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Cyclic behavior of 3D moment connections subjected to bidirectional load: Experimental approach

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ABSTRACT

This paper presents an experimental research to assess the cyclic behavior of a bolted moment connection with the use of optimized end-plate connected to built-up box column subassemblies subjected to bidirectional and unidirectional loading. Seven real scale specimens were tested: three specimens with four beams connected to column as interior joint, two specimens with two beams connected as corner joint configuration and two specimens with two beams connected to column as interior joint, according the protocol established in AISC Seismic provisions. The seismic performance was evaluated in terms of hysteretic behavior, failure mechanism, stiffness and dissipated energy. The joints studied were manufactured from of hot-rolled I-beams and square built-up box columns. The elements of connection such as bolts, welding, outer stiffeners and end-plates were designed to remain in elastic range from the flexural expected capacity of beams. The results showed that the required minimum moment of 0.8Mp at 0.04 rad of drift angle was achieved for all specimens tested. However, a higher stiffness and resistance was reached in configuration with unidirectional load (interior joint) in comparison to joint subjected to bidirectional load (corner joint). The damage was concentrated uniquely in beams, while an elastic behavior in columns and connection components was reached for 0.04 rad of drift angle. Finally, this moment connection configuration can be used as an alternative to design buildings with special moment frames under bidirectional loading considering the cyclic behavior of joints evaluated.

Keywords: bidirectional loading, built-up box columns, cyclic behavior, end-plate, experimental research, steel moment connection.

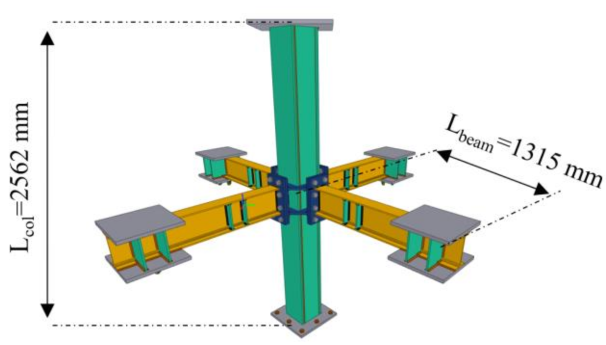
1. INTRODUCTION

In the last two decades, the use of planar special moment frames (SMF) has been studied in several countries considering the use of wide-flange columns with strong-axis bending and to prevent bending in the weak-axis. On the other hand, in Japan the use of hollow structural section (HSS) columns and moment connections in all directions is a common practice in multistory buildings and box columns in high-rise buildings [1]. The use of tubular columns is addressed for cases where biaxial bending is unavoidable, therefore, engineers should design using boxed wide-flange columns, cruciform columns, or built-up tubular columns, which often require internal or external diaphragms [2]. Additionally, the redundancy of Japanese SMF is directly related from the cost practice, because the increased redundancy results in a reduction of steel weight due to the continuity of members. Moment connections are used at beam-to-column connections about both axes of the column. Typically, the Japanese buildings use beams with depths of 400mm to 600 mm, resulting in more redundant structures and greater seismic ductility [3].

Recently, resilient steel structures have made it possible to reduce post-earthquake losses and avoided the use of demolition of structures. In this sense, society has benefited from the continued use of its buildings thanks to the application of this concept in the design of structures. Consequently, the structural resilience has received greater interest in civil engineering academia and industry worldwide [4]. Based on this concept and keeping its application in the seismic design of steel connections, the numerical and experimental study conducted by [5], end-plate connections with tubular columns can reach rotation levels of up to 0.05 rad with good seismic performance. Subsequently, in order to extend this study, numerical investigations conducted by [6][7][8] studied the behavior under cyclic loading and the bidirectional effect of moment connections with wide-flange, box-column and HSS-columns, respectively. The results of the study performed by [6], show that, in spite of complying with a design by capacity, the flexural resistance of the weak axis decreases the stiffness and rotational capacity of the connection, limiting the cyclic response of the joint.

However, studies by [7] and [8] show that the use of tubular connections allows achieving a good performance when biaxial bending loads are required. Similarly, the level of axial loading and the number of moment-connected beams must be considered in the seismic design of the joint, which is not explicitly required by [9]. Finally, the column panel resistance and the column-strong-weak beam criterion are directly dependent on the axial load and the number of connected moment beams. The numerical studies developed by [6][7][8] were performed from a calibration with the experimental study performed by [5]. In this sense, few experimental and numerical studies have been performed considering biaxial bending in 3D joint configurations. A brief description of the studies on this topic is described as follow. In U.S. practice, the use of Seismic provisions [9] and Prequalified connections [10] are mandatory. Specifically, in the code [10], 13 prequalified connections for use in moment frames located in seismic zone are allowed. However, there is no non-patent protected bolted connection that allows the use of tubular columns. Existing connections, in addition to being patent protected, require the use and/or combination of columns with limited column width or prefabricated parts. Therefore, other studies have sought to develop solutions in this area. An experimental study conducted by [11], evaluated the behavior of end-plate moment connections subjected to biaxial effect in the beam. Results show a loss of initial stiffness of up to 19%.

