Improved Configuration of Weak-Axis Connections in Seismic Steel Moment Frames

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ABSTRACT :

Steel moment frame buildings have a number of beam-to-column weak axis connections (WACs). Seismic design of moment frame buildings expects these WACs also to accommodate 4-6% drift levels that the building sustains during strong earthquake shaking. Very few studies are reported in literature on seismic behavior of WACs; it is not clear if the current WACs can accommodate these drifts. The configuration of WACs currently in use has major deficiencies, like constrained flow of forces between beam and column. This paper presents an improved beam-to-column WAC configuration, which overcomes many deficiencies present in current WAC configuration. Displacement-based finite element analyses are performed to estimate the drift capacities of current and proposed WACs; the proposed connection is seen to perform much better.

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KEYWORDS: Steel frames, weak axis connections, seismic design, moment connections, drift capacities

1. INTRODUCTION

Steel Moment resisting frames (MRF) are considered to be the most efficient load resisting systems for seismic loads. In the event of a major earthquake the demand for energy dissipation on steel MRF's designed according to current building code requirements is very high. Good performance of steel moment resisting frame buildings during strong earthquake shaking largely depends on the efficiency of connections between structural members to transfer loads without premature brittle failures. The capacity of an MRF to dissipate energy will therefore primarily depend on the adequacy of the strength and the ductility of the MRF at beam-to-column joints.

Steel MRF buildings usually have frames oriented in orthogonal directions. In either of these frames, when I-sections are used for structural members, two sets of beam-to-column connections are possible, namely, (a) the *strong-axis* connection, where the beam frames into the column flange, and (b) the *weak-axis* connection, where the beam frames into the web of the column. Since the moment of inertia and the plastic moment capacity of I-sections are usually higher along their strong-axis, the strong-axis is preferred in connections that need to transfer moments as well as shears. But the connections along the weak axis too are subjected to the same seismic forces. To enhance the redundancy in the frames it is imperative to design the weak axis connections (WAC) also to sustain 4% drift levels. The existing prevalent WAC configuration leaves a lot to be desired. The challenge is to design this WAC also to sustain the seismic forces. This paper presents an analytical understanding of the stress distribution and flow of forces near and at the weak axis beam-to-column joint. The weak axis welded moment shear connection between AISC W-beam and column sections are studied through a typical exterior beam-column sub assemblage. Displacement based inelastic finite element analysis is carried out progressively to determine the path of flow of forces, stress concentration and failure zones. Based on this understanding a reinforced connection configuration is proposed.

2. BACKGROUND

Research on beam to column moment connections has been focused in the past on column flange connections where the beam frames into the flange. However, another type of moment resisting connection commonly found in steel frames on which little research has been conducted, is the column web connection. Information regarding weak axis connection is largely missing and the question is still more intricate for 3-D connections (Yardimici et al, 1996). Static testing of early web connections under symmetric loading was reported (Graham et al., 1959) in which four-way beam-to-column moment connection tests were conducted. Research on unsymmetric web connections under repeated and reversed loading was reported (Popov and Pinkney, 1969). Because of the lack of full-scale testing conducted on such connections, the understanding of these web connections is very limited. In order to acquire a better knowledge on the behavior of web moment connections, a testing program of these moment-resisting steel beam-to-column web connections was initiated at Lehigh University (Rentschler and Chen, 1975) in the mid seventies under the guidance of the American Iron and Steel Institute and the Welding Research Council. The main aim of this program is to provide theoretical predictions and experimental results on web moment connections under static loading using welding, bolting and a combination of welding and bolting as connection media.
In this connection, the beam is attached to the column perpendicular to the plane of the column web. The action of the beam bending moment tends to bend the column about its weak axis. The analysis and design of this type of connection is comparably more difficult than that for flange connections, because the maximum strength of the connection assemblage may be limited by the formation of plastic hinges in the column or in the beam, by the formation of a yield line type of mechanism in the column web, by the development of local buckling of the column flanges and web, and by the fracture of material of the assemblage (Chen and Lui, 1988). Yield line mechanism depends on the width of beam flange, depth of the beam, and column web thickness [G.P.Rentschler et al, 1980and 1982].While the mechanics of analysis and design do not differ from strong axis connection, the details of the configuration, connection design as well as ductility considerations required are significantly different (AISC, 2001). Based on experimental studies conducted on welded flange connections potentially weak areas may occur in weak axis connections. In the beam-to-column connection such areas may be the column flange and the column web. The transmission of high localized forces in the column may cause local yielding and local buckling. These failure modes may be decisive for the moment resistance of a connection. The weak areas identified were high strain concentration areas at the junction of connection plates with the column flanges and the junction between the connection plate and the beam flanges (Driscoll and Beedle, 1982). The suggestions recommended based on the study were incorporated in the present study and a comparison with the proposed connection configuration was conducted.

