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A steel frame connection: I-beam to SHS column via bolted T-stub

This paper presents the outcome of an experimental study carried out by the authors on the performance of a connection between an I-beam and a square hollow section (SHS) column. In the light of practical and economic bolted field applications and transportation without damage by leaving the column exterior without projections, an appropriate joint type composed of T-stub connecting elements was studied. These connecting elements were bolted with long partially threaded studs passing through the SHS column. In order to observe the performance of the joint, four full-scale beam-column tee-joint specimens were tested under monotonic and cyclic loading in two groups. As a parameter, the rear face of the SHS column in the area of the connection was reinforced by backing plates in the second group of specimens. The results obtained are provided in terms of moment-rotation relationship and energy dissipation capacity. All specimens reached beam plastic load level at high deformation levels. Reinforcing the SHS column rear face with a backing plate resulted in an increase in strength and initial stiffness but a reduction in the energy dissipation capacity of the joint. Considering the overall cyclic performances, both joints maintained high plastic rotations in adequate resistance levels with acceptable energy dissipation capacities; consequently, they are suitable for use as semi-rigid partial strength joints in simply designed, braced steel frames in seismic regions or in unbraced steel frames in regions with less seismic activity. Additional component tests were carried out to understand better the component behaviour of the hollow section face in bending and to try to design component modelling parameters. The influence of bolt spacing and the distance between bolt rows on the resistance of the square hollow section face in bending was examined. As expected, when the bolts were closer to the sidewall, the resistance of the face increased.

1 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 these details suffer from the disadvantage of requiring fittings to the column, which can be costly to fabricate and make the section more difficult to transport without damage. The ideal system is one that allows site bolting, leaves the column exterior without projections, is similar to traditional joints between beams and open column and uses ordinary bolts.

France et al. conducted series of tests to investigate the moment capacity and rotational stiffness of simple and moment connections bolted to tubular columns using flowdrill connectors [1], [2], [3]. Reversed cyclic tests showed that the connections 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. On the other hand, the disadvantage of these blind connections is that 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 prefer not to work with such close tolerance levels.

The research work of *Shih-Wei Peng* [4], together with *J. Ricles* et al. [5], also examined the cyclic performance of a joint between an I-beam and a concrete-filled tubular column with a split-tee bolted connection detail. Based on their comparison of specimen response with the AISC Seismic Provisions and FEMA recommendations, split-tee connection details 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 [6] were much focused on bolted connections in relation to semi-rigid connection research. Then, the subsequent research [7] gathered all the information available to the designer and helpful for the design of a wide range of structural steel joints connecting hollow and/or open sections. Simple design aids (called design sheets), which are more appropriate to daily practice, have been prepared for a number of selected joint configurations. These are complemented by worked examples. Most of this research was analytical and based on experimental studies under static loading conditions. Further complementary studies need to be carried out, also under cyclic conditions.

