TESTS OF THREE SEISMIC RETROFIT TECHNIQUES FOR RIVETED STIFFENED SEAT ANGLE CONNECTIONS

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Abstract

Typical riveted stiffened seat angle connections taken from an 83 years old building were tested to investigate the actual hysteretic behavior and potential moment resistance of these connections and to determine how they could be efficiently retrofitted with a minimum amount of structural modifications. 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. This study shows that, in their as-is condition, these existing connections can develop a considerable moment resistance, but their 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. Three retrofitting schemes are proposed to improve the connection's hysteretic behavior. These are steel fuse-plates, ductile knee-braces and a "selective welding" approach. The steel-band and fuse-plates retrofit scheme permits to enhance connection strength and ductility, without requiring removal of the column concrete cover. The two other strategies require removal of the column concrete fireproofing in the vicinity of the connections. The effectiveness of those retrofit techniques was experimentally demonstrated.

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Introduction

Riveted stiffened seat angle connections, commonly used as rigid connections in old steel frames, have been categorized as flexible connections by practicing engineers for many decades now. Although such connections, are no longer used in today's moment resisting steel frame connections in seismic regions, there exists many old buildings originally built using this type of connections, 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 eventually lead to the demolition or the need to perform major seismic retrofit for many steel buildings.

In that perspective, this paper first reports on the full scale testing of existing riveted stiffened seat angle connections obtained from an existing building to establish the potential resistance of these connections and their typical hysteretic behavior. Then follow experimental results for three ductile retrofitting techniques proposed to enhance the hysteretic performance of these existing riveted stiffened seat angle connections: (i) The addition of ductile knee braces; (ii) a selective welding approach; (iii) the addition of steel fuse-plates.

Experimental approach

In this study, the specimens were primarily 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, F_y, of 225 MPa, and average tensile strength, F_u, 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. This indicates that rivets are comparable to ASTM A502 grade 1 rivets. Note that the steel columns of these specimens were originally enclosed in concrete added for fire proofing purposes. Tests showed this original concrete to have 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 hystretic behavior for cases where the yielding of columns would not be an issue. Results in this paper must be interpreted in that context.

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 when concrete was removed to expose the connections. Tests results for column concrete left in place are shown in Fig. 3a. In both cases, severe pinching of the hysteretic curves is clearly observed, even in the early stages of loading. The first specimen (Fig. 2) experienced a maximum positive moment (M^+_{max}) of 81.1 kN·m and corresponding maximum rotation (θ^+_{max}) of 21.28×10^{-3} radian. During the seventh cycle, failure occurred due to shear failure of a rivet in the seat angle when maximum negative moment (M^-_{max}) reached -139 kN·m, at a maximum negative rotation (θ^-_{max}) of -27.9×10^{-3} radian. Similar behavior was observed for the second specimen, the column concrete spalling in large slabs and providing no significant additional resistance (Fig. 7).

Hysteretic behavior of the existing (nonretrofitted) connections

The M-θ hysteretic curves of the tested connections show a distinct pinching. The main causes of this pinching 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).

Ductile knee-brace retrofit

The specimen retrofitted using the proposed ductile knee-brace technique is schematically shown in Fig.4. In the design of the braces, the objective is to maximize the energy dissipation of the knee bracing system. This desired performance must be achieved within the practical constraints normally encountered when operating on actual buildings. Detailed design guidelines are proposed in Sarraf and Bruneau (1996). In summary:

- Braces must be long enough to be connected properly to both beams and column (i.e., workability condition), but not too long as to become too intrusive. Judgment must be exercised to determine a reasonable practical range of brace lengths.

- To maximize capacity and energy absorption of the members in compression, their slenderness ratio should be kept as low as possible.
- To have an efficient, reliable and easily repairable energy dissipating knee bracing system, it is desirable to have all plastic hinges form in the compression member itself rather than in the gusset plates or other parts. Moreover, the braces must be designed to avoid local buckling or torsional buckling prior to formation of the plastic hinges in the compression member.
- The braces must act as a weak link, i.e., they must yield and dissipate energy. If overly strong braces are used, there would be risk of forming plastic hinges in the connected columns, which would defeat the intended purposes.

