Seismic retrofit of concrete-encased riveted stiffened seat angle connections

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Keywords: Seismic retrofit, semi-rigid connections, concrete-encased, stiffened seat, ductile steel fuse, riveted, ductility, hysteretic behavior.

ABSTRACT: Typical riveted stiffened seat angle connections taken from an 83 years old building were tested to investigate their actual hysteretic behavior and potential moment resistance. Existing columns had been embedded in low-strength concrete, as typically done at the turn of the century as a fireproofing measure. Connections were tested with and without that concrete fireproofing. Results show that such existing connections can develop a considerable moment resistance, but pinched hysteretic curves indicate they have a relatively low energy dissipation capability. A steel-band and fuse-plates retrofit scheme developed to enhance connection strength and ductility without requiring removal of the column concrete cover is shown to greatly enhance the hysteretic behavior of that connection. The special structural fuses eventually failed due to excessive inelastic buckling after having exhibited good hysteretic energy dissipation capacity.

1. INTRODUCTION

A type of riveted stiffened seat angle connections frequently found in buildings constructed in the seismic zones of Eastern North America prior to the introduction of earthquake resistant design requirements in codes is shown in Fig. 1. Although such connections are no longer used in today's moment resisting steel frames, there exists many old buildings built using this type of connection, and whose seismic survival is essential. Engineers, when required to assess the seismic resistance of such buildings, would typically ignore the lateral resistance of frames having this type of connection, which translates into a greater perception of seismic vulnerability, and could lead to the demolition or the need to perform major seismic retrofit for many steel buildings.

In that perspective, some riveted stiffened seat angle connections taken from an existing building were tested to investigate their actual hysteretic behavior and potential moment resistance. In that building, columns had been embedded in low-strength concrete, as typically done at the turn of the century as a fireproofing measure. Connections were tested with and without that concrete fireproofing. Results show that such existing connections can develop a considerable moment resistance, but pinched hysteretic curves indicate they have a relatively low energy dissipation capability. Connections embedment into the concrete of the columns was found to be of little benefit, contrary to what has been alleged by other researchers (Roeder et al. 1996). These tests also illustrate that the inherent ductility of the steel material alone is not sufficient to ensure satisfactory seismic performance of the connection during severe earthquakes.

Three retrofitting schemes have been proposed to improve the connection's hysteretic behavior, and their adequacy has been verified experimentally. This paper focuses on a steel-band and fuse-plates retrofit scheme that was developed to enhance connection strength and ductility without requiring removal of the column concrete cover. This seismic rehabilitation technique requires that special steel fuse-plates be added to connect the beam flanges to a special steel band wrapped around the column concrete. Tests showed that cyclic behavior was greatly enhanced by that retrofit measure. The special structural fuses eventually failed due to excessive inelastic buckling after having exhibited good hysteretic energy dissipation capacity.

In this paper, experimentally obtained hysteretic curves are presented, improvements in the behavior of connections are noted, and comparison with analytical predictions are made.

2. EXPERIMENTAL APPROACH

In this study, the specimens were part of a steel frame of the Daly Building, which was constructed in 1910 on the corner of Rideau and Sussex streets in downtown Ottawa and demolished in 1992. A limited number of specimens were obtained from the building, and most have been used to test seismic retrofit strategies. Standard ASTM E8 tests revealed that the specimens were of mild steel, with average yield point, Fy, of 225 MPa, and average tensile strength, Fu, of 400 MPa. Steel was also found to weldable. Experimentally obtained yield and tensile strength of rivets tested as part of existing connections were 258 MPa and 483 MPa respectively (Sarraf and Bruneau 1996). This indicates that rivets are comparable to ASTM A502 grade 1 rivets. As mentioned earlier, the steel columns of these specimens were originally enclosed in concrete for fire proofing purposes, and tests of this original concrete indicated a compressive resistance of 8MPa. The concrete was removed for one of the tested original connections and two of the retrofitted connections.

In each test of this experimental program, two identical connections of the specimen were simultaneously tested, one on each side of the column. The effect of column behavior on the results was minimized by applying an identical moment to the connection on both sides of the column, using the beams as double cantilevers in a symmetric manner for this purpose. It is important to realize that the chosen test set-up is not intended to simulate the effects of earthquakes on the columns of this sub-assembly, as the applied loading creates no shear or bending in the columns. However, the test set-up definitely allows to simultaneously test cyclically two identical connections per specimen and investigate their hysteretic behavior for cases where the yielding of columns would not be an issue. Results in this paper must be interpreted in that context.

