Proceedings of the
Annual Stability Conference
Structural Stability Research Council
Louisville, Kentucky, April 13-16, 2021

Large scale testing of WF beam to HSS column connections for seismic applications

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Abstract
This research study aims to evaluate the cyclic response of wide-flange beam – to – rectangular HSS column connections through experimental tests. Six steel moment connections are tested with three different configurations of the diaphragms: through diaphragm connections, external diaphragm connections, and welded flange plate connections. These tests aim to characterize the behavior of each connection in terms of the failure mode, strength, stiffness and ductility, and thus concluding if they qualify for applications in special or intermediate steel moment frames located in seismic zones.

1. Introduction
Steel moment resisting frames (S-MRF) are still a typical steel structural system used worldwide due to its architectural and structural advantages, including an aesthetically pleasant appearance, simple fabrication and erection, inherent ability to resist both gravity and lateral forces, and dissipate energy in ductile manner when cyclic forces are induced. Typical columns in S-MRF are either rolled wide-flange (WF) or built-up box cross-sections; however, hollow structural sections (HSS) have many desirable properties which make them suitable for use as columns in the S-MRF system for both non-seismic and seismic regions. HSS columns have been widely used in many building applications in seismic regions, and much experimental and analytical research has been conducted worldwide on connections to HSS columns (*e.g*. Kurobane 1981, Lui 1985, Tabuchi *et al*. 1986, Ting *et al*. 1991, Shanmugan *et al*. 1993, Lu 1997, Kurobane 1998, Takanashi *et al*. 2000, Azuma *et al*. 2006, Goswamiet *et al*. 2008, Malaga-Chuquitaype *et al*. 2010, Gomez-Bernal *et al*. 2011, Yamada *et al*. 2014, AlHendi and Celikag 2015, Zhang and Shu 2015, Pan *et al*. 2017, among others). However, in some countries, there is a lack of data on the effect of the local fabrication standard practices and detailing on the cyclic behavior of beam – to – HSS column connections.

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This research study aims to evaluate the seismic response of rolled wide-flange beam – to – rectangular HSS column connections through experimental tests. The main goal of these tests is evaluating if these connections qualify for special moment frames (SMF) or intermediate moment frames (IMF). It is worth mentioning that this research study was developed in Mexico with the aim of evaluating the strength and ductility of WF beam to HSS column connections that are currently design and fabricated to local practices, but for which there is no experimental evidence of the desired ductile behavior under cyclic forces.

2. Standard practices in Mexico

Typical low-rise and mid-rise steel buildings in Mexico consists of moment frames in the two orthogonal directions, where the ductility is promoted through structural redundancy by using moment connections for all beam-to-column joints, and thus all frames within the building are considered the seismic force-resisting system (SFRS) since all frames are typically considered in the design to provide the required resistance to the seismic forces. For high-rise buildings, the application of a dual structural system (*e.g.,* steel braces or shear walls) is needed for drift control. Therefore, the columns within the SFRS system usually requires the use of closed cross-sections (typically built-up box cross-sections or box wide-flange cross-sections), with very few alternative open sections (*e.g*., flanged cruciform cross-sections). These practices aim not only to provide the required strength for the biaxial bending, but also to connect beams to the four column sides. Thus, most beam-to-column moment connections in Mexico are welded flange plate connections (WFP), which use cover plates CJP welded to the column and fillet welded to the beam as seen in Fig. 1(a). This WFP connections have also been adopted in the local practices with HSS columns that, in contrast to built-up box columns, the internal diaphragms are not placed in most cases since the cover plates are misguidedly assumed as diaphragm plates.

In Japan, similarly to Mexico, structural engineers have promoted structural ductility and redundancy in S-MRF by beam-to-column moment connections in the two orthogonal directions. However, it should be noted that the equipment used in the Japanese shops are of an advanced robotic type, which ensures high accuracy for easy assembly, and a high quality in the fabrication of steel structures. Unlike the US and Japan, the performance of typically used moment connections have not been evaluated under a strict protocol of experimental tests simulating seismic demands, ensuring reliability for SMF and IMF under the local seismic ground motions.

(a) With built-up box columns (b) With HSS columns

Figure 1. Typical WF beam – to – column WFP moment connections in Mexico

3. Experimental program

As stated before, this research study aims to evaluate the seismic response of wide-flange beam – to – rectangular HSS column connections by experimental tests. The main goal of these tests is evaluating if these connections qualify for SMF or IMF for seismic applications.

3.1 Test setup

The test specimens have a T-shaped configuration, as seen in Fig. 2, that represent a sub-assembly of an exterior connection in a steel moment frame subjected to earthquake forces simulated through the cyclic protocol from the AISC Seismic Provisions (AISC 2016).