Several numerical and experimental studies on moment connections with I-beams and tubular columns (HSS or built-up box section) have been conducted [12][13][14][15][16][17][18][19][20][21][22]. In these studies, requirements of flexural strength, ductility and remaining capacity at 4% drift angle, established in AISC 341 [9], were satisfied. In addition, the components of the studied connections remain in the elastic range when the maximum inelastic incursions are reached. Although bidirectional resistance is studied in some cases, a few experimental tests were performed and considered the bidirectional effect.Recently, the cyclic behavior of welded moment connections with HSS columns was studied by [23]. In this research, experimental subassemblies subjected to bidirectional loading were performed. The results show a good performance in specimens tested. Posteriorly, similar study conducted by [24] considering the influence of reinforcement concrete (RC) slabs under bidirectional loading in moment connections was performed. The RC slab increased the plastic resistance of the specimens in order to 20%. An equation was proposed to quantify the slab effect on the plastic strength of the specimens. Therefore, additional studies are required to provide evidence on the cyclic behavior of bolted moment connections with built-up box columns subjected to bidirectional loading. The goals of this paper are to assess the cyclic behavior of enhanced end-plate moment connection with built-up box column configurations subjected to bidirectional and unidirectional loading and to verify the differences of joint behavior of two loading modes. In this sense, 3D joints with four I-beams connected to box-columns, 3D joints with two I-beams connected to box-columns as corner joint, and 2D joints with two I-beams connected to box-columns as interior joint were tested (see Figure 1). The experimental research reported in this study helps to determine their application in seismic structures, since bolted moment connections not patented considering biaxial effect are not deemed for their use in Chilean and AISC codes. The next section presents a description of experimental program, the third section show the results obtained, and the fourth section the main conclusions obtained.



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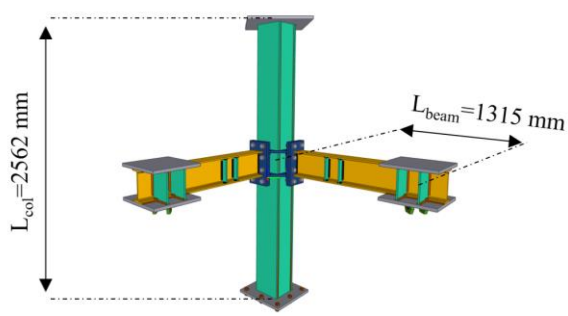

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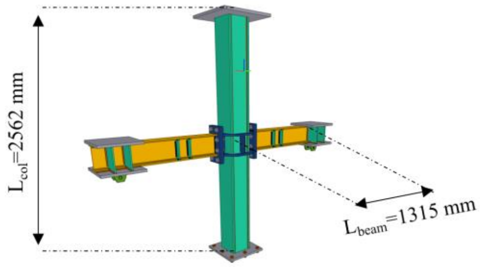

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Figure 1. Schematic view of joint configurations studied: (a) 4B interior joint configuration, (b) 2BC corner joint configuration, and (c) 2BI interior joint configuration. Units in [mm].

2. EXPERIMENTAL PROGRAM

2.1 Description of specimens

In this investigation, external and internal joint configurations of a steel building are evaluated. The size of the beams and columns come from the trial-and-error design of a 4-story office building located in Santiago-Chile, according to the Chilean seismic code NCh433 [25]. According to [25], the building is located in an intermediate seismic zone with peak ground acceleration PGA = 0.30g and in a type B soil, characterized by a shear wave velocity Vs30 ≥ 500 m/s and simple compressive strength of the soil σ > 0.4 MPa. The drift limit established in [25] is 0.002 for verifications in the elastic range. Lateral bracings were not considered and moment frames in both directions with rigid joints were designed following the philosophy established in Seismic provisions [9], complying the strong column-weak beam criterion and the elastic behavior of the web panel zone shear. The design of the connections follows the procedure indicated in [5] and [26], in which, a design by capacity is considered to bolts, welding and diaphragms, from the maximum probable moment of the beams (details in Figure 2).