3. EXPERIMENTAL RESULTS FOR EXISTING (NONRETROFITTED) CONNECTION

Details of the nonretrofitted joint connection tested are shown in Fig. 1. The moment-rotation (M-θ) relationship of the two identical connections was chosen as a good descriptive and quantitative expression of the hysteretic behavior and resistance for this type of connection. The rotation values for the two connections were independently measured. They were found to be nearly identical. For the following, an arbitrary sign convention is adopted for which positive moments produce tension in the top angles and compression in seat angles, and negative moments do the opposite.

The M-θ relationship of the connections is shown in Fig. 2. Test results when concrete was removed to expose the connections are presented elsewhere (Sarraf and Bruneau 1996). Severe pinching of the hysteretic curve is clearly observed, even in the early stages of loading. Failure occurred due to shear failure of a rivet in the seat angle under negative moment. The column concrete spalled in large slabs and provided no significant additional resistance (Fig. 3).

The main causes for the pinching observed in the M-θ hysteretic curves are:

- Slippage at the rivet holes due to lack of tight fit inherent to riveting practices in the past (mismatch of center of rivet and rivet holes during field riveting, and diametric shrinkage of rivet after their cooling), and insufficient frictional resistance between the connected parts (low clamping force of rivets after cooling).

- Rocking of the vertical leg of the top angle over the column flange as that angle deforms into a convex shape and rivet elongates, per a mechanism described by Sarraf and Bruneau (1996).
- Lack of integrity between parts of the stiffened seat connection, resulting in separation of the seat angle and stiffener angles under reversed loading for example.

Experimentally obtained results were comprehensively compared with various available analytical models, differences obtained were rationalized, and models were improved (Sarraf and Bruneau 1996).

4. SEISMIC RETROFIT STRATEGIES

Two effective seismic retrofit strategies applicable when the column is not encased in concrete (or when that encasement is removed) have been tested by Sarraf and Bruneau (1996). A ductile knee-brace technique relies on sacrificial brace members designed to have efficient and reliable energy dissipation characteristics. This retrofit system can be easily repairable following an earthquake. A second technique, called selective welding retrofit and applicable only when the existing structure is of a weldable type of steel, relies on a judicious application of welding and replacement of a few rivets by high-strength bolts to enhance the performance of the existing connections by eliminating known weaknesses while keeping those inherently good energy dissipating mechanisms already present.

The steel-band and fuse-plates retrofit scheme, presented here, was developed to enhance the strength and ductility of the existing connections, without requiring removal of the column concrete fire-proofing. For that purpose, the rectangular column immediately above and below the beam flanges were "wrapped" by steel plates. Specially detailed steel plates shaped like fuses (or dogbones) were connected to the steel-band around the column at one end, and welded to the beam flanges at their other end (Fig. 4). Yielding of these ductile steel fuse-plates over their section of least area provide the desirable energy dissipation mechanism.

To obtain maximum energy dissipation from the new connecting elements, while respecting the construction constraints expected during retrofit of an existing buildings, the following design guidelines were followed:

- The steel-band/fuse-plate assembly must be sized to minimize obstruction with the structural elements that could be present in the vicinity of the connection (i.e. joists supported by beams, floor elements immediately above the beams, etc.).
- The energy dissipators must be designed to permit development of the expected story drifts (design drifts should be selected to avoid large P-Δ effects that have a negative impact on global frame stability).
- The fuses must be designed with the lowest slenderness ratio possible to maximize hysteretic energy dissipation in compression (in addition to the energy dissipation in tension).
- The strength of the fuse-plate must be selected sufficiently low to prevent development of undesirable failure mechanisms (i.e. plastic hinges in columns), and yet sufficiently high to limit the expected ductility demands and provide a sizeable energy dissipation.

Taking the above into consideration, the fuse-plates were designed to resist alone a yield moment of 200 KN•m, a value significantly above that of the original connection and comparable in magnitude to the resistance of the ductile diaphragm retrofit technique tested earlier by Sarraf and Bruneau (1996).