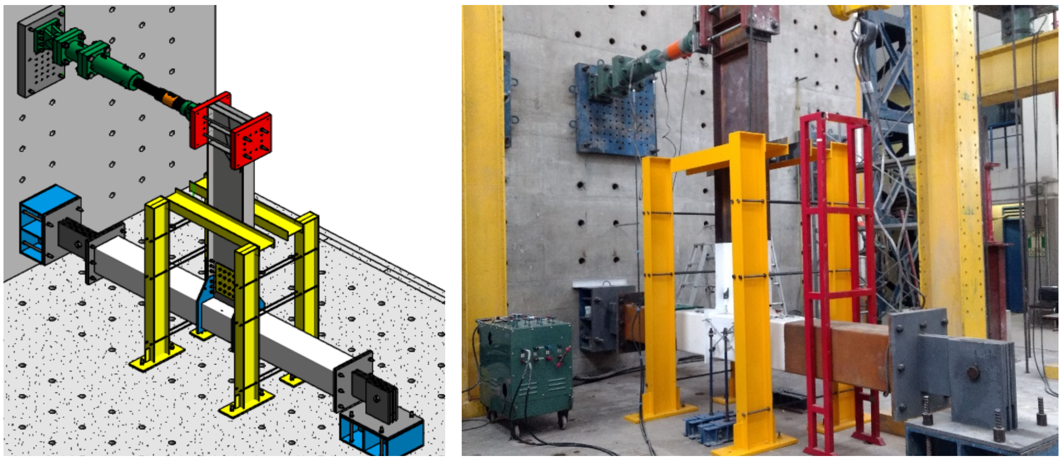


Figure 2. Test setup

3.2 Specimen matrix

As summarized in Table 1, six steel moment connections are tested with HSS columns and three different diaphragms configurations, where two specimens are designed as through diaphragm connections (labels 1T, 2T), two exterior diaphragm connections (labels 1E, 2E), and two flush exterior plate connections (labels 1F, 2F). Table 1 summarizes the most important parameters for the steel beam, the steel column, and the diaphragms. All specimens have a W610×101 (W24×68) ASTM A992 steel beam that connects to the midspan of an HSS406×406×16 (HSS16×16×$^{5}/_{8}$) ASTM A500 grade B steel column; the column in specimen 2F has a wall thickness of 8 mm ($^{5}/_{16}$ inches). All the diaphragm plates are ASTM A572 grade 50.

Table 1. Specimen matrix

<table>
<tr><th rowspan="2">Label</th><th colspan="3">Column</th><th colspan="2">Beam</th><th colspan="3">Diaphragm plate</th></tr>
<tr><th>Cross section</th><th>b/t</th><th>Standard</th><th>Cross section</th><th>Standard</th><th>Type</th><th>Thickness</th><th>Standard</th></tr>
<tr><td>1T</td><td rowspan="5">HSS406×406×16
(HSS16×16×$^{5}/_{8}$)</td><td rowspan="5">24.5</td><td rowspan="6">ASTM
A500-B</td><td rowspan="6">W610×101
(W24×68)</td><td rowspan="6">ASTM
A992</td><td rowspan="2">Through
diaphragm</td><td rowspan="2">19 mm
($^{3}/_{4}$ in.)</td><td rowspan="6">ASTM
A572-50</td></tr>
<tr><td>2T</td></tr>
<tr><td>1E</td><td rowspan="2">Exterior
diaphragm</td><td rowspan="2">22 mm
($^{7}/_{8}$ in.)</td></tr>
<tr><td>2E</td></tr>
<tr><td>1F</td><td rowspan="2">Flush
exterior
plate</td><td>22 mm
($^{7}/_{8}$ in.)</td></tr>
<tr><td>2F</td><td>HSS406×406×8
(HSS16×16×$^{5}/_{16}$)</td><td>52.0</td><td>19 mm
($^{3}/_{4}$ in.)</td></tr>
</table>