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

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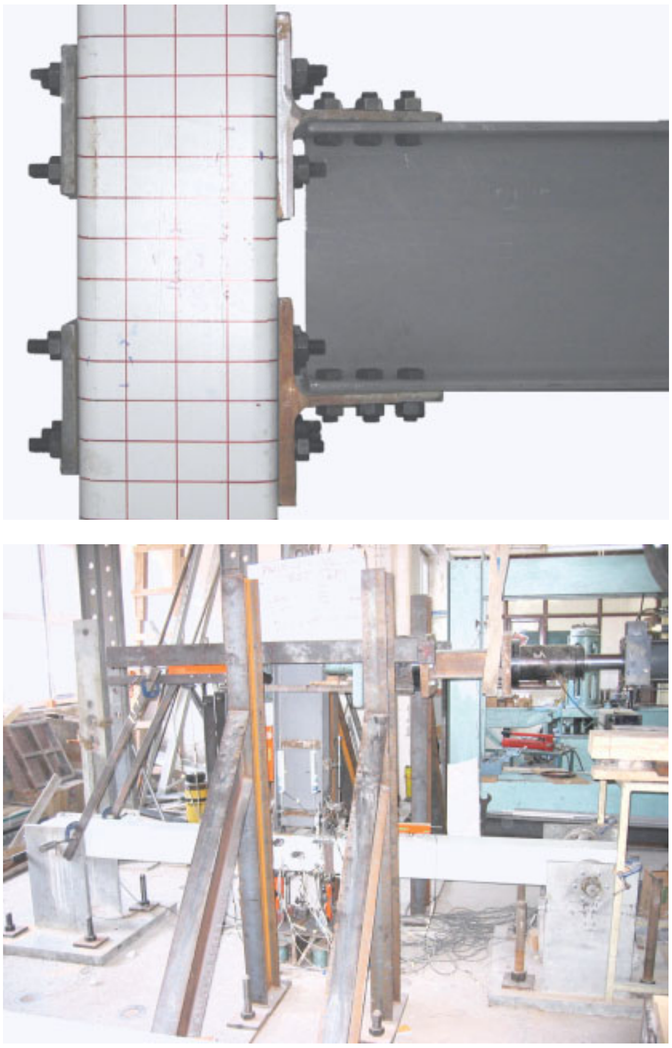

Fig. 1. *The connection studied and the test setup*

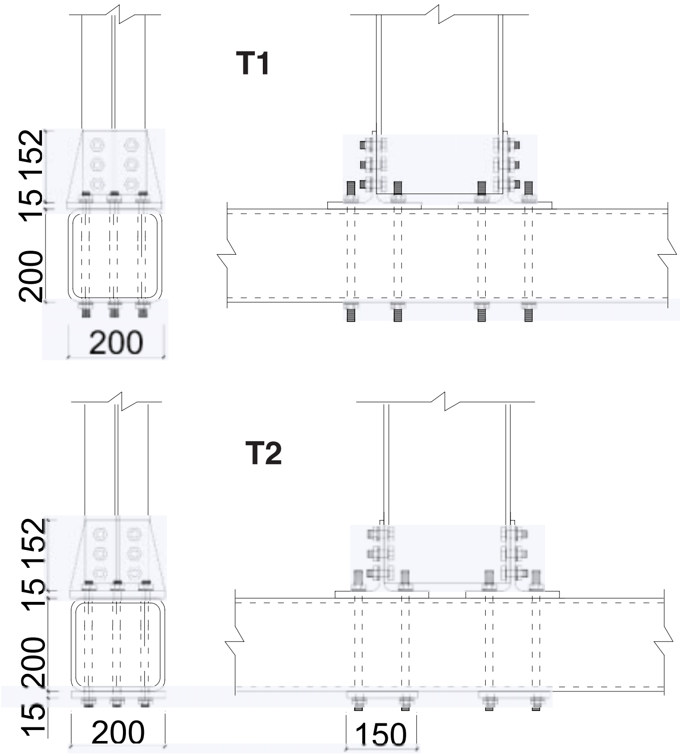

Fig. 2. Connection detail drawings of T1- and T2-type specimens

hollow section columns, as given in Fig. 1, have been studied [8] and their performance is presented in this paper. Subsequently, in order to determine the component modelling parameters, the behaviour of a square hollow section face subjected to inward bending was examined.

2 Summary of the experimental study

T-joints are representative of outer joints in a real framework. The height of the column was chosen so that it represents roughly the depth of one storey. Throughout the experiments, all necessary measurements for determining the amplitude of all the components of the joint deformability were recorded.

The T-joint test specimens consisted of an IPE 270 beam in grade S275JR joined to a 200 × 10 mm cold-formed square hollow section in grade S235JRH by means of bolted T-stub connecting elements fabricated from a split HEB 200 profile in grade S275JR. Each top and bottom T-stub was connected by five long, partially threaded 8.8 grade M16 studs in two rows passing through the column and six 8.8 grade M16 × 45 bolts to the beam flanges. The thread lengths of the bolts were chosen such that the shear plane passes through the unthreaded portion of the bolt.