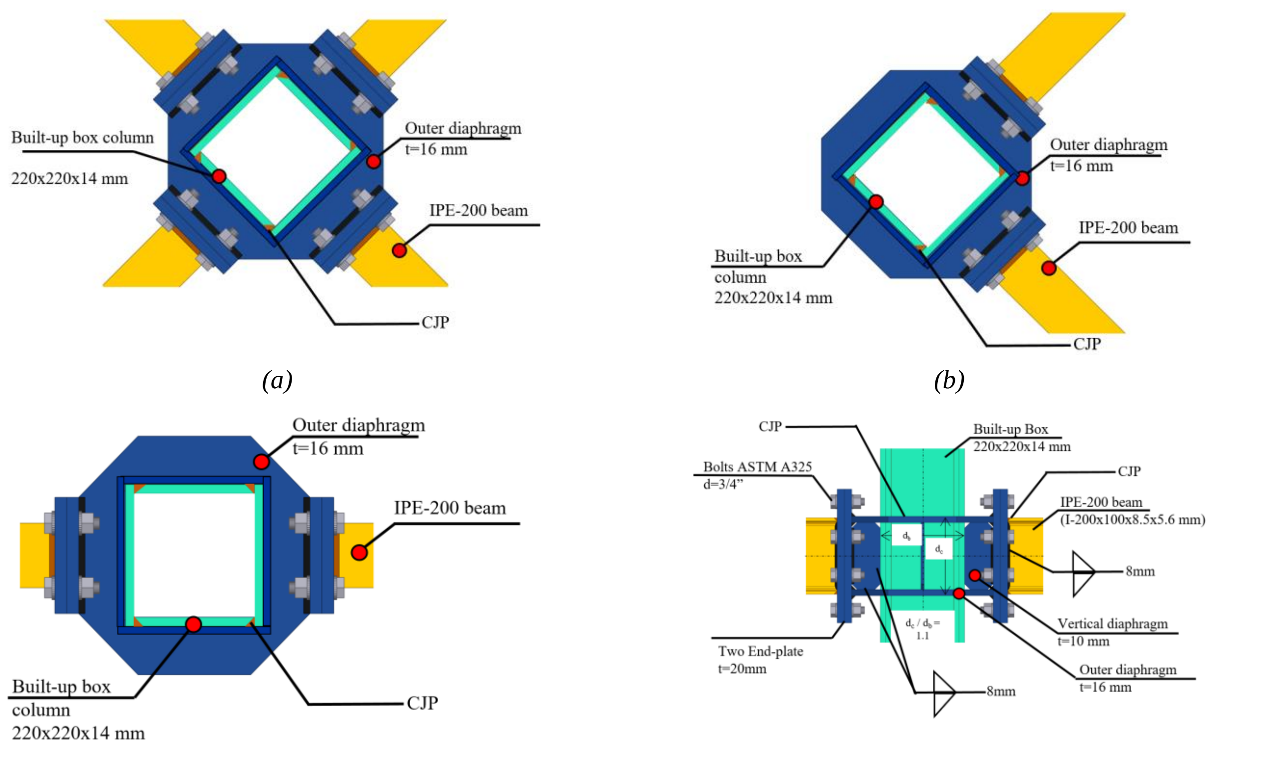
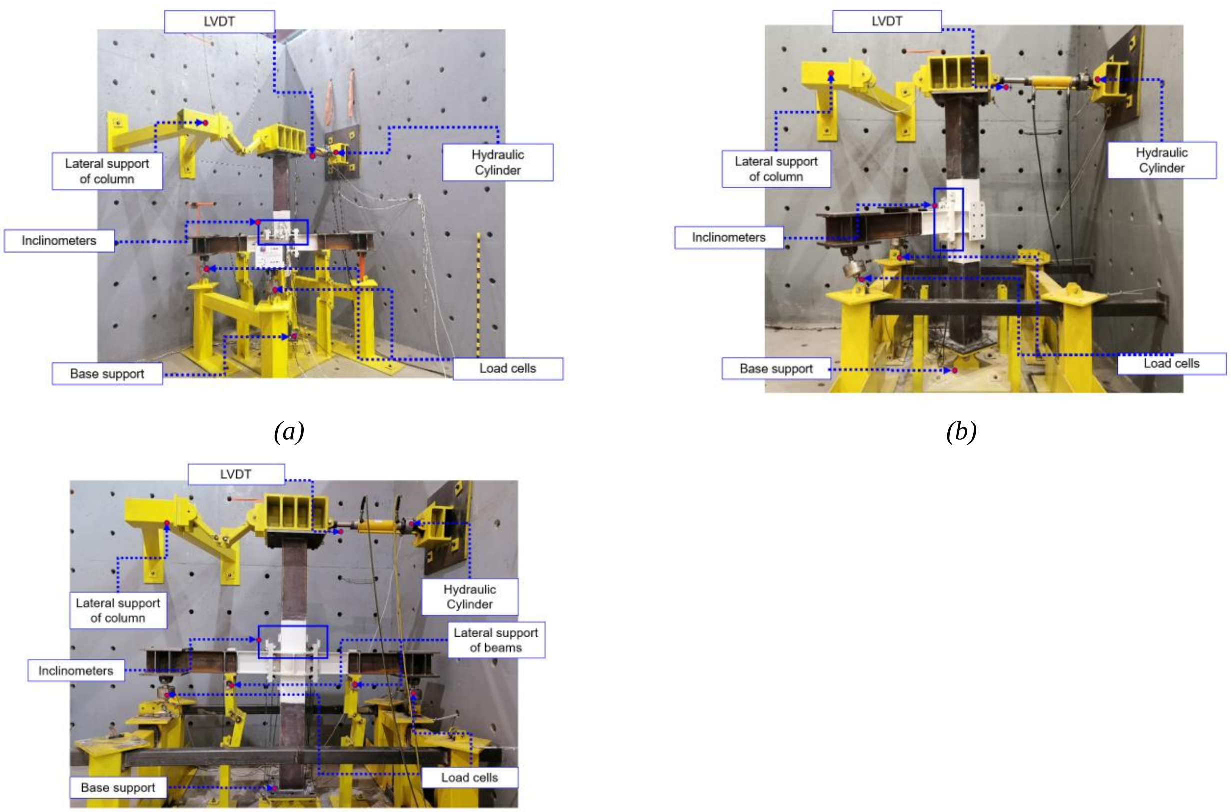


Figure 2. Details of components in configurations studied: (a) Plan view of 4B interior joint, (b) Plan view of 2BC corner joint, (c) Plan view of 2BI interior joint, and (d) elevation view of typical end-plate moment connection studied.

To evaluate the cyclic behavior of the steel joints, 3 types of joints were studied. The 3D joints with four I-beams connected to box-columns (4B-interior joint) and 3D joints with two I-beams connected to box-columns (2C-corner joint) were evaluated. Also, 2D joints with two I-beams connected to box-columns (2I-interior joint) were tested. Therefore, a direct comparison of the cyclic response between 2C-corner and 2I-interior joint was performed. To consider the repeatability of the physical phenomenon, 2 tests with similar conditions were contemplated except for the 4B-interior joint, which 3 specimens were tested (see Figure 1). Finally, based on a preliminary numerical study [26], the failure of the specimens to be tested is controlled by a ductile failure concentrated in the beam by bending due to the maximum imposed drift. The failure moment M_{pr} was calculated according to [10] and the moment ratio at beam-to-column connection defined by Eq. (E3-1) of AISC-341 [9].

2.2Test setup and instrumentation

Seven specimens were tested in the Laboratory of Structures of the Department of Civil Engineering at the Universidad Católica de la Santísima Concepción (UCSC), Chile. In general, the assembly of the joints studied consists of the application of a cyclic load at the upper end of the column. Pinned restraints are used in the end of columns and beams. The load is applied by a 25 tonf actuator connected to the reaction wall. The reaction wall is L-shaped and allows load application on one side and lateral restraint on the other side simultaneously. This lateral restraint is used to avoid the out-of-plane displacement by the load applied. In the reaction slab, column and beam supports are applied. The lateral restraints of the beams are fabricated from channel sections and connected by pins at the bottom end of the beam. For the 4B-interior joint and 2C-exterior joint configurations the same experimental setup was used, while for the 2I-interior joint configuration an additional setup was required. In the Figure 3, schematic view and plan view of joint configurations tested are shown.



