TESTS AND MODELING OF T-STUB BOLTED I-BEAM TO SHS-COLUMN FRAME CONNECTIONS

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1. ABSTRACT

This paper is focused on the connection performance of an I-beam to square hollow section column by presenting its behaviour under monotonic and cyclic loading conditions based on the experimental study carried out at Bogazici University Structural Laboratory. In view of practical and economic bolted field application, an appropriate joint type which is composed of t-stub connecting elements bolted by using long partially threaded studs through hollow section column has been studied. Considering the overall performances, the tested joints maintained high plastic rotations in adequate resistance levels with acceptable energy dissipation capacities; consequently are suitable for the use as semi-rigid partial strength joints in simply designed braced steel frames in seismic areas or in unbraced steel frames in less seismic areas. This paper also presents further experimental and analytical studies on component modeling of the joint. Taking into account the present study, proposed model gives good agreement with the test results in resistance point of view.

2. INTRODUCTION

Typical connection details for simple framing between tubular columns and open section beams usually employ a fitting welded to the column which supports the beam and allows clearance for site bolting. Most of them suffer from the disadvantage of requiring fittings to the column which can prove costly to fabricate and make the section more difficult to transport without damage. The ideal system is the one which allows site bolting, leaves the column exterior without projections, is similar to traditional beam to open column joints and uses ordinary bolts.

France et al. [1] conducted series of tests to investigate the moment capacity and rotational stiffness of simple and moment connections bolted to tubular columns using flowdrill connectors. Reversed cyclic tests showed the connections to behave in a manner suitable for use as either pinned or partial strength connections for simply designed braced steel frames. Failure of the joints was by bolt pull-out, although this only occurred after the column face had undergone gross deformation. Considering the disadvantages of these blind connections, the flexibility of the square hollow section (SHS) face may limit the moment capacity of the connection when thin walls and narrow bolt gauges are employed. For that reason, some fabricators may not prefer to work in those close tolerance levels.

In the research work of Shih-Wei Peng [2], the cyclic performance of an I-beam to concrete filled tubular column joint with split-tee bolted connection detail was examined. Based on their comparison of specimen response with the AISC Seismic Provisions and FEMA recommendations, this connection detail appear to be suitable for seismic resistant design.

Recent studies in CIDECT (Comité International pour le Développement et l'Etude de la Construction Tubulaire) research works were much focused on bolted connections in relation with semi-rigid connection research.[3] Latest research work gathered all the information available to the designer to be helpful for the design of a wide range of structural steel joints connecting hollow and/or open sections. According to analytical research based on the experimental studies under static loading conditions, simple design sheets more appropriate to daily practice were developed by using component modeling for some selected joint configurations and complemented by worked examples.

Considering the easy bolted field application and leaving the column exterior without projections, similar to traditional beam to open column joints, t-stub connections bolted by using longer partially threaded studs through hollow section columns, as given in *Fig. 1*, has been studied[4] and its performance is presented in this paper.

3. EXPERIMENTAL PERFORMANCE OF THE JOINT

Tee-joints, which are representative of outer joints in a real framework, have been experimentally studied. (*Fig. 2*) The height of the column is chosen so that it represents roughly the depth of one storey.

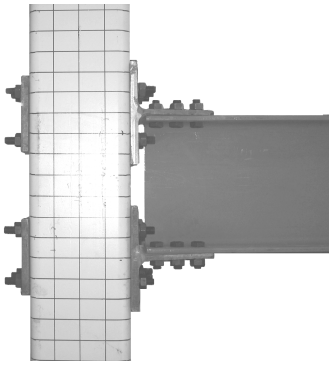

Fig. 1: Studied connection

Fig. 2: Test set-up

The specimens consisted of IPE 270 beam in steel grade S275JR joined to 200x10 mm cold –formed square hollow section in steel grade S235JRH by means of bolted t-stub connecting elements which was fabricated out of split HEB 200 profile in steel grade S275JR. Each top and bottom t-stubs were connected by five long partially threaded 8.8 grade M 16 studs in two rows through the column and by six 8.8 grade M16x45 bolts to the beam flanges.