Considering the above requirements and assumptions, the knee braces added to the existing specimen were selected to be standard double angles 25×25×6.4 mm (1"×1"×1/4"), designed to develop their ductile plastic mechanism when subjected to a moment of about 190 kN·m, a strength comparable to that of the original stiffened seat angle connection and which also precludes column failure modes.

Selective welding retrofit

Whenever an existing steel structure is of a weldable type of steel, it appears logical to attempt enhancing the cyclic behavior of the connection by welding. However, converting a semi-rigid connection into a fully rigid, if possible, could create a very dangerous situation by inducing plastic hinges in the column (weak column/strong beam failure mode). Instead, based on an understanding of the cyclic performance of this type of connection as a result of the tests reported herein, a more judicious application of welding is possible to greatly enhance the performance of the existing connections, eliminating known weaknesses while keeping those inherently good energy dissipating mechanisms already present. This procedure is termed "selective welding" approach to the retrofit of riveted stiffened seat angle connections.

The connection details of the existing specimen retrofitted using the proposed selective welding technique is somewhat different from that used previously since the column is a built-up shape made of two channels in this case; the steel column was originally enclosed in concrete added for fire proofing purposes, but was exposed at the connection level to be able to perform the proposed retrofit. The retrofit consists of three distinct tasks, as illustrated in Fig.4, and described as follows:

- Replace selected rivets by A490 high strength bolts of 19 mm (3/4") diameter. None of the other 48 rivets were replaced.
- Perform selective welding on stiffener angles.
- Perform selective welding on the beam.

Steel fuse-plates retrofit

A steel-band and fuse-plates retrofit scheme 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. 6). 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 other proposed retrofit techniques.

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.

Experimental observations

Ductile knee braced connections

The resulting M-θ hysteretic curve for the knee braced specimen presented in Fig.5a is based on the averaged rotation developed in the two connections of the specimen. Here, positive moments are assumed to cause tension in the top knee braces and compression in the bottom knee braces. The specimen in this test 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 show a small amount of pinching when large rotations at the joint are developed. This is due to the fact that once a knee brace 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 member to yield and elongate; after each yielding excursion, the stress-free elongated member is longer, and has to buckle just to be able to fit back into the original distance between its supports (gusset plates) connected to fixed points on the beams and the column. Consequently, over some range of rotations, all members can be buckled when the specimen is returned at its original position, and then, the capacity of the knee braces is temporarily provided mostly by the compressive members. This is why stocky braces which can resist loads and dissipate energy while buckling are preferable, even though tension yielding will always eventually develop.

Selective welded connection

The resulting M-θ hysteretic curve for the selective welding retrofitted specimen, based on the average rotation of connections on both sides of the column, is presented in Fig.5b. The specimen was subjected to cyclic loads up to maximum positive moment, M^{+}_{max}, of 74 kN·m and maximum negative moment, M^{-}_{max}, 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. Overall behavior improved as follows:

- Absence of visible yielding or cracks in the welds selectively made to retrofit the connection showed that these welds can effectively prevent slippage at the holes of the rivets connecting the top and bottom seat angles to the beams under action of the shear forces produced there by the moment at the connection. In addition to this, these welds increase the specimen's moment capacity by preventing premature bearing failure in these otherwise bearing-type connections.
- As expected in this experiment, the high strength bolts 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.

- As very clearly observed during the test, welding of the stiffener angles to the leg of the seat angles cause 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.

Steel plate-fuses retrofit

The resulting M-8 hysteretic curve for this retrofitted specimen is presented in Fig.3b. The specimen 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 $\theta = 7.5\text{x}10^{-3}$ 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 plate-fuses behaved as expected, undergoing alternating cycles of tension-yielding followed by severe plastic buckling (Fig. 8). The experiment ended after more than 20 cycles when a steel plate-fuse ruptured in tension. Although the retrofit procedure increased the strength of the connection quite significantly, the hysteretic curves still exhibited a considerable amount of pinching. This can be attributed to development of the existing connection strength simultaneously to yielding of the steel plate-fuses, and the elongation of the fuse-plates under repeated buckling/yielding.