3.3 Material properties

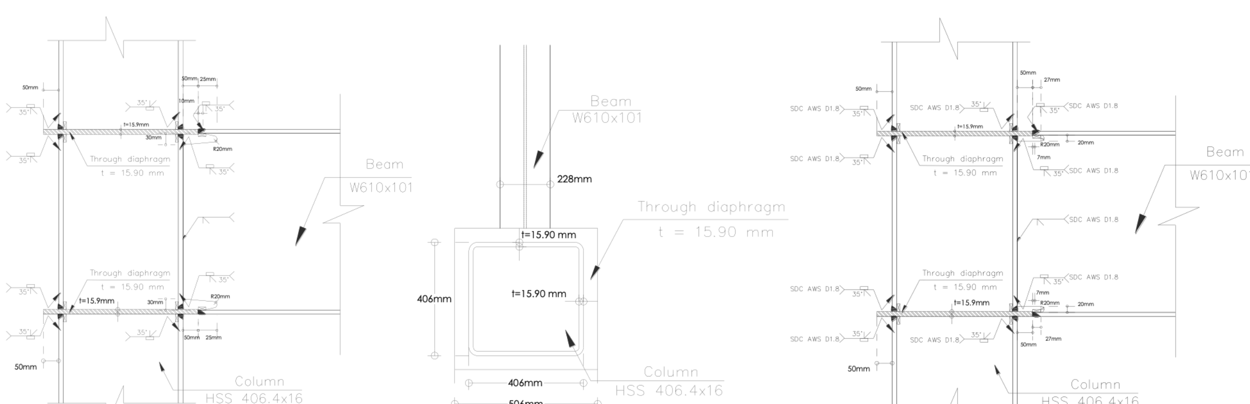
Tensile coupon tests were conducted for the steel used in these six specimens. Table 2 shows for each designated member and standard, the nominal specified minimum yield stress, F_y, and tensile strength, F_u, as well as the actual yield stress, F_{yct}, and tensile strength, F_{uct}, as determined from the coupon tests, and lastly, the actual-to-nominal ratios.

Table 2. Coupon testing results

Member	Standard	Nominal		Coupon tests		Actual-to-nominal ratio	
		F_y (MPa)	F_u (MPa)	F_{yct} (MPa)	F_{uct} (MPa)	F_{yct} / F_y	F_{uct} / F_u
W610×101 (W24×68)	ASTM A992	345	450	392	494	1.14	1.10
HSS406×406×16 (HSS16×16×$^5/_8$)	ASTM A500-B	315	400	362	423	1.15	1.06
16 mm plate ($^5/_8$ in. plate)	ASTM A572-50	345	450	411	447	1.19	0.99

3.4 Description of the test specimens

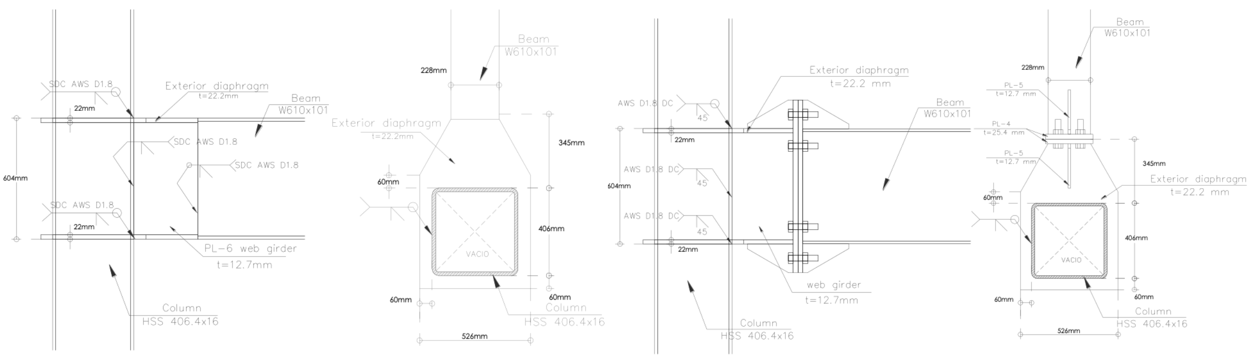
Specimens 1T and 2T were fabricated as through diaphragm connections, a detail that is widely used and very popular in Japan. These connections were designed following the provisions of the specifications AIJ (2005) and CIDECT (1996). The HSS column in these specimens requires two cuts and edges beveled for the CJP welding with the through diaphragms, which is also CJP welded to the wide-flange beam with access holes in both flanges. Unlike the practice in Japan where automatic beveling machine and robotic welding are used, these specimens were fabricated using the local numerical control equipment (CNC), where the edges were cut with a bevel torch and the CJP welding was applied with a semiautomatic FCAW process. Details with the fabrication of these two specimens are shown in Fig. 3. The main differences in these two specimens are related to the access hole detailing and the welding type used within the connection. Specimen 1T adopted the access hole geometry as recommended in Section 6.11.1.2 of AWS D1.8/D1.8M (AWS 2016), and the electrode satisfies the seismic supplement AWS D1.8/D1.8M (AWS 2016). On the other hand, specimen 2T adopted the access hole geometry as recommended in the AIJ (2005) and CIDECT (1996), and the electrode satisfies the seismic supplement for demand critical welding AWS D1.8-DC (AWS 2016).



