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STRUCTURAL ASSESSMENT OF STEEL BEAM-TO-COLUMN CONNECTIONS SUBJECTED TO CYCLIC LOADING BY NONLINEAR FINITE ELEMENT ANALYSIS

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Abstract

The non-linear behavior of four steel beam-to-column connections subjected to cyclic loading, commonly used for steel moment frames in Mexico, are assessed with the technique of the finite element (FE) method, with the aim to qualify their use for seismic applications as special (SMF) or intermediate (IMF) moment frames. The FE setup of the connection model has a T-shaped configuration, which represents a sub-assembly of an exterior connection in a steel moment frame subjected to earthquake forces simulated by the cyclic loading protocol from the AISC seismic provisions [1]. The non-linear FE analysis of 3D connection models were carried out using the software ABAQUS. Experimental tests of the connections evaluated herein will validate these numerical simulations, extend it to other cases of analysis, and improve it by including the fracture prediction for the structural steel and welds.

Keywords: beam-to-column, connections, steel, moment frame, seismic, cyclic.

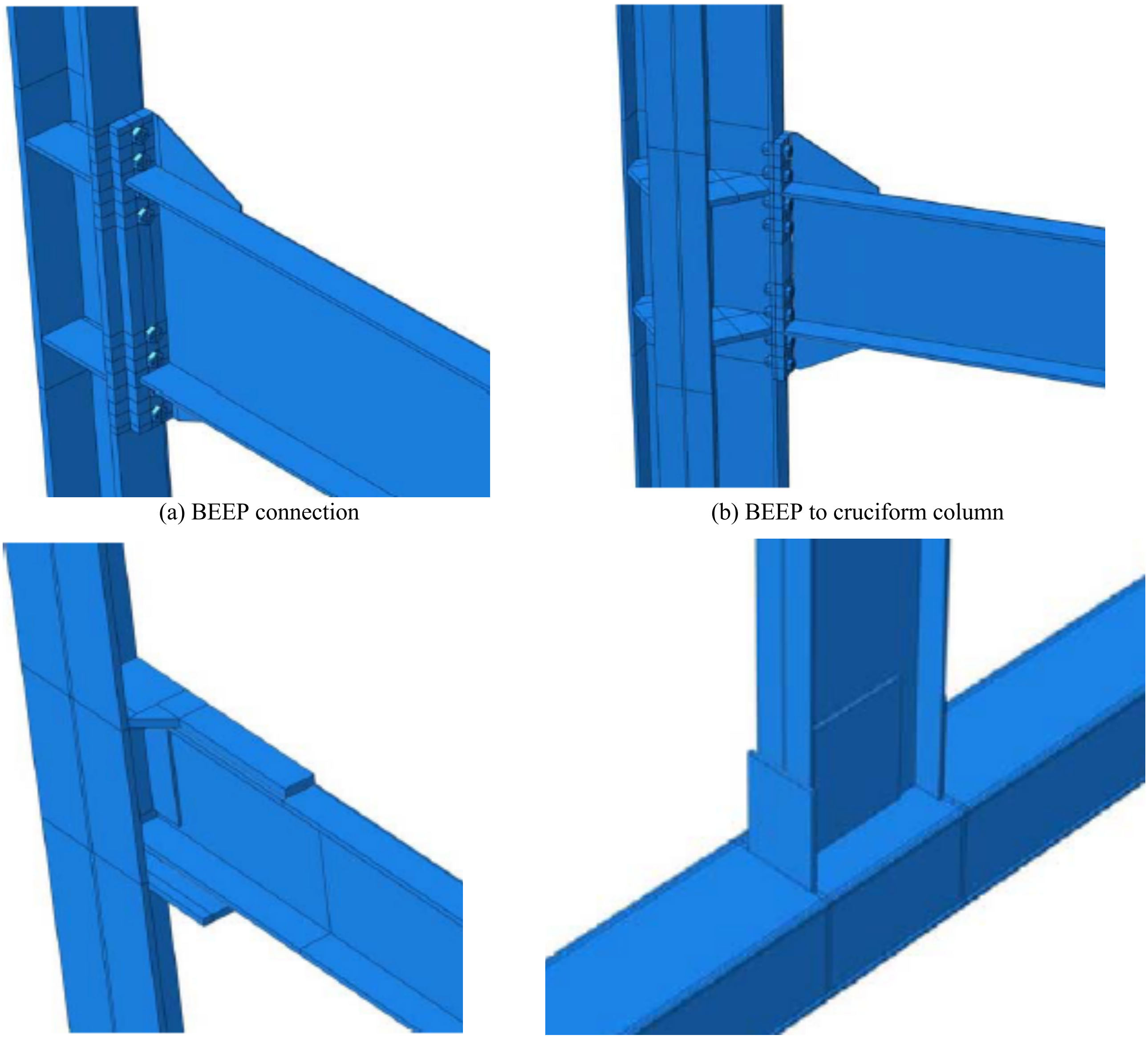
1 Introduction

This paper studies the behavior of some beam-to-column connections that are typical used in steel moment-resistant frames (MRF) in Mexico. A capacity design in special moment frame (SMF) and intermediate moment frame (IMF) systems is promoted with a *strong column - weak beam* plastic mechanism, a design philosophy under which in the occurrence of strong earthquakes, plastic hinges are allowed within the beams, and with limited shear yielding in the panel zone. The members in steel moment frames must be connected using rigid connections that allows the system to develop the expected ductility in design, *i.e.* at least 0.04 drift for SMF and 0.02 drift for IMF according to local standards [2], which is similar in other international seismic provisions, e.g. [1].

This research evaluates the connections that have been used and qualified in other countries (e.g. [3]), with the necessary adaptations that are commonly applied in the local standard practice [4]. Some of these adaptations consist of using columns with either hollow cross-sections (HSS), built-up box cross-sections, built-up flanged cruciform cross-sections, or H shape cross-sections with the beam connected to the column web. One of the main issues of using these local adaptations is related to the fact that there is not enough analytical or experimental evidence whether these would be adequate in the application of steel special or intermediate moment frames. Thus, the main contribution of this work lies in the analytical evaluation of some connections locally adapted based on non-linear analysis with finite elements (in the absence of experimental data) and, based on these analyses, establish whether or not they qualify for seismic applications.