The tests were carried out in two steps. Each specimen was tested in a monotonic way and in a reversed cyclic way according to the ECCS Recommended Testing Procedure [9]. The reinforcing effect of a backing plate on the rear face of the hollow section was observed.

Table 1. Specimen configuration

Specimen	T1M	T2M	T1C	T2C
Loading type	monotonic	monotonic	cyclic	cyclic
Backing plate	without	with	without	with

2.1 Evaluation of the test results

The failure of both monotonic specimens was due to the rupture of the threaded studs. On the other hand, in the cyclic tests, the specimens were loaded incrementally, which did not stress the studs severely and the tests ended with the failure of the T-stub progressively by low-cycle fatigue after an acceptable number of cycles.

Both specimens showed similar initial stiffness levels by comparing their monotonic and cyclic behaviours (Fig. 4). As expected, the ultimate moment and the corresponding rotation were lower in the case of cyclic actions due to low-cycle fatigue and the plastic excursions due to plastic reversals, from positive to negative range. Loss of strength due to cyclic actions was quite limited and only remarkable in the last cycles just before specimen failure. That is why both specimens demonstrated acceptable behaviour up until higher rotation levels, achieving 0.10 rad for T1C and 0.06 rad for T2C without a sudden loss in resistance or deterioration in the behaviour of the connection.

Considering the deformability of the joint, shear deformations were much less compared with the connection deformations governed by the deformations at the T-stub

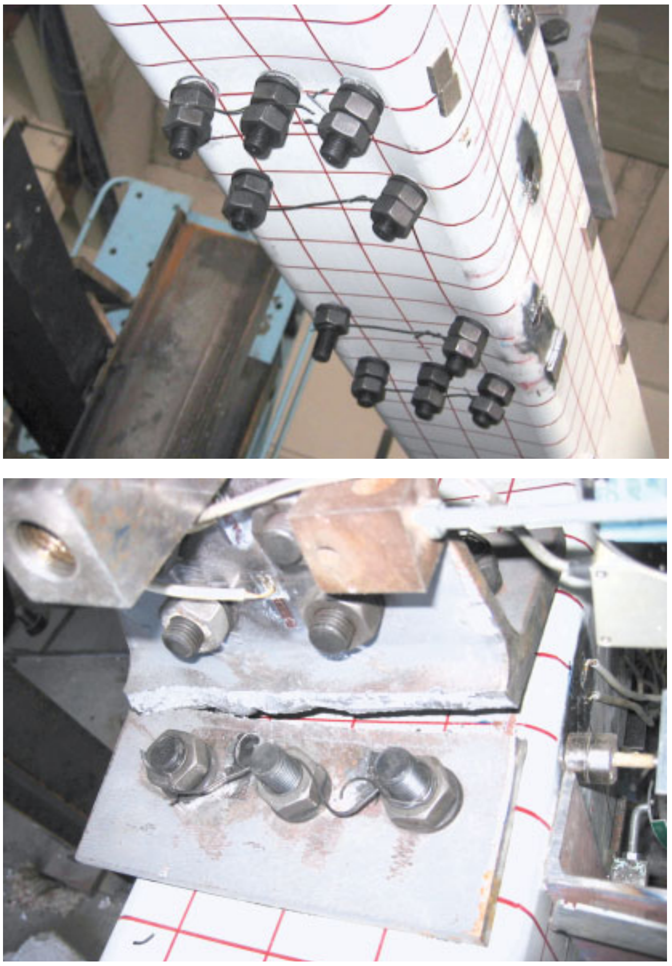

Fig. 3. Deformation of hollow section rear face and the complete failure of a T-stub stem