(a) Specimen 1T (b) Lateral view (c) Specimen 2T

Figure 3. Specimens 1T and 2T with through diaphragm connections

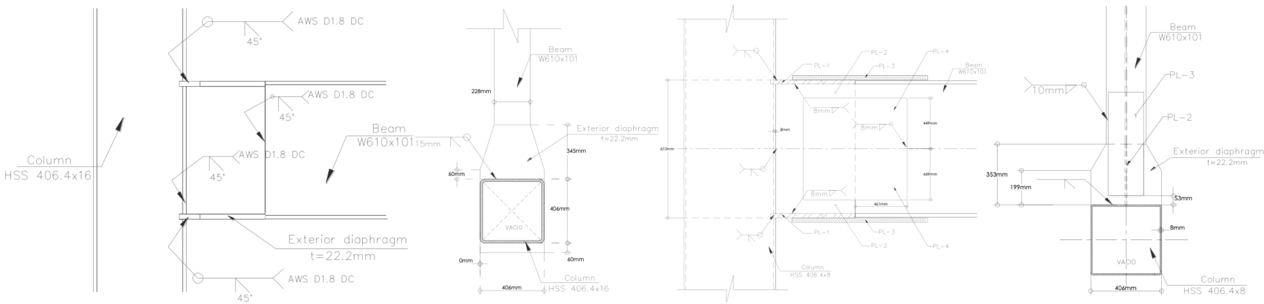
Specimens 1E and 2E are exterior diaphragm connections, configuration that has been also used in some Asian and European countries. These connections were also designed following the provisions of the specifications AIJ (2005) and CIDECT (1996). These specimens require an exterior plate diaphragm for each flange beam, which should be cut and placed (in one or more pieces) around the column with beveled edges for the CJP welding within the exterior diaphragms and the column. Unlike the practice in Japan where automatic beveling machine and robotic welding are used, these specimens were fabricated using the local numerical control equipment (CNC), where the edges were cut with a bevel torch and the CJP welding was applied with a semiautomatic FCAW process. The connection between the exterior plate diaphragm and the wide-flange beam can be joined based on different connection types, such as directly welded as shown in Fig. 4(a) for the specimen 1E, or with a bolted stiffened extended end-plate connection as shown in Fig. 4(b) for the specimen 2E. Although specimen1E has less fabrication processes than in specimen 2E, the first has the disadvantage of on-site welding between the beam and the exterior diaphragm. The welding for both specimens uses an electrode that satisfies the seismic supplement for demand critical welding AWS D1.8-DC (AWS 2016).



(a) Specimen 1E (b) Specimen 2E

Figure 4. Specimens 1E and 2E with exterior diaphragm connections

Specimens 1F and 2F were fabricated with flush exterior plates. These specimens are similar to the designed as exterior diaphragm connections, with the main difference than the exterior plates are cut with an isosceles-trapezoidal shape and one flush side with the same width than the HSS column; in addition, the wall thickness of the HSS column in specimen 2F is half thinner (*i.e.,* 8 mm or $^{5}/_{16}$ inches) than the wall thickness of the HSS column of the other five specimens (*i.e.,* 16 mm or $^{5}/_{8}$ inches). These specimens 1F and 2F require a flush exterior plate for each beam flange and a web plate, which should be cut and placed in each column face with beveled edges and CJP welding. Also, these specimens were fabricated using the local numerical control equipment (CNC), where the edges are cut with a bevel torch and the CJP welding was applied with a semiautomatic FCAW process. The connection between the exterior plates and the beam can be joined based on different connection types, such as directly welded as shown in Fig. 5(a) for the specimen 1F, or as a welded splice with cover plates as shown in Fig. 5(b) for the specimen 2S. The welding parts in both specimens use electrode that satisfies the seismic supplement for demand critical welding AWS D1.8-DC (AWS 2016). Note that specimens 1E (Fig. 4(a)) and 1F (Fig. 5(a)) are almost identical (*i.e.*, equal geometry, cross-section sizes, materials, fabrication process), with the only one difference regarding the diaphragm width (equal to the column width for specimen 1F, and wider for the specimen 1E).