Non-linear analyzes with finite elements of beam-to-column connections are made using the Abaqus software. These analyzes implicitly include material and geometric non-linearities, in addition to the fact that they can simulate the development of steel yielding and local buckling, as well as panel zone yielding. A limitation of these analyzes is that plasticity constitutive models do not consider the fracture for both the steel and welds, so the results in this work are reliable as long as fracture does not occur at either the steel or the welds [4].

The evaluated connections (Fig. 1) have similar configurations to some of the prequalified connections given in the document AISC 358-16 [3], but with adjustments that are made in the local practice, including cases of steel beams connected to either built-up box columns or built-up cruciform columns, or connected to the web in I or H columns generating weak-axis bending. The connections that are evaluated in this work include the following four cases: (a) Bolted extended end-plate (BEEP) connection as shown in Fig. 1(a); this model was useful for the calibration, since the numerical results are compared to the experimental data obtained from testing [7]. (b) BEEP connection to a flanged cruciform column; this model is similar to the previous connection case but using a built-up flanged cruciform column (from two wide flange shapes). (c) Welded flange plate (WFP) connection, which uses cover plates fillet welded longitudinally to the beam flanges, and then welded to the web column producing weak axis bending. (d) WFP connection using a built-up box column, which also uses covers plates that are fillet welded to the beam flanges and groove CJP welded to the column; the inner diaphragm is fillet welded in three sides with the box column, but the welding between the fourth side of the column and the inner diaphragm is evaluated with some variants including: (1) fully welded, (2) partially welded using plug welds, and (3) no welded.



(a) BEEP connection

(b) BEEP to cruciform column

(c) WFP to column web

(d) WFP to box column

Fig. 1 – Evaluated connections using non-linear finite element analysis [4]

2 Finite element modelling validation

In this section, the results of the finite element analysis performed for the first proposed connection are described, and the comparison of this predicted numerical response with the data measured from the test of a full-scale connection reported by Sumner *et al.* [10] is given. Both the experimental specimen and the numerical model have the same geometry, cross-sections and material properties, and thus, this analysis was useful for calibration and validation of the FE Analysis.

2.1 Specimen description

This connection is calibrated with the connections 8ES-1$^1/_4$-1$^3/_4$-30 tested by Sumner *et al.* [10]. This beam-to-column connection consists of a W30×99 beam connected to a W14×193 column, both ASTM A572 Gr. 50, and using a strong bolted extended end-plate connection (*i.e.* end-plate of ASTM A36 and 1$^3/_4$ in. thick, and eight bolts per flange of ASTM A490 with 1$^1/_4$ in. diameter). This connection, shown in Fig. 2, is submitted to the load protocol proposed by the SAC project [5] (Fig. 3) and adapted in the AISC Seismic Provisions [1]. In this quasi-static test, where the displacements are applied in low speed, the inertial effects can be neglected and so the problem can be solved through iterations in an implicit type procedure [6].

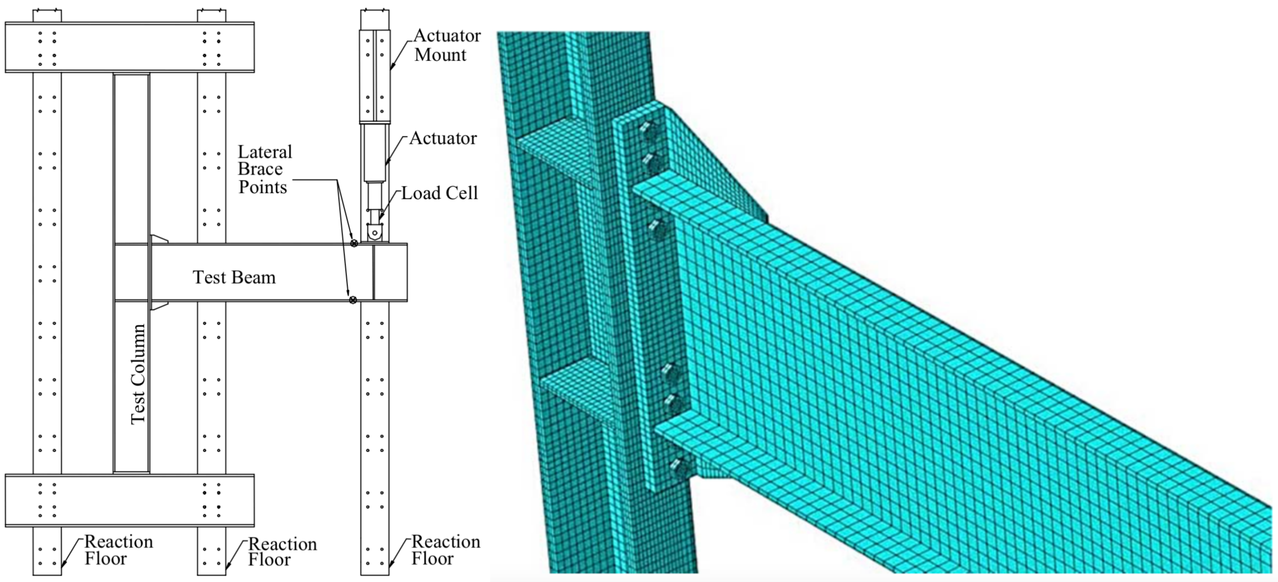


Fig. 2 – Test setup of the specimen tested by Sumner *et al.* [10] and the FE model [4]