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 seismic retrofit strategies (selective welding, ductile knee braces, and steel fuse-plates) are effective solution that enhance moment capacity and significantly improve the hysteretic energy dissipation capability of these riveted stiffened seat angle connections. 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-A issues when using these retrofit solutions.

Analytical models for the connections retrofitted using selective welding and ductile knee-braces are presented in Sarraf and Bruneau (1996). A research report on the tests of steel fuse-plate tests is in preparation at the time of this writing.

References

Sarraf, M., Bruneau, M., 1996. Cyclic testing of existing and retrofitted riveted stiffened seat angle connections. *ASCE Structural Journal,* Vol. 122, No.7, pp.762-775.

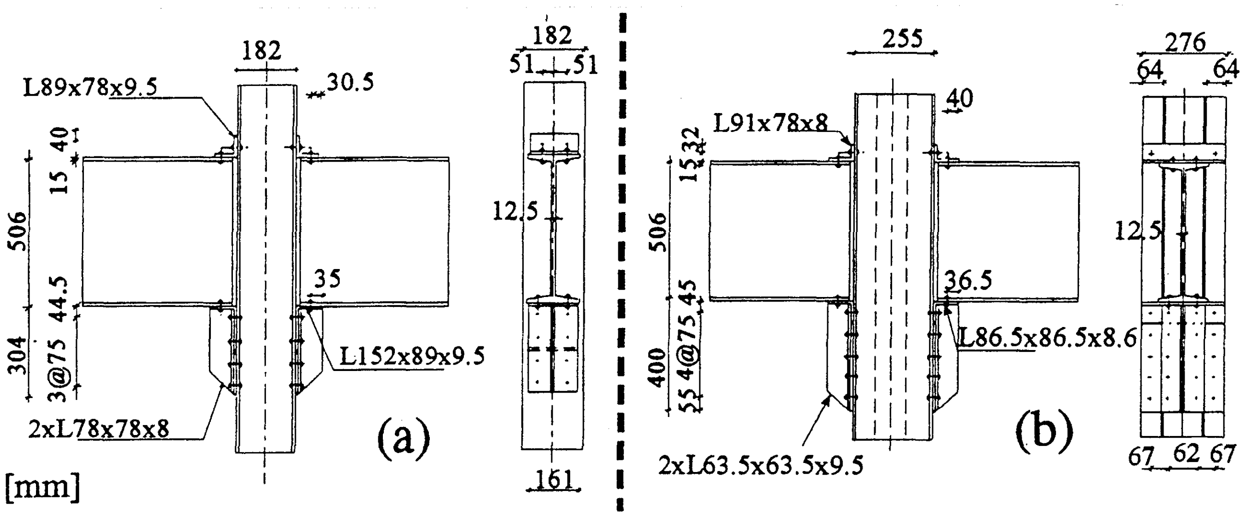

Figure 1. Detail of riveted stiffened seat angle connections: (a) Single column detail; (b) built-up column detail.