Rounded edges were provided at both ends of the fuse section to avoid large stress concentrations in these areas. The steel-bands were designed to remain elastic. For ease of installation, and to minimize the amount of field welding required, the steel-band/fuse-plate assembly was conceived such that two halves of the assembly could be first welded in-shop. The two pieces were transported to the site and joined together with complete penetration welds. The end of the fuse-plates were welded to the beams, using standard SMAW (shielded metal arc welding) with E70 electrodes. Then, grout was poured behind the steel-band to ensure uniform contact with the column face. The grout used was of a quick setting type with a 28 day compressive resistance of 30 MPa. This retrofit was accomplished with the column in a vertical position to simulate actual on-site conditions. Welding was chosen to minimize the length of plate required, but a bolted connection could just as easily have been designed. Note that if this bolted option is considered, the bolts should be designed as slip critical and care must be taken to avoid any net area tensile yielding in the vicinity of the bolts.

Note that in all cases, converting a semi-rigid connection into a fully rigid one, besides being often complex if at all possible, is not necessarily a satisfactory solution as it could force plastic hinges to form in the column (resulting in an undesirable weak column/strong beam failure mode).

5. EXPERIMENTAL OBSERVATIONS

The resulting M-θ hysteretic curve for specimen retrofitted with steel fuse-plates is presented in Fig.5. Recall that all results presented are based on the averaged rotation developed in the two connections of each specimen. Here, positive moments are assumed to cause tension in the top fuses and compression in the bottom ones.

The specimen with steel fuse-plates was subjected to cyclic loads up to nearly equal maximum positive and negative moments of 250 kN·m. These loads respectively caused maximum rotations of approximately 0.03 radian. Yielding of the the steel fuse-plates was detected at an applied moment of M = 205 kN•m and a corresponding rotation of $\theta = 7.5 \times 10^{-3}$ radian, as evinced by the appearance of small cracks in the paint along the neck of the fuse plate, and recorded by strain gages at that location.

The steel fuse-plates behaved as expected, undergoing alternating cycles of tension-yielding followed by severe plastic buckling (Fig. 6). The experiment ended after more than 20 cycles when a steel fuse-plate ruptured in tension. Although the retrofit procedure increased the strength of the connection quite significantly, the hysteretic curves still exhibited pinching. This can be attributed to development of the existing connection strength simultaneously to yielding of the steel fuse-plates, and elongation of the fuse-plates under repeated buckling/yielding. Once a fuse-plate buckles, its residual deformation and out-of-straightness cannot be completely eliminated when subjected to tension in the next half cycle. Moreover, tensile load causes the plate to yield and elongate; after each yielding excursion, the stress-free elongated plate is thus longer, and has to buckle just to be able to fit back into the original distance between its connection points on the beams and the column. Consequently, over some range of rotations, all members can be slightly buckled when the specimen is returned at its original position, and then, the capacity of the fuse-plates assembly is temporarily provided mostly by the compressive members. This is why stocky plates which can resist loads and dissipate energy while buckling are preferable, even though tension yielding will always eventually develop.

For comparison purposes, the resulting M-θ hysteretic curve obtained for the specimens retrofitted with knee braces and selective welding retrofit are presented in Fig.7a and 7b respectively. The specimen with knee braces reached the maximum moment, M_{max}, of 197 kN·m and developed maximum rotation, θ_{max}, of 29.8×10^{-3} rad. The experiment ended after five cycles, when severe buckling deformations of the knee braces and large rotations were observed and continuation of the test would likely not have generated any new information. The hysteretic loops of the knee braces also show a small amount of pinching when large rotations at the joint are developed, because the cyclic behavior of braces resembles that of the fuse-plates described earlier.

The specimen retrofitted using selective welding was subjected to cyclic loads up to maximum positive moment of 74 kN·m and maximum negative moment of -136 kN·m. These loads respectively caused maximum positive rotation, θ^{+}_{max}, of 25.9×10-3 rad and negative rotation, θ^{-}_{max}, of -38.8×10-3 rad. The experiment ended after 10 cycles when the applied negative moments caused relatively large inelastic deformations in the seat angles as well as formation of plastic hinges and buckling of the stiffener angles. Hysteretic curves for this connection indicate relatively low pinching and, consequently, good energy dissipation, mainly because welding of the stiffener angles to the leg of the seat angles caused these two parts to move together. As such, they both contribute to the flexural resistance, whether the connection is subjected to positive or negative moments. Moreover, the few high strength bolts installed to replace rivets at critical locations provided a sufficiently large clamping force and did not yield, preventing the creation of possible gaps which can cause pinching due to rocking of the vertical legs over the column flanges.