(a) Specimen 1F (b) Specimen 2F

Figure 5. Specimens 1F and 2F with flush exterior plate connections

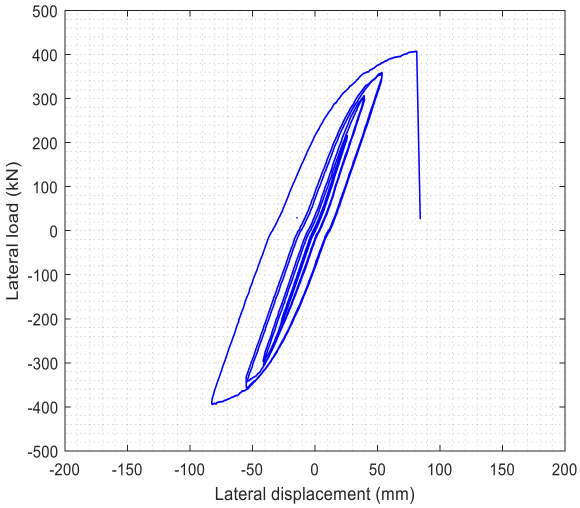
4. Test results and discussion

4.1 Load – displacement response

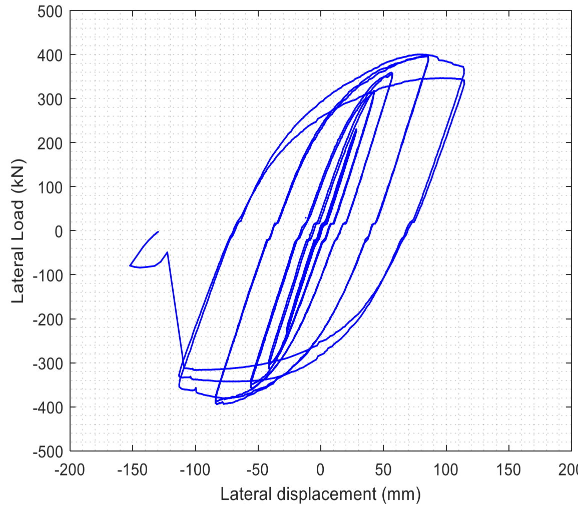
Fig. 6 shows the global response in terms of the applied load *vs*. displacement curves. The six hysteresis curves are developed using the load cell measurements and the horizontal displacement of the pin measured by the LVDT, both placed at the actuator centerline axis. Based on these plots, the following observations are highlighted:

- Specimens with through diaphragm connections 1T and 2T have about the same stiffness and strength. However, specimen 2T developed higher ductility due to better detailing, including the use of demand critical welds as per AWS D1.8-DC (AWS 2016), preheat and a stricter temperature control of the welding parts, and with some changes in the access hole shape and size. Specimen 1T exhibited a fracture at the CJP welding between the through diaphragm and the flange beam. In contrast, specimen 2T showed a fracture at the flange beam near the through diaphragm.
- Although all the specimens have the same beam and column (with the exception of specimen 2E), but with different connection and diaphragm type, the plots show that the stronger and the more ductile connections were those with exterior diaphragm. In particular Specimen 1E, which is directly welded to the beam, reached 170 mm of lateral displacement (equivalent to 0.06 rad of story drift angle), before the flange beam fractured after developing a large amount of yielding and local buckling. Specimen 2E, with an exterior diaphragm connected by a bolted stiffened extended end-plate, also showed satisfactory performance, with a large amount of yielding and local buckling at the beam end, and finally a fracture in the beam near the stiffener.
- Specimens 1F and 2F developed the lowest strength and ductility due to premature failures by fractures at the column corners. In particular, Specimen 2F, with the highest slenderness, showed a reduction in the elastic stiffness earlier due to these fractures at the column corners, and reached a very low strength. Specimen 1F exhibited yielding at the column at about 0.02 rad of story drift angle, and the specimen failed when fractures at the corners of the HSS columns occurred at near 0.03 rad of story drift angle.

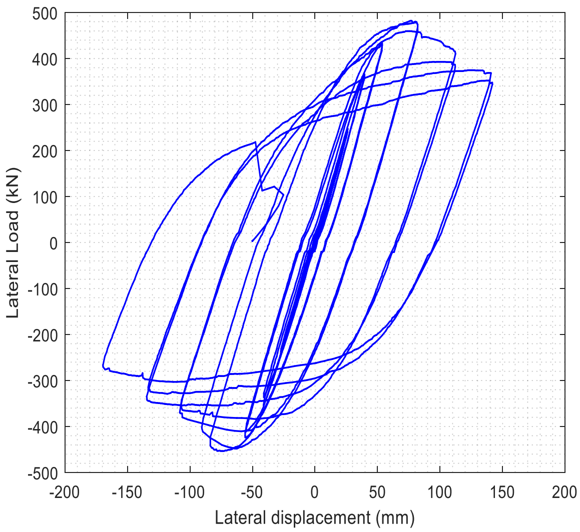
It should be noted that specimens 1E and 1F are identical with the only one exception of the diaphragm width. However, the ultimate strength, failure modes, and above all the ductility, are much better for the specimen with the external diaphragm 1E, which emphasizes the importance of the supplementary width in the diaphragm that aims to better distribute the force path within the connection and reduce the stress concentration at the CJP welding and the column.



