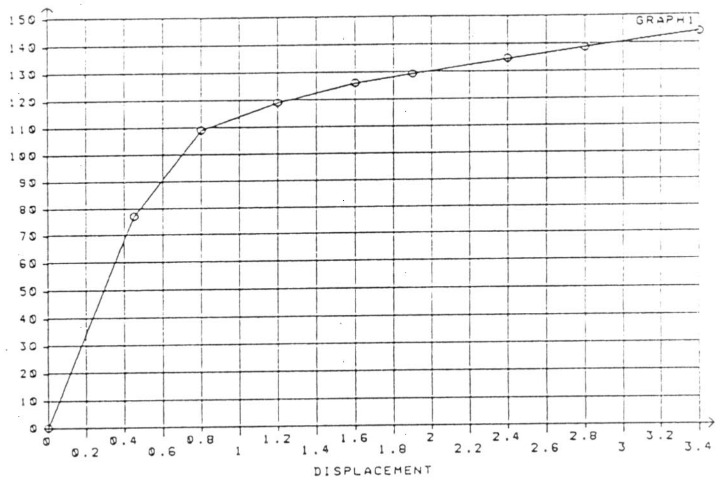

Fig. 3. FEM, Force vs. Disp.

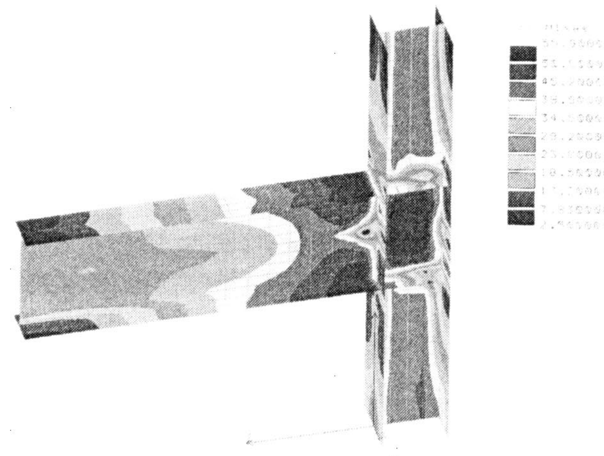

Fig. 4. FEM, Stress Contours

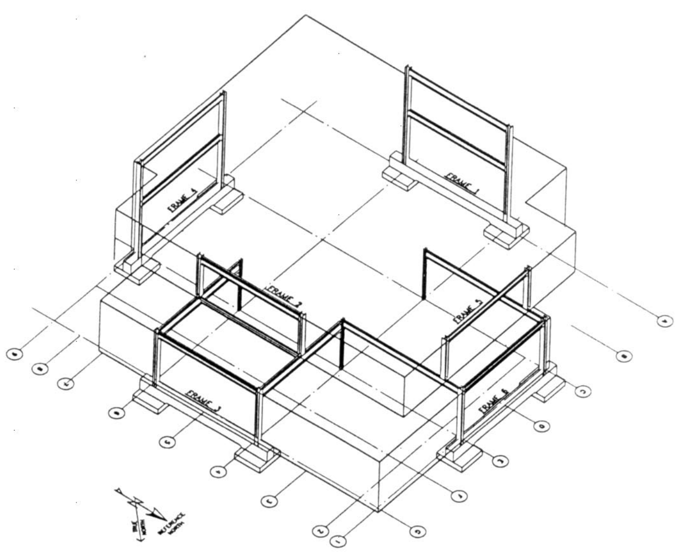

Fig. 5. Lateral Force Framing

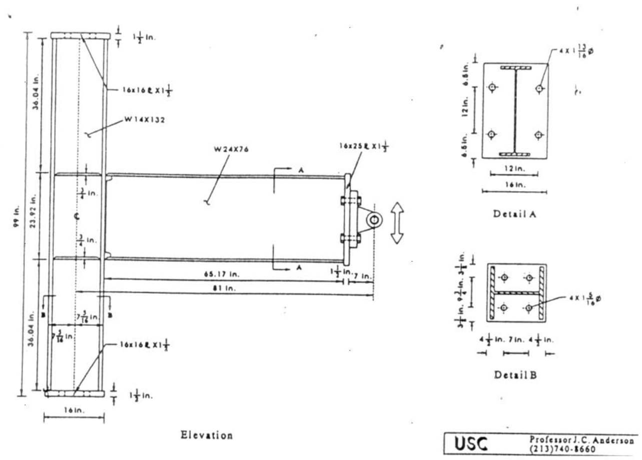

Fig. 6. Test Configuration

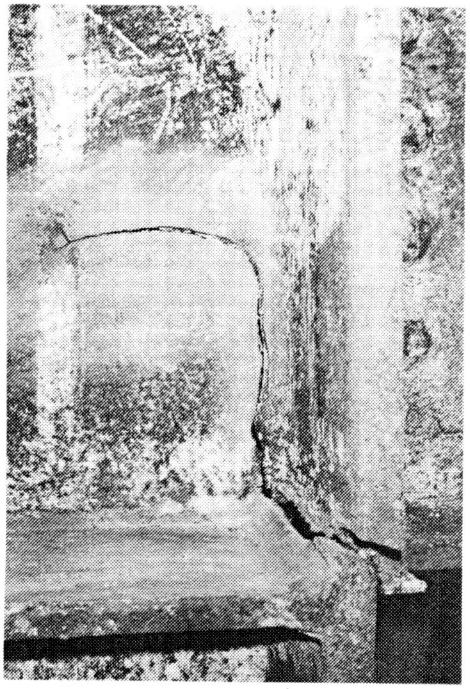

Fig. 7. Panel Zone Crack

