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Experimental performance of double built-up T moment connections under cyclic loading

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ABSTRACT

Prequalification of the Double Split Tee (DST) connection has been conducted using hot-rolled shapes, whose availability is still limited in several countries. In addition, the range of available T web and flange thicknesses is narrow for these shapes. Thus, the use of built-up T-stubs on the DST connection seems to be a promising alternative, considering that they offer more freedom of sizing and allow a better use of the material. The experimental research reported here aimed to validate the use of built-up tees instead of hot rolled tees in DST moment connections. Four beam-to-column full-scale double built-up T (DBT) moment connections were subjected to cyclic loading following the prescribed loading history in AISC seismic provisions. The parameters varied among the specimens were the T flange thickness, the type of weld between the T flange and the stem (fillet weld or full penetration groove weld), and the column web thickness, in order to observe the most significant limit states of the connection, namely development of plastic hinges in beams, panel zone plastification, and T flange prying. The results indicate that the connection can sustain 4% drift without strength degradation, whether it is designed for T flange prying or beam hinging. In the case of the former, only limited permanent deformation of the flange is observed, while for the latter significant plastic deformation of the beam can be achieved with little or no damage of the T. The type of weld used has no significant effect on the connection performance.

1. Introduction

Prior to the earthquake Northridge (1994) and Kobe (1995), the use of partial or total weld between beams and columns had acceptable behavior. However, once these events occurred, numerous steel moment-frame buildings experienced brittle fractures in the beam-to-column connections, regardless of their age and/or number of stories, behaving differently from what was anticipated [1]. In particular, the 1994 Northridge Earthquake in California, USA, evidenced the shortcomings in the existing design recommendations for steel construction at that time. This alarmed the engineering community, resulting in an important FEMA-funded investigation project called SAC Steel Project and conducted by the SAC Joint Venture, the American Institute of Steel Construction (AISC), the American Iron and Steel Institute (AISI), and the National Institute of Standards and Technology (NIST).

Over the years, seismic design has focused on providing a ductile response of steel structures. Before 1994 the typical moment connection was the Welded Unreinforced Flange (WUF), which was supposedly capable of developing a ductile response by inducing large plastic rotations without significant strength degradation. However, after the devastating Northridge earthquake, the research carried out by the SAC Steel Project led FEMA to publish a document with recommendations for the design of new steel moment frame buildings [2]. This document included a list of prequalified moment-frame connections with design procedures and limitations, wherein the Double Split Tee (DST) Connection can be found.

The DST Connection consists of two T shapes, called T-stubs, bolted to the beam and column flanges, and a shear tab welded to the column flange and bolted to the beam web, as shown in Fig. 1. It is considered a full-strength and partially restrained connection, i.e., it can develop the full expected plastic moment of the beam itself and its deformation increases the calculated drift of the frame by more than 10%, respectively. A study conducted at the Georgia Institute of Technology [3,4] was the main influence for the FEMA 350 [2] recommendations for DST connections and eventually led to the prequalification of the connection in [5]. The study considered T-stubs, both individually and in full-scale bolted connections, cut from hot-rolled wide flange sections, leaving built-up T-stubs out of the connection prequalification. The experimental part of the research program included forty eight T-stubs specimens subjected to monotonic and cyclic loading and six full-scale beam-column connection specimens subjected to cyclic loading. It was reported that the energy was dissipated through bending of the T flanges, caused by combined tension and prying, and friction between the T stem and the beam flange, initiated after significant plastic hinging had developed in the beams in the case of the connection specimens.

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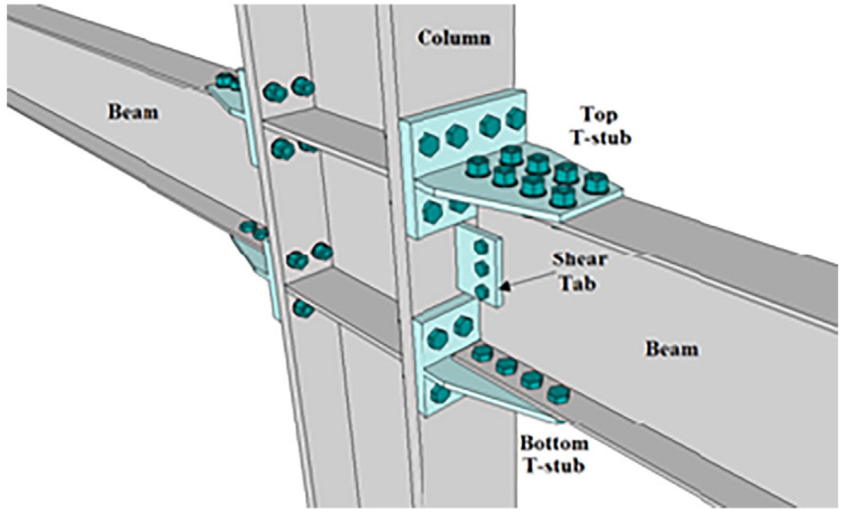

Fig. 1. DST connection.