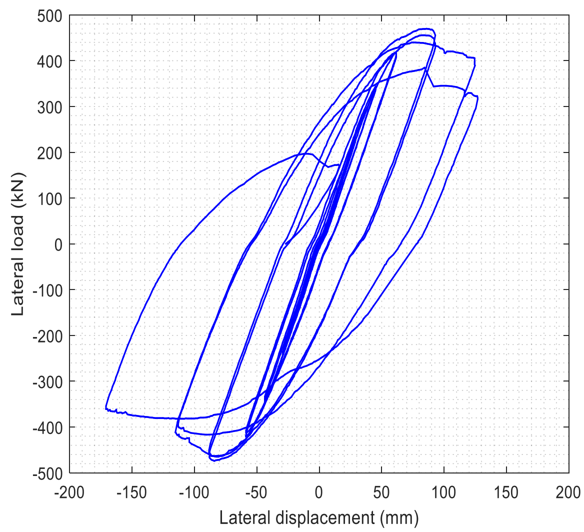
(a) Specimen 1T



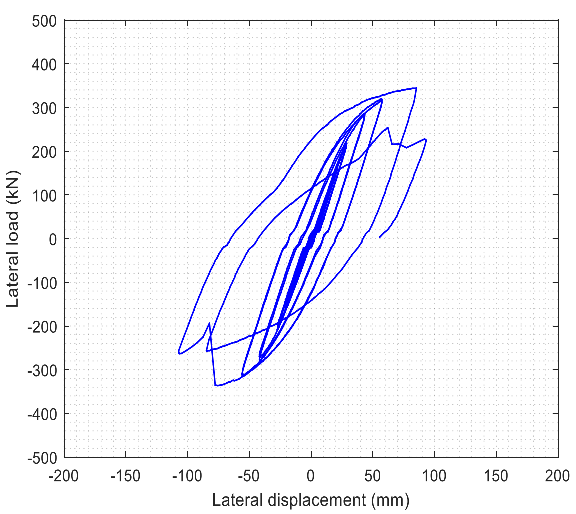
(b) Specimen 2T



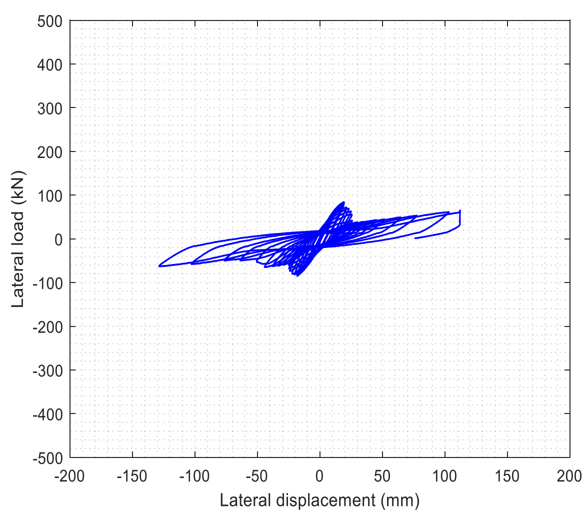
(c) Specimen 1E



(d) Specimen 2E



(e) Specimen 1F



(f) Specimen 2F

Figure 6. Load – displacement curves

4.2 Moment – story drift angle response

Fig. 7 shows the moment – story drift angle response obtained for each specimen. The moment was calculated multiplying the external load applied by the actuator by the distance from the actuator to the column centerline axis. The story drift angle was calculated dividing the lateral displacement given at the free beam end by the distance from the actuator to the column centerline axis. Based on this figure, the following additional observations are highlighted:

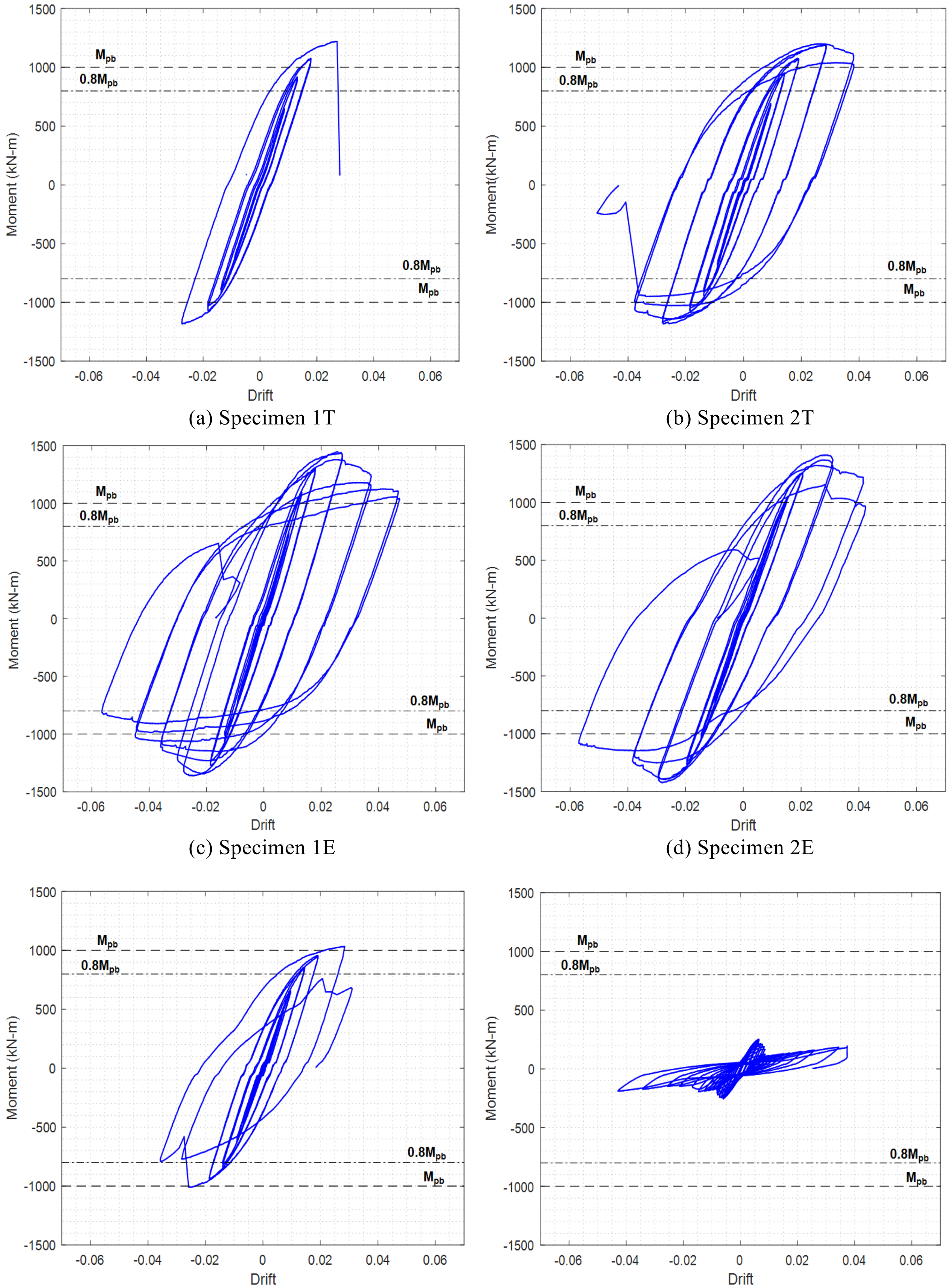
- All the connections, with the exception of specimen 2S, reached the target strength given by the plastic strength of the steel beam, M_{pb} (dashed lines in Fig. 7). The partial strength of the specimen 2T is due to early fractures at the slender column near the corners.

- As stated before, specimens with through diaphragms (Specimens 1T and 2T) have about the same stiffness and strength. However, specimen 2T developed higher ductility due to better detailing. Fracture at the CJP welding between the through diaphragm and the flange beam occurred at 0.03 rad of story drift angle (after the two cycles of 0.02 rad) for Specimen 1T, while the Specimen 2T fractured at the flange beam after 0.04 rad of story drift angle. At the ultimate story drift angle for these two specimens, the moment strengths of both connections are higher than $0.8M_{pb}$ of the connected beam (dot-dashed lines in Fig. 7). Therefore, through diaphragm connections qualify for both SMF and IMF, only if this connection is designed and fabricated with a strict detailing as applied in this specimen.

- As noted, although all the specimens have the same beam and column (except specimen 2E) with different connection and diaphragm type, the stronger and the more ductile connections were those with exterior diaphragm. Both connections 1E and 2E reached and passed the target story drift angle (two cycles of 0.04 rad) with enough flexural strength (higher than $0.8M_{pb}$). Therefore, exterior diaphragm connections qualify for both SMF and IMF, only if this connection is designed and fabricated with the detailing as applied in this specimen.