6. CONCLUSIONS

From this experimental and analytical study of the hysteretic behavior of existing riveted stiffened seat angle connections and three proposed techniques for their seismic retrofit, it can be concluded that:

- Although riveted stiffened seat angle connections have not been designed to resist moments, they can develop a considerable moment capacity and exhibit a relatively ductile hysteretic behavior which could be beneficially considered when evaluating frames built of these connections and subjected to small and moderate earthquakes. However, they exhibit pinched hysteretic curves, and retrofitting may be desirable.
- The proposed steel fuse-plate seismic retrofit strategy is an effective solution that enhance moment capacity and significantly improves the hysteretic energy dissipation capability of these riveted stiffened seat angle connections, and which does not require removal of the column concrete encasement. (Note that selective welding, and ductile knee braces retrofits are also satisfactory, but only applicable when steel columns are exposed). However, awaiting the results of non-linear inelastic analysis of full structures, engineers are cautioned to use judgment and pay a particular attention to drift and P-Δ issues when using these retrofit solutions.

Details on the experimental and analytical work presented here are presented in Sarraf and Bruneau (1996), and Bisson and Bruneau (1998).

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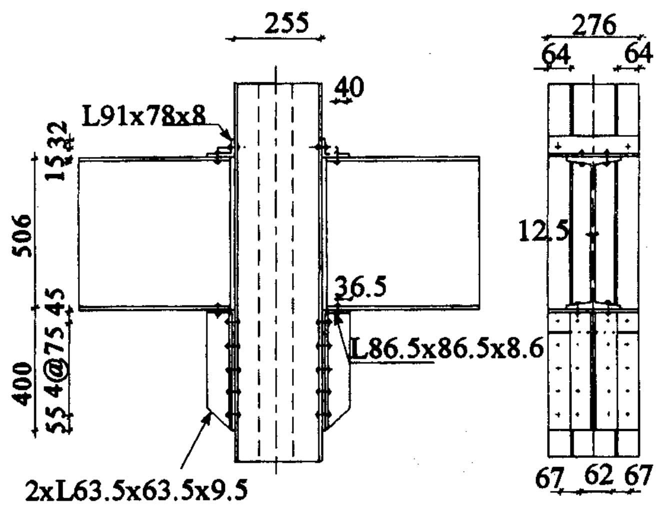


Figure 1. Detail of riveted stiffened seat angle connections - built-up steel column detail without concrete encasement.

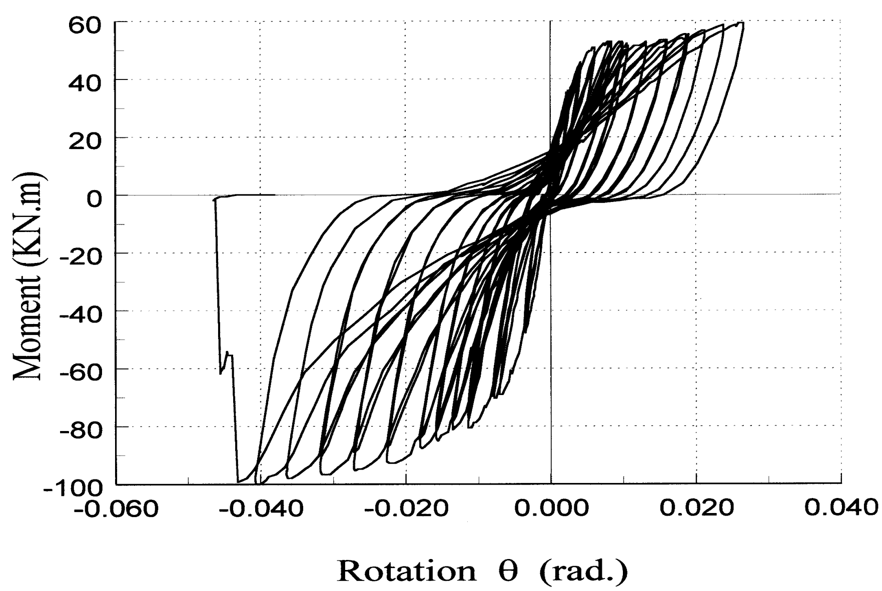


Figure 2. M-θ hysteretic curve of existing connection with column fire-proofing left in place.

