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IMPROVING THE PERFORMANCE OF STEEL BEAM-COLUMN MOMENT-RESISTANT CONNECTIONS

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SUMMARY

The behaviour of steel beam-column, moment-resistant connections has been under intense review since the Northridge earthquake of January, 1994. These connections are traditionally designed assuming that the beam moment is applied to the column primarily through a top and bottom beam flange force couple while the web connection transfers the applied shear. However, these assumptions may not be appropriate to accurately represent the stress distribution within the connection. Therefore, the demand on the various connecting elements, particularly the full penetration groove welds, is not adequately predicted.

A re-examination of the load path through the steel beam-column moment-resistant connection and the resulting demand on the connecting elements for various connecting element configurations is pursued. Three-dimensional finite element models of test specimens used to represent the behaviour of typical beam-column connections at an exterior column of a multi-story steel frame are developed. Loaded through a displacement history applied to the beam free end consistent with previous and concurrent experimental tests, results from the analytical models indicate that in transferring the beam moment to the column the stress distribution in the beam flange-to-column flange connecting weld is not uniform across the beam flange width with a maximum at the center of the beam web significantly higher than that at the edge of the beam flange. It is believed that, with an improved understanding of the load path through the connection and the resulting demands on the connecting elements, it is possible to achieve, through appropriate design of reinforcing elements, significant strength and rotational capacity of steel beam-column moment-resistant connections.

INTRODUCTION

In the days following the Northridge, CA earthquake of January 17, 1994, reports of brittle failures in steel connections began to surface, first from buildings under construction at the time of the earthquake, and then from additional buildings where fireproofing and finish material were removed during investigations prompted by the earlier steel fracture reports [EERI Preliminary Reconnaissance Report, 1994]. As a result, with connection performance contrary to that expected, several investigations began with the purposes of explaining how the failures occurred, determining how to repair existing damaged buildings, and investigating various design alternatives in order to achieve more satisfactory connection performance. In general, these investigations have suggested that not any one issue can be isolated as the sole cause of the poor connection performance. Moment connection behaviour is a complex interaction of several parameters.

In a typical bolted-web, welded-flange (BWWF) moment-resistant connection, the full penetration flange welds are highly stressed under beam moment so that, for a BWWF connection to successfully resist earthquake loads, the high quality of these welds is essential. The maximum beam moment that can be developed in the beam-column connection depends on several parameters and may be limited by the panel zone shear strength or the column bending strength. It is important to note that many of the connection failures that occurred during the Northridge earthquake fractured at stress levels in the beam clearly below first yield although some panel zone yielding had taken place [NIST, 1997; FEMA, 1995]. In order to achieve improved performance of these

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connections, a better understanding of the effect of connection member material properties and connection configuration on the load path, or stress distribution, through the connection is needed.

MODELING CONNECTION BEHAVIOR

Figure 1a shows a schematic of a typical connection test specimen. The type of specimen shown is used to represent the behaviour of a typical beam-column connection at an exterior column of a multi-story frame. In a typical experiment of an exterior column connection assembly, the test specimen column is restrained vertically at the bottom and restrained horizontally at the top and bottom. The specimen is loaded through a hydraulic actuator at the beam tip. The beam tip is then moved through a vertical displacement history, simulating the loading of a connection as part of a moment frame subjected to a lateral load history. The displacement history can be applied so as to employ either very low or high strain rates.

