Design of Welded Steel Moment Connections Using Truss Analogy

SUBHASH C. GOEL, KYOUNG-HYEOG LEE, and BOŽIDAR STOJADINOVIĆ

INTRODUCTION

A new design procedure for fully-restrained welded beam-to-column moment connections is presented in this paper. This procedure is based on the truss analogy approach developed by the authors (Goel, Stojadinovic, and Lee, 1996 and 1997). The truss analogy approach accounts for boundary effects occurring in the connection because the configuration of the connection restrains the deformation of the connected elements (Lee, Goel, and Stojadinovic, 1997). The resulting stress distribution, or "load path" can be modeled using a truss. The diagonals of the truss model represent the resultants of normal and shear stresses in the web of the beam that go through the regions near the beam flange welds.

A new connection design is also presented in this paper. This connection configuration is designed by following the load path model using truss analogy. The idea behind the new connection design is to indirectly connect the beam to the column. The connection elements are to be placed near the top and bottom flanges of the beam to handle the forces in the truss chords and diagonals. Each flange connection comprises a horizontal flange plate and a set of vertical ribs designed to transfer the force in the truss members. The middle portion of the beam web need not be connected to the column except for field erection purposes because it is virtually stress-free.

The combination of the new design procedure and new connection design is intended to ensure adequate ductility of the moment-resisting frame. To achieve this, it is desirable that plastic hinges form in the beams at a safe distance away from the beam-to-column connections. Thus, the new connection is designed to develop the maximum expected flexural and shear strength of the beam at the location of the plastic hinge. It does so by keeping all connection elements elastic.

The design procedure and new connection design were validated by conducting a proof-test on two connection specimens made with a W30×99 A572 Gr. 50 beam and a W14×257 A572 Gr. 50 column. The proof-tests were conducted following the SAC Joint Venture Phase 2 connection testing protocol (SAC Joint Venture, 1997). Both specimens performed very well. All plastic deformation was contained within the beam plastic hinge, while the connection elements, the connection panel zone and the column remained elastic as intended by design. The specimens sustained four complete cycles at 4 percent total drift without any undesirable deterioration of connection strength. Therefore, the new connection design satisfies current AISC requirements for use in Special Moment-Resisting Frames.

PREVIOUS WORK

Development of the truss analogy based design procedure and the proposed connection design started during earlier tests on two smaller size specimens.

The first test was done to prove the truss analogy design concept (Goel et al., 1997). The specimen, consisting of a W21×50 beam and a W12×152 column, behaved very well. The goal of the second test was to illustrate the application of the proposed design procedure for repair or upgrading of existing pre-Northridge connections. The second specimen, featuring a W24×68 A572 Gr. 50 beam and a W14×120 A572 Gr. 50 column, was first designed using pre-Northridge connection details and tested as Specimen 1.2 of the SAC Joint Venture Task 7.02 Test Program. After brittle fracture of the flange weld in a column divot pullout mode, this specimen was retrofitted by detaching the beam from the column. New flange plates and vertical ribs, designed as described in this paper, were then added. The retrofitted Specimen 1.2R successfully achieved plastic rotations of approximately 0.04 radian without fracture, but with extensive yielding and local buckling of the beam web and flanges occurring away from the connection. The test was ended due to severe lateral-torsional deformation of the beam and column of the specimen.

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DESIGN PROCEDURE

The force demand on the moment connection was calculated using the design parameters suggested in the SAC Joint Venture Interim Guidelines (FEMA, 1995 and 1997). Effects of gravity loads were not considered. Therefore, the expected plastic flexural strength of the beam M_{pr} at the location of the plastic hinge, including material overstrength, is computed by using Equation (6-1) of the Interim Guidelines:

$$M_{pr} = \beta M_p = \beta\, Z_b\, F_y \tag{1}$$

where: β is a factor that adjusts the nominal plastic moment to the expected plastic moment of the beam to compensate for modeling uncertainty, strain hardening at expected plastic rotation levels, and differences between nominal and expected mean yield strength of the beam material; F_y is the nominal yield strength of the beam; and Z_b is the plastic section modulus of the beam. The moment adjustment factor β is defined as:

$$\beta = 0.95\left[1.1\,\frac{F_{ym}}{F_y}\right] \tag{2}$$

where, 1.1 is a strain hardening factor, and F_{ym} is the expected yield strength of the beam web based on mill test statistical data. For example, the adjustment factor for Grade 50 steel is:

$$\beta = 0.95\left[1.1\,\frac{58\text{ ksi}}{50\text{ ksi}}\right] = 1.2 \tag{3}$$

The beam-to-column moment connection details are designed using the truss analogy approach and the AISC-LRFD Specification (AISC, 1994 and 1995). The expected plastic flexural strength of the beam M_{pr} is computed by using Equation 1. This moment will be developed at the beam plastic hinge, which is expected to form at a distance equal to one depth of the beam away from the column face. The shear force at the plastic hinge can be determined from the free body diagram shown in Figure 1.

$$V_c = \frac{2M_{pr}}{L_b - 2d_b} \tag{4}$$

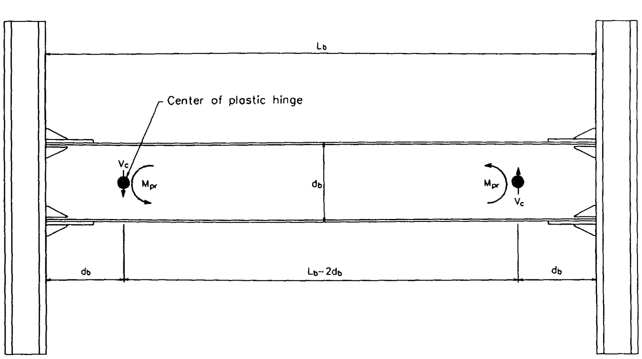


Fig. 1. Calculation of shear force at the plastic hinge.

where, L_b is the clear length of the beam between the column faces and d_b is the depth of the beam. Alternatively, the same shear force can be calculated by using the design truss model in Figure 2, as:

$$V_c = \frac{M_{pr}}{L_b/2 - d_b} \tag{5}$$

assuming that the depth of the truss is equal to the beam depth d_b.

