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EXPERIMENTAL PERFORMANCE OF DOUBLE BUILT-UP T MOMENT CONNECTIONS

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Abstract

The brittle failures detected during the 1994 Northridge earthquake on steel moment-frame buildings prompted significant investigations on steel moment connections. This work resulted in the proposal of several prequalified connections, including the Double Split Tee (DST) connection, to be used in beam-to-column moment joints of seismic resistant steel buildings. The DST connection is a partially restrained-full strength connection, which consists of two T-stubs bolted to the column flange and the beam top and bottom flanges. The prequalification process was conducted using hot-rolled shapes, whose availability is still limited in Chile and other countries. In addition, the range of usable T web and flange thicknesses is narrow for these shapes. These reasons led to study the use of built-up T-stubs as an alternative on the DST connection, considering that they offer a larger freedom of sizing and improved material utilization. This experimental research aims to validate the use of current design recommendations for the DST connection when built-up T shapes are used instead, by testing three beam-to-column full-scale welded T moment connections under cyclic loading. The parameter varied between the specimens was the T flange thickness, in order to observe the most significant limit states of the connection. The experimental data allowed the identification of the failure modes of the beam and connection, which were the development of plastic hinges and T flange prying effect, respectively. The results indicate that the major contributors to energy dissipation in the connection were the T flange prying and plastic hinging mechanisms, as predicted, and that the three Beam-to-Column connections accomplish the requirements for use in Special Moment Frames, in terms of acceptable strength degradation at a story drift angle of 0.04 rad, with no visible damage on the T-stubs' welds.

Keywords: Moment frame connection; failure modes; partially restrained.

1. Introduction

The 1994 Northridge Earthquake in California, USA, brought to the table the shortcomings in the existing design recommendations for steel construction at that time. 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. This alarmed the engineering community, resulting in an important FEMA-funded investigation project called SAC Steel Project and conducted by the SAC Joint Venture, AISC, the American Iron and Steel Institute and the National Institute of Standards and Technology.

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 this response by inducing large plastic rotations without significant strength degradation. But after this devastating earthquake, the research carried out by the SAC Steel Project led FEMA to publish FEMA 350: Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings [1]. This guideline includes 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. This connection was prequalified thanks to the study conducted by Swanson and Leon (2000) at the Georgia Institute of Technology. The T-stubs they studied, both individually and in full-scale bolted connections, were all cut from hot-rolled wide flange sections, leaving built-up T-stubs out of the connection prequalification. 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.

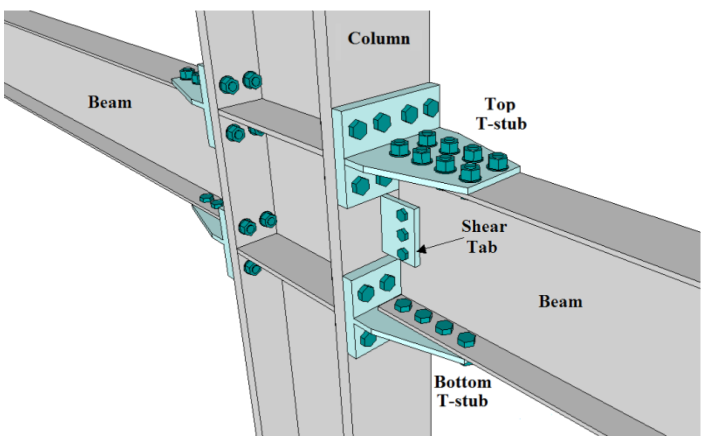


Fig. 1 – DST Connection

This research is part of an ongoing study at the University of Chile regarding welded T-stubs, whose main objective is to document experimental data associated to the global performance of four full-scale double built-up T connections under cyclic loading conditions, although the results of only two of them are included herein. 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.

2. Previous related research

As previously stated, the main study that influenced the FEMA 350 [1] recommendations is Swanson and Leon's [2] [3] experimental research program, which included 48 T-stubs specimens subjected to monotonic and cyclic loading and six full-scale beam-column connection specimens subjected to cyclic loading. 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.

Piluso et al. [4] 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. [5] 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 [6] 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.