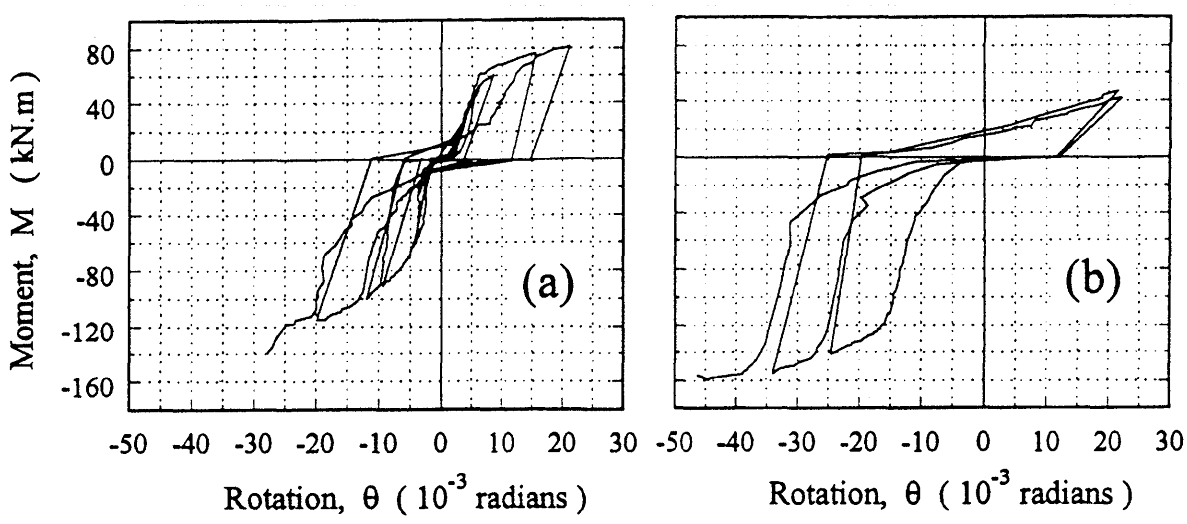

Figure 2. Hysteretic curves of existing connection: (a) Original connections; (b) Further tested after sheared rivets replaced by bolts.

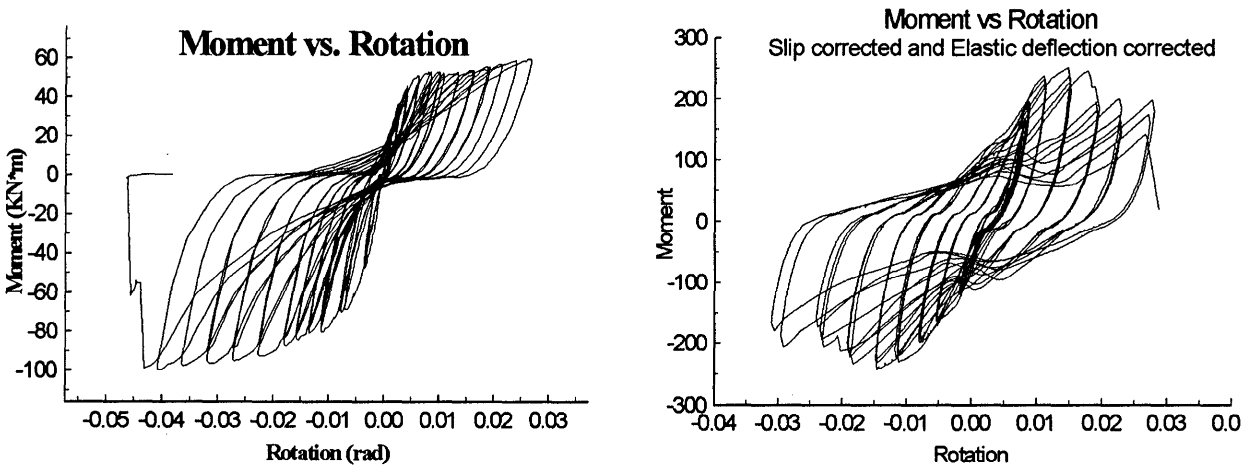

Figure 3. M-8 hysteretic curves of (a) Existing connections with column fire-proofing left in place; (b) Connection retrofitted with ductile steel fuse-plates.