The moment portion of the horizontal force in the beam flanges is calculated by dividing the expected flexural strength of the beam plastic hinge by the distance between the centroids of the top and bottom beam flanges. Thus, the horizontal moment couple forces T and C at the plastic hinge are:

$$T = C = \frac{M_{pr}}{d_b - t_f} \tag{6}$$

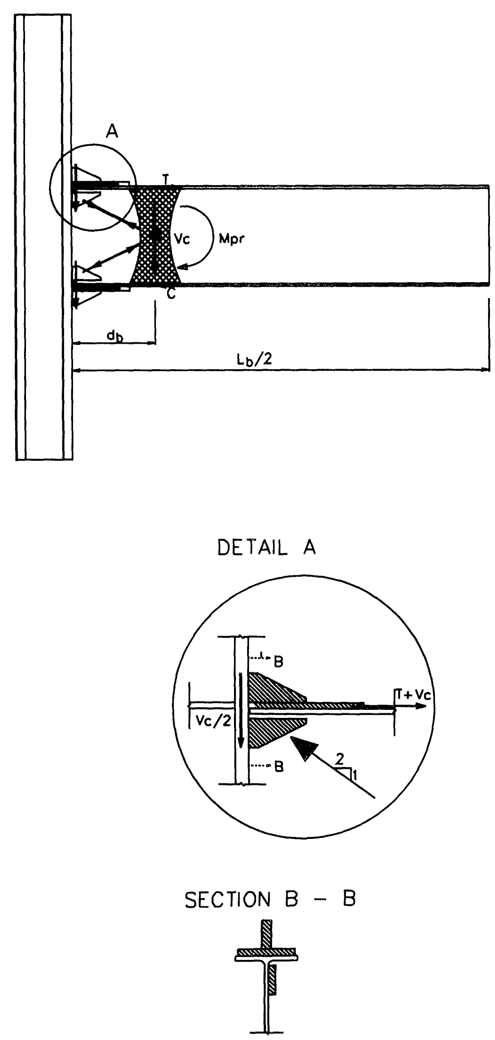


Fig. 2. Truss model of force flow for design of the connection.

The top and bottom corners of the connection of the beam at the column face should be designed to resist the end reaction forces of the model shown in Figure 2. The reactions of the truss model can be calculated either from the member forces of the truss model, or from the free body diagram of the truss model. If the top and bottom flange connections are identical, the vertical shear force V_c can be equally divided between the top and bottom corners. Thus, the design horizontal and vertical reaction forces for each corner of the beam are $T+V_c$ and $V_c/2$, respectively.

Design of the connection follows the "load path" suggested by the truss model. The beam will not be directly connected to the column at any point. Instead, a horizontal flange plate and a set of vertical rib plates will be used to transmit the reaction forces in each corner of the beam. The design procedure for the top and the bottom corner connection is the same. The test results in this study show that the connection performance is not sensitive to small differences in detailing of the connections at the top and bottom of the beam.

A comprehensive parametric finite element study was conducted to investigate the distribution of stresses in connection elements (Lee et al., 1997). Two findings are directly related to the proposed design procedure. First, the moment arm between the resultants of normal stresses transmitted through the connection elements at the top and the bottom of the connection is approximately equal to $(d_b - t_f)$. This is the same as the moment arm used to compute the horizontal reactions T and C of the truss model in Equation 6. Second, it is accurate enough to assume that the horizontal reaction force in each beam corner is divided between the horizontal flange plate and the vertical ribs in proportion to their areas.

The required area of the horizontal flange plate A_h is:

$$A_h = \frac{\alpha_h(T + V_c)}{\phi F_y} \tag{7}$$

where: α_h is the ratio of the horizontal reaction force desired to be transmitted through the flange plates to the total horizontal force $(0 < \alpha_h < 1)$; F_y is the nominal yield stress for the flange plates; and ϕ is the strength resistance factor, taken as 0.9 for yielding in tension according to the AISC-LRFD Specification.

The value of α_h depends on the ratio of the area of the horizontal flange plate and the combined area of the vertical ribs. The parametric study (Lee et al., 1997) showed that a value of α_h between 0.3 and 0.7 is reasonable for use in design. A larger value of α_h would require a larger portion of the horizontal reaction force to be transmitted through the horizontal flange plate. Consequently, a thicker flange plate and thinner vertical ribs will be needed. Similarly, smaller values of α_h would imply thicker vertical rib plates and thinner flange plate. A balance needs to be maintained. The parametric study (Lee et al., 1997) also showed that the design of the connection is not very sensitive to the variation of α_h due to the particular choice of flange plate and rib sizes.

The required area of the vertical rib plates A_v at one corner of the connection is calculated by combining normal stress f_t due to the remaining portion of the horizontal reaction force (not transmitted by the flange plate) and the shear stress f_v due to the vertical reaction force. The vertical reaction force $V_c/2$ in one corner of the connection is assumed to be equally distributed between the inner and the outer vertical ribs. Normal stress f_t and shear stress f_v in the vertical ribs are:

$$f_t = \frac{(1 - \alpha_h)(T + V_c)}{A_v} \tag{8}$$

$$f_v = \frac{0.5V_c}{A_v} \tag{9}$$

An LRFD formulation of the von Mises' yield criterion suggested by Goel (1986) is used to calculate the required area of the vertical rib plates A_v. Assuming that the horizontal flange plate and the vertical ribs are made of the same grade of steel, their combined von Mises stress limit is given by:

$$f_t^{\,2} + 3f_v^{\,2} = (\phi F_y)^2 \tag{10}$$

Area of the vertical rib plates A_v can be calculated from:

$$\left[\frac{(1 - \alpha_h)(T + V_c)}{A_v}\right]^2 + 3\left[\frac{0.5V_c}{A_v}\right]^2 = (\phi F_y)^2 \tag{11}$$

obtained by substituting Equations 8 and 9 into Equation 10.