(a) *(b)*

(c)

Figure 3. Experimental setup: (a) View of setup for 4B joint configurations, (b) view of setup for 2BC joint configurations, (c) view of setup for 2BI joint configurations.

The columns were fabricated from ASTM A572 Gr.50 [27] steel plates, the beams (hot-rolled sections), pins, end-plates and stiffeners were made from ASTM A36 steel [28], while ASTM A325 [29] was specified for bolts. A complete joint penetration weld CJP was used to join the beam to end-plate, and fillet weld was used to join stiffeners to column. A bolt pretension according to [30] was used. The mechanical properties are used typically in numerical studies such as [31] and [32], playing an important role in the representation of the physical phenomenon. A summary of the material properties is shown in the Table 1.

Table 1. Material properties for steel.

Element	Steel type	Thickness (mm)	Yield strength (MPa)	Ultimate strength (MPa)	Yield strain (mm/mm)	Ultimate strain (mm/mm)
Beam flange	A36	8.5	351	454	0.0018	0.08
Beam flange	A36	8.5	349	432	0.0016	0.07
Beam flange	A36	8.5	347	439	0.0016	0.07
Beam web	A36	5.2	307	403	0.0014	0.10
Beam web	A36	5.2	332	407	0.0016	0.07
Beam web	A36	5.2	322	410	0.0015	0.20
Column wall	A572 Gr.50	14	354	550	0.0018	0.16
Column wall	A572 Gr.50	14	512*	575	0.0024	0.10
Column wall	A572 Gr.50	14	393	559	0.0020	0.26

***Note:** (*) the value obtained is higher than the expected yielding value. It is recommended to discard this value for use in numerical models.*

In order to obtain the results of interest, load cells, inclinometers and displacement sensors (LVDT) were used. A load cell located at the load application point, and load cells at the end of each beam were placed to obtain the forces at those points. The displacement of the column at the load application point was measured through an LVDT and the beam plastic rotations and panel rotation were measured from the use of inclinometers.

2.3 Loading protocol

The loading protocol is applied according to the established in Seismic provisions [9]. The bidirectional effect was obtained from the configuration of specimens and the location of the load. The configuration with load applied in the upper end of column is more unfavorable than configurations where the load is applied directly to the beams according to the study performed by [26].

3. EXPERIMENTAL RESULTS

In this section, the results of the experimental study in terms of cyclic behavior, failure mechanism, dissipation energy and stiffness are shown. In this sense, force vs. displacement hysteresis curves of the system, normalized moment vs. rotation curves, accumulated dissipated energy per tested configuration, normalized tangent stiffness vs. rotation and secant stiffness vs. rotation are reported, similar to procedure used in [5][8][26]. A large number of data was obtained, therefore, uniquely the system and west-beam data will be shown and analyzed. The initial stiffness *Ko* was obtained by dividing the moment in the elastic loading and unloading branch by the rotation at those levels, the dissipated energy was obtained from the closed area in the hysteretic loop per cycle, the maximum moment (Max) is the maximum moment achieved in each joint configuration tested, the plastic moment (Mp) is obtained from the $Mp=FyZx$, according the established in [30], the $M_{0.04}$ is the moment obtained at 4% drift angle, and the maximum rotation (θ_{max}) is the maximum rotation reached in the test according the configuration tested.

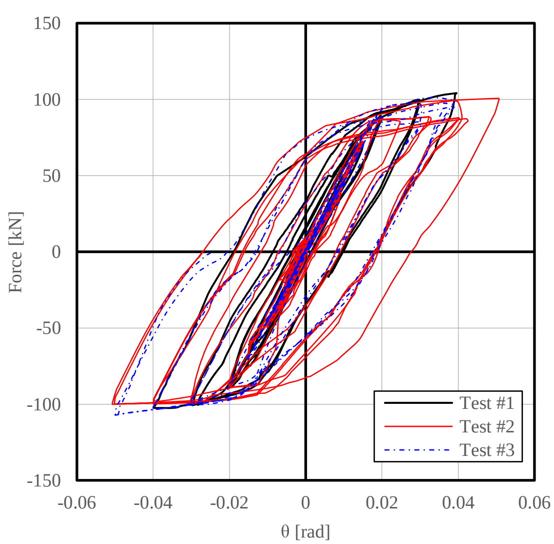
3.1 Hysteresis curve

The hysteretic behavior of the tested specimens is shown in Figure 4. Force-displacement curves of the global system and normalized moment-rotation curves of the west beam are performed. Tests #1-#2-#3 show the 4B-interior joint with four beams per column, the Tests #4-#5 correspond to the 2C-corner joint with two beams per column, and Tests #6-#7 present the 2I-interior joint with two beams per column. The force-displacement curve of joint 4B shows a stable hysteretic cycle without degradation of strength and stiffness. Test #1 reached a maximum rotation of 4%, however due to actuator movement problems no further force could be applied. On the other hand, Test #2 and #3 reached a maximum drift of 5% and similar behavior to Test# 1. Nevertheless, Test #2 presented a disturbance in the measurements of the positive rotations due to the contact between a nut and the eye bar of load cell in one of the beams. Despite of this problem during the test, a hysteretic pinching mechanism was not reached.

Tests #4-#5 exhibits a low stiffness in comparison to Tests #6-#7, despite having a similar number of beams and columns. This demonstrates the influence of the biaxial effect on the behavior of the joint under cyclic loading, which can be affected by a distortion of the web panel zone shear if not considered in the joint design. In this sense, a higher force was required to deform Tests #6-#7 specimens, which reached a force of 83 kN with respect to the 59 kN achieved by Tests #4-#5. The repeatability of the tests was obtained, showing that the phenomenon is reproducible for the similar conditions.