Figure 3. Behavior of non-retrofitted specimen with concrete encasement: (top) Visible cracking at early stages of testing; (bottom) Spalling of column fire-proofing near end of test.

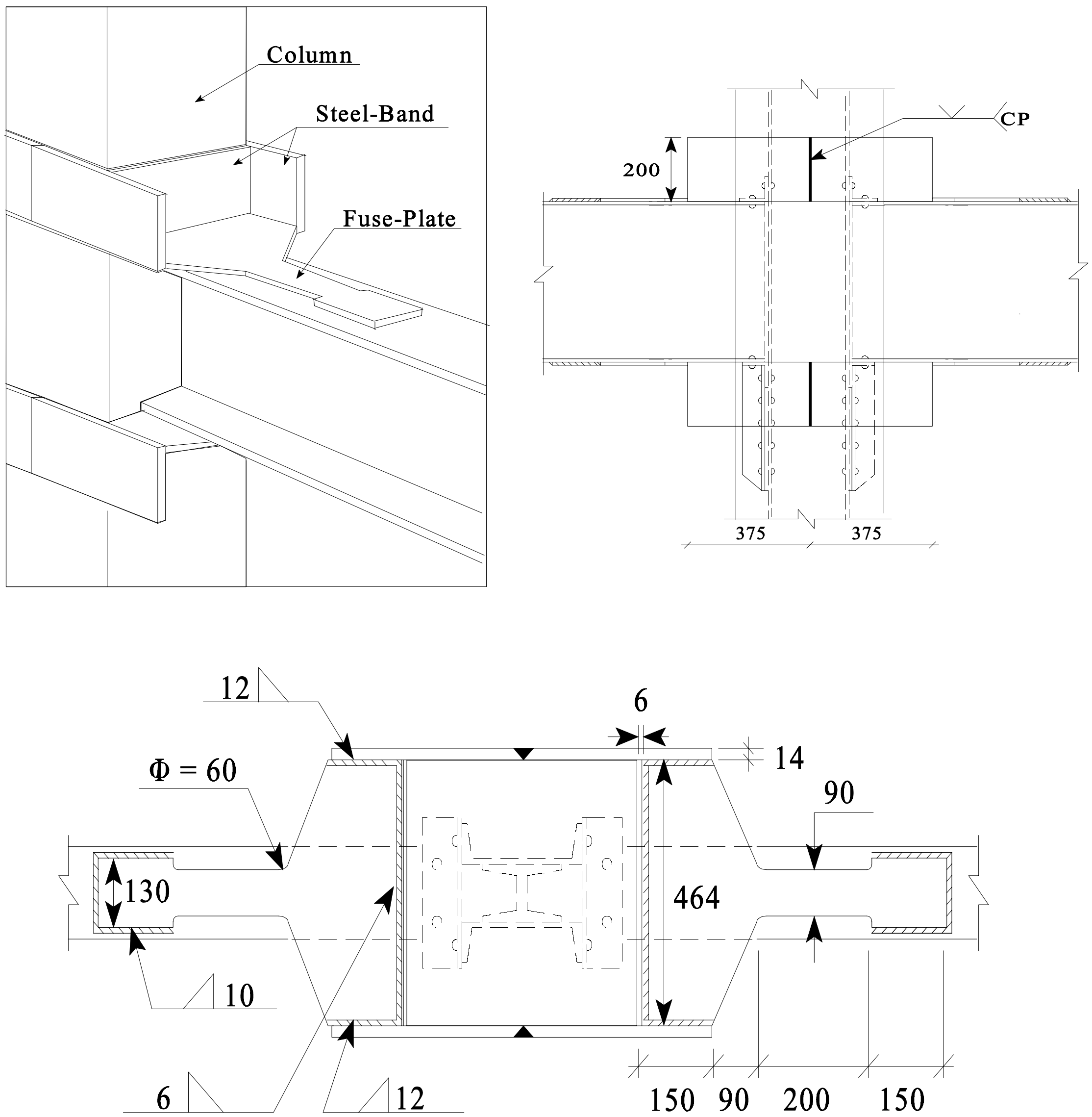


Figure 4. Ductile steel fuse-plates detail.