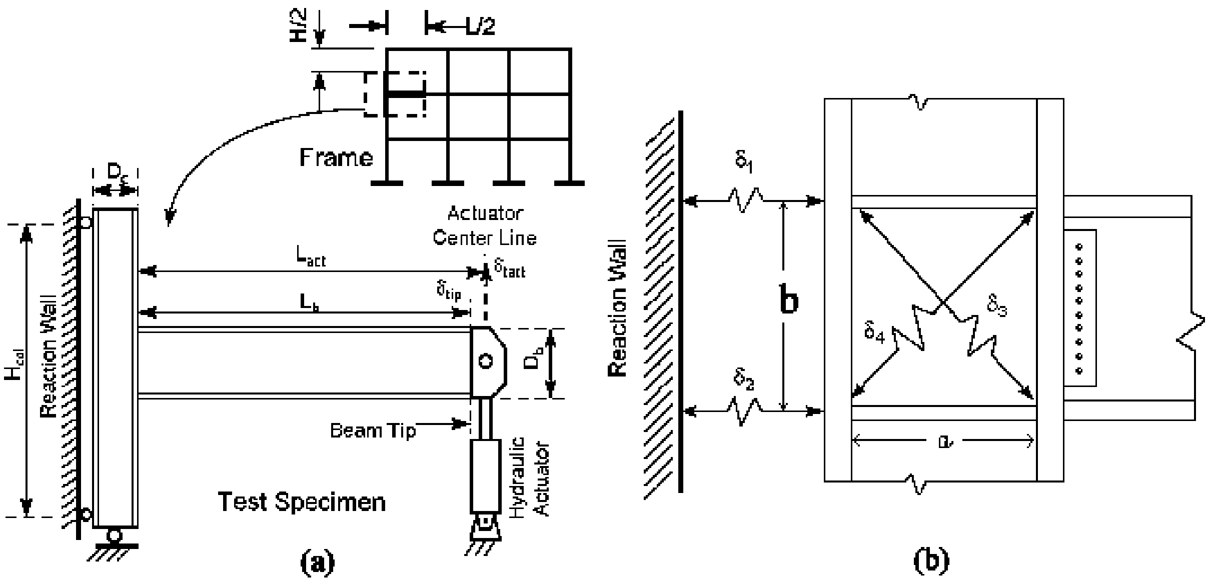

Figure 1. (a) Schematic of a typical connection test specimen, (b) Schematic illustrating moment connection deformation parameters [SAC, 1996].

Several parameters are measured and recorded throughout the displacement history. Most notably, the force applied at the beam tip, *P*, is recorded, as well as the beam tip displacement, δ_{tip} , the lateral column deflection at the top and bottom of the panel zone, δ_1 and δ_2 , respectively, and the diagonal deformation of the panel zone, δ_3 and δ_4 . From these measurements, the contribution of various connection assembly components to the total assembly deformation can be evaluated through the use of several deformation parameters. These parameters are the panel zone rotation angle, γ , the beam tip deflection due to panel zone rotation, δ_γ , the column rotation angle, θ_c , the beam tip deflection due to column rotation, δ_c , the beam rotation, θ_b , and the beam tip deflection due to beam rotation, δ_b . Referring to Figure 1b, the panel zone rotation angle, γ , and the column rotation, θ_c , can be calculated as

$$\gamma = \frac{\sqrt{a^2 + b^2}}{D_b}(\delta_3 - \delta_4) \tag{1}$$

and

$$\theta_c = \frac{\delta_1 - \delta_2}{D_b} - \gamma\left(1 - \frac{D_b}{H_{col}}\right) \tag{2}$$

where a is the horizontal panel zone dimension, b is the vertical panel zone dimension, D_b is the beam depth, and H_{col} is the column height. The corresponding displacements, δ_γ and δ_c, are then determined such that

$$\delta_\gamma = \gamma L - \frac{\gamma D_b}{H_{col}}\left(L_b + \frac{D_c}{2}\right) \qquad [3]$$

and

$$\delta_c = \theta_c\left(L_b + \frac{D_c}{2}\right) \qquad [4]$$

The beam displacement, δ_b, and the beam rotation, θ_b, are then determined as

$$\delta_b = \delta_{tip} - \delta_c - \delta_\gamma \qquad [5]$$

and

$$\theta_b = \frac{\delta_b}{L_{clear} - \frac{D_b}{4}} \qquad [6]$$

where L_{clear} is the distance from the point of load application to the end of the haunch or cover plate, or to the column face if no haunch or cover plate exists [SAC, 1996].

In assessing the performance of these connections, SAC [FEMA Interim Guidelines, 1995] recommends that the connections be able to achieve a plastic rotational capacity of 0.03 radian. This recommendation is usually taken to mean the beam plastic rotation, θ_{bp}, should exceed 0.03 radian. In addition, the connections must be capable of developing the plastic moment capacity of the beam. In tests conducted at the National Taiwan University prior to the Northridge earthquake, results indicated that the ultimate moment capacities of beam-to-box column BWWF connections can be predicted solely from the beam flange flexural strength, [Tsai and Liu, 1992].