Several experimental and numerical studies on bolted T-stubs connections can be found in the literature. Piluso et al. [6] tested 11 back-to-back T stubs loaded through the stems under monotonic tensile load up to the failure. This experimental program did not consider the effect of failure modes associated with the T-stem or the shear bolts, because the grips of the testing machine grabbed directly the T stems. A fracture of the weld between stem and flange was observed in the single welded specimen tested by these authors, starting in the center and propagating toward the flange edges, which was blamed on the stress concentration generated by 3D effects in the central part of the T flange.

Girão Coelho et al. [7] tested 32 back-to-back built-up T specimens loaded through the stems under monotonic tensile load. Although most specimens failed by fracture of the tension bolts after bending deformation of the T flange, a few failed by cracking in the heat affected zone in the T-flange. The authors blamed these failures in the residual stresses and modified microstructure in the heat affected zone, which was highly dependent on the type of electrode used and the hydrogen content.

Hantouche et al. [8] tested 24 built-up T stub specimens of uniform size with a flange thickness of 50 mm and a stem thickness of either 19 or 32 mm, fabricated using fillet or complete joint penetration (CJP) welds, under axial monotonic and cyclic loads. In general, the thinner stem specimens failed by net section fracture of the stem, while the thicker stem specimens failed by tension bolt fracture with only initial yielding of the flange. The authors concluded that T-stubs fabricated using either fillet welds or CJP welds performed adequately under both loading protocols.

There are several countries, including Chile, with an absence of hot-rolled sections, reason that motivates further study on the use of welded T-stubs on the type of connection mentioned. Additionally, they have advantages of improved material utilization and freedom of sizing. Herrera et al. [9] numerically studied welded T-stubs subjected to monotonic tensile loads by creating a finite element model using ANSYS. They concluded it is difficult to make yielding of the T stem the controlling limit state because of the weld existence, and that it may be more appropriate allowing for a controlled prying of the T flange. Herrera et al. [10] conducted the experimental stage of the numerical study. They tested two series of 11 welded T-stubs under a monotonically increasing tensile load, were the majority failed by fracture of the tension bolts after significant yielding of the T flange due to the prying of the T flange, except for the thinner stem specimens, which failed due to tensile fracture on the net section of the T stem. The strength was larger than the strength predicted by current design recommendations, and the failure mode for the thinner stems predicted by the specifications did not match the failure mode observed in the experiments. A relevant fact is that none of the specimens tested showed signs of weld damage, encouraging the use of welded T sections.

Bravo and Herrera [11] tested two series of 10 welded T-stubs under cyclic loading conditions, with the same geometry of the ones reported in [10]. It was concluded that the performances of welded and hot-rolled T-stubs were comparable, since the observed phenomena were controlled by the same parameters. The T flange is the element that contributes the most to the total deformation and ultimate strength. Additionally, it was recommended to use a flange-to-stem thickness ratio near 1.25 in order to allow deformation of the T flange without reaching the yielding of the tension bolts.

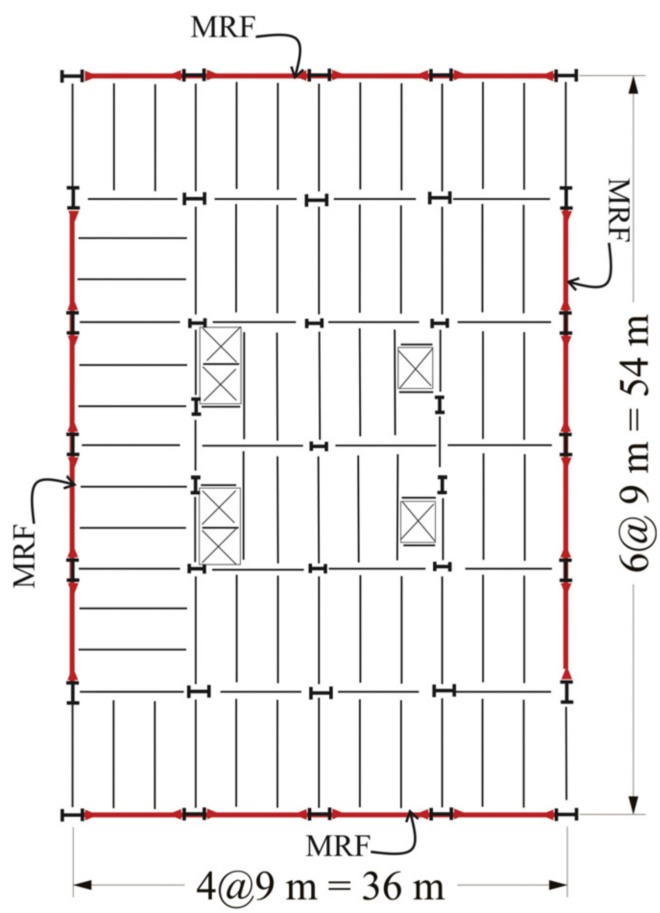

Fig. 2. Plan view of the analyzed perimeter moment resisting steel frames building.