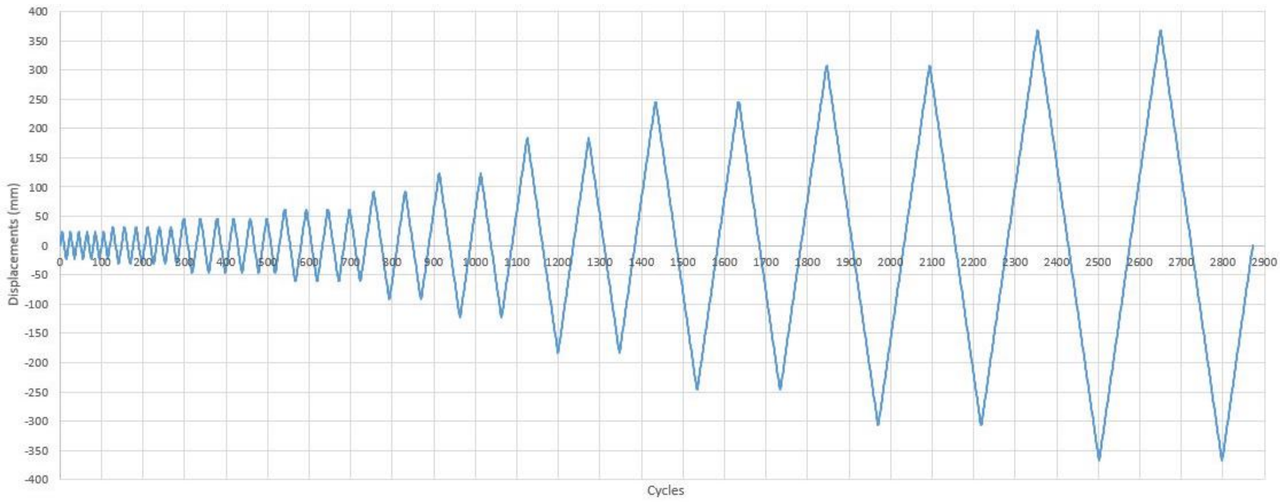


Fig. 3 – Load protocol applied in displacement control [5]

In order to validate and calibrate the finite element analysis, the numerical simulation results are compared with the connection previously evaluated experimentally by Sumner *et al*. [10]. This connection is mainly chosen because of the extensive documentation reported by the authors, which allows to develop an accurate finite element model of the specimen.

2.2 Analysis assumptions

Steel is modelled as a plastic material, considering a combined hardening plasticity model, which is recommended for modelling metals under cyclic loads. This combined hardening constitutive model capture the Bauschinger effects [8]. The true stress-strain curve is obtained based on the material properties reported by Sumner [7], where the true values are obtained by operating the engineering values from the material tests using Eq. (1) and Eq. (2) [6].

$$\varepsilon_t = \ln\left(1+\varepsilon_e\right) \tag{1}$$

$$\sigma_t = \sigma_i\left(1+\varepsilon_e\right) \tag{2}$$

All the finite elements are 3D deformable continuous solids with eight nodes and reduced integration technique (C3D8R). These elements are suitable for modelling complex non-linear problems involving contacts, plasticity and large deformations. The connection geometry and the load have an axis of symmetry, so only half of the connection specimen is modelled, which saves computational cost in the non-linear analysis. In the FE model, the degrees of freedom perpendicular to the plane of symmetry were constrained. Both residual stresses and initial imperfections are ignored in the FEA modelling. The pretension force applied to the bolts is assigned within the FE models.

Also, the interactions and contacts between the components of the connection are assigned. As shown in Fig. 4, the welds are represented as tie type contacts joining the nodes and restricting the degrees of freedom in contact, which eases convergence during the analysis, although this fact discard a possible weld fracture. The tie restrictions shown in Fig. 4 include those welds that join: (a) beam to end-plate, (b) stiffeners to beam flanges and end-plate, (c) doubler plate to column, and (d) continuity plates to both column and doubler plate.

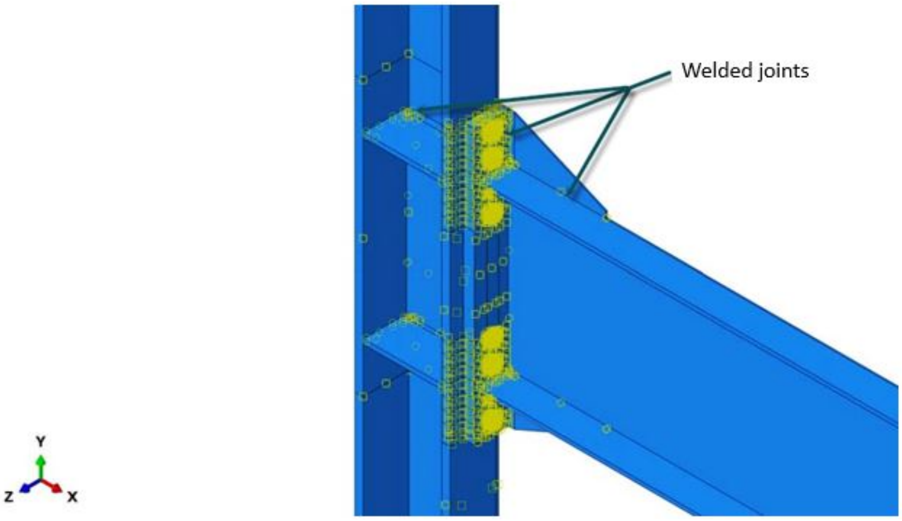


Fig. 4 – Tie restrictions for welded joints

Regarding the bolts, both head and nuts are modelled with an hexahedral FE, and the diameter of the hole is considered 1.6 mm. larger than the bolt shank diameter [4] with no friction accounted between the bolt shank and the steel around the holes at both columns and end-plates, as shown in Fig. 5. However, the contacts between (a) the bolt heads with the end-plate, (b) the nuts with the internal face of the column flange, and (c) the end-plate with the exterior face of the column flange, assume hard contact formulation in the normal direction, and "penalty" friction formulation in the tangential direction as shown in Fig. 6, using a friction coefficient of 0.3 assuming a class A surface.