The length of the flange plate and the dimensions of the rib plates are governed by the required fillet weld lengths. In addition, the length of the horizontal flange plate depends on the position of the beam plastic hinge, assumed to be one beam depth away from the column face. If the required length of the flange plate exceeds about 80 percent of the beam depth, the beam will form a plastic hinge further than one beam depth away from the column. In that case, the horizontal length of the truss model in Figure 2 needs to be adjusted for the new position of the beam plastic hinge, and connection reaction forces must be recalculated. Fortunately, if the beam plastic hinge forms within the assumed truss length (d_b), design connection reaction forces in the truss model will be conservative. The length of the flange plates can, to a large extent, be controlled by adjusting the size of the flange plate fillet welds.

The required flange plate fillet weld size and length can be calculated by using the vector sum of the normal force

$$N_f = \frac{1}{2}(1 + \alpha_h)(T + V_c) \tag{12}$$

and the shear force

$$V_f = \frac{V_c}{4} \tag{13}$$

transferred through this weld. Normal force N_f is the sum of the normal force in the flange plate

$$N_{fp} = \alpha_h(T + V_c) \tag{14}$$

and the normal force in the outer vertical ribs,

$$N_{ovr} = \frac{1}{2}(1 - \alpha_h)\,(T + V_c) = N_{ivr} \tag{15}$$

The normal force in the inner vertical rib N_{ivr} is the same as the normal force in the outer vertical rib N_{ovr}. The connection shear force is assumed to be equally divided among the inner and outer vertical ribs. Thus:

$$V_{ovr} = V_{ivr} = \frac{V_c}{4} \tag{16}$$

DESIGN OF THE TEST SPECIMENS

The proof test specimens were made using a W30×99 A572 Gr. 50 beam and a W14×257 A572 Gr. 50 column. Grade 50 plates were used for all connection elements: two flange plates, two vertical rib plates and a C-shaped shear tab. The half span of the beam was 134 in. and the length of the column was 144 in. The location of the plastic hinge was assumed to be at a distance d_b = 29.65 in. from the face of the column. Thus, expected plastic flexural strength of the beam M_{pr} is:

$$M_{pr} = 1.2F_yZ_b = 18{,}720 \text{ kip in.} \tag{17}$$

The statics of the connection truss model gives:

$$T = \frac{M_{pr}}{d_b - t_f} = \frac{18{,}720}{29.65 - 0.67} = 646 \text{ kips} \tag{18}$$

and

$$V_c = \frac{M_{pr}}{\frac{L_b}{2} - d_b} = \frac{18{,}720}{134 - 29.65} = 180 \text{ kips} \tag{19}$$

Thus, the design reaction forces $(T+V_c)$ and $V_c/2$ are 826 kips and 90 kips, respectively. Using $\alpha_h = 0.5$, the required area of the horizontal flange plate A_h is:

$$A_h = \frac{\alpha_h(T + V_c)}{\phi F_y} = \frac{0.5 \times 826}{0.9 \times 50} = 9.18 \text{ in.}^2 \tag{20}$$

Therefore, a 8.5 in. × 16 in. × 1 in. Gr. 50 plate (A_h = 8.5 in.^{2}) was used for the top flange plate and a 12.5 in. × 16 in. × ¾ in. Gr. 50 plate (A_h = 9.375 in.^{2}) was used for the bottom flange plate. Such a small difference in the area of the two flange plates is not considered as significant. The required length of the flange plates is 16 in., which is smaller than 80 percent of d_b, well within the assumed length of the truss model.

The total required area of the vertical rib plates, A_v, is calculated as follows:

$$f_t = \frac{(1 - \alpha_h)\,(T + V_c)}{A_v} = \frac{0.5 \times 826}{A_v} = \frac{413}{A_v} \tag{21}$$

$$f_v = \frac{0.5V_c}{A_v} = \frac{90}{A_v} \tag{22}$$

Applying Equation 10:

$$\left[\frac{413}{A_v}\right]^2 + 3\left[\frac{90}{A_v}\right]^2 = (\phi F_y)^2 = (0.9 \times 50)^2 \tag{23}$$

the required area A_v was found to be 9.81 in.^{2} One Gr. 50 4.5 in. × 9 in. × 1 in. triangular outer vertical rib plate and one 4.5 in. × 9 in. × ½ in. Gr. 50 triangular inner vertical rib plate could have been used. However, in order to reduce the number of connection elements and to provide ease of erection and field welding, a one-piece 1-in. thick C-shaped shear tab plate was used instead of the top and bottom inner vertical ribs.

The outer vertical rib was welded to the column using a complete joint penetration weld. The fillet welds between the outer vertical rib and the flange plate were sized for the normal force N_{ovr} (Equation 15) and shear force V_{ovr} (Equation 16). The shear tab was welded only near its top and bottom corners of the web to mimic inner vertical ribs. The required fillet weld sizes and lengths were computed to transfer the design normal force N_{ivr} (Equation 15) and shear force V_{ivr} (Equation 16). Connection details are shown in Figures 3 and 4.