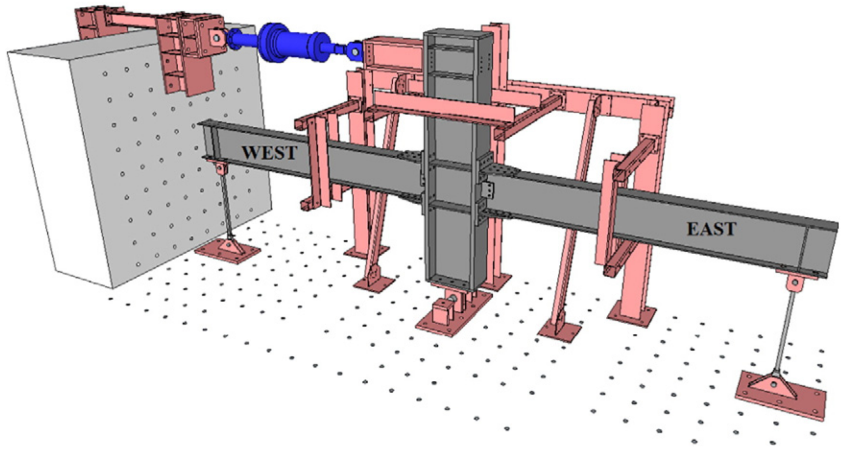

Fig. 3. Experimental setup.

The main goal of this paper is to document experimental data associated to the global performance of full-scale double built-up T connections under cyclic loading conditions. Four different connections specimens, designed for use in Special Moment Frames (SMF), in conformance with the FEMA [2] and AISC Seismic Provisions [12] and Specification [13] for structural steel buildings, are tested in which a different mode of failure is induced on each specimen: (a) inelastic deformation in the connection; (b) plastic hinge formation on the beams; (c) panel zone shear yielding in the column; and (d) panel zone shear yielding in the column together with plastic hinge formation on the beams. In addition, two types of welds were used to fabricate the T stubs (fillet welds and full penetration welds) to study the effect of this detail. The size of the connections is determined by a trial and error process to satisfy the Chilean seismic design code [14] for the case of a 12-story residential building structured on moment frames. This data helps determine their suitability for use in seismic resistant structures, and provides the basis for which partially-restrained connections could be designed in the future, since the current Chilean seismic design codes prohibits their use in steel rigid frames of industrial structures, and leaves it uncertain for residential buildings.

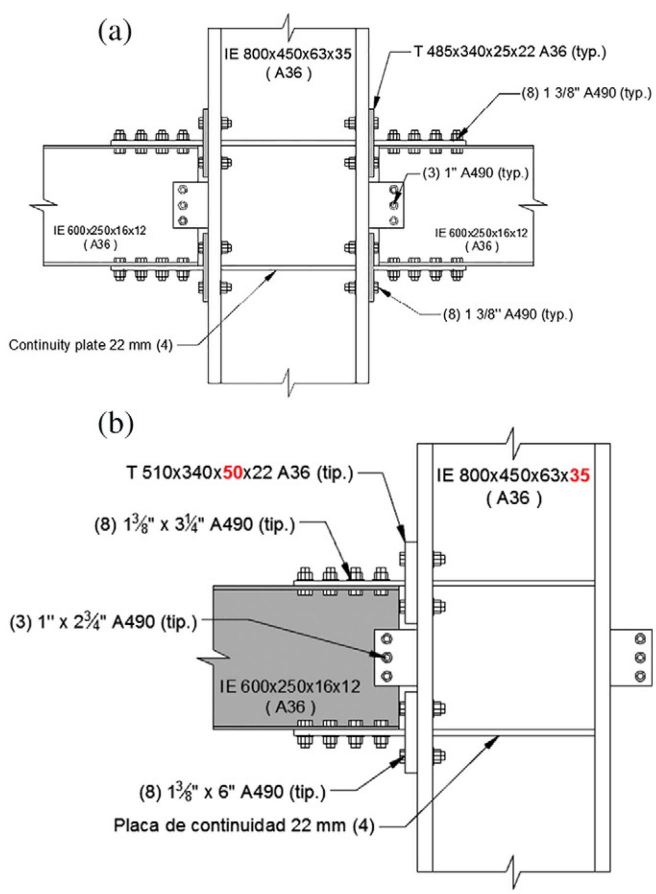

Fig. 4. Connection details: a) SS-01; b) SS-02.