Regarding research conducted at the University of Chile, Desjouis [7] studied welded T-stubs subjected to monotonic tensile loads by creating a finite element model using ANSYS. He 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. Gómez [8] conducted the experimental stage of Desjouis' numerical study. He 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.

Muñoz [9][9] developed design recommendations for this type of connection. The most relevant is that the flange-to-stem thickness ratio should be low, allowing the flange to dissipate energy and produce large rotations, making the whole T section to participate. Bravo [10] tested two series of 10 welded T-stubs under cyclic loading conditions, with the same geometry of the ones tested by Gómez [8]. 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.

3. Experimental Test Setup

Three tests were carried out in the experimental laboratory of structures of the Department of Civil Engineering at the University of Chile. The experimental setup consisted in a 3.6 [m] high test-column with a pinned base and one 4.3 [m] long test-beam fixed on each side to it, as seen in Fig. 2. These beams were supported at their other end on a rigid instrumented link. In order to mantain safety and to restrain the specimens' out-of-plane movement, a reaction frame anchored to the reaction floor was designed by Núñez [11]. The cyclic load was applied by a 1000 kN actuator anchored to the reaction wall (Fig. 3) and driven by a displacement control software, following the FEMA/SAC loading history, which is the same that appears in Chapter K of the AISC seismic provisions [12].

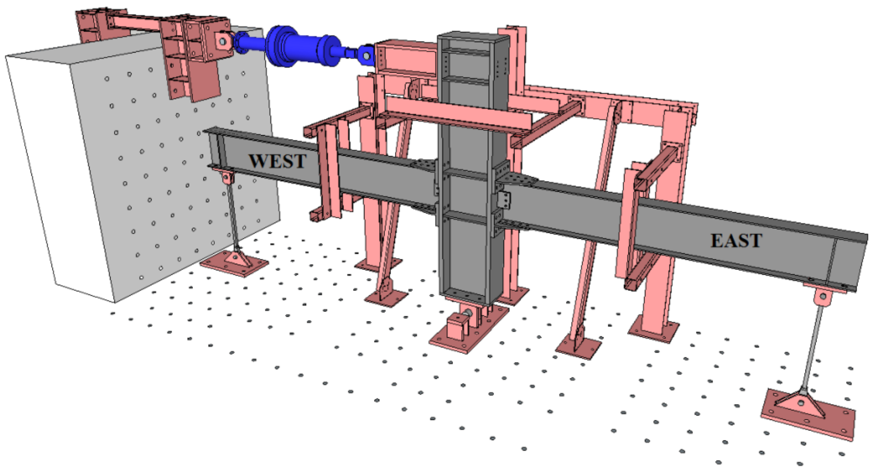


Fig. 2 – Experimental Setup

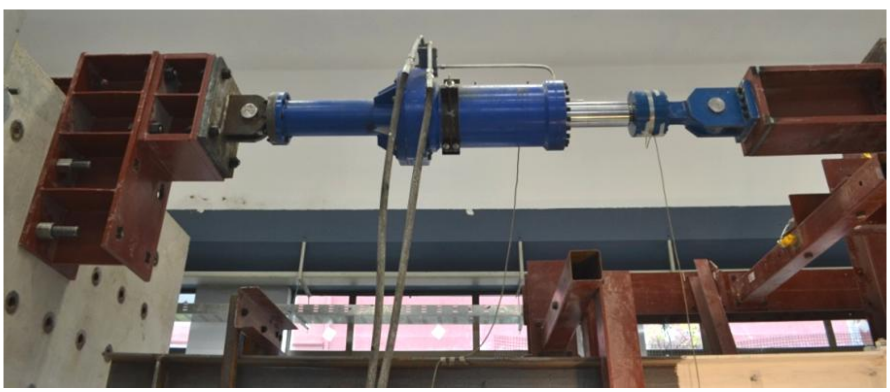


Fig. 3 – Hydraulic actuator mounted at its initial position

4. Description of Specimens

The two different specimens presented herein were designed for use in Special Moment Frames (SMF), in conformace with the AISC seismic provisions [12] and specification [13] for structural steel buildings. A W24x84 beam and a W36x194 column were chosen from the design made by Alarcón [14] of a 12-story residential building structured on moment frames. However, these elements' sections were adjusted to induce different failure modes on each specimen: (a) inelastic deformation in the connection and (b) plastic hinge formation on the beams. The T-stubs' dimensions were also changed from the original connection detail, maintaining the diameter and number of bolts.