Each type of specimen (*Fig. 3*) has been tested in a monotonic and reversed cyclic way according to Recommended Testing Procedure of ECCS [5]. The reinforcing effect of backing plate to the rear face of the hollow section has been observed.

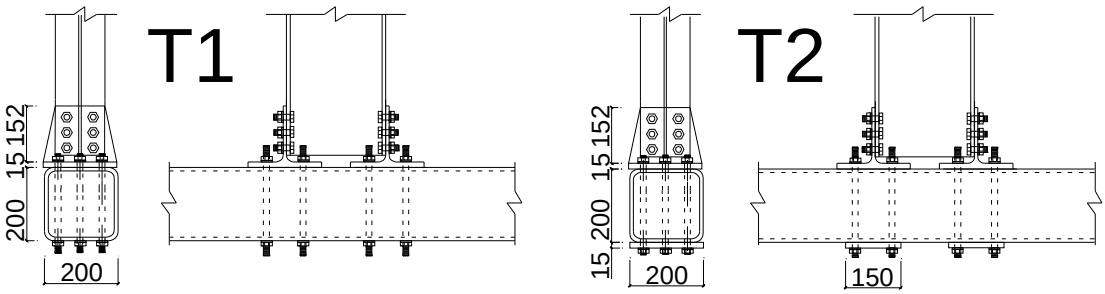

Fig. 3: Connection detail drawings of T1 and T2 type of specimens

3.1 EVALUATION OF THE TEST RESULTS

The failure of both monotonic specimens was due to the rupture of the threaded studs. (*Fig. 4*). The connection deformations were governed by the deformations at the t-stub (including the elongation of the partially threaded studs) and at the hollow section column rear face(including deformation of the column sidewall, across its depth). Backing plate reduced the deformation at the column face successfully.

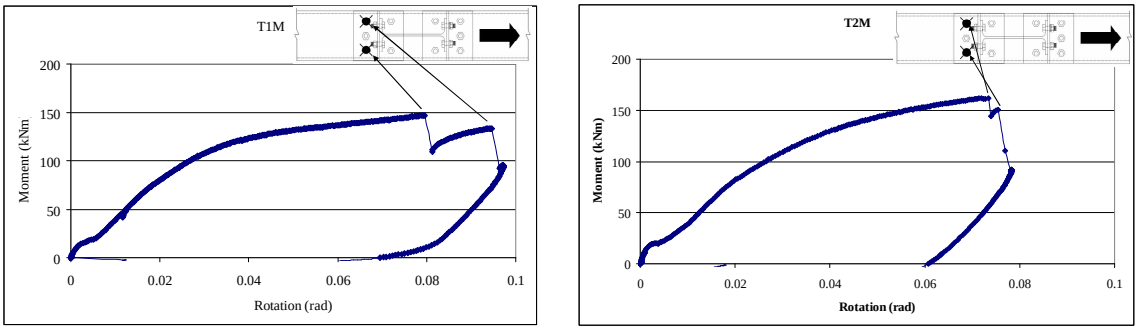

Fig. 4: Components of connection deformation of the specimen T1M and T2M

On the other hand in the cyclic tests, the displacement amplitudes of cycles were increased incrementally which did not stress the studs severely and the tests ended with the failure by low-cycle fatigue of the t-stub progressively after tolerable amount of cycles. (*Fig. 5*) It is not possible to say that both specimens exhibited a stable cyclic response and reliable energy-dissipation capacity under repeated loading. Strength deterioration phenomena due to cyclic actions were only remarkable at higher rotation levels (0.10 rad for T1C and 0.06

rad for T2C) without a sudden loss in resistance. Besides there was remarkable stiffness degradation especially after the column rear face of the specimens started deforming due to the opening gab during loading process between t-stub with respect to the column face. Hysteretic pinching behaviour started with bolt slips at t-stub-beam interface and due to the flexibility of SHS column face, this became more intense.