2. Experiments

2.1. Specimens size

As previously stated, the size of the tested connections is determined by a trial and error process to satisfy the Chilean seismic design code [14] for the case of a 12-story residential building structured on special moment resisting frames (SMRF). The seismic demand was determined by considering soil type A, seismic zone A, and a response modification factor of 7 (for SMRF) according to the classification established in NCh 433 [14]. This code defines 3 seismic zones for the entire country, approximately from East to West, using the peak ground acceleration (PGA) as the discerning variable: zone 1 has the lowest seismic hazard (0.2 g) and corresponds to the region close to the Andes Mountains; Zone 2 has intermediate seismic hazard (0.3 g), corresponding roughly to the area between the Andes and the Coastal Mountain Range; and Zone 3 has the highest seismic hazard (0.4 g), being the closest to the coast. Six soil types are defined in the code: A, rock or cemented soil, characterized by a shear wave velocity over 900 m/s and RQD above 50% (for rock, according to ASTM D6032) or a simple compressive strength of the soil over 10 MPa (for cemented soil); B, soft/fractured rock or very dense/firm soil, characterized by a shear wave velocity over 500 m/s and simple compressive strength of the soil over 0.4 MPa (for fine soils) or an SPT with an N-value above 50 (for granular soils); C, dense/firm soil, characterized by a shear wave velocity over 350 m/s and simple compressive strength of the soil over 0.3 MPa (for fine soils) or an SPT with an N-value above 40 (for granular soils); D, medium dense/firm soil, characterized by a shear wave velocity over 180 m/s and an undrained shear strength over 0.05 MPa (for fine soils) or an SPT with an N-value above 30 (for granular soils); E, medium compacity/consistency soil, characterized by a shear wave velocity under 180 m/s and an undrained shear strength under 0.05 MPa (for fine soils) or an SPT with an N-value above 20 (for granular soils); and F, special soils, requiring ad-hoc studies. The seismic zone and soils type were chosen to obtain a design that could be tested in the lab; NCh433 has a stringent drift limit that usually controls the design of MRF buildings. For modelling purposes, the effect of the connection on the lateral response of the building is included by using an equivalent bending stiffness of the beam (EI_{eq}) as suggested in [2]. The building has perimeter moment

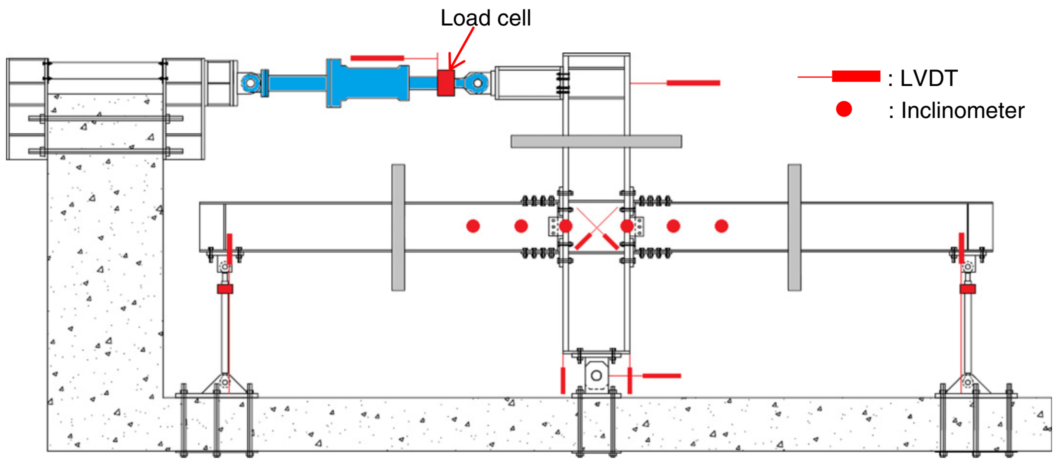

Fig. 5. Experimental instrumentation.

resisting steel frames and interior gravity frames with a rectangular plan of 36 m by 54 m as depicted in Fig. 2, and uniform 3.5 m story heights. The distance between centerlines of the frames columns is 9 m in both directions as also shown in Fig. 2. For the sake of simplifying the analyses, hinge joint are assumed between perpendicular beams of the building; thus, numerical analyses are performed on idealized 2-D perimeter frames. Based on the results of these simulations, it was concluded that the sizes of the beams are controlled by the maximum interstory drifts allowed in [14] (0.2%) and the sizes of the columns are set to meet the criteria of strong column-weak beam established in [2]. As a result, a W24 × 84 beam and a W36 × 194 column were chosen to conduct the experimental program. Full details of the design process of the connections can be found in [15].

2.2. Test setup

Five tests were carried out in the Experimental Laboratory of Structures of the Department of Civil Engineering at the University of Chile. The test specimens consisted of a 3.6 [m] high test-column with a pinned base with either one or two 4.3 [m] long beams fixed on each side to it, as seen in Fig. 3. These beams were supported at their other end on a rigid instrumented link. A reaction frame anchored to the reaction floor ([16]) restrained the specimens' out-of-plane movement. The cyclic load was applied by a 1000 kN actuator anchored to the reaction wall (shown in blue in Fig. 3) and driven by a displacement control software, following the loading history in Chapter K of the AISC seismic provisions [12]. As previously stated, W24 × 84 and W36 × 194 sections were selected to construct the beam and column of the test specimens respectively. However, the thicknesses of these elements were adjusted to induce two different failure modes: (a) inelastic deformation in the connection and (b) plastic hinge formation on the beams. The thicknesses of the T-stub flanges and stems were also modified from the original connection detail, maintaining the diameter and number of bolts. FEMA 350 and previous research recommend inducing these limit states, because they provide sources of energy dissipation while incurring in the inelastic range of the elements. Therefore, they provide enough deformation capacity to enable a ductile response.

All structural elements (beams, columns and T-stubs) were made from ASTM A36 steel. The average properties obtained from coupon tests ([17]) were a yield strength of $F_y = 293$ MPa and a tensile strength $F_u = 445$ MPa. The bolts were 35 mm (1⅜″) diameter ASTM A490 bolts, with a yield strength $F_y = 1149$ MPa and a tensile strength $F_u = 1246$ MPa, values also obtained from coupon tests. There were eight shear bolts and eight tension bolts in each T-stub. Two different types of welds were used for the T-stubs: full-penetration welds and fillet welds.