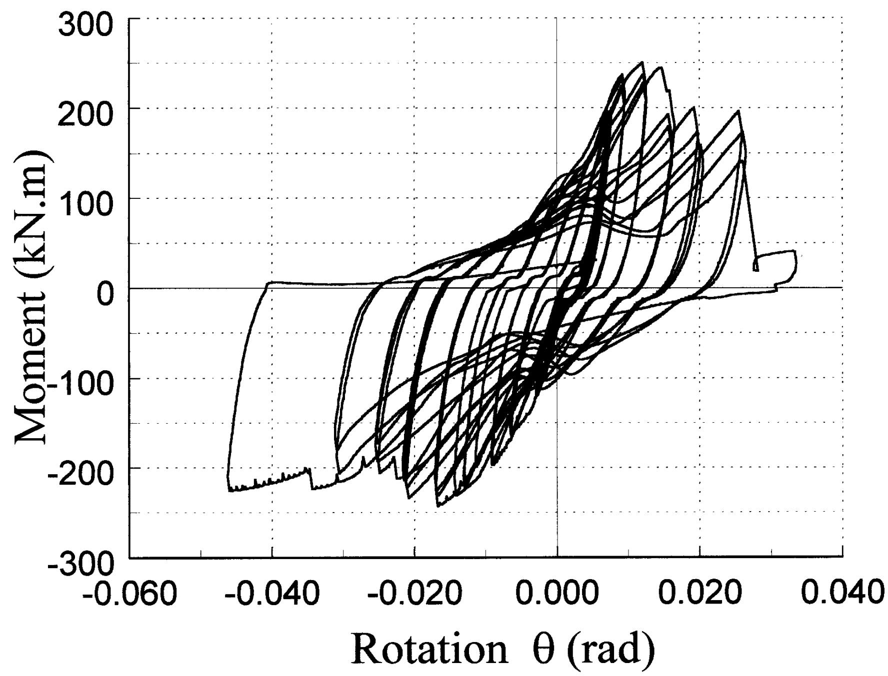
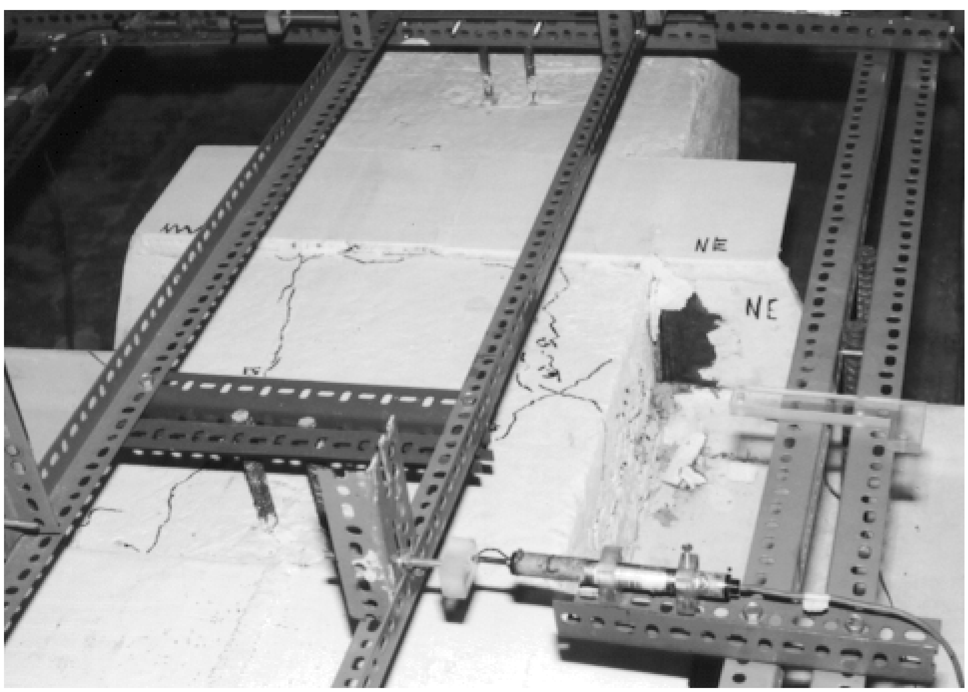