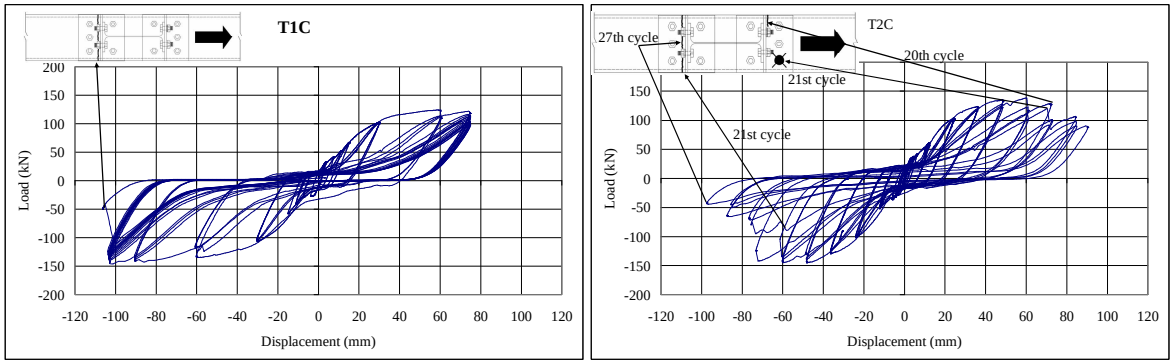

Fig. 5: Load vs displacement curve of the specimens T1C and T2C

Non-dimensional monotonic behaviour of the specimens with the boundary curves of Eurocode 3 [6] are given in *Fig. 6*. Although their stiffness levels were not very high, both specimens failed after reaching plastic moment capacity of the connected beam (142 kNm). Therefore the specimens T1M and T2M can be classified as semi-rigid-full-strength joint according to the joint classification of Eurocode 3.

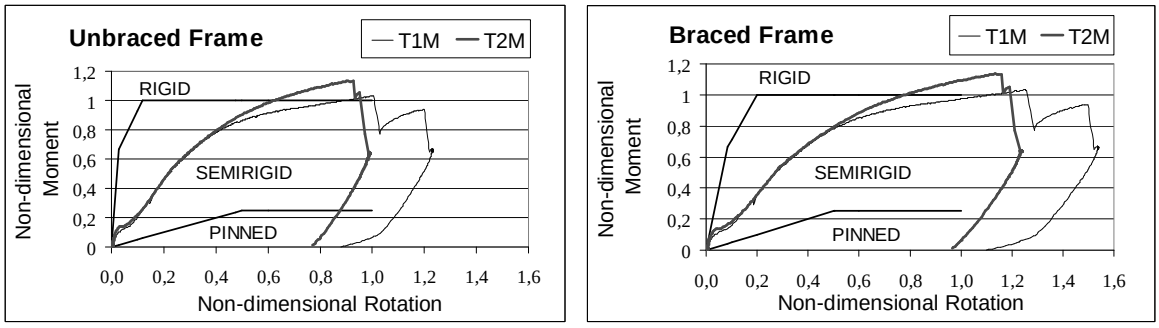

Fig. 6: Joint classification of the specimens according to Eurocode 3

Total dissipated energies through out the tests T1C and T2C were 107 and 90 kNm, respectively. Deformations at t-stub played a significant role in dissipation of the energy and stiffening effect of backing plate reduced SHS column face deformation levels. Highest level of energies was dissipated in the first cycles of the group cycles. (*Fig. 7*)

In order to compare the behaviour of tested joints with other types of joints, non-dimensional strength (the ratio between the maximum force applied at the end of the cantilever specimen during the test and the force leading to the yielding of the beam) and non-dimensional dissipated energy (the ratio between the total dissipated energy of the joint during the test and the energy leading to the yielding of the beam) parameters can be used.[7]. The cyclic and static performance of tested joints are quite satisfactory compared to other types of joints; stronger than bolted angle and bolted plate splice joints with having acceptable energy dissipation capacity.(*Fig. 8*)