In order to the behavior obtained from the normalized moment-rotation curves, a very similar behavior was achieved by Tests #1-#2-#3, regardless of some slight differences in the unloading and reloading branch for Test #1. Additionally, bending moments > 0.8Mp at 0.04 rad drift angle were obtained in all 3 tests, which is required according to minimum requirement established in Seismic provisions [9] for moment connections used in special moment frames. On the other hand, the Tests #4-#5-#6-#7 also satisfy this requirement, however values above 1.5Mp are reached for Tests #6-#7, which show once again, that when the biaxial effect is considered, a loss of strength and stiffness can affect the performance of the connection under cyclic loading. As is observed in Tests #4-#5, values of 1.07Mp and 1.05Mp were achieved. These stiffness losses must be considered in the design of joints subjected to biaxial loading.

A comparison of cyclic response between 2BC and 2BI joint configurations is shown. The 2BI configuration achieved higher stiffness and higher strength compared to the 2BC configuration. This confirms that despite having the same number of connected members, configurations with bidirectional effect develop a resistance up to 50% lower than configurations without bidirectional effect. In addition, 2BI configurations develop higher stiffness than 2BC configurations. Analytically, this phenomenon can be observed from the Equivalent Load-Displacement Method (ELDM) proposed by [18]. The ELDM for 2D and 3D joints based on the above equivalent column top hysteresis curves, the hysteresis behaviors of different joint configurations can be compared through of different parameters such as stiffness, equivalent damping and energy dissipation.



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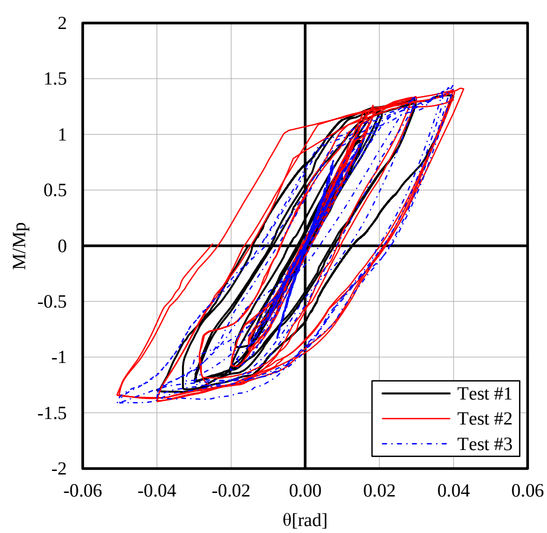

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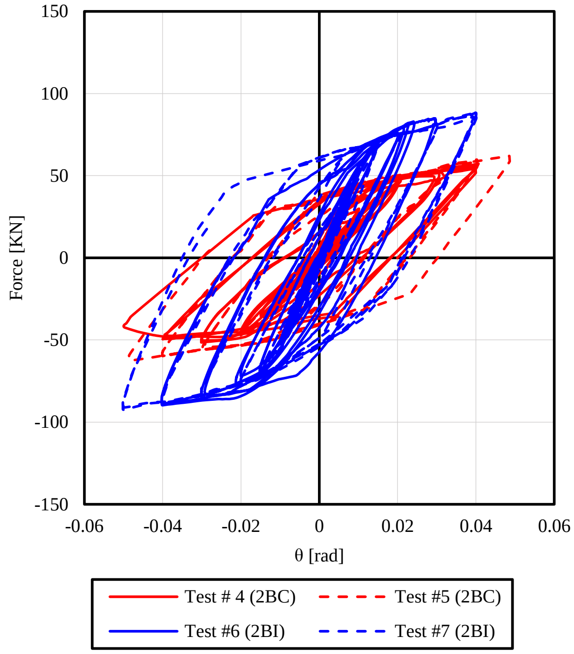

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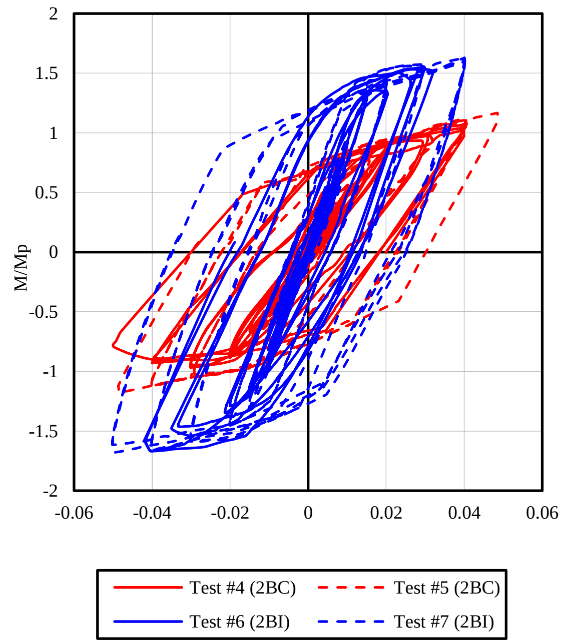
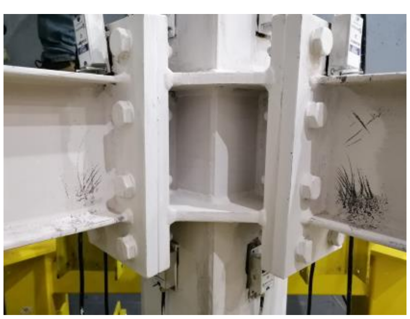


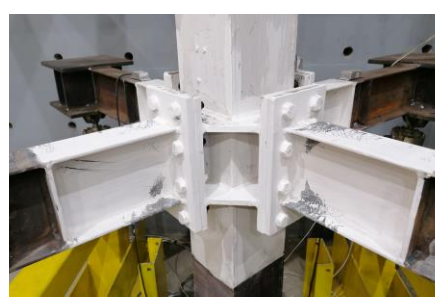
(d)

Figure 4. Force-Rotation curves of global system and Normalized Moment-Rotation curves of west beam.