The relationship between the strength demand at the beam-column connection and the beam plastic rotation was studied by Tsai and Liu [1992] using an analytical model of cantilevered beams subjected to a concentrated load applied monotonically at the beam free end. Using a tri-linear stress-strain steel material model, Tsai and Liu determined that the relationship between the fixed end moment, M_J, and the beam plastic rotation, θ_p, can be expressed as

$$a = \frac{M_J}{M_p} = \frac{3}{2}(1 - u\nu) + \frac{1}{3}\nu(2 + f) + \frac{E\nu(2+f)}{2F_y r}\theta_p \qquad [7]$$

where $u = \varepsilon_{st}/\varepsilon_y$, $\nu = E_{st}/E$, $f = Z_f/Z$, $r = L/(d - 2t_f)$, ε_{st} is the strain range of perfect plasticity, ε_y is the yield strain, E_{st} is the strain hardening modulus, L is the beam length, d is the beam depth, and t_f is the beam flange thickness. a is the moment amplification factor and represents the effect due to strain hardening. For a plastic rotational capacity $\theta_p = 0.015$, considered a minimum for a typical beam-column connection in a moment resistant frame [Tsai and Popov, 1988], Tsai and Liu showed that a ranges from 1.05 to 1.28 and is more sensitive to the beam length-to-depth ratio, r, and the critical strain ratio, u, than to the modulus ratios, f and ν so that the smaller the beam length-to-depth ratio, the larger the flexural strength demand that results at the support. Furthermore, since it is believed that the amplified fixed end moment is deformation path dependent, this end moment growth can be accelerated under cyclic loads [Tsai and Liu, 1992].

METHODS OF IMPROVING CONNECTION PERFORMANCE

Since the flexural capacity of the BWWF moment connection is limited by the beam flange flexural strength and the flexural demand on the connection is likely greater than the beam plastic moment capacity, the strength criterion, $Z_f F_u \geq aZF_y$, must be met in order for the moment connection to sustain the beam bending moment as inelastic beam rotation develops. Thus, with $Z_f F_u$ as the design strength of the connection, and aZF_y as the generalized load effect, an LRFD criterion for the strength limit state of seismic BWWF moment connections can be stated as

$$Z_f \left(F_u\right)_n \geq \alpha Z\left(F_y\right)_n \quad [8]$$

where $(F_y)_n$ is the nominal tensile yield strength, $(F_u)_n$ is the nominal ultimate tensile strength, and α is the design coefficient such that

$$\alpha = \phi \frac{\mu_{F_y} /(F_y)_n}{\mu_{F_u} /(F_u)_n} a \quad [9]$$

where a is from Equation [7], ϕ is from Ang and Tang's [1984] probabilistic model, μ_{F_y} is the mean value of the yield strength, and μ_{F_u} is the mean value of the ultimate strength. Generally, then, the two main methods of enhancing BWWF connection performance are either to enhance the strength capacity by increasing the effective flange modulus at the connection, such as through the use of cover plates or vertical rib plates and haunches, or to reduce the strength demand by decreasing the effective beam section modulus near the connection, such as is done by shaving the beam width like a "dogbone". Indeed, most of the design alternatives considered in the research following the Northridge earthquake easily fall into either of these two categories.

ASSESSING CONNECTION ENHANCEMENT

For either of the two methods of connection enhancement, several factors must be considered. These factors can be described as either geometric properties or material properties. Geometric properties include the relative beam and column member sizes, the location of stress raisers, and areas of member element restraint and three-dimensional stresses and strains. Material properties include beam-to-column relative strength and column flange through-thickness properties. It is important to keep in mind that these geometric and material factors are inter-related. That is, the existence and size of a weld defect such as a slag inclusion, for example, may be considered a geometric property. Yet, the effect of this weld defect on the connection performance will depend, not only on the nature of the applied load, but also on the fracture toughness of the weld material and/or base metal.

Geometric Properties

Weld Flaw Location

In a study of connections that failed during the Northridge earthquake, it was found that many of the fractures at the bottom beam flange groove weld initiated from weld defects many of which were of sizes that would have been rejected per AWS D1.1 if the defects were discovered during construction. In particular, slag inclusions were commonly found in the middle of the flange width, likely due to the necessity of interrupting the weld because of the beam web. Furthermore, it is important to note that the distribution of bending stress across the width of the beam flange is not uniform and is higher at the center where the majority of weld flaws were found [NIST, 1997].

High Triaxiality

As is the case for most BWWF connections, the full penetration groove weld is highly restrained through its length as well as in the transverse direction. Thus, while the steel and the weld metal are ductile, the beam flange-weld-column flange connection is extremely rigid and any fracture that occurred in this region would be expected to be brittle [Tsai and Popov, 1995].