The limit states induced are recommended by FEMA 350 and previous research, because they provide sources of energy dissipation while incursing in the inelastic range of the elements. Therefore, they generate enough deformations to enable a ductile response.

ASTM A36 steel was used for the beams, columns and T-stubs. The measured properties obtained from coupon tests 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 of F_y=1149 MPa and a tensile strength of F_u=1246 MPa, values also obtained from coupon tests. There were eight shear bolts and eight tension bolts in each T-stub. The specified welds of the T-stubs were varied on each side of the beam-to-column joint: full-penetration type on one side, and fillet type on the other.

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. Since this, it was decided to conduct two tests of specimen SS-02 to still use the beam and T-stubs that were originally intended to be connected on the east side. Specimen SS-01 is shown in Fig. 5 ready for testing, where the lateral restrictions for beams and columns and one of the instrumented links can be observed.

The failure moment at the face of the column of specimen SS-01 was determined following the equation (3-61) of FEMA 350 , while for specimen SS-02 it was defined as 1.2·M_{yf} (equation (3-54) of FEMA 350), where M_{yf} is the yield moment of the beam:

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

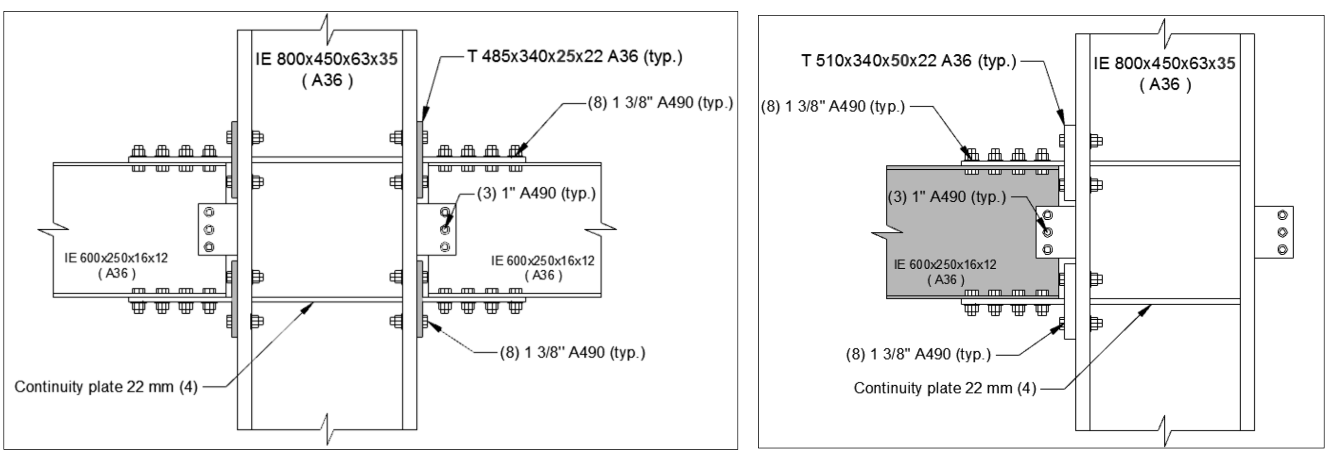
$$C_y = \frac{1}{C_{pr}\frac{Z_e}{S_b}} \tag{2}$$

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

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

$$M_{pr} = C_{pr}R_yZ_eF_y \tag{5}$$

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; 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.



(a) SS-01 (b) SS-02

Fig. 4 – Connection details

Fig. 5 – Specimen SS-01 ready for testing (left) and instrumented link (right)

5. Results

5.1. 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 its 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 fixed for the next test.

The Moment-Rotation curves for both beams are shown in Fig. 6 and Fig. 7, where the 80% of their plastic moment M_p is indicated as a limit for strength degradation. The maximum moment reached by each specimen at the column face was over the limit mentioned, with no strength degradation experienced by either of them.