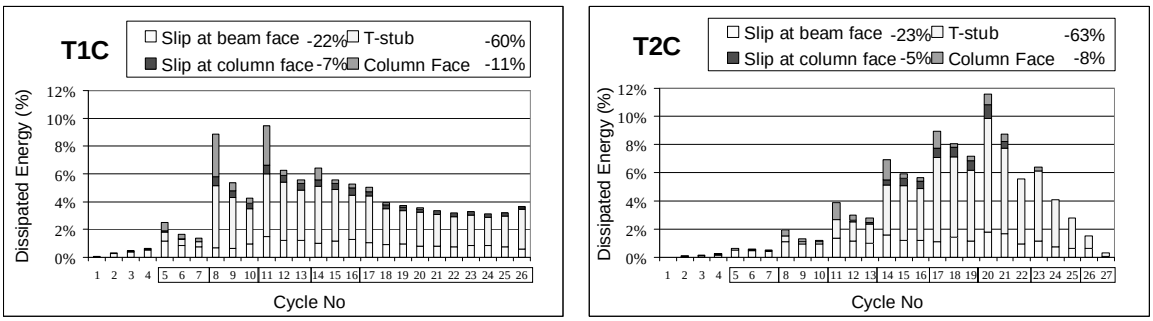

Fig. 7: Distribution of total dissipated energy of connection on connection components

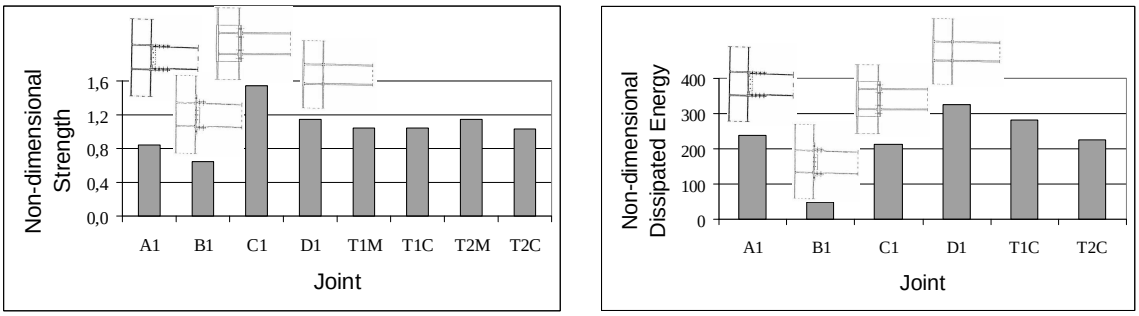

Fig. 8: Non-dimensional strength and dissipated energy of the joints

4. MODELING OF THE JOINT

In this type of I-beam to SHS column connections, the deformation of SHS column face plays an important role in overall performance of the joints. In order to observe the behaviour of SHS face in bending, further parametric experimental study has been done.[4]

4.1 PERFORMANCE OF SQUARE HOLLOW SECTION FACE IN BENDING

The SHS face has been pressed inside by six bolts in different spacing configurations. A gradually increased load has been applied on the bolts up to failure. (*Fig.9*)

All tests have been carried on until rigid plate touched the face of the hollow section. Until that point, the specimens behaved in ductile manner by providing high deformation capacity without major loss of strength (almost perfectly plastic). (*Fig. 11*)

4.2 MODELING OF SQUARE HOLLOW SECTION FACE IN BENDING

The model for square hollow section face in bending used here is based on the method described in Eurocode 3 Part 1-8 with proposed supplementary dimensional definitions. Considering the deformation similarities, equivalent t-stub is assumed as shown in *Fig. 13*. Since the corners are not as rigid as t-stub stem, it is more reasonable to take m as $b_1 - 0.6\ r$ (b_1 is the distance from bolt hole center to the sidewall and r is the external corner radius of square hollow section). The multiplier 0.6 might be related to $1\text{-}t_c/r$ or

*1.5*t_c, where t_c is the thickness of hollow section. These assumptions should be verified in further analytical or experimental studies.