It should be noted that the flange plates were designed only for the axial force α_h $(T+V_c)$. Longitudinal and transverse bending stresses caused by the outer vertical rib plates were considered secondary and therefore were neglected in design. This assumption was verified through finite element analysis prior to testing. No signs of excessive stresses in the flange plates were observed during the tests. Nevertheless, caution should be exercised in this regard. Designers should investigate whether or not to include such flexural stresses in design of the flange plates, particularly when the connection features relatively thin flange plates and thick vertical ribs, corresponding to values of α_h smaller than 0.3.

CONNECTION DETAILS

The beam is not directly connected to the column. Instead, the connection is indirect, through horizontal flange plates and vertical rib plates. There is a 1-in. gap between the column flange and the beam (Figure 3). The beam flanges are not welded to the column flange (Figure 4). Furthermore, the horizontal sides of the C-shaped shear tab are not welded to the beam web along the k-line (Figure 3). This weld configuration minimizes the deformation restraints imposed on the beam and lets it deform as freely as possible.

Each horizontal flange plate was connected to the column flange using a complete joint penetration groove weld. The remaining three sides of the flange plate were fillet-welded to the beam flange. These fillet welds were stopped 0.5 in. from the corners of the flange plate as shown in Figure 4. The

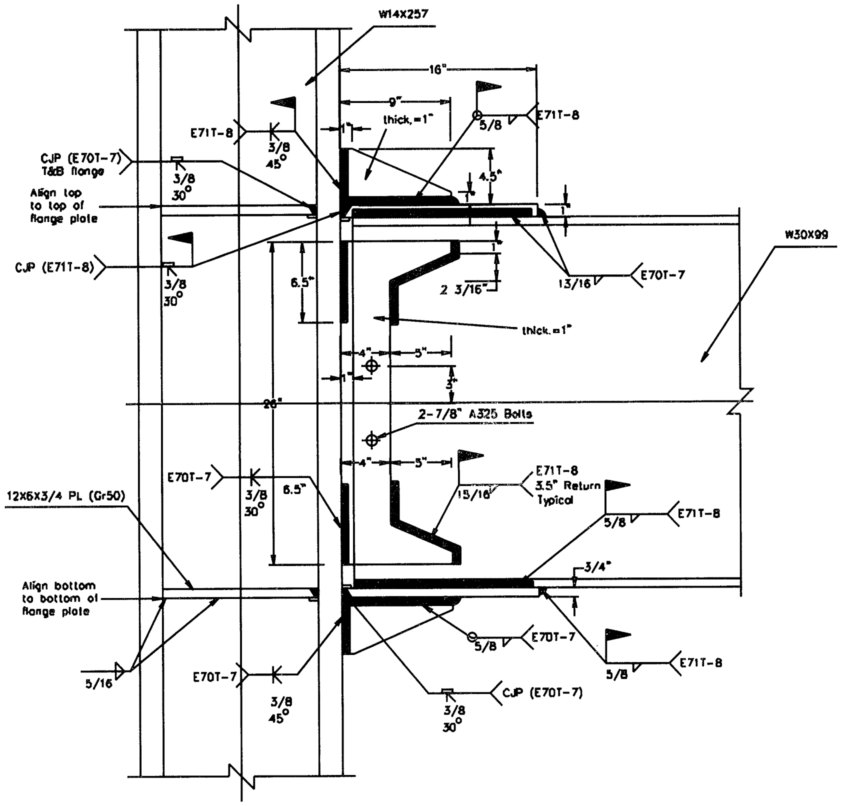
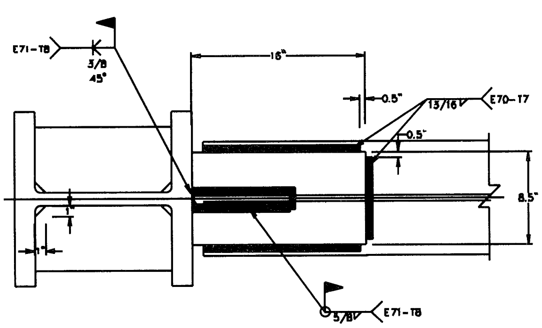
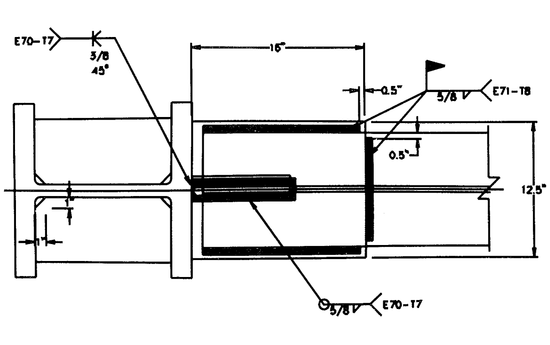


Fig. 3. Side view of the connection.



TOP FLANGE



BOTTOM FLANGE

Fig. 4. Plan view of the connection.

vertical sides of the outer vertical ribs were groove-welded to the column flange, while the horizontal sides were fillet-welded to the flange plates. The vertical side of the C-shaped shear tab was groove-welded to the column flange, while the oblique sides were fillet-welded to the beam web. The length of the shear tab groove welds and the sizes and lengths of the shear tab fillet welds were designed to carry the portions of the normal and shear forces transmitted through them. Since the middle portion of the shear tab is practically stress-free no weld was provided there. Two erection bolts were provided in the shear tab.

Several welding operations can be done in the shop to streamline erection in the field. The bottom flange plate and its vertical rib, and the C-shaped shear tab, can be shop-welded to the column. The top flange plate and its vertical rib can also be shop-welded to the beam flange, albeit with adequate consideration of cross-section tolerances. The remaining welds need to be made in the field. All of these are fillet welds except for the complete joint penetration groove welds between the top flange plate and its vertical rib and the column. Furthermore, the backing bars and the run-off tabs of the complete joint penetration welds between the flange plates and the column do not have to be removed.