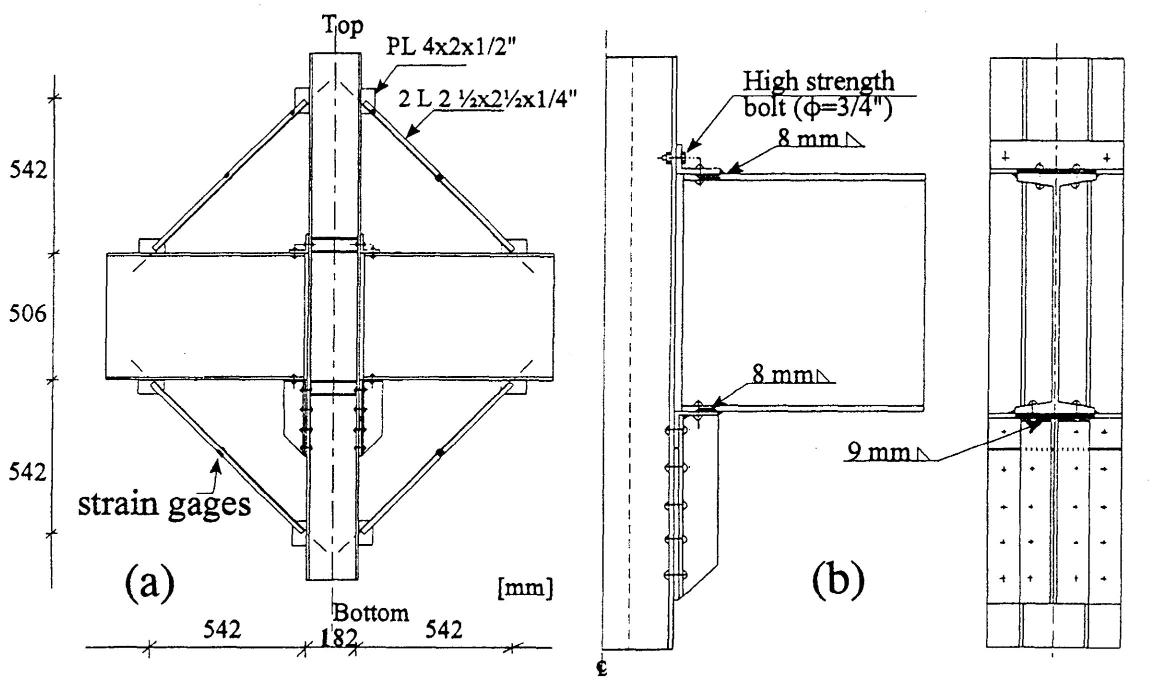

Figure 4. Retrofit details; (a) ductile knee braces; (b) selective welding.

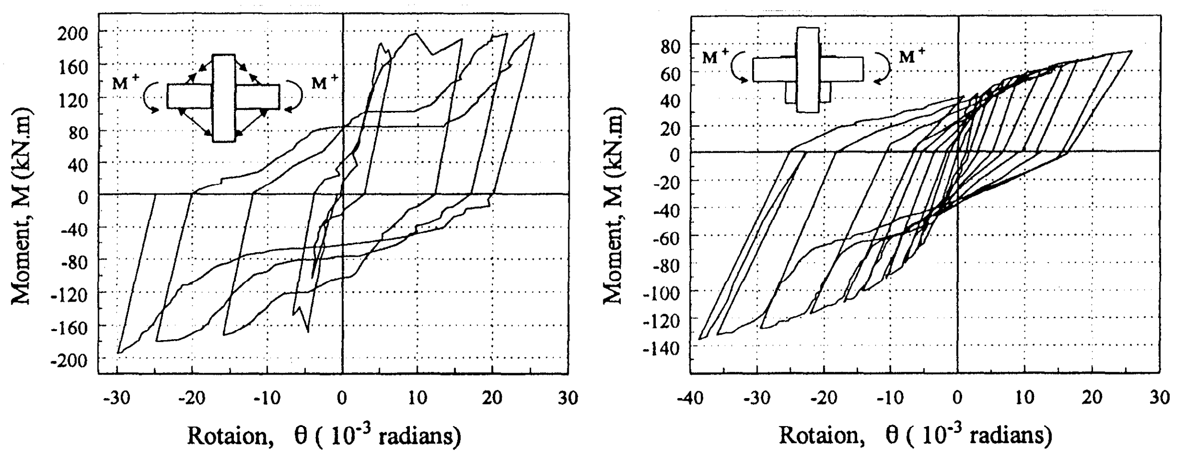

Figure 5. M-6 hysteretic curves for specimens with: (a) knee braces and (b) selective welding.