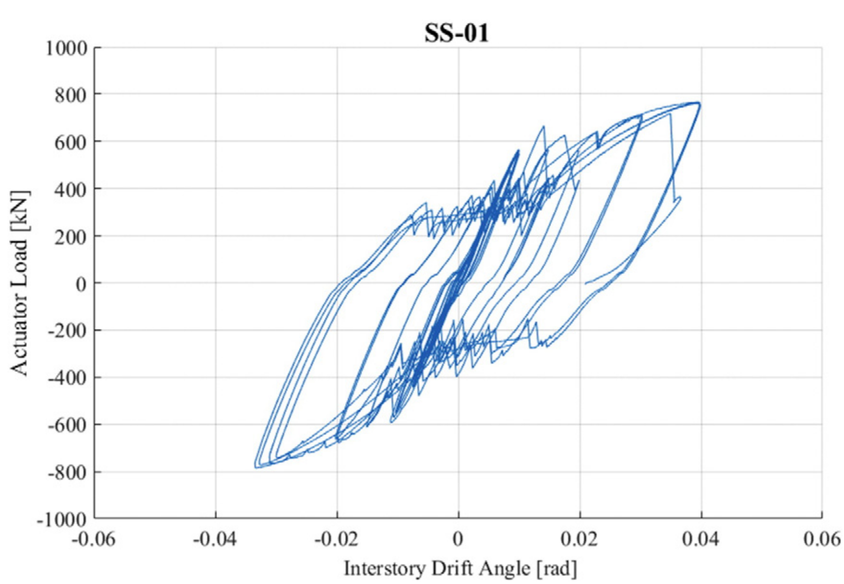

Fig. 6. Applied load vs. interstory drift angle for test SS-01.

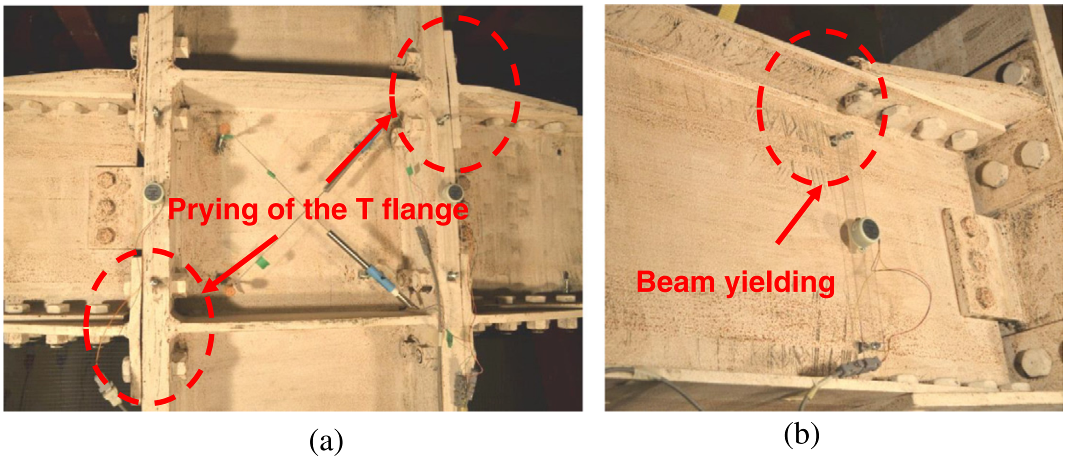

Fig. 7. Test SS-01: (a) Prying on bottom-west and top-east T-stubs at 0.03 rad drift angle; (b) yielding initiation on beams at end of test.

Fig. 4 shows the connection detail of each specimen, where SS stands for Structural System. The differences between them were the thickness of the T-stub flange (t_{fT}) and the number of beams. The east beam could not be included in SS-02 because the threads of one of the load cells supporting one of the instrumented links fractured during the test of SS-01, precluding its further use. Therefore, it was initially decided to conduct two tests of specimen SS-02 (a and b), using the beam and T-stubs that were originally intended to be connected on the east side. Considering that the beams from test SS-01 and the T-stubs from test SS-02a and b showed no signs of damage, it was decided to conduct two additional tests (c and d) reusing the column from the previous tests, the beams from test SS-01 and the T-stubs from tests SS-02a and SS-02b. However, a new set of bolts was used, since A490 bolts should not be reused.

The failure moment M_f at the face of the column of specimen SS-01 was determined following Eq. (3-61) of FEMA 350, while for specimen SS-02 it was defined as $1.2 \cdot M_{yf}$ (Eq. (3-54) of FEMA 350), where M_{yf} is the yield moment of the beam defined by Eqs. (1) to (5):

$$M_{yf} = C_y \cdot M_f \tag{1}$$

$$C_y = \frac{1}{C_{pr}\frac{Z_e}{S_b}} \tag{2}$$

$$C_{pr} = \min\left\{\frac{F_y + F_u}{2F_y}; 1.2\right\} \tag{3}$$

$$M_f = M_{pr} + V_p \cdot x \tag{4}$$

$$M_{pr} = C_{pr} R_y Z_e F_y \tag{5}$$

where C_{pr} = peak connection strength coefficient; F_y = specified minimum yield stress of the beam; F_u = specified minimum tensile strength of the beam; R_y = ratio of the expected yield stress to the specified minimum yield stress of the beam, equal to 1.3 for A36 plates; S_b = elastic section modulus of the beam at the zone of plastic hinging; and Z_e = effective plastic section modulus of the beam at the zone of plastic hinging.