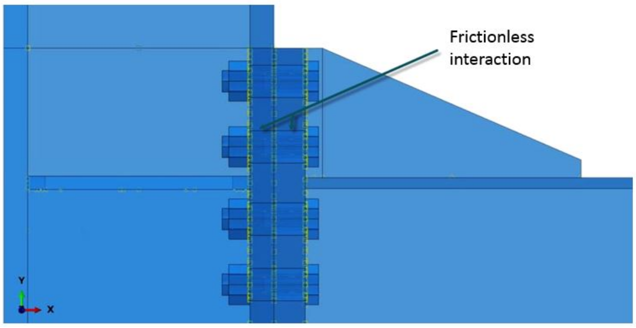


Fig. 5 – Friction-free interactions between bolt shank and holes of end-plates

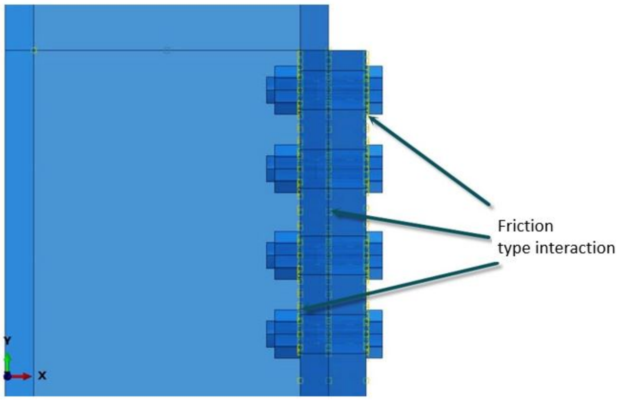


Fig. 6 – Connection friction type interactions

Free boundary conditions are assumed at both columns ends as shown in Fig. 7, where all the nodes at the cross-section were linked to the center point (i.*e.* RP1 and RP2) by a rigid body constrain. The latter was similarly implemented at the beam free end (*i.e.* RP3), where the load protocol in displacement control was applied. Also, degrees of freedom perpendicular to the plane of symmetry were constrained as shows in this figure.

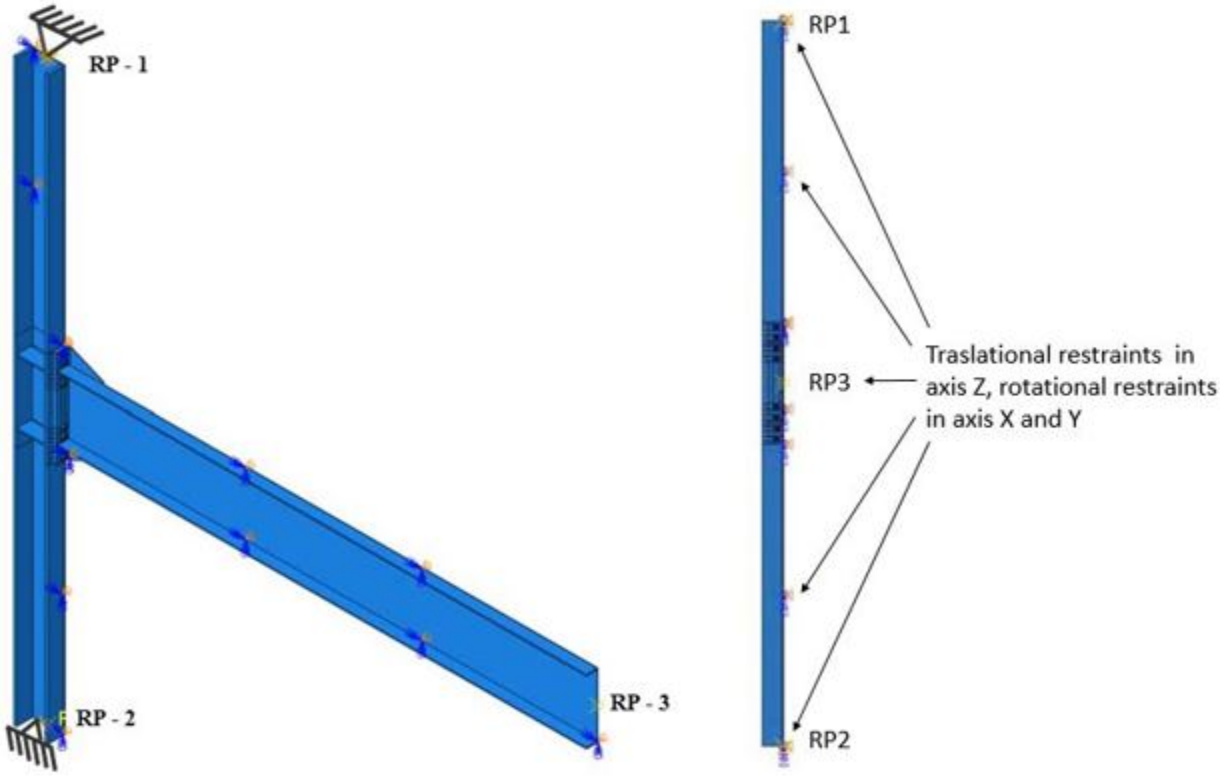


Fig. 7 – Boundary conditions and constrained nodes

Each part of the connection was properly partitioned in order to have a structured mesh, and thus improving the convergence during the analysis. The finite elements used in this work, as shown in Fig. 8., are 3D hexahedral with eight nodes and reduced integration technique (i.e. C3D8R) [6].

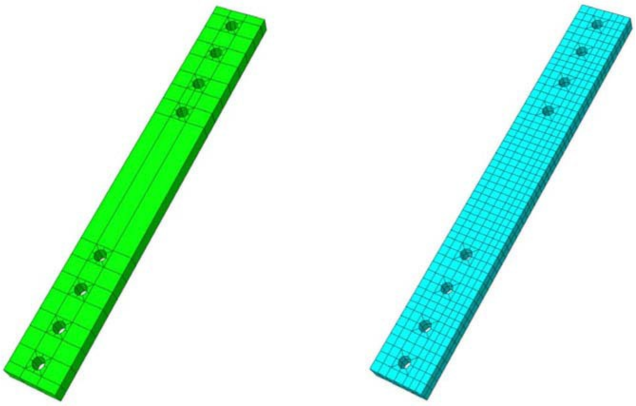


Fig. 8 – Example of a partitioned part that use a structured mesh.