Concrete Slab

Although the effects of the concrete slab are typically not considered in the design of steel moment-resistant frames, ignoring the additional strength and restraint of the composite floor is unconservative. The interaction of the beam and the slab results in the neutral axis shifting upward toward the slab. This higher neutral axis results in larger strains at the bottom flange compared to the beam top flange and may partly explain the predominance of bottom flange failures during the Northridge earthquake [Tsai and Popov, 1995; Leon, Hajjar, and Shield, 1996].

Material Properties

Relative Connection Member Strength

The relative strength of the beam, column, and panel zone members of a connection has a significant effect on the distribution of plastic deformations. Typically, moment resistant frames are designed using a weak beam-strong column design philosophy so that it is expected that plastic hinges form in the beam to dissipate the transmitted ground motion energy. Also, yielding of the column panel zone can be a stable energy dissipation method but excessive panel zone shear yielding, due to a relatively weak panel zone, may not be desirable since it may lead to local kinking at the panel zone corners resulting in high strains and curvatures in the region of the beam flange welds [Krawinkler, 1978].

Through-Thickness Properties

Rolled structural steel shapes are not isotropic in that rolled steel members show greater ductility in the direction of rolling. In fact, some experiments have shown that the ductility in the through-thickness direction (orthogonal to the direction of rolling) was only half as large as that in the direction of rolling [Adams and Popov, 1976].

ANALYSIS OF STEEL BEAM-COLUMN MOMENT CONNECTIONS

In an effort to better understand the behaviour and performance of bolted-web welded-flange steel moment connections, an analytical model of one of the specimens experimentally tested as part of Phase I of the SAC Joint Venture at the Earthquake Engineering Research Center at the University of California at Berkeley was developed. The analytical model consists of a 3-dimensional finite element model. The finite element model was loaded through a static displacement history applied at the free end of the beam and results were compared with those of the SAC specimen. The primary focus in the development of this model was to capture as much connection detail as possible so as to obtain an accurate representation of the stress and strain distribution throughout the connection.

Model Definition

Mesh

The model consists of 8-noded solid elements and is shown in Figure 2. The beam web, column web, and shear tab elements employed a standard 8-noded brick element formulation while the flange elements used an 8-noded incompatible mode formulation due to the improved capability of the incompatible mode formulation to model bending of thin plates [HKS, 1997]. Also, this model considers the top and bottom cover plates to be rigidly attached to the beam flanges, so that the cover plate and the flange share nodes along the interface. The beam web and shear tab were connected similarly. Note that this may not accurately represent the load path through these member connections and may yield a different stress distribution, particularly at the beam flange-to-column flange weld.