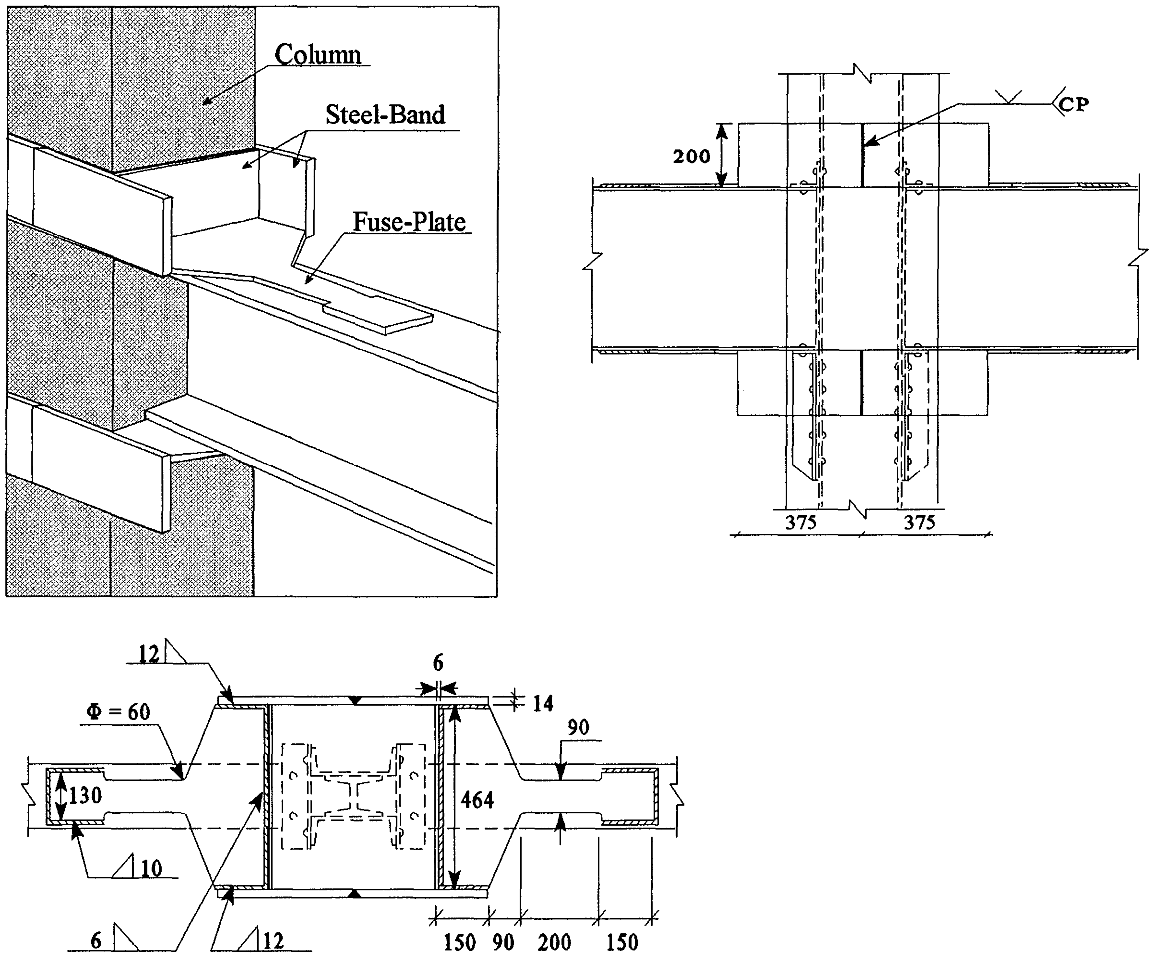

Figure 6 . Ductile steel fuse-plates detail.

Figure 7. Behavior of non-retrofitted specimen with concrete encasement: (a) Visible cracking at early stages of testing; (b) Spalling of column fire-proofing near end of test.