2.3. Instrumentation

The instrumentation was selected based on the results of interest, namely: applied load-top displacement, moment-rotation of beam plastic hinge and connections, and shear versus deformation of the panel zone. Therefore, LVDTs were used to record the top and support displacements as well as the panel zone deformation, inclinometers

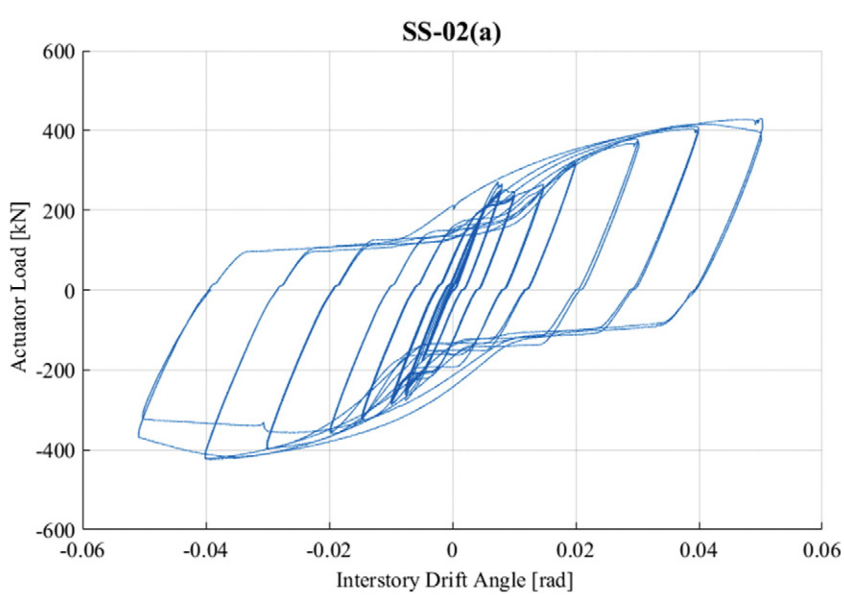

Fig. 8. Applied load vs. interstory drift angle for test SS-02a.

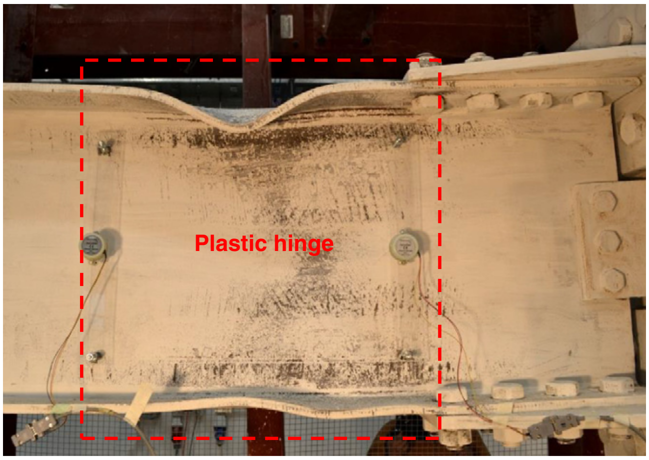

Fig. 9. Plastic hinge development at 0.05 rad interstory drift angle (cycle 2) in test SS-2(a).

to record the total rotation of the beam and the connection, and load cells to record the applied load and reactions on the beam-ends. The location of these sensors is shown in Fig. 5, where the red circles indicate the inclinometers, the thin red rectangles with a line indicate the LVDTs, and the thick red rectangles with a black border are the load cells.

3. Results

3.1. Test SS-01

Specimen SS-01 was expected to experience inelastic deformation in the T-stub flanges, allowing a prying effect to take place on the tension bolts. While conducting the test, the actuator ran out of stroke in the west direction at 3 percent drift, but could reach 4 percent drift in the east direction. This issue was caused by an error in the length of the extension piece between the actuator head and the column top. The piece was fixed for the next tests.

The applied load-interstory drift angle curve for the test is shown in Fig. 6. The maximum moment reached by each beam at the column face, calculated as the reaction on the link multiplied by the beam length, exceeded the minimum of 0.8 Mp of the beam established by the AISC Seismic Provisions [10], with no strength degradation experienced by either of them.

The prying of the T-stub flanges was notorious at an interstory drift angle of 0.03, as seen in Fig. 7a, instant at which yielding initiation on the beams was also observed (Fig. 7b). The west T-stubs were fabricated with fillet welds and the east T-stubs with complete joint penetration welds, neither of which presenting any visible damage.