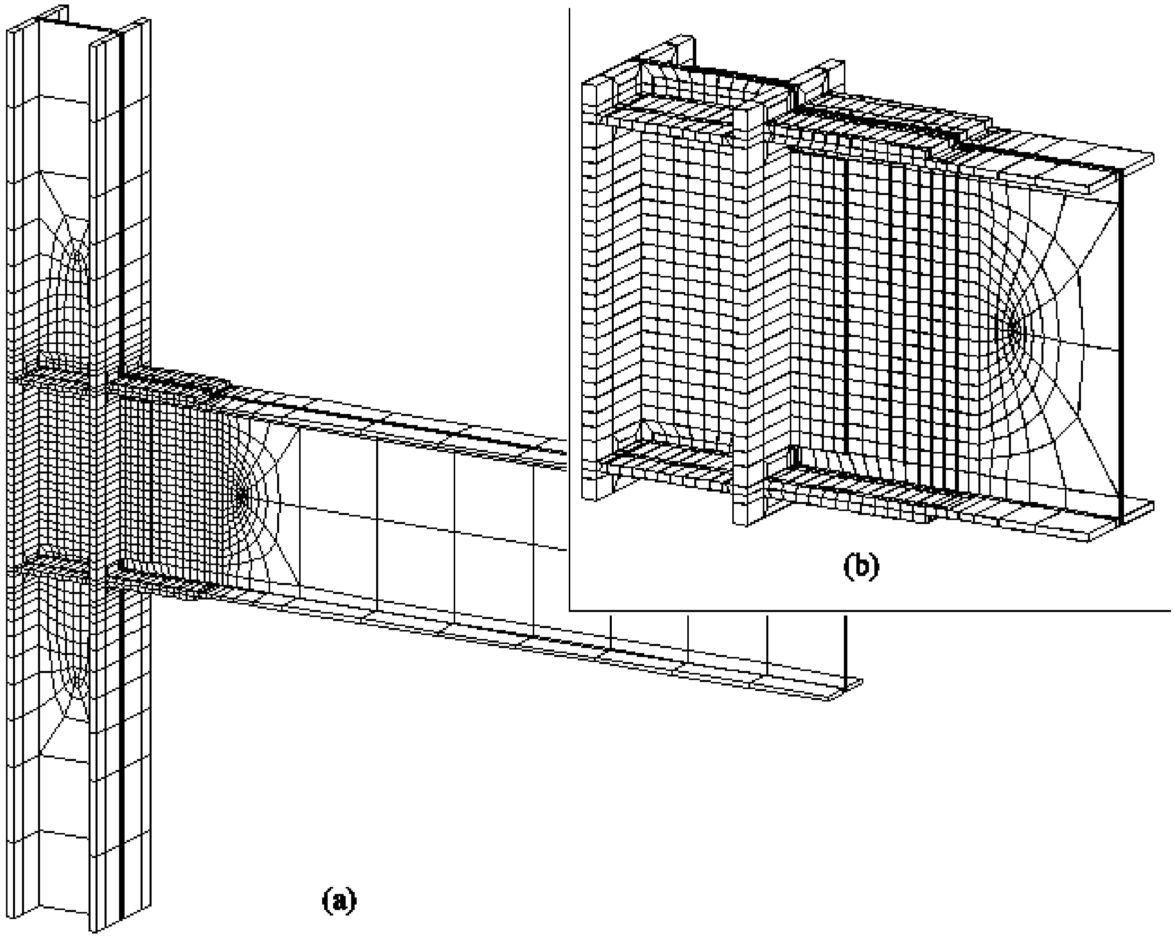

Figure 2. (a) Finite element model mesh of the SAC connection detail. (b) Detailed view of the mesh in the panel zone area.

A second model that is identical to the first model except that the top and bottom cover plates, along with the continuity plates, and the corresponding welds were removed, was also developed. This second, unreinforced connection model, was developed as a comparison to the connection reinforced with cover plates.

Material Properties

The material model used for the beam and column is a classical metal plasticity model with isotropic hardening. The beam consists of A36 isotropic steel with an elastic modulus of 29000 ksi (200000 MPa). The beam flange yield strength is 50.3 ksi (346.8 MPa) with an ultimate strength of 70.9 ksi (488.9 MPa) at 20% plastic strain while the beam web yield strength is 55.7 ksi (384.1 MPa) with an ultimate strength of 71.9 ksi (495.8 MPa) at 20% plastic strain. The column consists of A572 Grade 50 isotropic steel with an elastic modulus of 29000 ksi (200000 MPa), a yield strength of 53.0 ksi (365.4 MPa), and an ultimate strength of 75.0 ksi (517.1 MPa) at 20% plastic strain. These material properties were used so as to match the values of tensile coupon tests of samples taken from specimen EERC-AN1 of the SAC Steel Project [SAC, 1996]. The material properties of the shear tab were chosen to match those of the beam web while the top and bottom cover plates and column continuity plates had material properties matching those of the column. The weld metal material properties consist of an elastic modulus of 8400 ksi (58000 MPa) and a yield strength of 70 ksi (482.7 MPa) and is considered to be elastic-perfectly plastic. Residual stresses due to rolling and the weld process are ignored in this model.

Applied Displacement History

The displacement history corresponds to the displacement history applied in the SAC experiment [SAC, 1996].

Results

Figures 3 shows the beam moment-plastic rotation relation of the reinforced and unreinforced connection models, where the moment is the moment at the column face. Note that the beam-moment plastic rotation hysterisis loop of the reinforced connection model is in relatively good agreement with the corresponding plot from the SAC experiment. Figure 4 shows the tensile stress in the beam bottom flange weld near the column face at various points across the flange width. Note that the stress at the center of the beam flange is significantly higher than near the edges of the flange.