The choice of the weld metal should be made considering that the connection elements are intended by design to remain elastic. Thus, the strains in the welds are expected to remain within the elastic range. The weld metal needs to be adequately matched to the base metal, while the notch-toughness of the weld metal is a matter of preference. The fabricator may choose the weld processes and electrodes suitable for shop welding and field welding in horizontal and all positions.

The above shop and field welding scheme was followed during the fabrication of the two proof-test specimens reported in this paper. The types of welding wire used for various groups of welds are shown in Figure 3. All shop welds were made using E70T-7 wire whereas all field welds were made using E71T-8 wire. The specimens were made with the column held vertically up and the beam held horizontally to simulate field welding conditions. Photographs in Figures 5 and 6 show two views of a specimen ready for field welding.

TEST SETUP AND RESULTS

Standard proof-tests were performed on two nominally identical specimens to demonstrate that the new connection satisfies the AISC seismic requirements for Special Moment-Resisting Frames and to validate the new design procedure. Figure 7 shows the test set-up. Each specimen represents a part of a moment-resisting frame under lateral seismic loading between the beam and column inflection points. Location of these inflection points was assumed at mid-spans. The specimens were rotated 90 degrees with respect to their position in a building and placed into the test fixture. The length of the beam from the center of the actuator to the column face was 134 in., and the length of the column between the supports was 144 in. The actuator load was applied at the beam inflection point, while the column ends were connected to nominally pinned supports.

The specimens were instrumented and tested according to the SAC Phase 2 connection testing protocol (SAC Joint Venture, 1997). The cyclic quasi-static displacement pattern applied to the specimens is shown in Figure 8. In both tests, the applied displacement was increased up to and including the 4 percent drift level. Connection plastic rotation was computed at the location of the beam plastic hinge. Maximum plastic rotation of 0.04 radian was reached in both specimens. The actuator load-displacement and the beam moment-plastic rotation response plots for the first specimen (AISC-MI1) are shown in Figures 9 and 10, respectively. The response plots

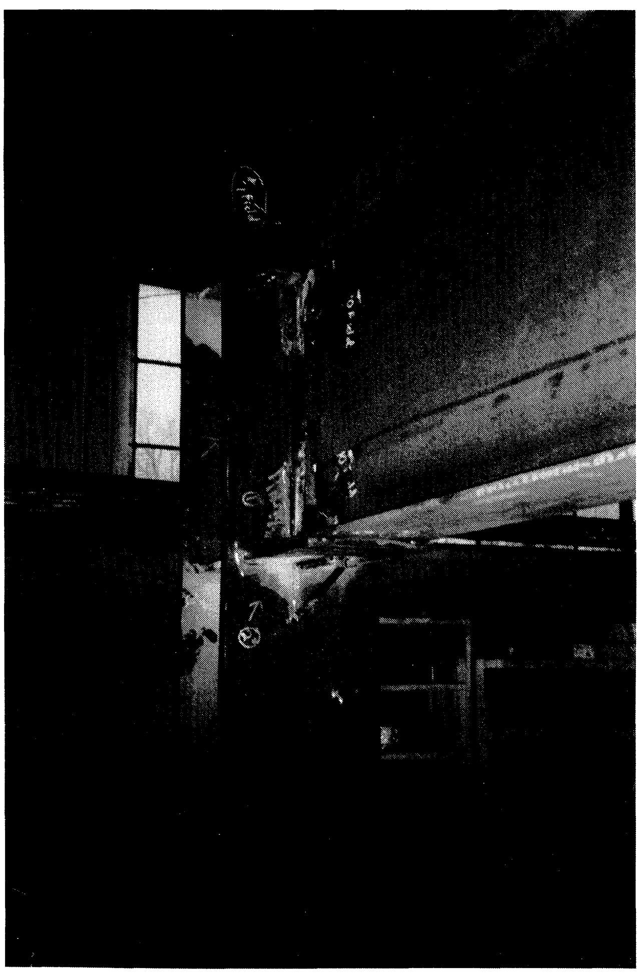


Fig. 5. Specimen ready for field welding: shear tab side.

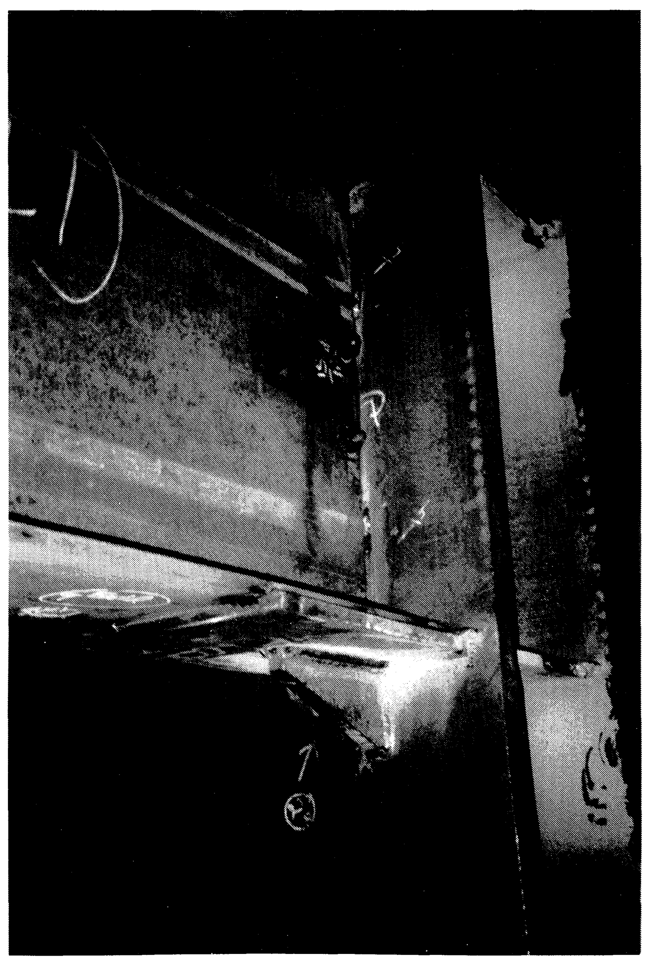


Fig. 6. Specimen ready for field welding: side opposite the shear tab.