3.2 Failure mechanism

Ductile, brittle and hybrid failure mechanisms can control the cyclic behavior of steel connections. In general, brittle failure mechanisms are associated to stress concentration in the column, buckling or distortion of the web panel zone shear, failure of welds, bolts or buckling of stiffeners. Hybrid failure mechanisms are a combination of both, ductile and brittle failure mechanisms. Figure 5 shows the damage for each specimen tested for two drift levels: a level of 0.01 rad equivalent to design drifts and 0.04 rad for maximum damage levels in beams according to [9]. In all tests, a ductile failure mechanism controlled by flexural strength of the beam is reached for all tests. Consequently, no brittle failures or hybrid failures were recorded, which is a consequence of the design level used when the biaxial effect is considered. Additionally, it is important to highlight that no damage to beams, columns or connection components was recorded for a drift of 0.01 rad, which is desirable in buildings located in seismic zones in Chile where the high probability of occurrence of earthquakes requires resilient structures capable of maintaining operability and safety levels. Finally, high levels of yielding were achieved in all beams for 5% of drift angle, however are level of damage not desirable in buildings.





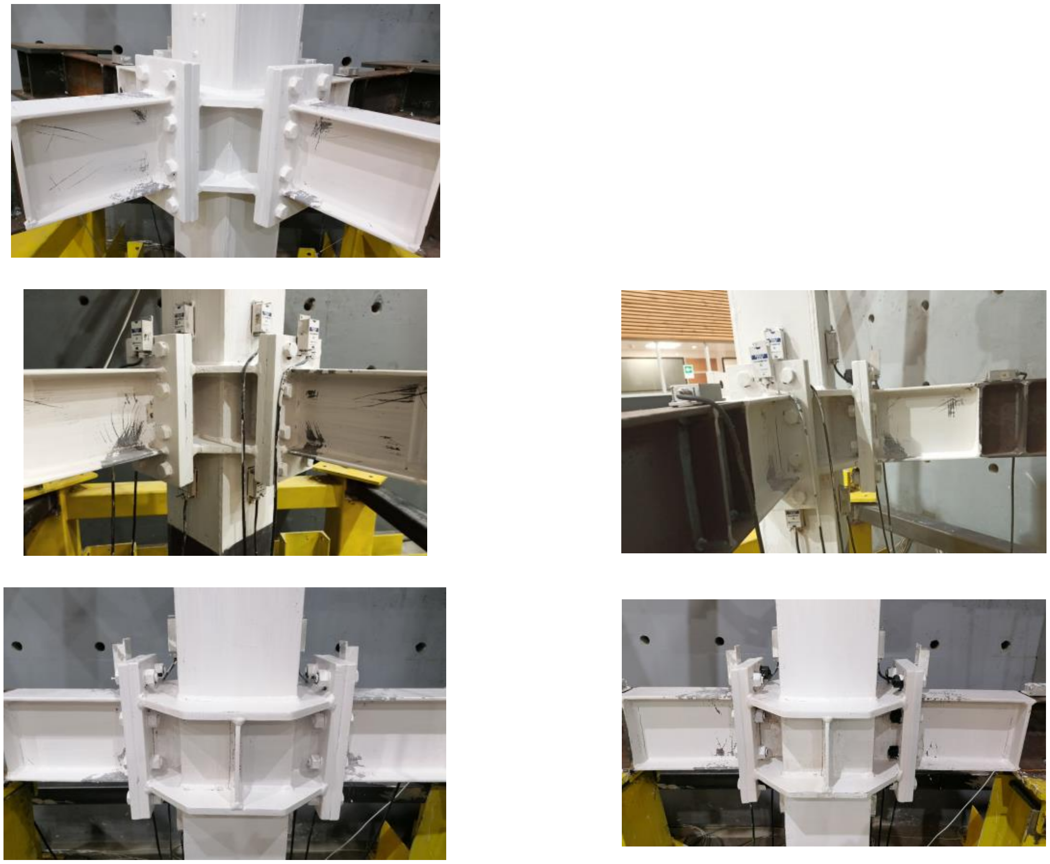


Figure 5. Damage of steel specimens tested at 4% drift ratio.

3.3 Energy dissipation

The accumulated dissipated energy is shown in Figure 6 for each specimen tested. In this sense, it is possible to compare the response between systems and evaluate their efficiency to dissipate energy injected into the system by inelastic deformation. Tests #1-#2-#3 reached higher energy levels compared to the rest of tests, because more beams are connected to the system. In Tests #1-#2-#3, up to 50% more dissipation was recorded, obtaining very similar values by the three tests, which is a sign of the repeatability of the physical phenomenon. The tests of 2C-corner joints and 2I-interior joints exhibited a similar behavior, and Tests #4 and #6 reached lower values in comparison to Tests #5 and #7, because these tests did not achieve the same level of deformation. However, a similar trend in terms of energy dissipation is obtained between specimens with similar levels of deformation. Finally, it can be observed that the 2C-corner joints develop up to 17% more dissipated energy with respect to the 2I-interior joints. Also, by simple inspection, it can be deduced that to develop a cycle of 0.05 rad from of 0.04 rad in joints with two beams requires dissipating an energy of at least 1.97 times the accumulative energy until at 4% drift angle. Therefore, it is noticeable that the main energy dissipated comes from the levels where damage is recorded.