2.3 Comparison between experimental and numerical response

The moment-drift curve computed from numerical simulation is shown in Fig. 9 (blue line), as well as the experimental curve (black line) reported by Sumner *et al*. [10]. This comparison shows that the finite element modelling provides a good prediction of the experimental response, with an overall good accuracy in strength, stiffness, and drift ductility. However, beyond 0.04 drift, the numerical simulation loses some accuracy of the strength degradation due to damage accumulation within the tested connection and some fractured welds, a fact attributed to the FE analysis cannot predict since it was not modelled.

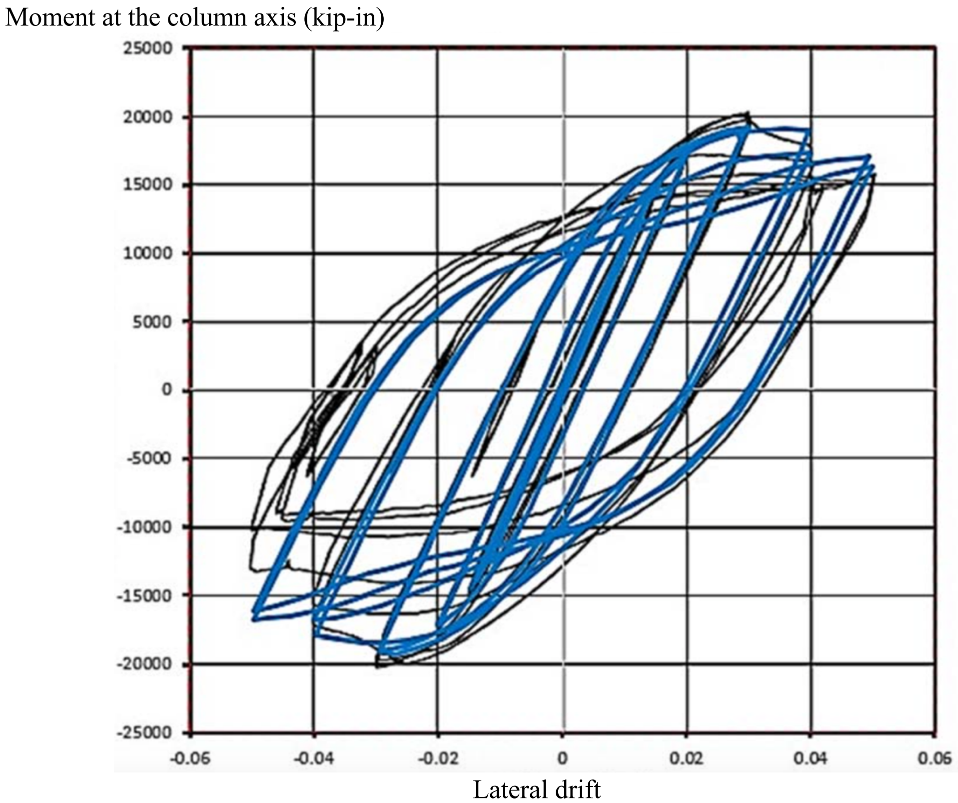
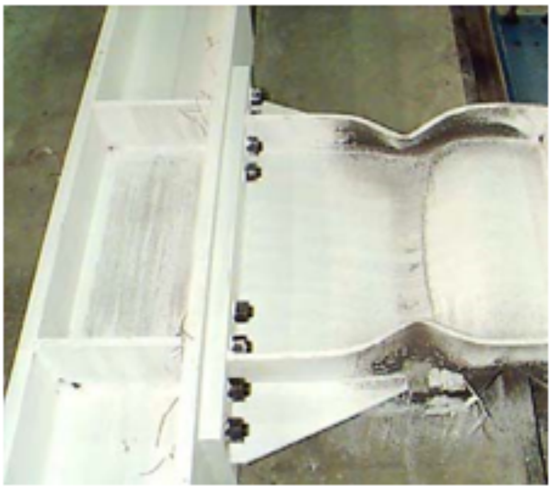
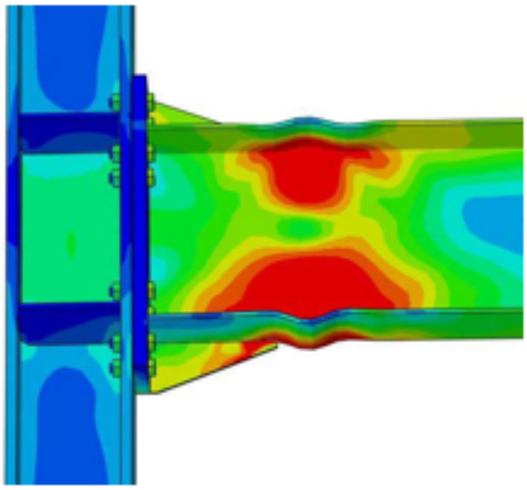


Fig. 9 – Moment-drift curves of the BEEP connection

In addition, the finite element modelling adequately predicts the failure modes for the connection, including extended yielding and local buckling of the beam flanges and web, and shear yielding at the panel zone. Fig. 10 shows a comparison of the damage at the end of the load protocol (a) exhibited and reported from the experimental test and (b) estimated from the numerical simulation.



(a) Experimental test [10]



(b) Numerical simulation [4]

Fig. 10 – Damage at the end of the load protocol.