3.2. Test SS-02a

Specimens SS-02 were expected to experience plastic hinging on the beam at the end of the connection. The applied load-interstory drift angle curve of the test is shown in Fig. 8. Yielding was experimentally observed on the beam in the first peak of 0.02 rad interstory drift angle cycles, which coincides, with the first significant deviation on the reloading curve for the second cycle at the same deformation. Notorious local buckling on the beam flanges was observed at 0.03 rad drift angle, together with a small but noticeable strength reduction for the second cycle at this deformation. In addition, the first noticeable degradation of the unloading stiffness occurs at this deformation. The moment reached at the column face was higher than the limit required at 0.04 rad drift angle (0.8 Mp). The test was finished after achieving a 0.05 rad drift angle (second cycle), instant at which the plastic hinge had notoriously developed, as shown in Fig. 9, with no visible damage on the CJP welds of the T-stubs.

3.3. Test SS-02b

The applied load-interstory drift angle curve of the test is shown in Fig. 10. Specimen SS-02b showed a similar performance to SS-02a, with the exception of the strength degradation for the second cycle at 3 percent drift. However, after reaching an interstory drift angle of 0.05 rad, it developed a fragile fracture in the beam top flange, where an initiation of a net section fracture occurred in the last row of the shear bolts, caused by the high deformation demands imposed by the buckling of the beam flange. Contrary to specimen SS-02a, the inelastic deformation was concentrated around the end of the T stem, which could explain the lack of strength degradation and the fracture. The test continued up to 0.05 rad, where the local buckling was very noticeable, as observed in Fig. 11. Significant strength degradation took place during the second cycle at 0.05 rad drift angle, when the fragile fracture shown in Fig. 11 occurred. Both T-stubs were

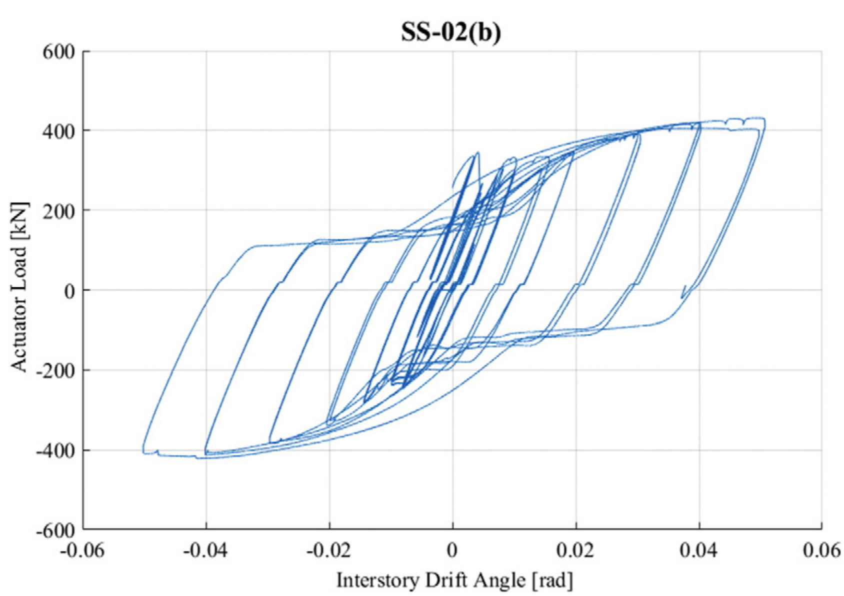

Fig. 10. Applied load vs. interstory drift angle for test SS-02b.

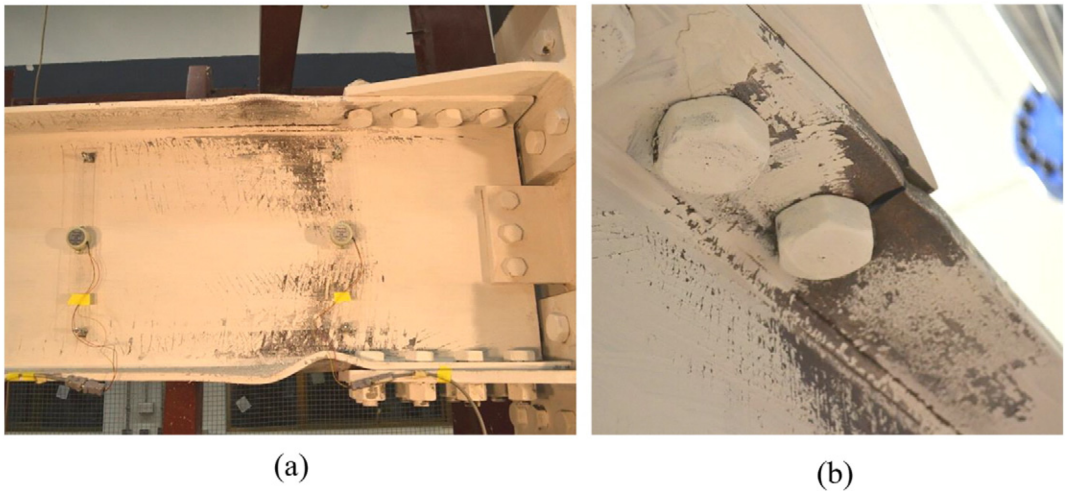

Fig. 11. Test SS-02(b): a) Plastic hinge development; b) fracture at 0.05 rad interstory drift angle.

fabricated using fillet welds, with no visible damage experienced by any of them.