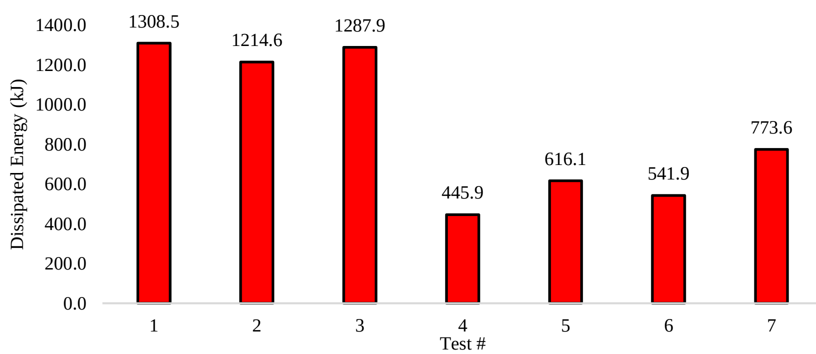


Figure 6. Dissipated energy for joint configurations tested.

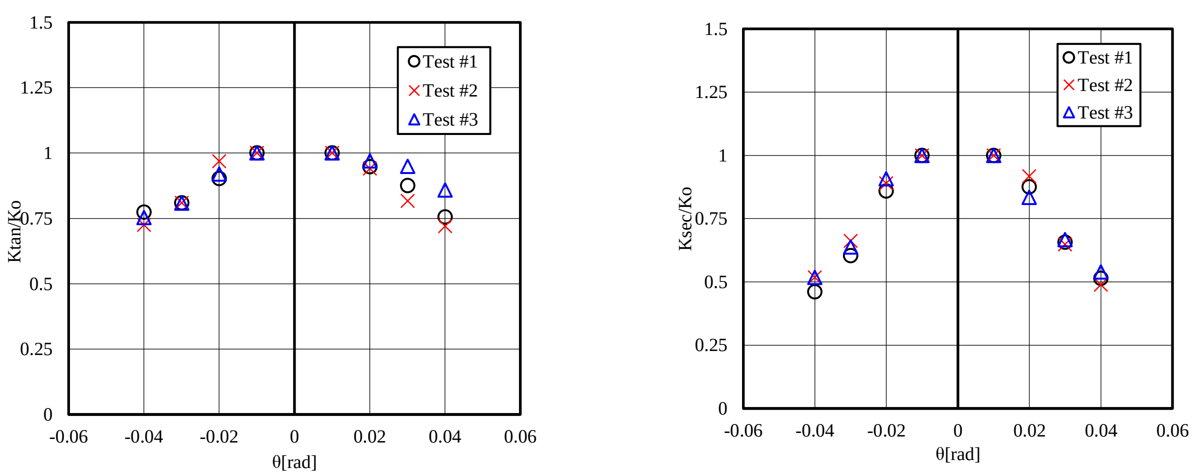
3.4 Stiffness

Measuring the tangent stiffness in the loading and unloading branch is one way to evaluate the stiffness degradation of steel joints. Connections with pinching develop stiffness losses, while connections without hysteretic pinching reach stiffness values similar to inelastic range measurements. In Figure 7, stiffness losses of 10% at 0.02 rad drift, which is considered acceptable according to seismic design codes. On the other hand, for a 0.04 rad drift angle, losses of 25% are achieved for all specimens tested. Similarly, Figure 7 shows the secant stiffness for all tests. This value can be used to assess the loss of strength in each cycle. The results obtained show that for 2% drifts, losses of up to 10% are developed in Tests #1-#2-#3, while for Tests #4-#5-#6-#7 losses of up to 25% are recorded. Comparing the joint configurations with two beams, Tests #4-#5 achieved a fast degradation compared to Tests #6-#7, which reached the same degradation levels but by softly. This represents another evidence of the differences between the behavior of in-plane joints with respect to biaxial joints.

3.5 Comparison of results with specification requirements

According to [10], eleven connections are permitted for use in steel moment frame buildings subjected to earthquakes. These connections have been prequalified following the protocols established in [9] and the seismic-resistant design philosophy. However, of these connections, only one typology (patented) can be used with tubular columns. In this sense, the advantage of using tubular columns is widely known due to their resistance in both directions. For this reason, full-scale tests of connections with tubular columns provides easily accessible information for the use of this type of connections without requiring the payment of a patent. The hysteretic behavior is generally evaluated in [9] from the connection flexural capacity of at least 80% of the plastic moment at 4% drift angle and controlled by ductile failure mechanisms without damage to the connection components. On the other hand, the tubular column (hollow structural section or built-up box section) may be connected by four sides of column. Also, the tubular column may be subjected to bidirectional loading effects, therefore, it is necessary that its performance be evaluated considering bidirectional loads and different configurations. In this sense, preliminary results from numerical studies performed by the authors have shown that the performance of the connections is affected as a larger number of beams are connected, especially when 2BC or three-beam joints are analyzed.

In this research, the results obtained confirm the hypotheses raised and introduce the need to consider the loss of stiffness and strength experienced by type 2BC joints in comparison with type 2BI joints. Additionally, the fulfillment of the strong column-weak beam criterion established in [9] considering the loss of column efficiency when out-of-plane beams are connected is an important design criterion that helps to comply with favorable designs. However, nonlinear dynamic analyses require more information such as a hysteresis curve to consider in this case the stiffness and strength degradation that some configurations such as 2BC can reach with respect to the 2BI joint. Finally, when capacity designs of the connections are performed, the required column resistance to support the resistance imposed by out-of-plane beams simultaneously with in-plane beams is considered in the design, and the criteria established in [9] are satisfied, a good behavior of the connections can be expected. However, for more sophisticated analyses such as nonlinear time-history analyses, it is necessary to calibrate the hysteretic behavior of the joint according to the configuration selected, considering the loss of strength and stiffness that joints with beams in both directions can achieve.