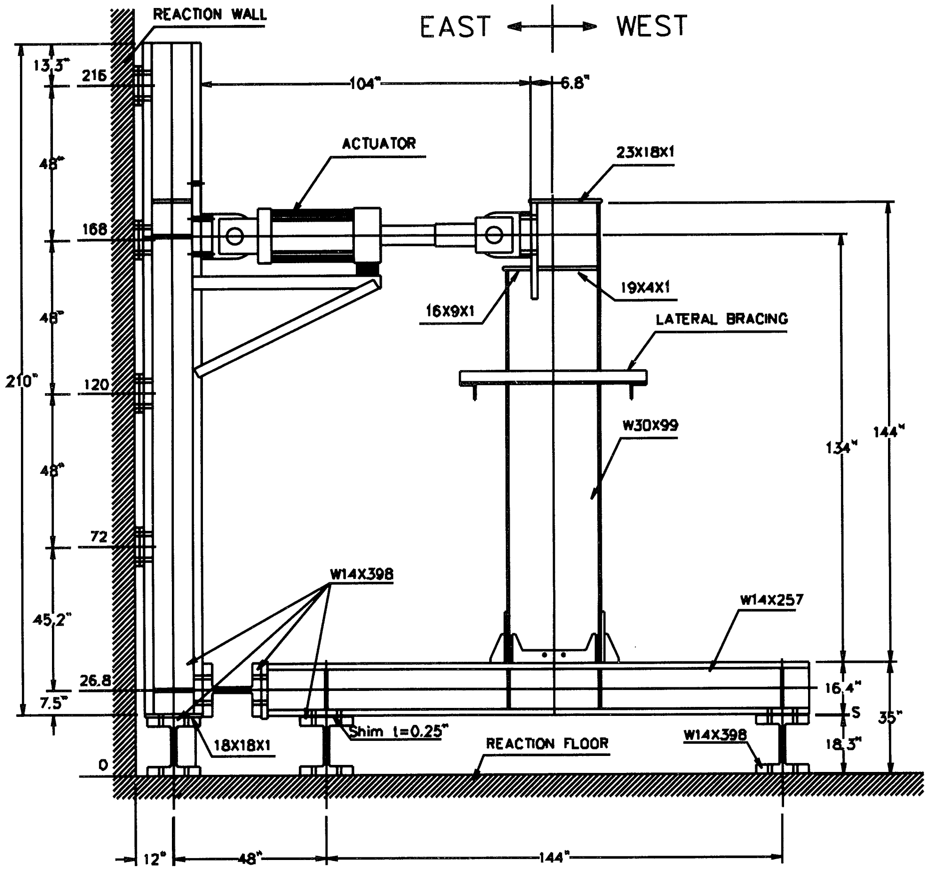


Fig. 7. Test setup for Specimens AISC-MI1 and AISC-MI2.

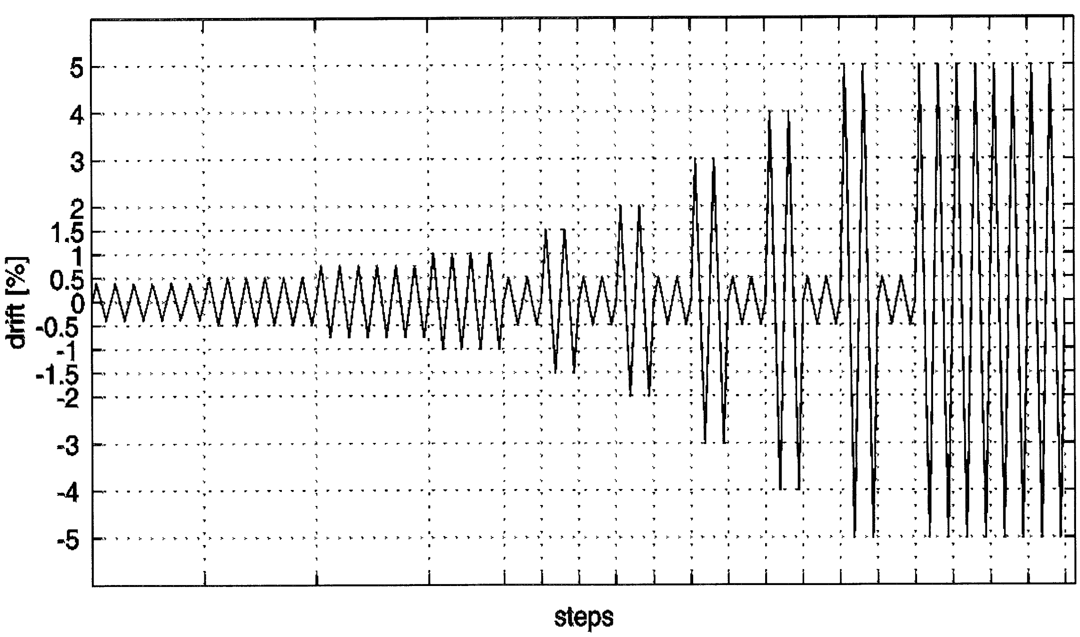


Fig. 8. Displacement history applied during tests.

for the second specimen (AISC-MI2) are shown in Figures 11 and 12.