- Specimens 1F and 2F developed not only the lowest strength and ductility due to premature failures by fractures at the column corners. Specimen 2F, with the highest *d/t* slenderness, showed particularly poor performance. Since the HSS column in specimen 2F classifies as a slender cross-section, this is not recommended for seismic applications. Although Specimen 1F completed the two cycles of 0.02 rad of story drift angle with enough flexural strength (higher than $0.8M_{pb}$ of the connected beam), its application is not recommended due to the failure mode characterized by fractures at the corners of the column. Instead, the application of connections with exterior diaphragm, which fabrication is similar, is recommended since higher strength, stiffness and ductility can be reached with this connection type.

- Although specimens 1E and 1F are identical, with the only one exception of the diaphragm width, only the specimen with the external diaphragm 1E qualifies as a ductile connection, which again emphasizes the importance of the supplementary width in the diaphragm that aims to better distribute the force path within the connection and reduce the stress concentration at the CJP welding and the column. Specimens 1F, which are of common use in Mexico, exhibited an inadequate brittle behaviour due to fractures at the welding and the column corners; thus, this connection type is not recommended for seismic applications.



(a) Specimen 1T (b) Specimen 2T

(c) Specimen 1E (d) Specimen 2E

(e) Specimen 1F (f) Specimen 2F

Figure 7. Moment – story drift angle curves

4.3 Summary of the experimental results

Table 3 summarizes the principal results of the six connection tests. In this table M_{pb} is the plastic moment strength of the beam W610×101 (W24×68), which is 1000 kN·m for all these specimens. M_u is the maximum or ultimate flexural strength of the connection, and θ_{conn} is the story drift angle set within the cyclic protocol that the connection completed before the failure occurrence. These results confirm that only the connections with through and exterior diaphragm that are designed and fabricated with ductile detailing and local practices (as specimens 2T, 1E and 2E) qualify as ductile connections suitable for SMF in seismic regions.

Table 3. Summary of the experimental results

Specimen	Column	b/t	Beam	M_u (kN·m)	M_u / M_{pb}	θ_{conn} (rad)
1T	HSS406×406×16 (HSS16×16×5/8)	24.5	W610×101 (W24×68) M_{pb} = 1000 kN·m	+1078 - 1078	1.08 1.08	0.02
2T				+ 1196 - 1186	1.15 1.13	0.04
1E				+ 1441 - 1353	1.44 1.35	0.05
2E				+ 1412 - 1421	1.41 1.42	0.04
1F				+ 1029 - 1000	1.03 1.00	0.015
2F	HSS406×406×8 (HSS16×16×5/16)	52.0		+ 255 - 255	0.26 0.26	0.01

5. Conclusions

This research study aims to evaluate, through experimental tests, the cyclic response of wide-flange beam – to – rectangular HSS column connections fabricated with the local standard practices in Mexico. Six steel moment connections are tested with different configurations and diaphragms, including through diaphragm connections with two different detailing levels, and both external diaphragm connections and flush exterior plate connections, each with two different configurations for the connection with the beam.

With the exception of specimen 2E, all the specimens have the same beam and column, and thus these have about the same stiffness and elastic strength. However, due to differences in the connection, diaphragm type, and differences in detailing, the tested connections exhibited different ultimate strength, ductility, and failure modes. Some of the detailing aspects that will improve the connection behaviour include the use of demand critical welding as per AWS D1.8-DC, preheat and a stricter temperature control of the welding parts and the access hole geometry.

The test results confirm that only the connections with through and exterior diaphragm that are designed and fabricated with ductile detailing and local practices (as specimens 2T, 1E and 2E) qualify as ductile connections suitable for SMF and IMF in seismic regions. In contrast to specimen 2T, the specimen 1T did not qualify due to a deficient detailing. Specimens 1F and 2F, which are of common use in Mexico, exhibited an inadequate brittle behaviour due to fractures at the welding and the column corners; based on these tests, this connection type is not recommended for seismic applications, although it can be reinforced with an exterior diaphragm connection, which fabrication is similar, but with higher strength, stiffness and ductility.

Acknowledgments
The work described in this paper is part of a collaborative research project developed by the Metropolitan University in Mexico (UAM), the Kyoto University in Japan, and Virginia Tech in the United States. The authors would like to thank Nippon Steel & Sumikin Metal Products Co., Ltd. (NSMP), Grupo BAYSA and Fabrestructuras for supporting this research. The National Science and Technology Council of Mexico (CONACyT) granted a PhD fellowship to the second author that is gratefully acknowledged. The tests described herein were carried out at the structures lab of the National Center for Disaster Prevention (CENAPRED); the valuable group effort of the CENAPRED personnel in the experimental program is greatly appreciated.

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