3 Finite element analysis of connections with no experimental data available

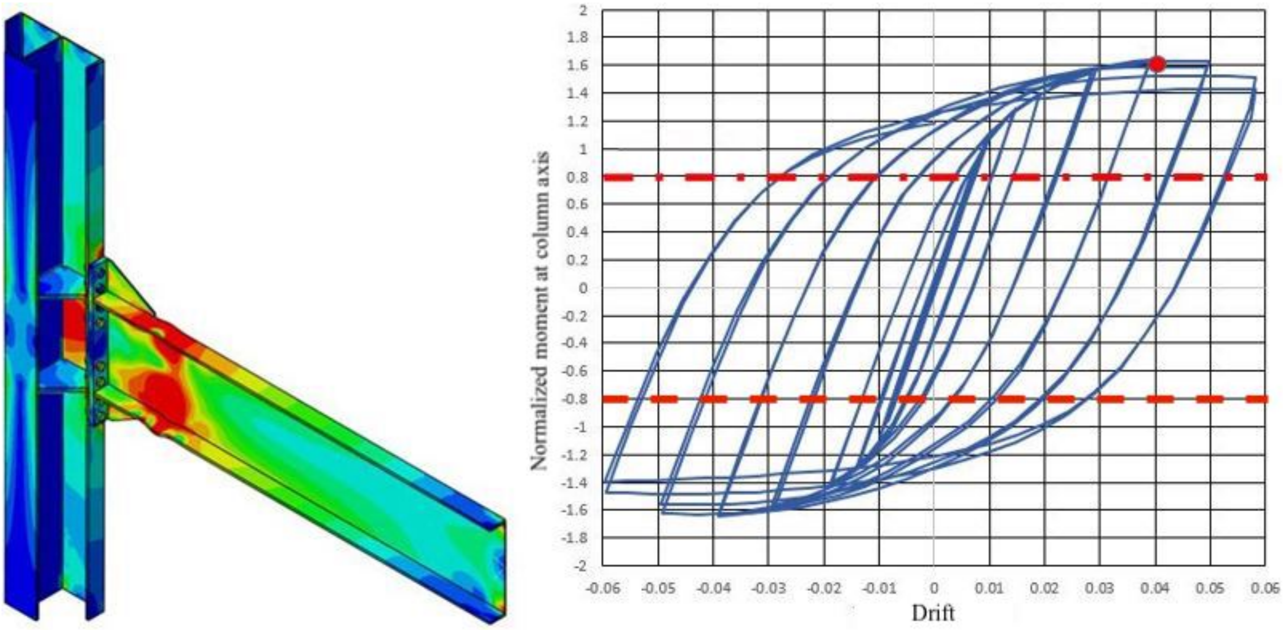
Finite element analysis is performed to other three beam-to-column connections that are typical used in steel MRF in Mexico, for which there is no available experimental data. The results of the numerical simulations are subjected to the standard cyclic loading protocol according to AISC 341-16 seismic provisions (Fig. 3). The results evaluated for each connection include the global lateral load – lateral displacement curve, the moment-drift curve, the assessment of limit states and, for each load increment, the distribution of principal stresses and strains, and the deformed shape. In the absence of experimental data, these FE analyzes are useful to evaluate whether or not these connections, as fabricated and design locally, qualify for use in special or intermediate moment frames. Thus, Figs. 11 to 13 show (a) the distribution of von Mises stresses, and (b) the hysteretic moment-drift curve for each evaluated connection. In these figures, the flexural capacity of the connection at the column axis is normalized with the plastic moment of the beam, M_p, and both $\pm 0.8M_p$ lines and the 0.04 drift are highlighted within these figures.

3.1 BEEP connection to cruciform column

This connection is very similar to the previous one, tested by Sumner *et al.* [10] and calibrated in this study, with the exception that the column is a built-up cruciform cross-section from two W24×76 ASTM A992. The beam is W24×76 ASTM A992, the end-plate is ASTM A36 and $1^1/_4$ in. thick, the continuity plates are ASTM A36 and 1 in. thick, the stiffeners are ASTM A36 and $^5/_8$ in. thick, and the bolts are ASTM A490, and eight bolts per flange of ASTM A490 with $1^1/_4$ in. diameter. This T configuration of the specimen is representative for an exterior beam-to-column connection, where the beam is 3.25 m. long, and the column is 3 m. long. Similar to the previous connection, C3D8R solid with eight nodes with reduced integration technique are used. Also, the interactions are assigned as tie contacts in all the welding parts, with normal and tangential contacts in the bolt heads and nuts, and frictionless at the bolt shank. Fig. 11 shows (a) the distribution of von Mises stresses, and (b) the hysteretic moment-drift curve computed for this connection. This simulation confirms that, as expected, this connection qualifies for seismic application in either IMF or SMF.