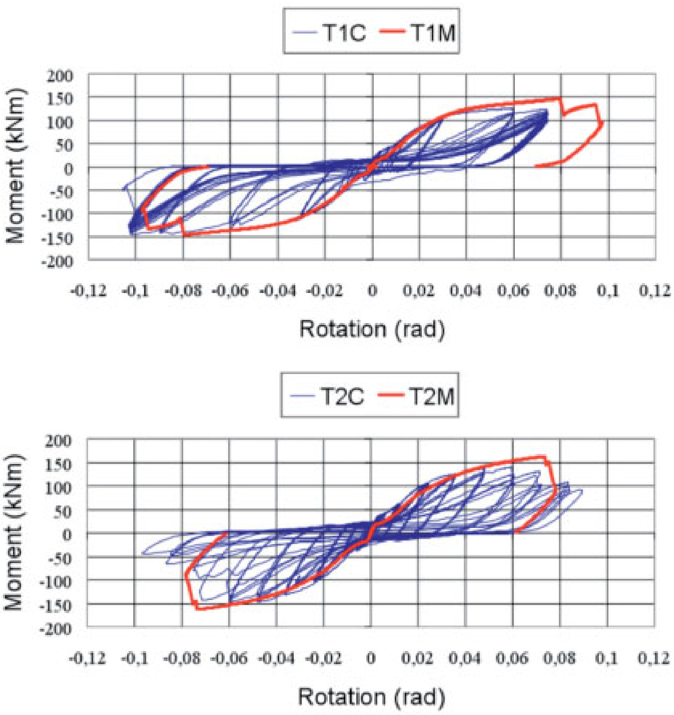

Fig. 4. Comparison of moment–rotation behaviour of monotonic and cyclic loaded specimens

(including the elongation of the partially threaded studs) and at the rear face of the hollow section column (including deformation of the column sidewall across its depth).