3. OBJECTIVE

The limited studies indicate that WAC analysis both experimental as well as analytical have been largely ignored. There were no clear recommendations regarding the flow of forces, configuration and, drift levels that these connections are capable of sustaining. The objectives of the present study are

(i) To understand the flow of forces from the beam to the column flanges for a Weak axis connection.
(ii) To identify the problem areas and the possible causes.
(iii) To propose a rectification for the problem zones through modified connection configuration.
(iv) To make a comparison of the proposed connection configuration with the existing prevalent connection configuration.

4. ANALYSIS SCHEME

The weak axis welded moment shear connection between AISC W-beam and column section was investigated through a typical exterior beam-column sub assemblage (W18x119 beam with W14x398 column). A typical joint sub assemblage consisted of a column of height equal to the center to center distance between the points of contra flexure both above and below the floor levels in consideration and a beam of span half the actual span length in the frame (Figure 1). The top and bottom ends of the column stubs were hinged. The beam end was free, and was subjected to the monotonic displacement loading. The beam was directly connected to the column web in the weak axis connection. Due to geometrical and loading symmetry only half the model was analyzed by introducing the requisite boundary conditions. All numerical investigations were done through displacement controlled nonlinear finite element analyses using ABAQUS 6.7. In all the finite element models, the beams were of grade ASTM A36 grade steel with yield strength F_y of 250 MPa, and the columns and connection elements were of ASTM 572 grade 50 steel with F_y of 345 MPa.The connecting weld material (E70 electrode) had yield strength 0f 345 MPa and ultimate tensile strength of 480 MPa at 20% elongation. All these materials had the same initial elastic modulus of elasticity E of 200GPa. Poisson's ratio was assumed to be 0.3 for all the three materials.

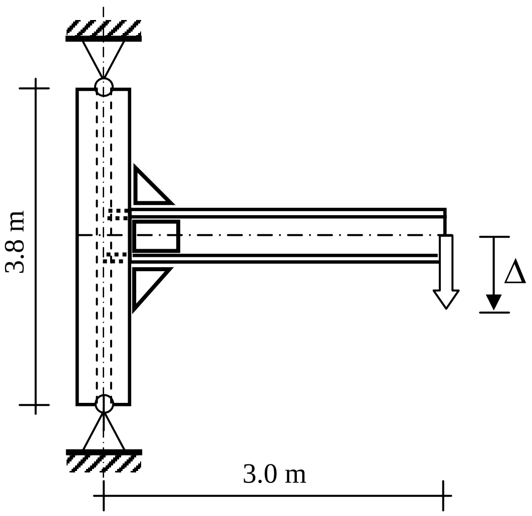
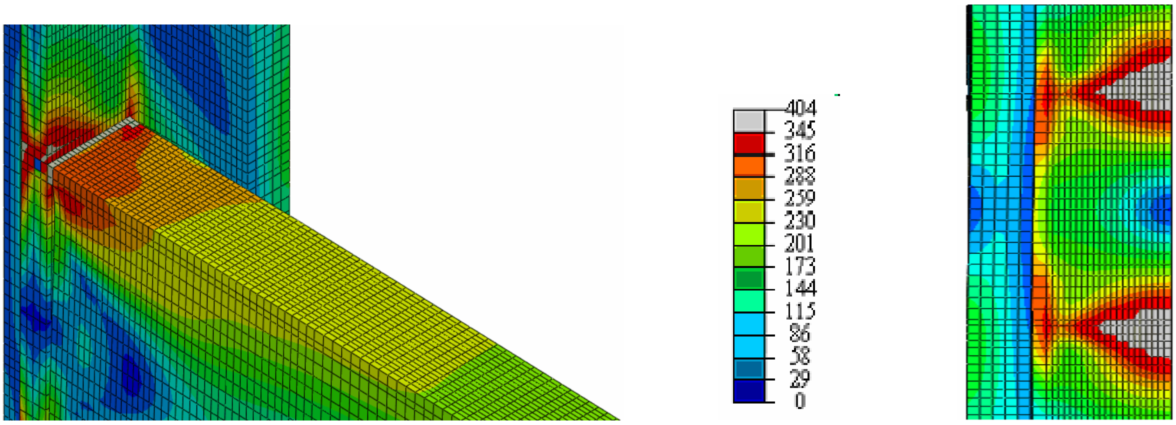