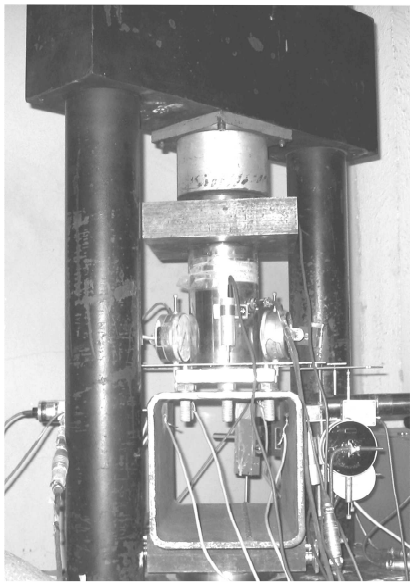

Fig. 9: Test set-up

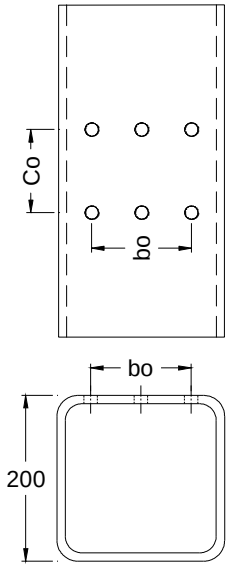

Fig. 10: Bolt locations

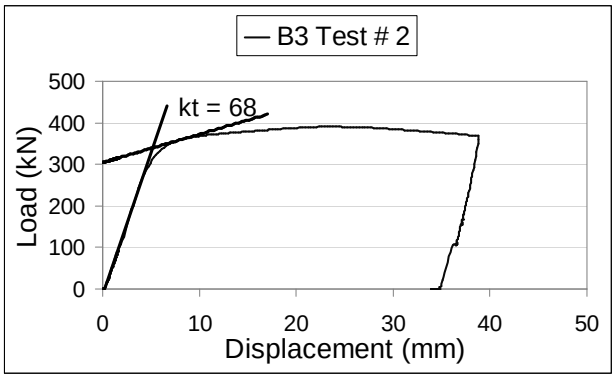

Fig. 11: Typical load vs displacement curve

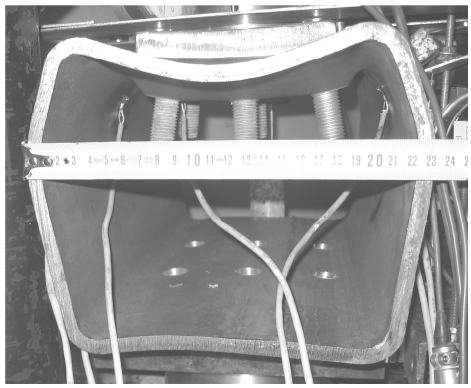

Fig. 12: Typical deformation pattern

Resistance of the square hollow section face in bending for the performed tests has been calculated based on single bolt-rows. The predictions according to Method 2 of design resistance calculation of a t-stub flange in Eurocode 3 Part 1-8 are very close to average test results in the level of 1-2 % of the yield load.(*Fig. 14*)

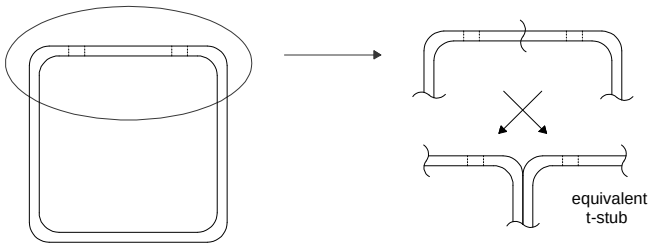

Fig. 13: Equivalent t-stub description of hollow section face in bending component