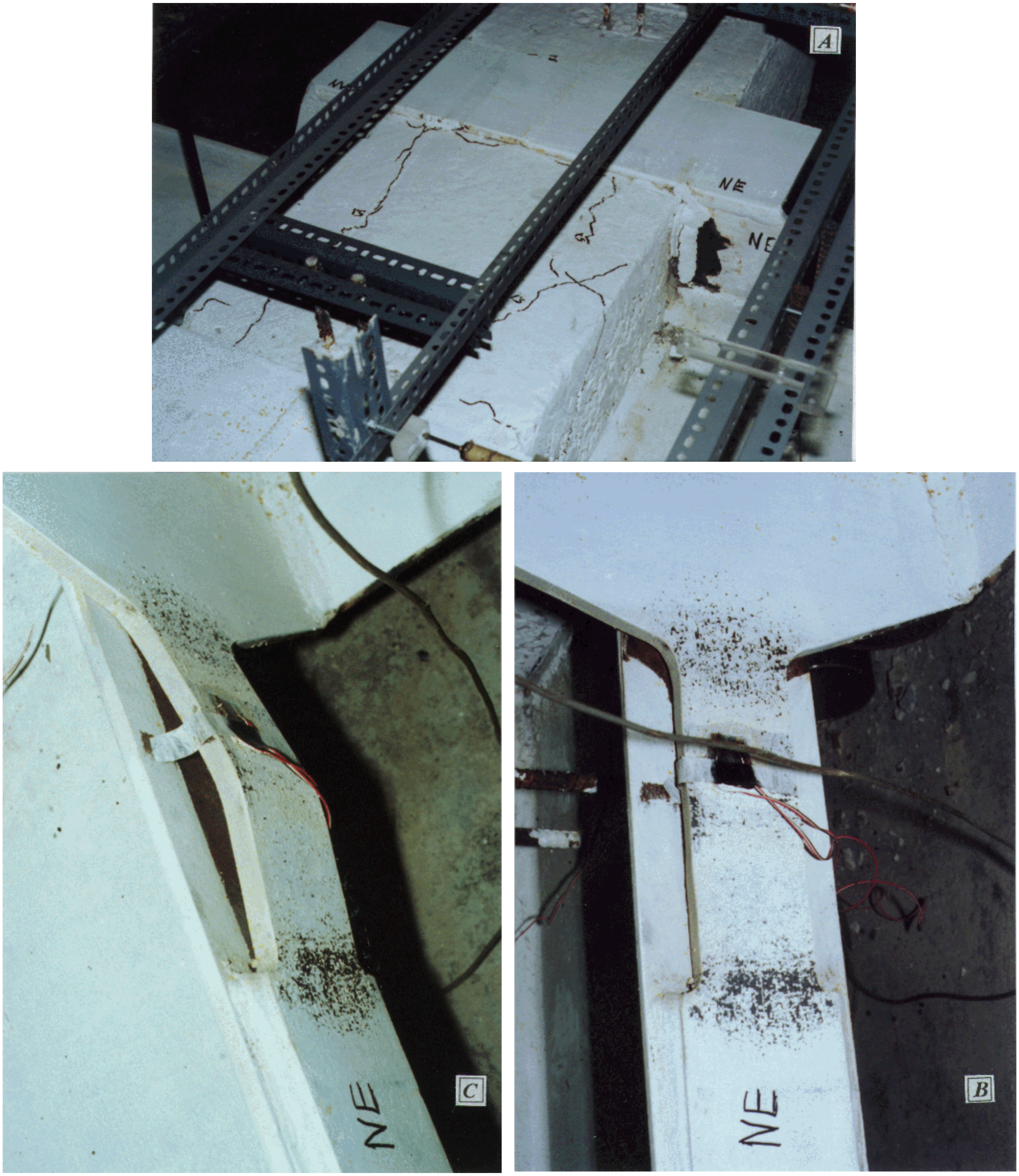

Figure 8. Testing of specimen retrofitted with ductile steel fuse-plates: (a) Development of cracking contained by steel bands; (b) Energy dissipation by tensile yielding of fuses; (c) Energy dissipation by ductile buckling of fuses.