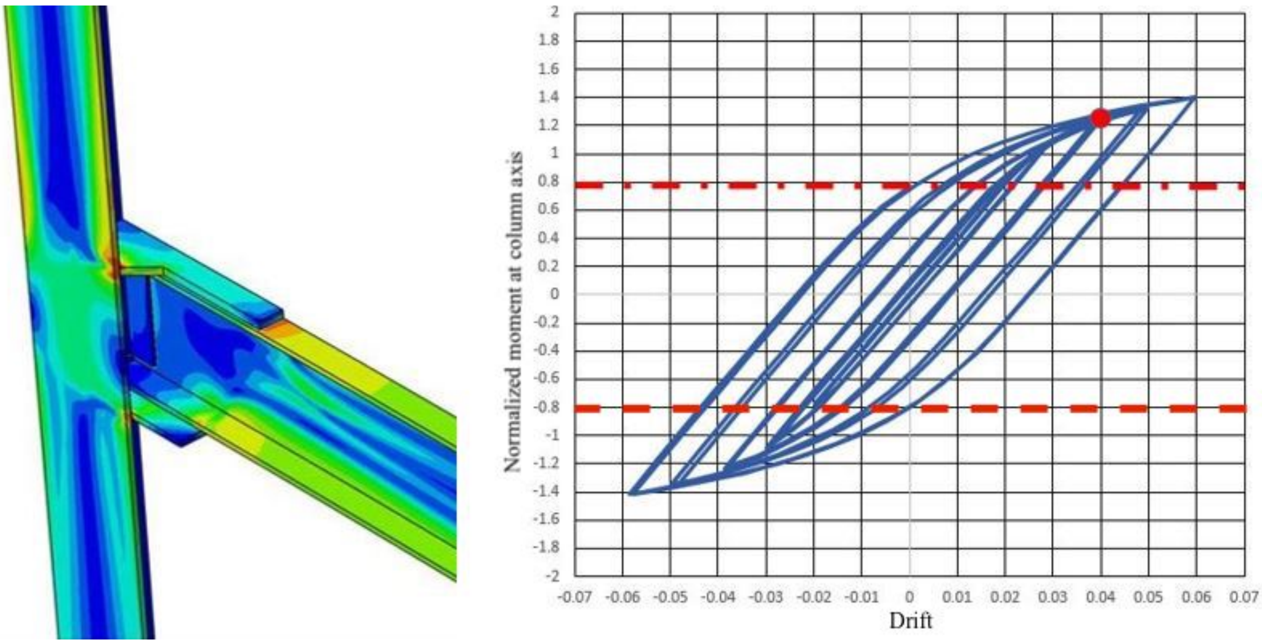
3.2 WFP connection to column web

The connection evaluated in this section is related to a connection used in a real building in Mexico. This connection, as designed and fabricated, is integrated by a beam (W16×50 ASTM A992) connected through the web of a column (W14×90 ASTM A992) in a T configuration. The welded flange plate (WFP) connection uses a 1 in. thick top cover plate and ¾ in. thick bottom cover plate ASTM A36, which are grooved welded to the column and fillet welded to the beam flanges. Also, two shear angles L4×4×5/16 ASTM A36 are fillet welded to both the beam web and the column web. Similar to the previous connection, C3D8R solid with eight nodes with reduced integration technique are used for the modelling of this connection, and tie contacts are assigned in all the welding parts. Fig. 12 shows (a) the distribution of von Mises stresses, and (b) the hysteretic moment-drift curve computed for this connection. Although this simulation points out that this connection might qualify for seismic application, Fig. 12(b) points out a low energy dissipation, in addition to the fact that the ductility in this connection is highly dependent of the welding ductility since metal fracture is not accounted in the FE model. Potential fractures at the welds or the heat affected zone (HAZ) might limit the ductility of the connection to a lower drift capacity and, then, make this connection not suitable for qualification.



(a) Von Mises stress distribution (b) Normalized moment – drift curve

Fig. 11 – Numerical results of the BEEP connection to cruciform column



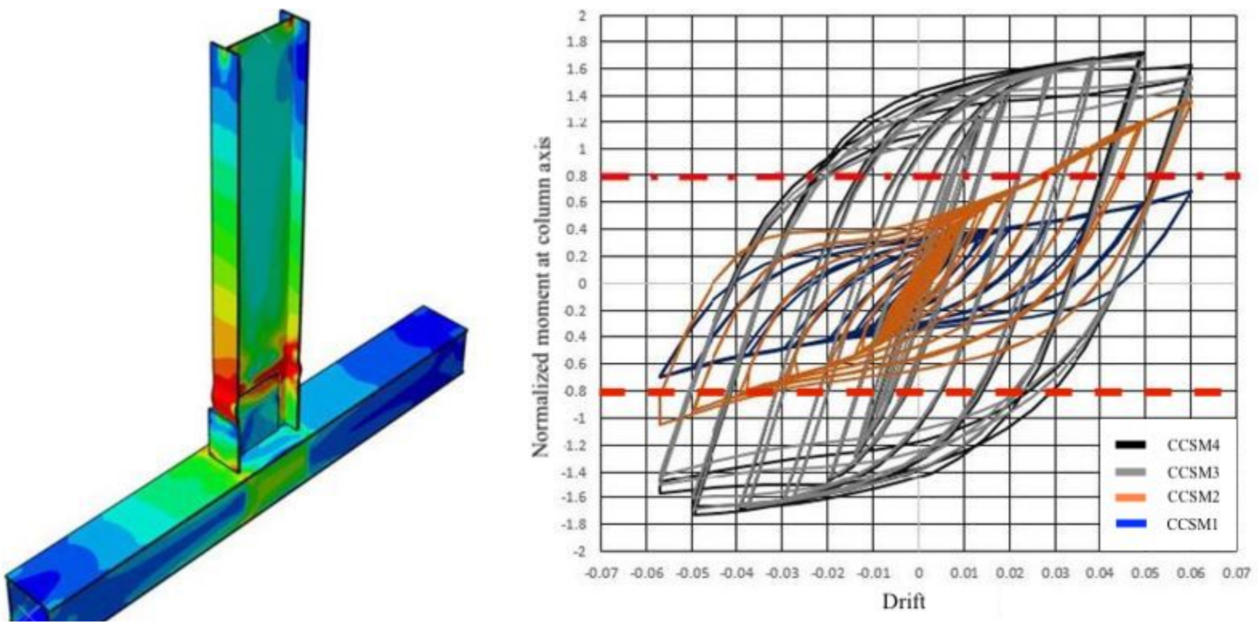
(a) Von Mises stress distribution (b) Normalized moment – drift curve

Fig. 12 – Numerical results of the WFP connection to column web

3.3 WFP connection to box column

This connection was selected in this study since it is widely used in steel moment frame buildings in Mexico. This connection is integrated by a beam (W24×68 ASTM A992) connected through a welded flange plate (WFP) connection to a built-up box column (4PL16×16×5/8 ASTM A572 Gr. 50) in a T configuration. A shear plate (PL20×10×1/2 ASTM A36) is fillet welded to both the beam web and the column web. Also, the WFP connection uses a 3/4 in. ASTM A36 thick cover plates, which are grooved welded to the column and fillet welded to the beam flanges. In addition, internal diaphragms (3/4 in. ASTM A36) are fillet welded in both sides of the diaphragm with three faces of the built-up box column; since the diaphragm cannot be fillet welded to the fourth face of the built-up box column once is placed, either slot or plug welds are frequently used in Mexico to join these parts. In this study, four different cases were taken regarding the ratio of the length of the slot weld, L_w, to the column width, b: (i) CCSM4 for L_w/b = 1.0 (i.e. the slot weld is slightly the same as the column width, black line). (ii) CCSM3 for L_w/b = 0.7 (gray line). (iii) CCSM2 for L_w/b = 0.4 (orange line). (iv) CCSM1 for L_w/b = 0.0 (no welded, blue line).