Both specimens exhibited almost identical behavior during the tests. Significant flange yielding just outside the flange plates was observed at a drift level of approximately 1 percent. Flange yielding continued to spread in subsequent load cycles. Local buckling occurred in the flanges first, at a drift level of approximately 2 percent. It was followed by first signs of web buckling at a drift level of approximately 3 percent. Lateral-torsional deformation of the beam, stemming from the plastic hinge region, started as the specimen drift was increased above the 3 percent level. The 4 percent drift cycles were repeated four times. Local buckling as well as lateral-torsional deformation became more severe during the last two cycles at 4 percent drift, accompanied by a gradual decrease in the strength of the specimen with each new cycle. The recorded hysteresis loops for both specimens are full and stable, testifying to the ductility provided by the connection. Photographs in Figures 13 and 14 show the deformations of specimens during the tests. No yielding was observed in any

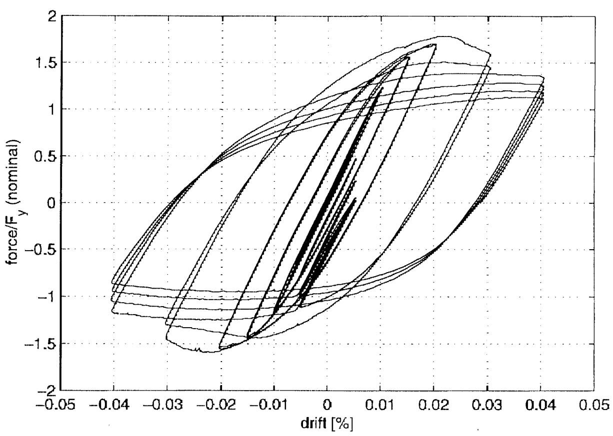


Fig. 9. Force-displacement response of Specimen AISC-MI1.

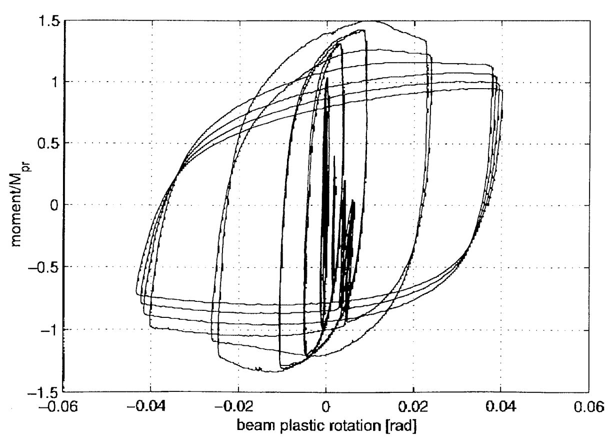


Fig. 10. Moment-beam plastic rotation response of Specimen AISC-MI1 at the location of the plastic hinge.

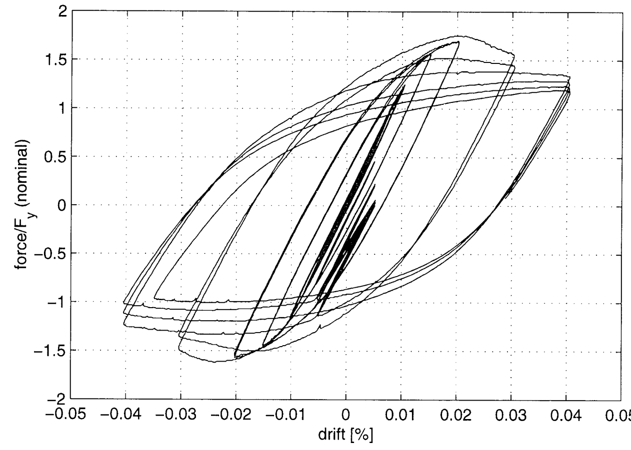


Fig. 11. Force-displacement response of Specimen AISC-MI2.

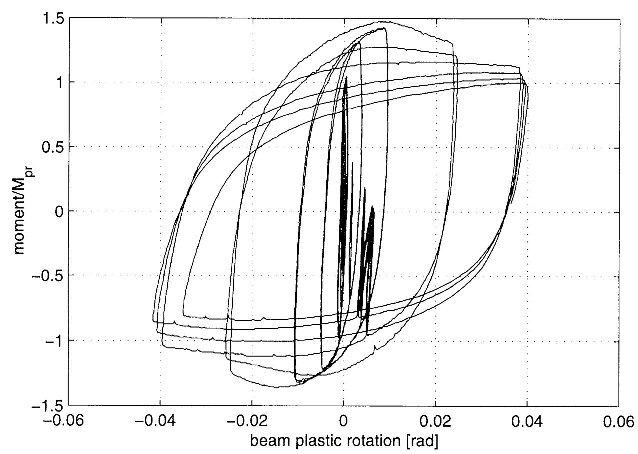


Fig. 12. Moment-beam plastic rotation response of Specimen AISC-MI2 at the location of the plastic hinge.

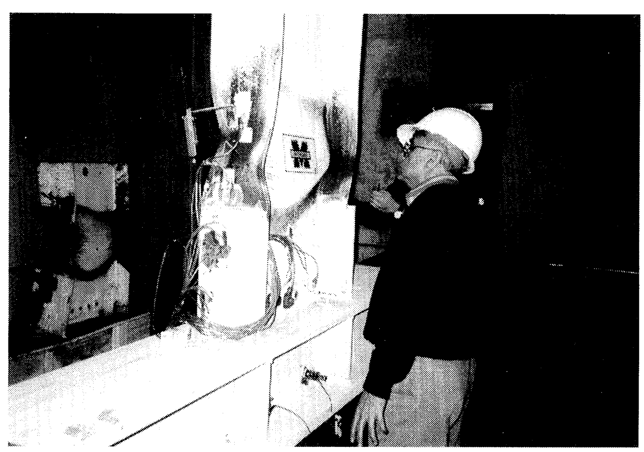


Fig. 13. Plastic hinge in the beam of Specimen AISC-MI1.

of the elements of the connection as intended by the design procedure. As expected, the column panel zone also remained elastic during the tests.