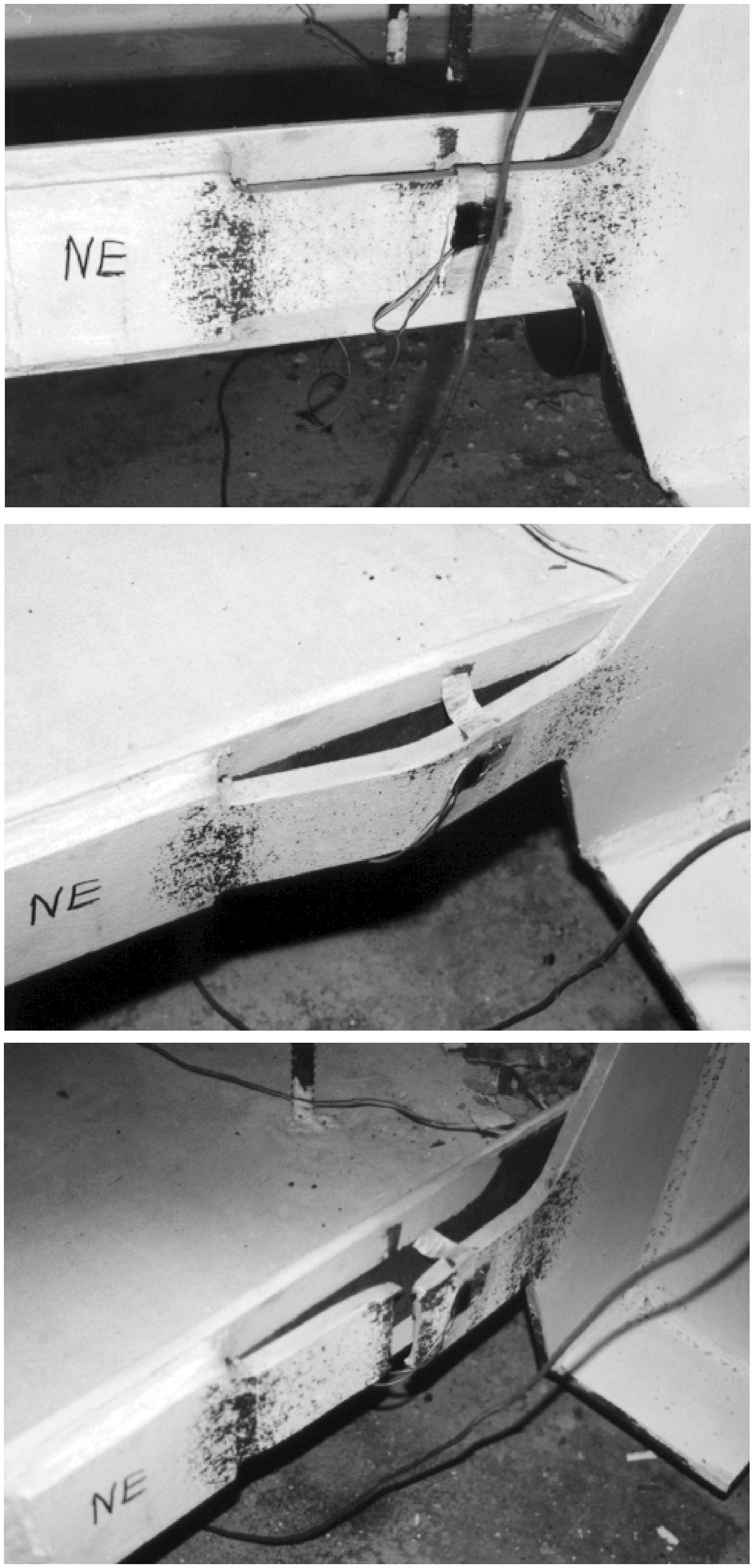