Similarly, this connection is modelled with C3D8R solid with eight nodes and reduced integration technique, and tie contacts are assigned in all the welding parts. Fig. 13 shows (a) the distribution of von Mises stresses, and (b) the hysteretic moment-drift curve computed for this connection for the four L_w/b cases (i.e. CCSM4 for L_w/b = 1.0, CCSM3 for L_w/b = 0.7, CCSM2 for L_w/b = 0.4, and CCSM1 for L_w/b = 0.0). As observed in this figure, the welding between the interior diaphragm and the fourth side of the column plate has a significant influence on the maximum strength, stiffness, energy dissipation, and ductility of the connection. For the case when the interior diaphragm is not welded with the fourth plate of the box column (i.e. CCSM4 for L_w/b = 0.0), the connection cannot reach the beam plastic moment. This simulation points out that this connection might qualify for seismic application only if the interior diaphragm is welded to the fourth plate of the box column plate with a sufficient length (e.g. as CCSM4 for L_w/b = 1.0 or CCSM3 for L_w/b = 0.7), this result is highly dependent of the welding ductility since metal fracture is not accounted in the FE model. Potential fractures at either welds or the heat affected zone (HAZ) might limit the ductility of the connection to a lower drift capacity and, therefore, make this connection not suitable for qualification. The use of demand critical welding as specified by AWS D1.8, as well as preheating, an strict temperature control, among other quality control (QC) and quality assurance (QA) requirements, will avoid early fractures at the critical welds of this connection, mainly between the internal diaphragm and the inner column plates, the cover plates and the outer column plates, and the beam flanges and the cover plates.



(a) Von Mises stress distribution (b) Normalized moment – drift curve

Fig. 13 – Numerical results of the WFP connection to box column.

4 Conclusions

This research study evaluates by means of finite elements modelling, a group of beam-to-column connections that are commonly used in the practice of Mexico within steel moment resisting frames. These connections have not been sufficiently studied, including the fact that there is a lack of experimental data from tests or other complementary studies that evaluate their potential application in seismic areas as either special or intermediate moment frames. Thus, the non-linear behavior of four steel beam-to-column connections subjected to cyclic loading that are commonly used for steel moment frames in Mexico are assessed with the technique of the FEM, with the aim to qualify their use for seismic applications as special or intermediate moment frames. The FE setup of the connection model has a T-shaped configuration, which represents a sub-assembly of an exterior connection in a steel moment frame subjected to earthquake forces simulated by the cyclic loading protocol from the AISC seismic provisions.

The four connections that are assessed and discussed within this paper have similar configurations to some of the prequalified connections documented by ANSI/AISC 341-16, but with the modifications that are made in the local practice, including columns with built-up box section, built-up cruciform section, and wide flange section with connection to the web. The assessed connections include the following four cases: (1) Bolted stiffened extended end-plate (BEEP) connection, with wide flange cross-sections for both the beam and the column; the numerical results of the FE modelling of this connection were calibrated with the experimental data test reported by Sumner and Murray. (2) Bolted stiffened extended end-plate connection, with a wide flange beam and a built-up cruciform column. (3) Welded flange plate (WFP) connection, with a wide flange beam to the web of a wide flange column. (4) Welded flange plate (WFP) connection, with a wide flange beam and a built-up box column with internal diaphragm plates; for this connection, the following cases of the welding between the inner diaphragm and the box column plates are assessed: (a) four sides fully welded, (b) three sides fully welded and the fourth side partially welded with plug welding, and (c) three sides fully welded and the fourth side unwelded. The later three connections have limited information regarding previous studies with both experimental tests and numerical simulations.

Although it is well known that non-linear finite element simulation of the connections does not substitute experimental tests, this modelling provides reasonable accurate results to assess the connection response, which include lateral drifts and/or displacements and susceptible zones where stress concentration, plasticity or local bucking may occur.

In general, only the BEEP connections to wide flange column and cruciform column, as well as the WFP connection to box column, exhibited ductile behavior when subjected to cyclic loading, and thus qualify for seismic applications as special or intermediate moment frames. However, due to limitations in the FE modelling that does not account for the material fracture, this ductile behavior highly depends on the compliance with an adequate detailing at the fabrication process, such as the use of demand critical welds, preheating, temperature control, among other quality control (QC) and quality assurance (QA) requirements. Numerical FE simulation of the connections evaluated herein will validate upcoming experimental test results, and also extend it to other cases of analysis, and improve it by including the fracture prediction for the structural steel and welds within the numerical simulations.

5 Acknowledgments

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6 References

[1] AISC (2016): ANSI-AISC 341-16. *Seismic provisions for structural steel buildings*. American Institute of Steel Construction (AISC). Chicago, Illinois, USA.

[2] NTC-DCEA (2017): *Norma Técnica Complementaria para el Diseño y Construcción de Estructuras de Acero* (In Spanish). Gaceta Oficial. Mexico.

[3] AISC (2016): *Prequalified connections for steel structural frames for special and intermediate steel moment frames for seismic applications*. American Institute of Steel Construction (AISC). Chicago, Illinois, USA.

[4] Sifuentes J.A. (2017): *Evaluación de conexiones a momento comunes en la práctica local para su uso en marcos dúctiles de acero* (In Spanish). Master thesis. Universidad Autonoma Metropolitana. Mexico.

[5] SAC Steel Project (1997): *Report SAC/BD-97/02. Protocol for Fabrication, Inspection, Testing and Documentation of Beam-Column Connections Tests and Other Experimental Specimens*. Sacramento, California, USA.

[6] Abaqus (2012): *Abaqus. Analysis Users Manual, Dassault Systemes*. Fremont, California, USA.

[7] Sumner E. (2003): *Unified design of extended end-plate moment connections subject to cyclic loading*, Master Thesis. Virginia Tech. Blacksburg, Virginia, USA.

[8] Bruneau M. (2011): *Ductile design of steel structures*. McGraw Hill,. 2nd edition.

[9] Balc R. (2012): *Finite element analysis of beam to column end plate bolted connection*. Vol. 1/2. Civil Engineering & Architecture, Acta Technica Napocensis, pp. 24-29.

[10] Sumner, E.A., Mays, T.W. and Murray, T.M. (2000): End-Plate Moment Connections: Test Results and Finite Element Method Validation. *Connections in Steel Structures IV*, Proceedings of the Fourth International Workshop, American Institute of Steel Construction, Chicago, IL, pp. 82–93.