Figure 1: *Exterior WAC joint assemblage*: Geometry and loading of the subassemblage investigated

5 RESULTS AND DISCUSSION

For the ductile behavior of the beam-column connection, welds which exhibit brittle failure should not yield. On the other hand, steel being a ductile material, strain hardens and mobilizes strength beyond yield strength. Normal stresses σ_{xx} and shear stresses τ_{xz} together participate in yielding the beam section causing plastic hinging to occur. Hence the design strategy to ensure seismic behavior is to locate the plastic hinge in the ductile beam away from the connection elements and brittle welds. Therefore the desired path for flow of forces keeping in line with the above understanding should be through the beam flanges to the column flanges without subjecting the connection elements and the associated welds to stress concentration. The investigation on unreinforced weak axis beam-column connection (Bare configuration) resulted in the following observations shown in Figure 3.2: (i) Complete Joint Penetration (CJP) groove welds connecting the beam to the column web yield at about 0.1% drift with the initiation at the tip of the beam flange (ii) Out of plane deformations of the beam flanges and the column web. Excessive column web buckling occured and there was no transfer of forces to the column (iii) Cold column flanges indicate no force flow to them indicating that beam flanges narrower than the clear distance between the column flanges do not provide a path for the flow of forces (iv) Connecting weld at the beam neutral axis showed no yielding indicating negligible transfer of force through the beam web to the column web



(a) (b)

Figure 2: *Stress contours of unreinforced WAC*: (a) Stress contours indicate early yielding of connecting welds, cold column flanges and, no transfer at the beam web level. (b) Excessive column web yielding and deformations.

This behavior indicated that the connection region required reinforcement to facilitate flow of forces to the column flanges from the beam, to prevent brittle weld failure, reduce stress concentration zones and push the plastic hinging into the beam away from the connection region.

Non linear finite element analysis was conducted in a progressive manner to address the issues identified in the analysis of the unreinforced (Bare) connection and to arrive at the proposed connection configuration which considerably rectifies them. Initially as per the literature guidelines, prevalent configuration with only cover plates resulted in stress concentration at the reentrant corner which is the junction of the cover plate with the column flange and no force flows to the column flanges. Beam panel zone had high stress regions closer to the beam flanges and entry fillet welds yield at 0.33% drift levels (Figure 3).

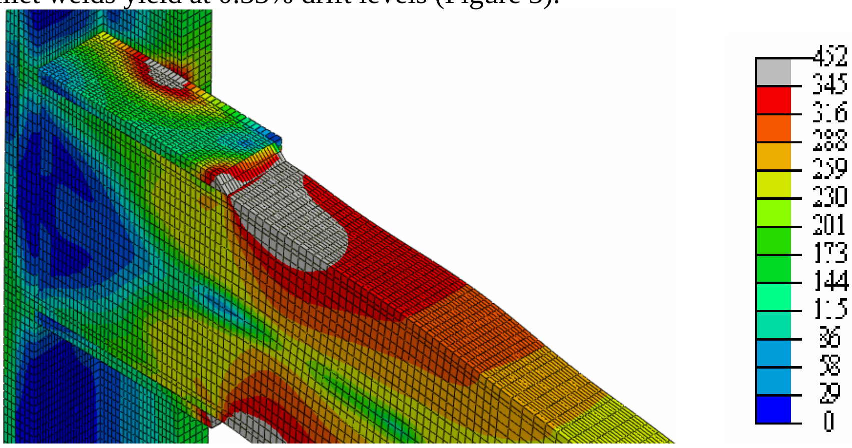


Figure 3: *Existing configuration for WAC*: Stress contours indicate high stress concentration at the entry fillet weld and the beam flange and column flange junction.

Hence the cover plates were modified as collar plates that go around the column flange to rectify the concentration of stress at the reentrant corner and the beam was also flared by providing *horizontal haunches* to the column flange tip to provide a horizontal plane for the flow of forces. Vertical inclined rib plates provide a vertical plane to allow the smooth flow of beam shear force to the column flanges. CJP welds connecting the rib plates to the column flanges were critical, hence to shift stress concentration from these welds and to stiffen the column flanges against out of plane deformation or buckling, vertical stiffener plates of same thickness as the rib plates were provided at the front and rear. To facilitate the transfer of beam web shear to the column flanges and to provide a planar continuity a beam web plate of the same thickness as the rib plates and height equal to the beam inner clear height was provided which reduced shear stress values in the beam panel zone. The proposed configuration and its behavior at 4% drift are shown in Figure 4.