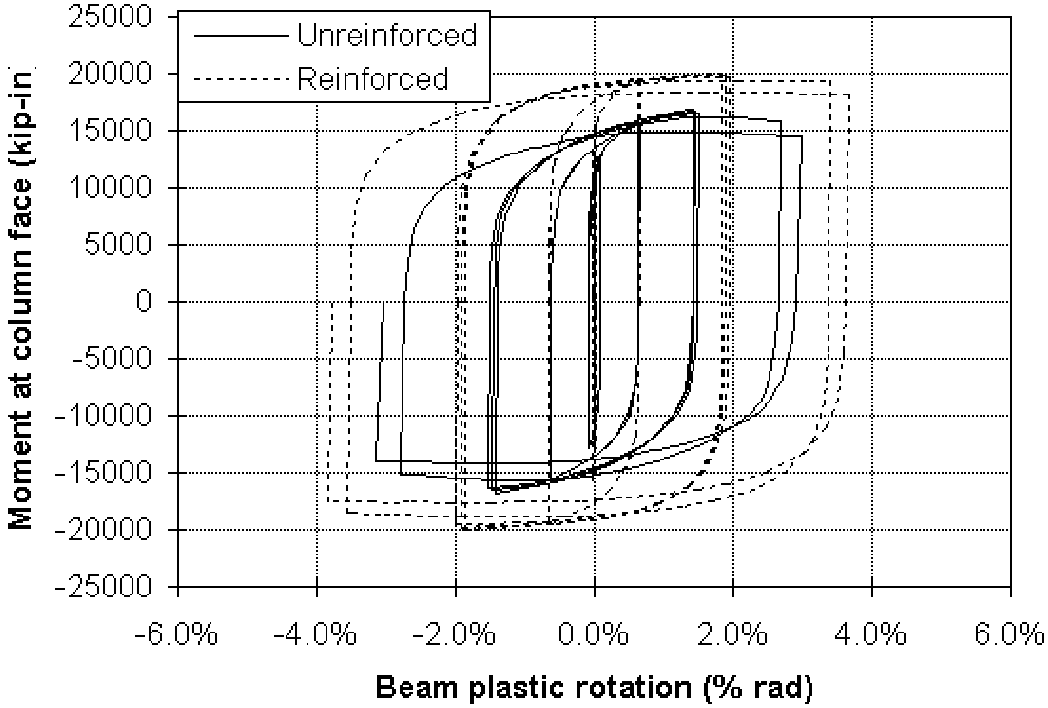

Figure 3. Comparison of the beam moment-plastic rotation relation between the reinforced and unreinforced connection model (1 kip-in = 113.0 N-mm).

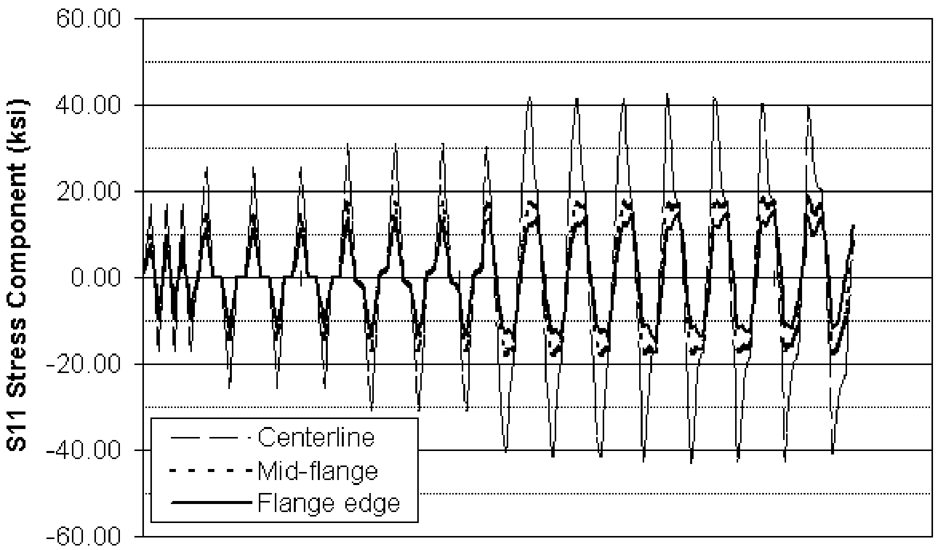

Figure 4. S11 stress components (beam axis direction) over the displacement history at midthickness of the lower beam flange/cover plate-to-column flange groove weld of the reinforced connection model at the beam web centerline, mid-flange, and near the flange edge (1 ksi = 6.895 MPa).

DISCUSSION

In an effort to further enhance the current FEM, future models may include modeling of the bolted shear tab to beam web connection as well as modeling of the interface between the cover plates and exterior beam flange surfaces. Furthermore, modeling of the residual stresses due to the rolling process and to the weld process can be included. Material modeling enhancements include modeling the through-thickness behavior of the column flange as well as more detailed plasticity effects through a stress-strain behavior model. It is believed that a model consisting of all such enhancements will represent the most accurate picture of connection performance.

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