The test was stopped to protect the test setup after out-of-plane buckling of the specimen became severe. Such deformation, caused by lateral-torsional buckling of the specimen, is shown in Figure 15. Lateral-torsional deformation seems to have been triggered by local buckling of the beam web in the plastic hinge region. However, reduction of connection strength due to lateral-torsional deformation was neither sudden nor dramatic. More connection rotation, and a better performance in general, may be expected in real buildings where the floor slab may provide a stiffer lateral support than the restraints used in the test setup.

It should be mentioned, however, that a small crack was observed at the end of the test of the Specimen AISC-MI1. The crack was located in the heat-affected zone in the top beam flange next to the transverse fillet weld of the flange plate. It occurred during the third and the fourth cycles at the 4 percent drift level and did not penetrate through the beam flange thickness during the test. The crack appeared to have been caused by severe flange twisting as a result of local and lateral-torsional buckling. It did not have a noticeable effect on the connection strength at the stage when the test was stopped. The second specimen (AISC-MI2) did not develop such a crack.

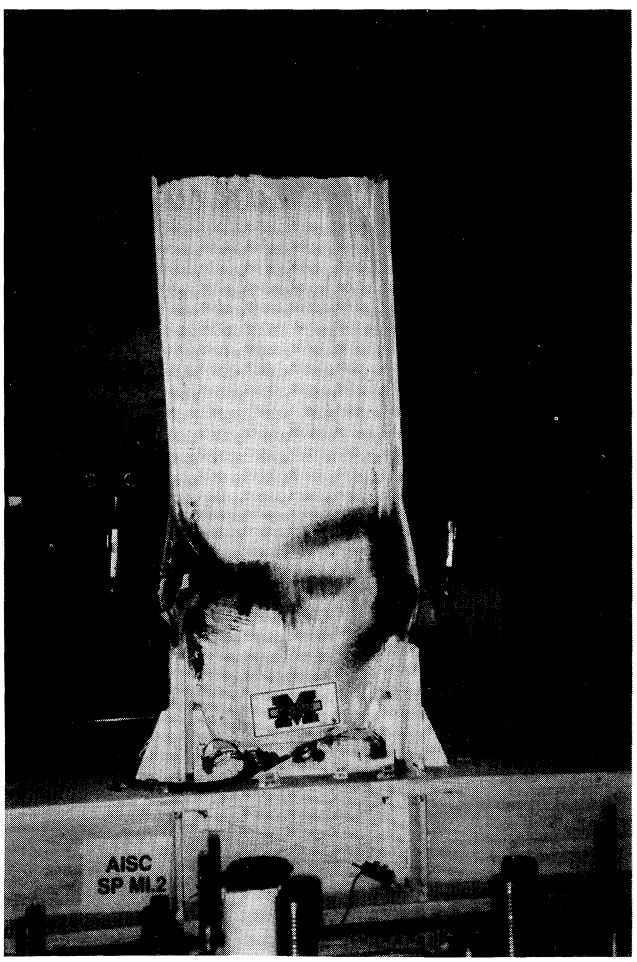


Fig. 14. Plastic hinge in the beam of Specimen AISC-MI2.

CONCLUSIONS

This study shows that the truss analogy approach provides a simple and practical method for design of welded beam-to-column moment connections. This new design approach was applied to a new connection design. The design procedure and the proposed connection design were successfully validated by two full-scale proof-tests.

Both proof-test specimens behaved as intended by design. The connection elements remained elastic without developing any undesirable local stress concentration. The column and the connection panel zone remained elastic, and did not contribute to plastic rotation capacity of the connection. A plastic hinge formed in the beam away from the connection. It was characterized by extensive yielding and local buckling of the beam web and beam flanges. The specimens sustained four consecutive cycles at a total drift of 4 percent without significant strength deterioration. Simultaneously, the beam plastic hinge developed plastic rotations of 0.04 radian. The test-ending limit state was severe lateral-torsional buckling of the beam. Therefore, the new connection satisfies current AISC requirements for use in Special Moment-Resisting Frames.

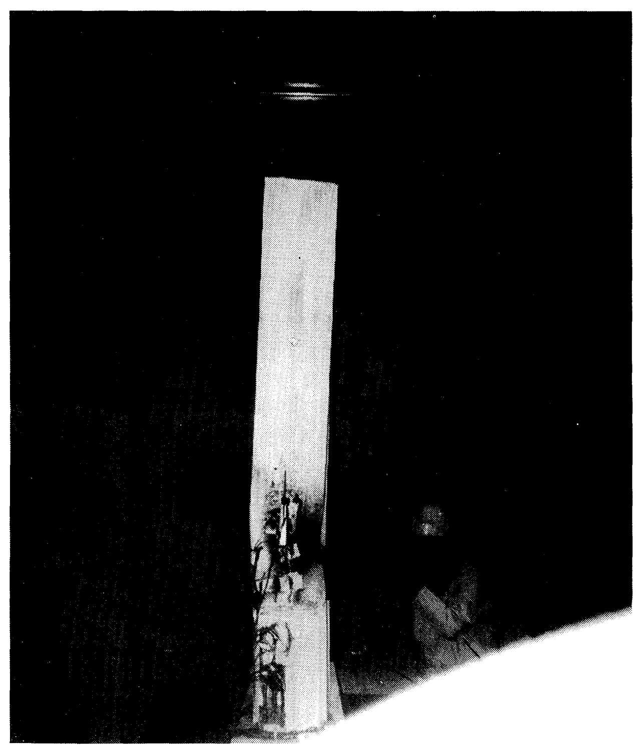


Fig. 15. Lateral-torsional deformation of a Specimen AISC-MI2 at 4 percent drift.

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