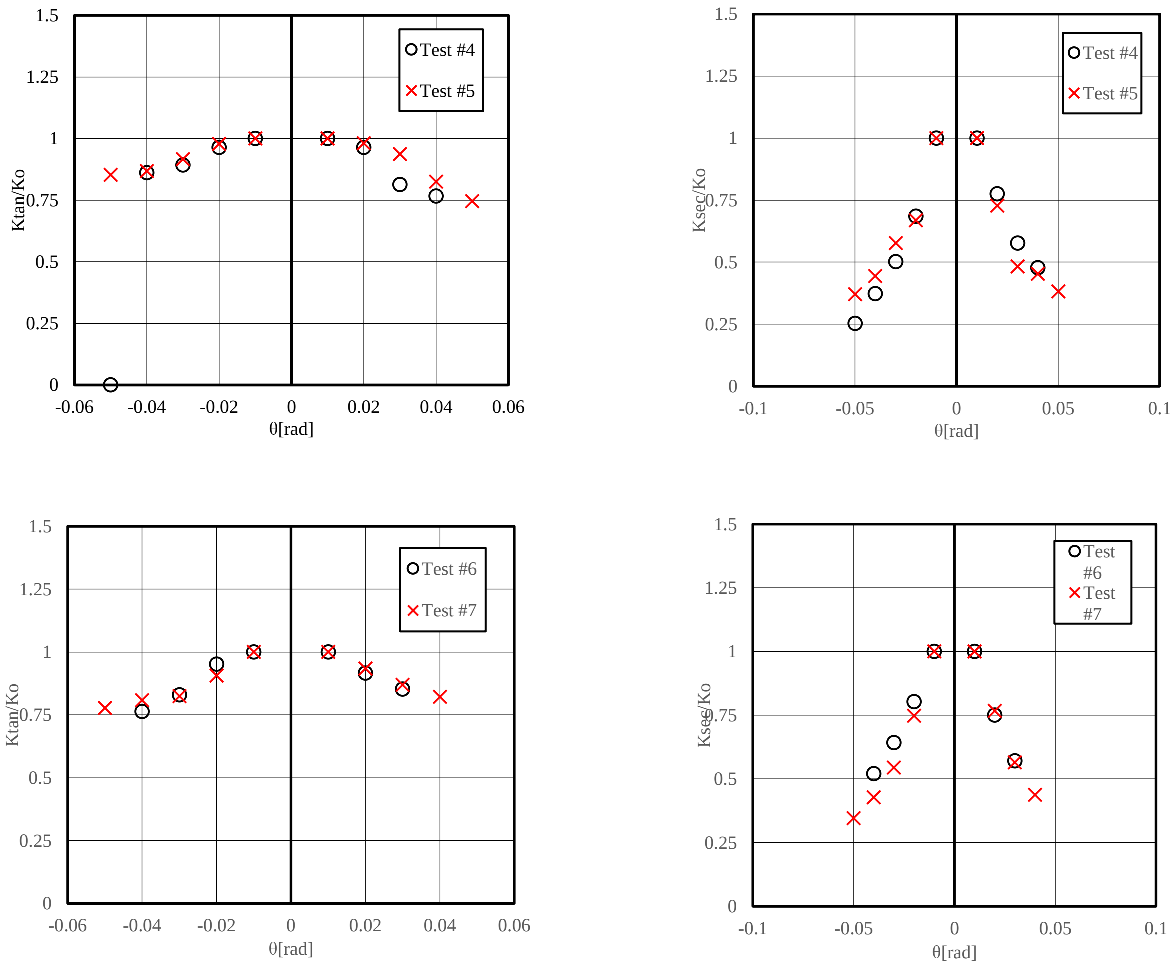


Figure 7. Tangent and Secant stiffness in specimens tested.

4. CONCLUSIONS

The cyclic behavior of interior and exterior steel moment connections connected to built-up box columns was assessed from an experimental research. Seven full-scale specimens of interior and exterior joint configurations were tested. Unidirectional and bidirectional load were used to obtain the cyclic response and their influence on hysteretic behavior of joint configurations. Finally, the evaluation was performed in terms of hysteretic behavior, failure mechanism, stiffness and dissipated energy. The main conclusions are mentioned as follow:

1. The hysteretic behavior exhibited by the specimens tested developed a stable behavior. Hysteretic pinching in the loop for the joint configurations studied was not obtained. The required minimum moment of 0.8Mp at 0.04 rad of drift angle is satisfied in all cases analyzed, according to the requirements established in Seismic provisions. However, a lower strength and stiffness was developed in the 2BC joint compared to the 2BI joint, therefore, the bidirectional effect must be considered in the hysteretic response of the connections when biaxial effects are expected.

2. The failure mechanism reported in all cases is controlled by a ductile failure. No damage in beams, columns and connection components was reached for a drift of 1%. This is desirable in buildings located in zones with high probability of seismic occurrence, which requires resilient structures capable of maintaining operability and safety levels. For a drift of 4% the damage is concentrated in beams.

3. A stiffness loss of 10% at 0.02 rad drift were achieved in 2BC joint configuration in comparison to 2BI joints. Also, a rapid degradation in 2BC joints with regard to 2BI joints were obtained. Therefore, this behavior is a evidence of the bidirectional effect in the cyclic response in steel joints.

4. Despite having two beams per joint, 2BI joints can achieve 25% more dissipated energy relative to 2BC joints. Therefore, this shows once again that the bidirectional effect can decrease the dissipated energy capacity and its efficiency in terms of performance.
5. The performance of joint configuration 4B reached higher values of dissipated energy and stable hysteretic cycles. However, the tangent stiffness decreases faster compared to the other tested configurations (2Bc and 2BI). Also, the secant stiffness reached higher values per rotation level compared to configurations 2BC and 2BI, which shows the efficiency of the confinement imposed by the beams on each face of the column.

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