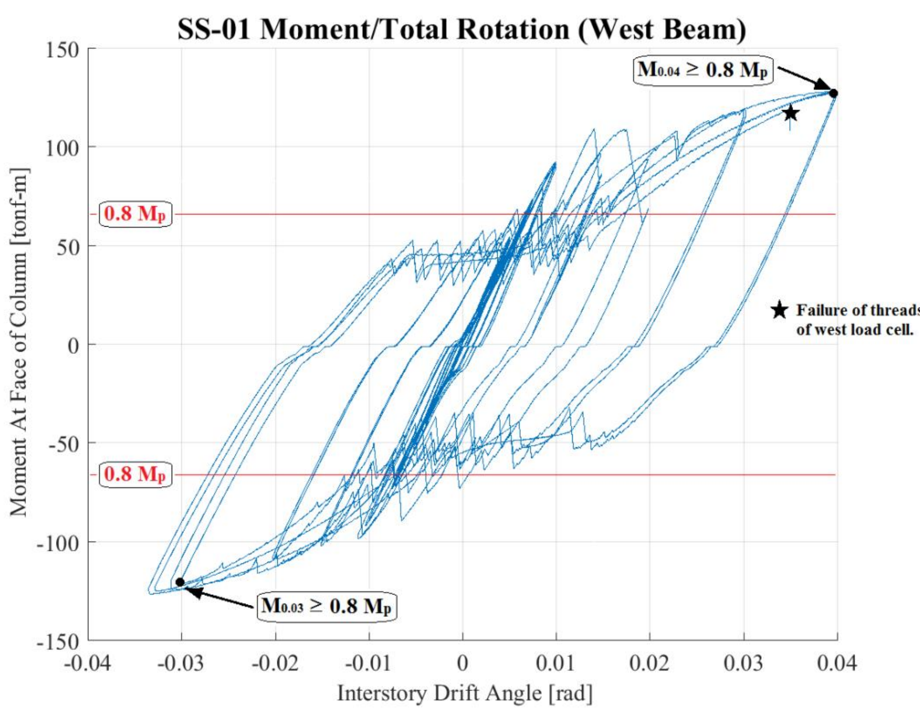


Fig. 6 – West beam moment at face of column vs. interstory drift angle for test SS-01

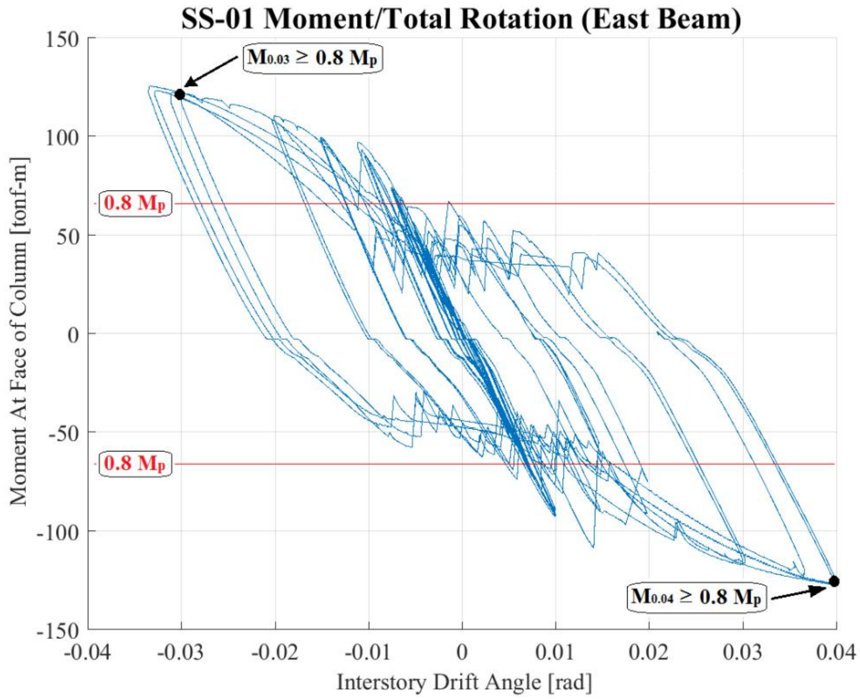


Fig. 7 – East beam moment at face of column vs. interstory drift angle for test SS-01

The prying action affecting the T-stubs was notorius at a interstory drift angle of 0.03, as seen in Fig. 8, instant at which yielding initiation on beams was also observed. The west T-stubs were fabricated with fillet welds and the east T-stubs with complete joint penetration welds, types that didn't present any visible damage.