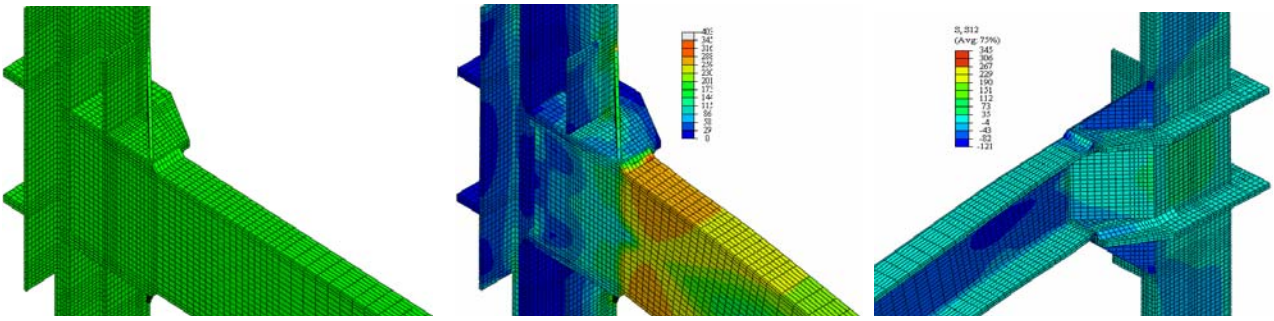


Figure 4: *Proposed WAC configuration*: (a) Proposed connection configuration showing the connection components (b)von Mises stress contours representing the effectiveness of the configuration at 4% drift (c)shear stress contours depicting the transfer of shear forces through the inclined rib plate.

The von Mises stress values in the connection region including in the critical welds were below yield value of 345 MPa.. The plastic hinge formation had been shifted to the beam which was initiated in the beam flanges and progressed to the web away from the connection region. The shear stress contours depicted a clear flow path of the shear forces through the inclined rib plates. The shear stress variation for the three cases was compared to show the shift of the initiation of plastic hinge into the beam in Figure 3.5. The configuration with only tapered cover plate (prevalent configuration) shifted the yielding into the beam but the critical welds yielded at very low drifts. The proposed configuration sustained 4% drifts without the connection elements and critical welds yielding. The beam panel zone also had values much below the yield values.

Thus the proposed configuration of collar plates, inclined rib plates and stiffeners effectively facilitated a smooth flow of forces from the beam to the column flanges.

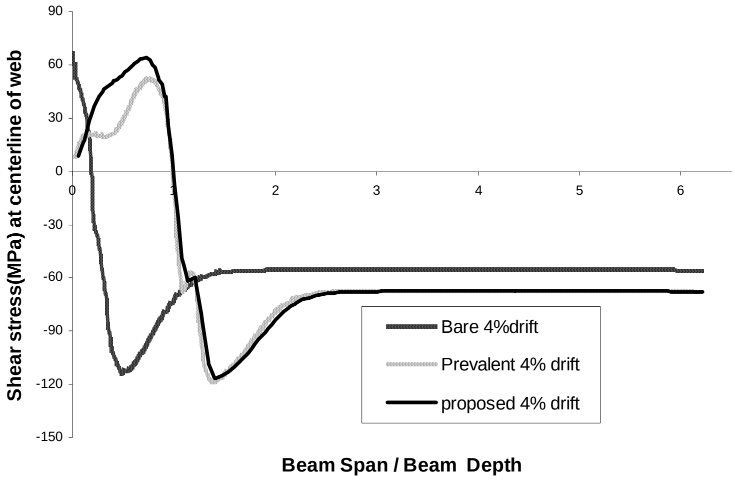


Figure 5: Shear stress variation along the beam span at the beam neutral axis for the three configurations studied.

6. CONCLUSIONS

Following were the main conclusions of the analytical study

(i) The proposed connection effectively sustained drift levels up to and beyond 4% as in strong axis connections

(ii) The critical entry fillet welds and CJP groove welds connecting the rib to the column flanges did not yield at even 4% drift levels

(iii) Local deformations in column were significantly reduced
(iv) The flow efficiency in the proposed connection was significantly higher as compared to the flow in the prevalent configuration.
(v) The proposed configuration of collar plates, inclined rib plates and stiffeners effectively facilitated a smooth flow of forces from beam to column flanges

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