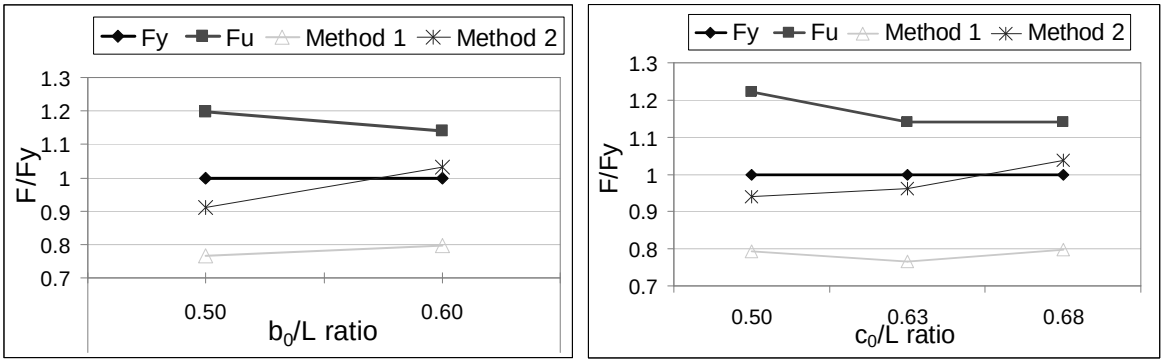

Fig. 14: Modeling results vs test results

4.3 COMPONENT MODELING OF THE TESTED SPECIMENS

Eurocode 3 covers most of the components mainly related to open sections (H or I profiles). However component properties of joints with hollow sections are still not yet covered. There is an ongoing analytical study proposing properties of the components related to hollow section connections. [3]

Component	**T1M**	**T2M**
	defined by	
SHS column web panel in shear	EC 3	EC 3
SHS column sidewall (web) in transverse compression	CIDECT	CIDECT
SHS in transverse compression - face failure in bending	EC 3+KY	EC 3+KY
Backing plate in bending	-	EC 3+KY
T-stub in bending	EC 3	EC 3
Bolts (partially threaded studs) in tension	EC 3	EC 3
Bolts in shear	EC 3	EC 3
Plate in bearing (plate in general, beam flange or web, column flange or face, end-plate, cleat or base plate)	EC 3	EC 3

Table 1: Related joint components

The weakest links and corresponding resistance values are given in *Table 2*. According to strain data of the tests, the yielding started in moment levels of which were in parallel with calculated elastic moment resistance (M_e) and plastic moment resistance (M_{Rd}).

Specimen	**Weakest component**	**M_e (kNm)**	**M_{Rd} (kNm)**	**Failure**	**M_{ult} (kNm)**	**M_{ult}/M_{max}**
T1M	SHS face in bending	67	100	Bolt failure with yielding of the face	124	0.84
T2M	SHS face in bending	76	114	Bolt failure with yielding of the face	124	0.77

Table 2: Calculated moment resistance of joints

5. CONCLUDING REMARKS

The studied joints achieved high plastic rotations without having a sudden loss of strength and reversed cyclic tests showed that the studied connections behaved in a manner suitable for use as either pinned or semi-rigid partial strength connections for simply designed braced steel frames in seismic areas or for unbraced steel frames in less seismic areas.

Reinforcing the HS column rear face by backing plate has provided an increase of strength and initial stiffness but a reduction of energy dissipation capacity of the joint. Energy dissipation capacities of the joints were acceptable and mainly governed by t-stub deformability. Column face only acted at first cycles of group cycles when specimens were loaded to the next displacement step. In order to improve cyclic performance, connection detail at the rear face has to be adapted so as to allow HS column face to participate more in energy dissipation by repeated inward and outward face deformations during cycles. Another improvement could be adding concrete inside HS column which would reduce column face deformation. This would improve the joint strength but not much the energy dissipation capacity, as again, energy would mainly be dissipated at the t-stub.

There is a relation in between bolt locations and the resistance of SHS face in bending component. The distance of edge bolts from sidewall certainly influences the resistance where the distance between bolt-rows does not influence the resistance primarily.

The resistance of SHS face in bending can be predicted according to the method described in Eurocode 3 with supplementary dimensional definitions. Considering the present study, proposed model gives good agreement with the test results in resistance point of view.

5. REFERENCES

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