Figure 5. M-θ hysteretic curves of connection retrofitted with ductile steel fuse-plates.



(a)

Figure 6. Testing of specimen with ductile steel fuse-plates: (a) Development of cracking contained by steel bands



(b)

(c)

(d)

Figure 6. Testing of specimen with ductile steel fuse-plates: (b) Energy dissipation by tensile yielding of fuses; (c) Energy dissipation by ductile buckling of fuses; (d) Fracture at end of test.

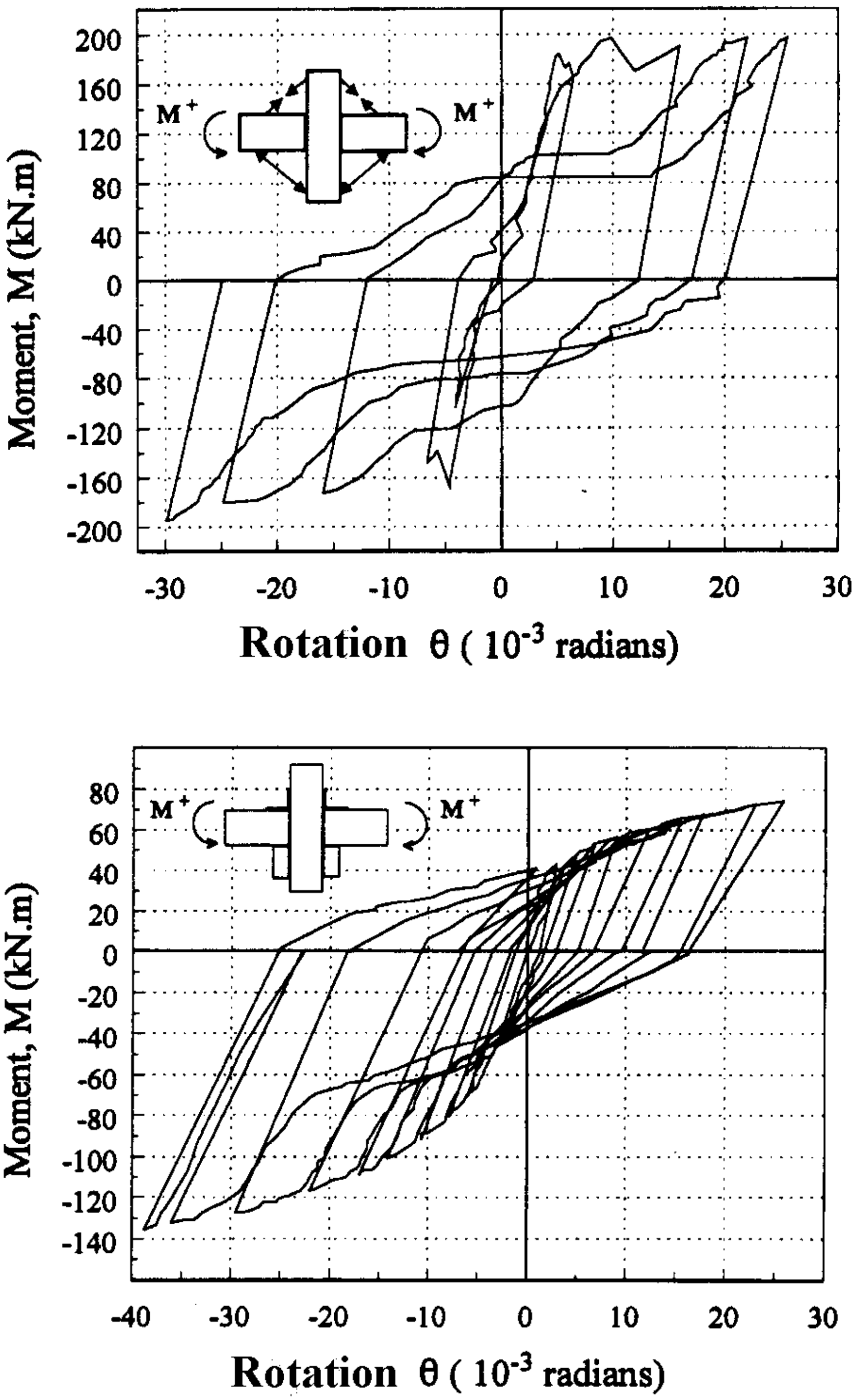


Figure 7. M-θ hysteretic curves of connections retrofitted with: (top) Ductile knee-braces; (bottom) Selective welding strategy (from Sarraf and Bruneau 1996).