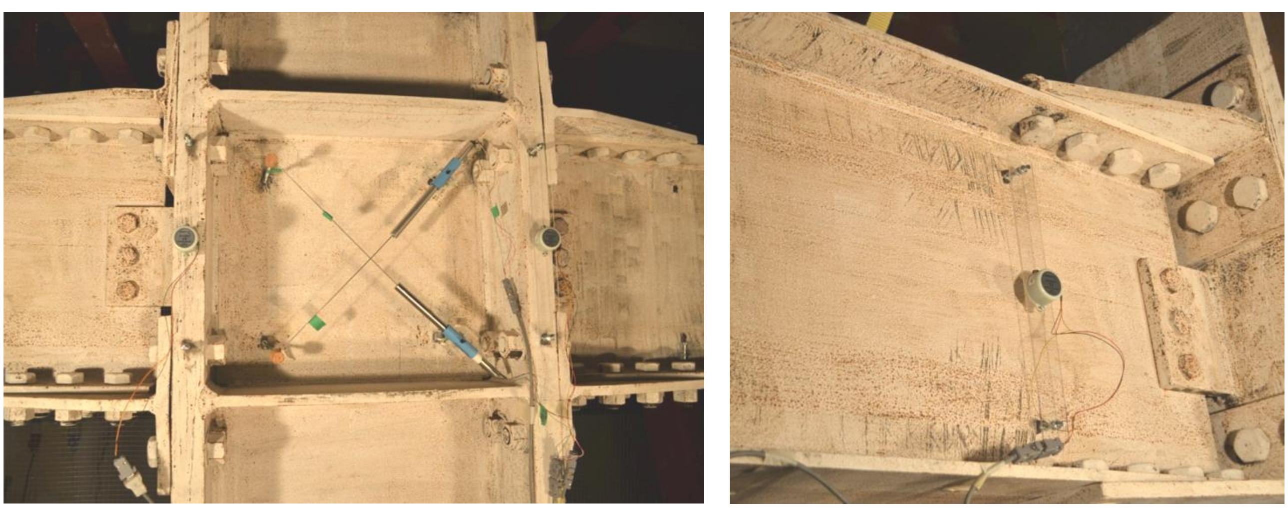


Fig. 8 – Test SS-01: Prying action on bottom-west and top-east T-stubs at 0.03 rad drift angle (left) and yielding initiation on beams at end of test (right)

5.2. Test SS-02(a)

During the test of specimen SS-02(a), yielding initiation on the beam was seen in the first peak of 0.02 rad interstory drift angle, and notorius local buckling on the beam flanges was experienced when 0.03 rad drift angle was reached. The Moment-Rotation curve of the beam is shown in Fig. 9, where the 80% of its plastic moment M_p is again indicated as a limit for strength degradation. The moment reached at the column face was higher than the limit mentioned at 0.04 rad drift angle. The test was finished after achieving a 0.05 rad drift angle, instant at which the plastic hinge had notoriously developed, as shown in Fig. 10, and with no visible damage on the T-stubs' complete joint penetration welds.