3.4. Tests SS-02c and SS-02d

The applied load-interstory drift angle curves of the tests are shown in Figs. 12 and 13. These specimens showed a performance similar to that of SS-02b, developing also a fragile fracture in the last row of the shear bolts in the beam top flange, discarding a possible effect of the type of weld on the occurrence of the fracture. No visible damage was observed on the welds either. Neither the strength nor the deformation capacity seem to have been affected by the loading cycles applied to the beams and the T-stubs in the previous tests, which reinforces the observation regarding the insignificant level of damage sustained by these components during the initial tests (SS-01, SS-02a, and SS-02b).

4. Conclusions

Five full-scale beam-to-column tests were conducted to study the response of the double built-up T moment connection under cyclic loading conditions. The specimens tested responded as expected, with yielding and plastic deformation occurring where the design intended. Therefore, the current design procedure for DST stipulated by AISC 358 [5] is applicable for this type of connection when using built-up T-stubs.

The SS-01 connection was capable of accommodating a story drift angle of 0.04 rad, achieving a flexural resistance at the column face significantly larger than that required by the AISC Seismic Provisions [12], without any strength degradation. However, the 0.04 rad drift could not be achieved in the west direction due to problems with the experimental setup. All SS-02 connections achieved a story drift angle in excess of 0.04 rad in both directions, with a flexural resistance at the column face also higher than that required by AISC Seismic Provisions [12]. Strength degradation was observed at 0.05 rad story drift angle. None of the welds used to fabricate the built-up T-stubs showed visible damage, corroborating what had been concluded from previous works [8–10] on this type of connection.

Regarding the failure mechanism of this type of connection, the study corroborated that it can be changed from plastic bending of the T flange to beam plastic hinging just by increasing the thickness of the T flanges.

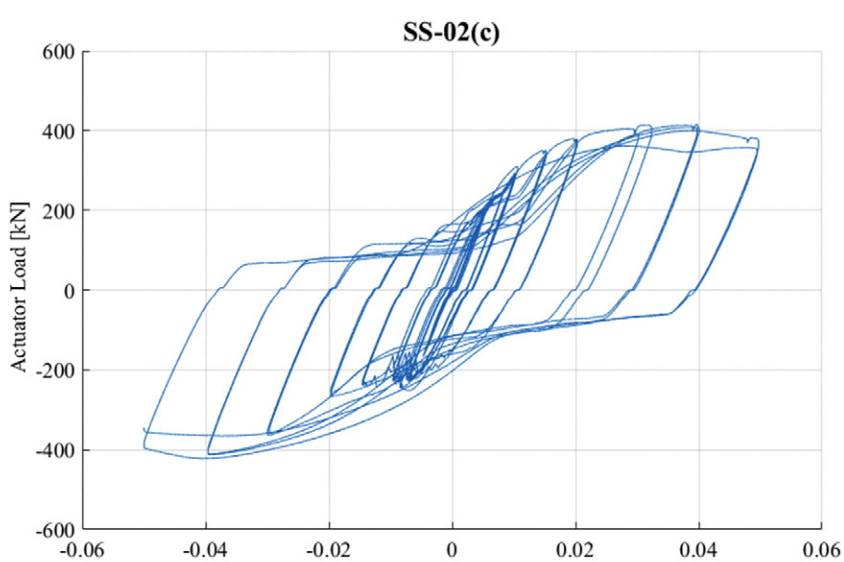

Fig. 12. Applied load vs. interstory drift angle for test SS-02c.

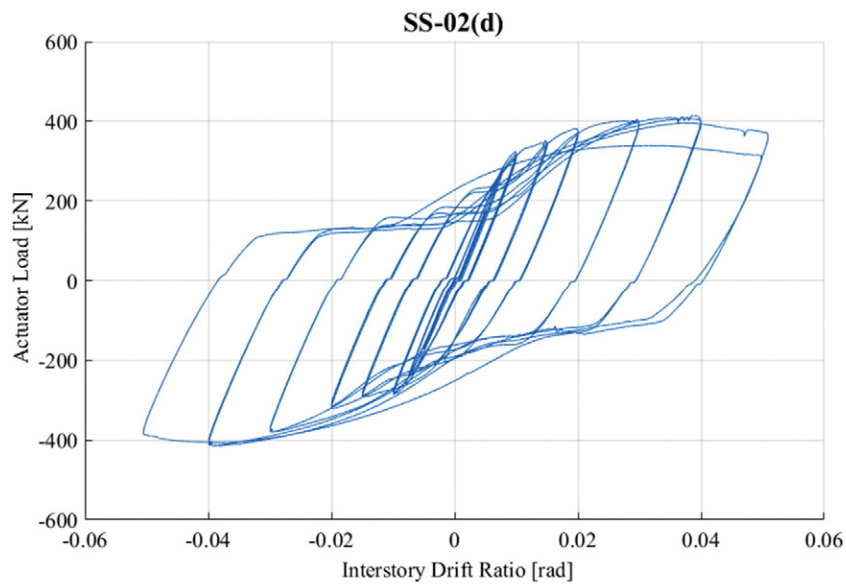

Fig. 13. Applied load vs. interstory drift angle for test SS-02d.

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