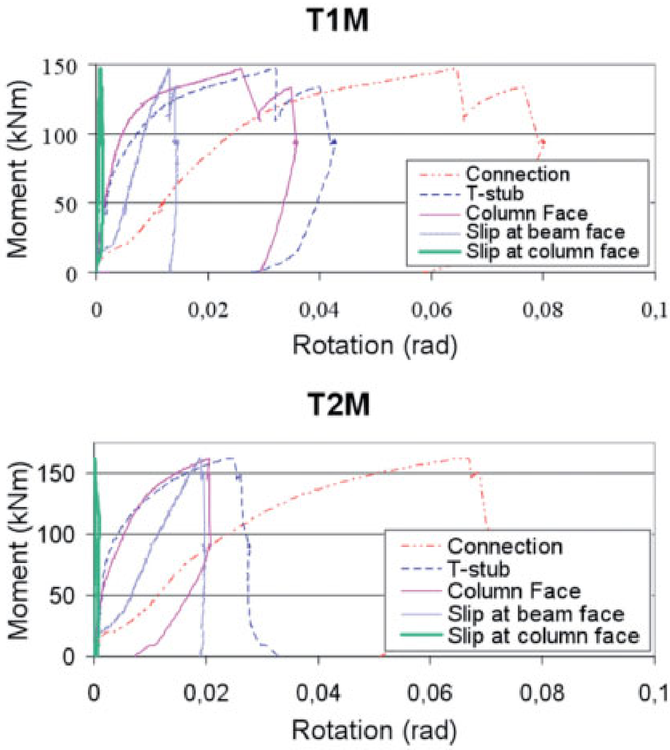

Fig. 5. Components of connection deformation for T1M and T2M specimens

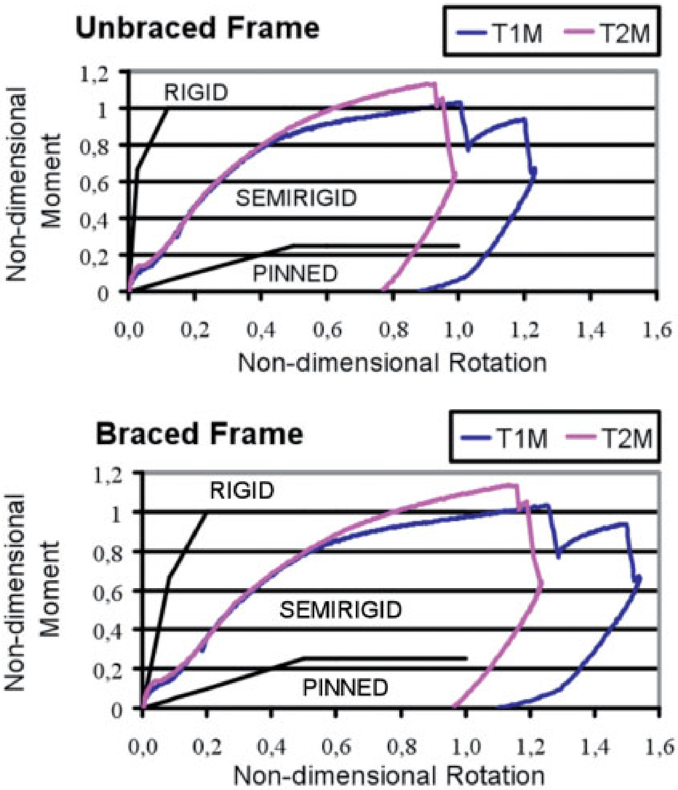

Fig. 6. Joint classification of the specimens according to Eurocode 3

At higher load levels, deformation of the T-stub was dominant. (Fig. 5). With T2M especially, the backing plate successfully reduced the deformation levels at the column face.

Non-dimensional monotonic behaviour of the specimens with the boundary curves of Eurocode 3 [10] are given in Fig. 6. Although their stiffness levels were not very high, both specimens failed after reaching the plastic moment capacity of the connected beam (142 kNm). Therefore, the T1M and T2M specimens can be classified as semi-rigid full-strength joints according to the joint classification of Eurocode 3.

It is not possible to say that both specimens exhibited a stable cyclic response and reliable energy dissipation capacity under repeated loading. Nevertheless, the strength levels of the specimens were as expected and the strength deterioration did not reach significant levels through an increase with the cycle deformation amplitude. On the other hand, there was remarkable stiffness degradation, especially after the rear column faces of the specimens started deforming due to the opening gap between T-stub with respect to the column face during the loading process. Due to its bolted nature, the hysteresis of the joint was influenced by bolt slippage at the T-stub/beam interface under cyclic loading. As a result, hysteretic pinching behaviour was observed.

Total dissipated energies throughout the tests T1C and T2C were 107 and 90 kNm respectively. Considering the joint components, the majority of the energy was dissipated by the connection. The energy dissipation due to deformation of the web shear panel was in the region of 6 % for T1C and 11 % for T2C. Distribution of the total dissipated energy on components of connection deformation is presented in Fig. 7. The highest level of energies was dissipated during the first cycles of the group cycles. It is obvious that deformations at the T-stub played a significant role in dissipating the energy during both tests. In test T2C especially, backing plates stiffened the column face and reduced its deformation levels, which reduced its contribution to energy dissipation.

Non-dimensional strength and non-dimensional dissipated energy parameters can be used to compare the behaviour of the joint with other types of joint [11]. Non-dimensional strength is defined as the ratio between the maximum force F_{max}, applied at the end of the cantilever specimen during the test, and the force F_p leading to the yielding of the beam ($F_p = M_{pb}/L_b$, where M_{pb} is the plastic moment of the beam computed on the basis of the actual value of the yield stress of the beam flanges and L_b is the length of the cantilever specimen). The non-dimensional dissipated energy is defined as

$$\bar{E} = \frac{E_{joint}}{F_p \nu_p}, \quad \nu_p = \frac{F_p L_b^3}{3EI_b} \tag{1}$$

where ν_p is the end displacement corresponding to F_p.

The non-dimensional strength and non-dimensional dissipated energy of the tested joints are quite satisfactory when compared with other types of joint; stronger than bolted angle and bolted plate splice joints, and acceptable energy dissipation capacity (Fig. 8).