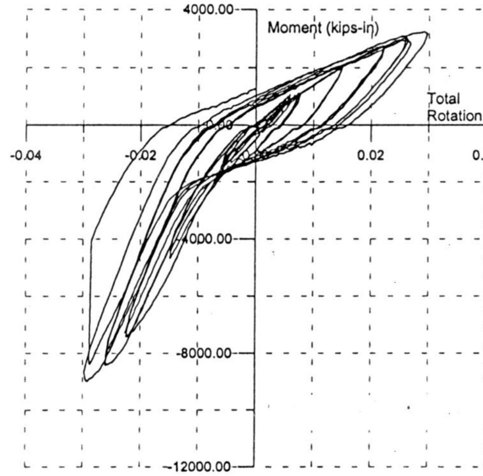

Fig. 8. Hysteretic Behavior, #4

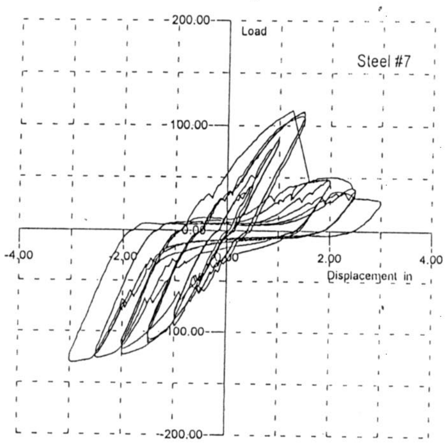

Fig. 9. Hysteretic Behavior, #5

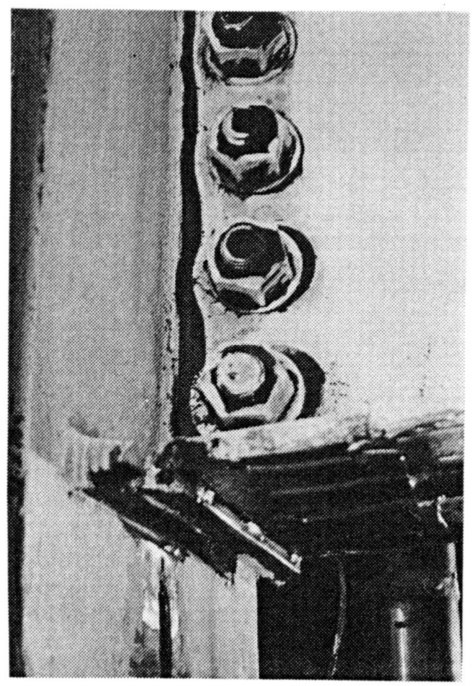

Fig. 10. Bolted Shear Tab, #5

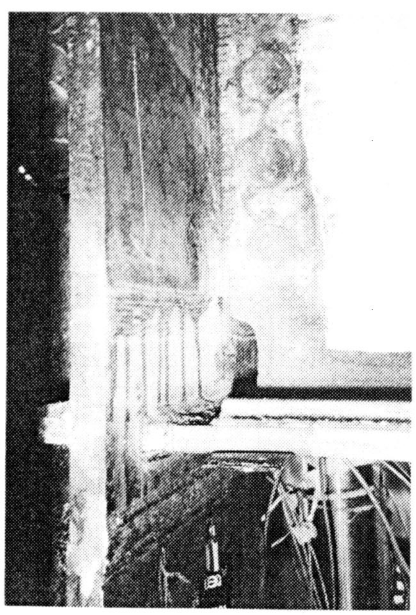

Fig. 11. Specimen #4R, Repaired

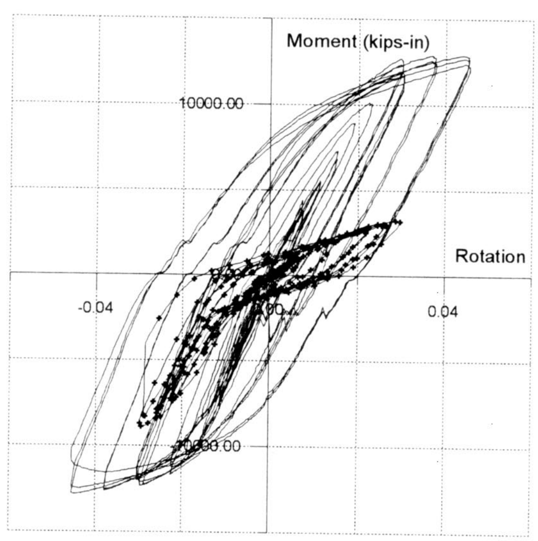

Fig. 12. Hysteretic Behavior
Initial vs. Repaired