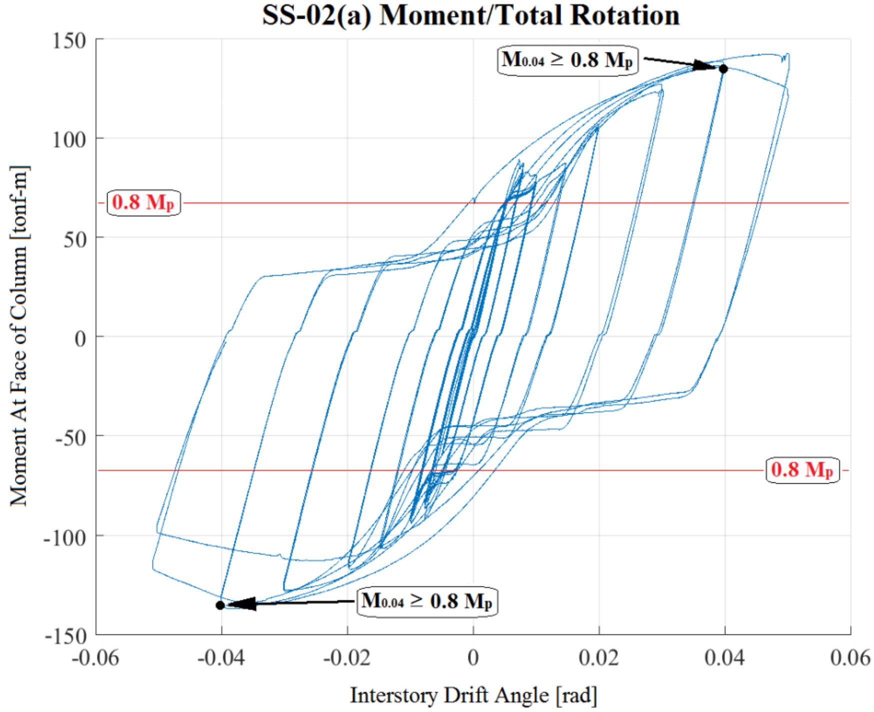


Fig. 9 – Moment at face of column vs. interstory drift angle for test SS-02

Fig. 10 – Plastic hinge development at 0.05 rad interstory drift angle (cycle 2) in test SS-02

5.3. Test SS-02(b)

Specimen SS-02(b) showed a behavior practically the same as SS-02(a). However, after reaching an interstory drift angle of 0.05 rad, it experienced a fragile fracture in the beam top flange, where an initiation of a net section fracture occured in the last row of the shear bolts. The Moment-Rotation curve of the beam is shown in Fig. 11, where it can be seen that the 80% of its plastic moment M_p is overpassed in excess at 0.04 rad drif angle. The test continued up to 0.05 rad, where the local buckling is very notorius, as observed in Fig. 12. Strength degradation took place during the second cycle at 0.05 rad drif angle, when the fragile fracture presented in Fig. 12 occured. Both T-stubs were fabricated using fillet welds, with no visible damage experienced by any of them.

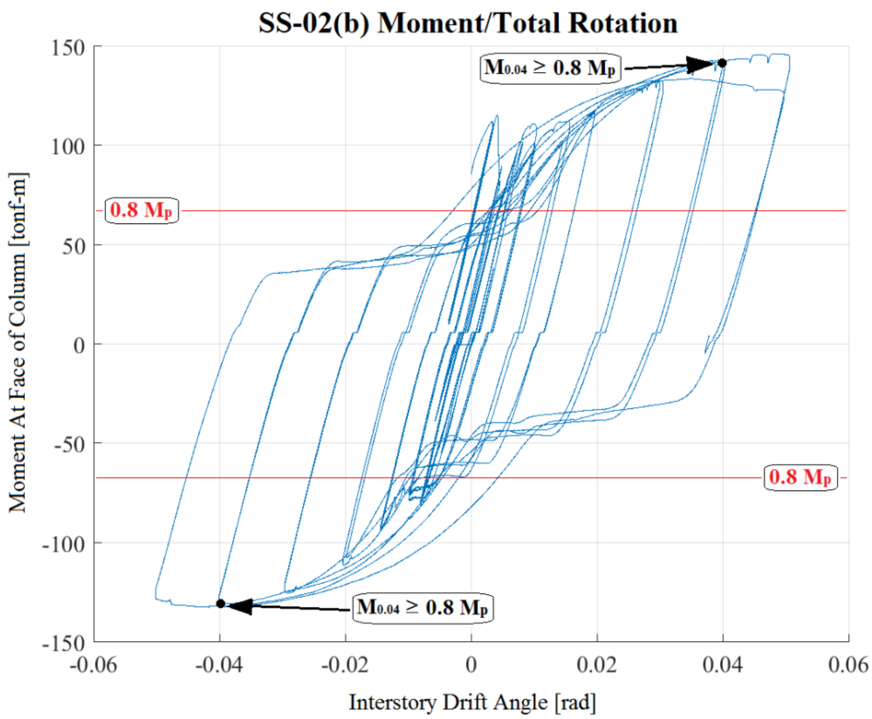
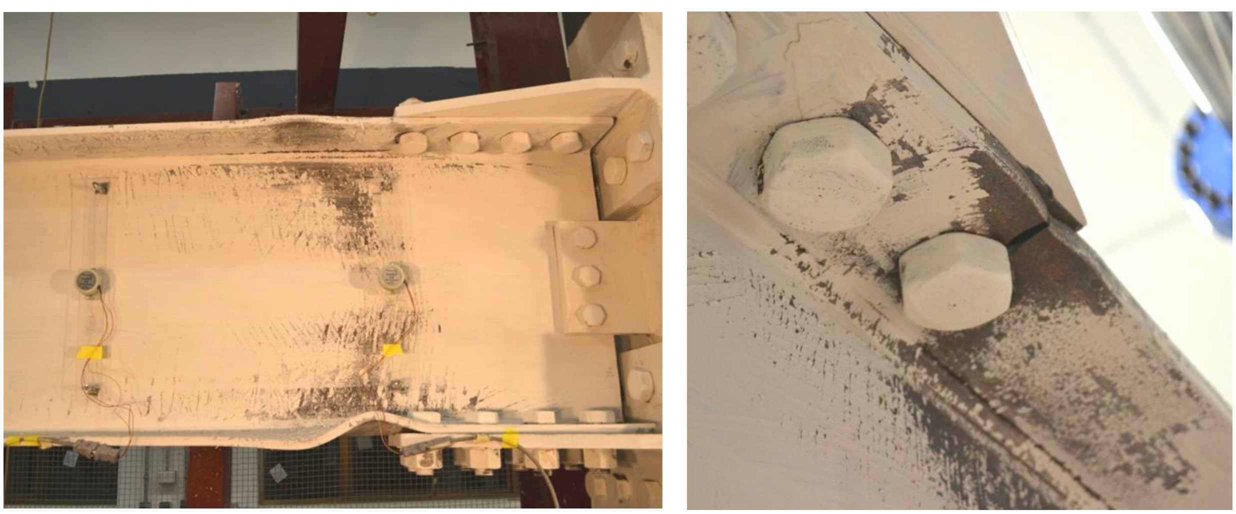


Fig. 11 – Moment at face of column vs. interstory drift angle for test SS-02



(a) (b)

Fig. 12 – (a) Plastic hinge development and (b) fragile failure at 0.05 rad interstory drift angle in test SS-02(b)

5.4. Comparison between predicted and actual M_{fail}

The predicted and actual values for the failure moment at the face of the column (M_{fail}) of each specimen are presented in Table 1. In test SS-01, the predicted failure moment is determined at initiation of plastic bending of the T-stubs flanges, therefore, the actual failure moment is the average yield moment identified in the Moment-

Rotation curves of both beams, being 1.32 times higher than the predicted one. On the other hand, in tests of SS-02 the actual failure moment experienced by the connection is 1.07 and 1.06 times above the predicted for SS-01(a) and SS-01(b), respectively, which was determined at 1.2 times the moment at onset of plastic hinge formation.

Table 1 – Comparison between predicted and actual M_{fail}

Test ID	Predicted Failure Mode	Predicted M_{fail} [ton·m]	Actual M_{fail} [ton·m]
SS-01	Yielding of the T flanges	67.6	89.3
SS-02(a)	Plastic hinging on beam	131.2	139.9
SS-02(b)	Plastic hinging on beam	131.2	139.4

6. Conclusions

An experimental study of two full-scale beam-to-column tests was conducted to determine the behavior 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 stipulated by FEMA 350 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 $0.80M_p$ of the connected beam, without any strength degradation. However, the 0.04 rad drift could not be achieved in the west direction due to equipment issues, as explained in section 5.1. Both 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 $0.80M_p$ of the connected beam. Strength degradation was observed at 0.05 rad story drift angle. None of weld types used to fabricate the welded T-stubs showed visible damage, favoring future investigations on this type of connection.

Finally, in order to design this connection so that inelastic behavior is controlled by flexural yielding of the beam, it is conservative to define the failure moment as the probable plastic beam moment at hinges transfered to the column face (M_f). The failure mechanism can be changed from plastic bending of the T flange to beam plastic hinging just by increasing the thickness of the T flanges.

7. Acknowledgements

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