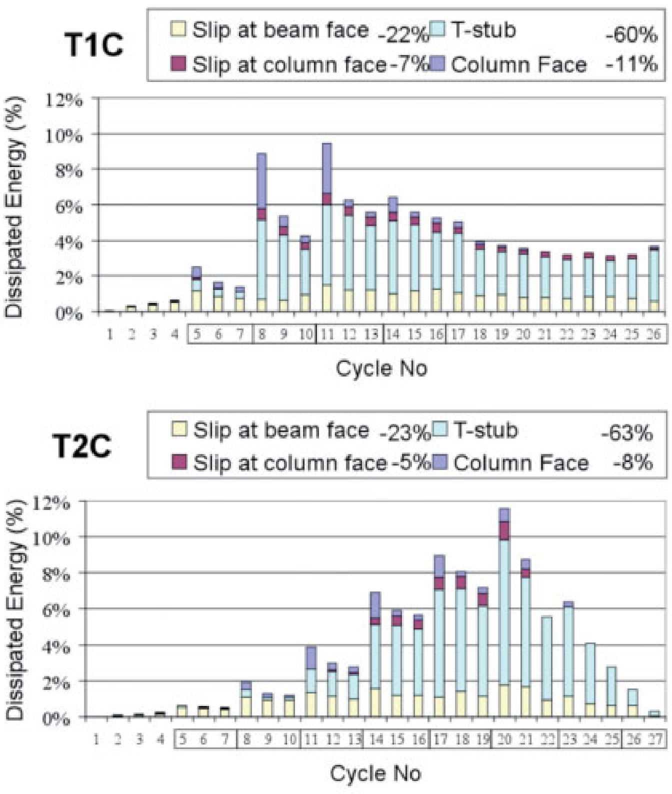

Fig. 7. Distribution of total dissipated energy of connection on connection components

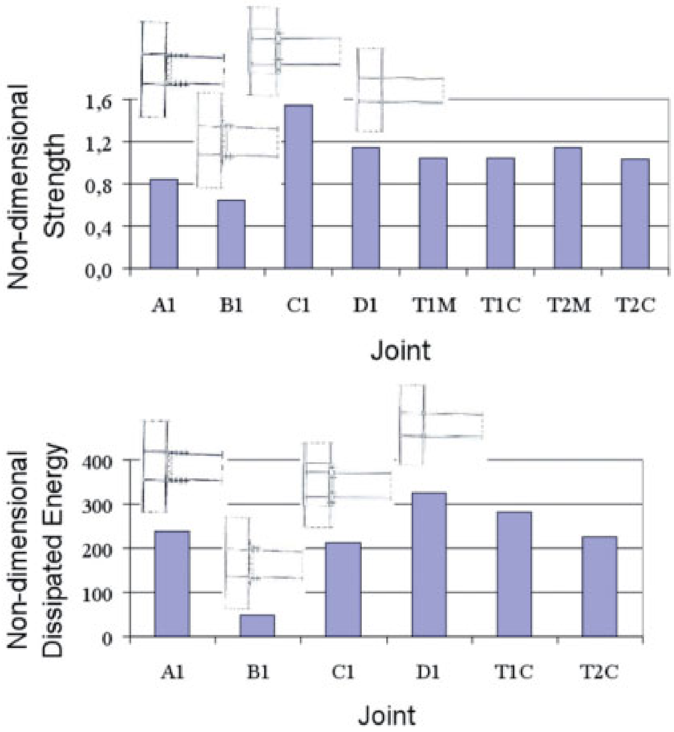

Fig. 8. Non-dimensional strength and dissipated energy of joints

3 Component modelling

Component tests were carried out to observe the component behaviour of the square hollow section face in bending. Six hexagonal bolts were pushed inside a square hollow section face which represents the column face.

A classical monotonic load increase test was carried out with a hydraulic press. In particular, initial stiffness, yielding push-through load capacity and ultimate push-through load capacity were evaluated. The influence of bolt spacing and the distance between the bolt rows on the stiffness and the resistance characteristic of the square hollow section face in bending were examined. A typical load v. displacement curve of tests and typical deformation pattern after tests are given Fig. 10.

When the bolts were closer to the sidewall, the stiffness and resistance of the face increased. On the other

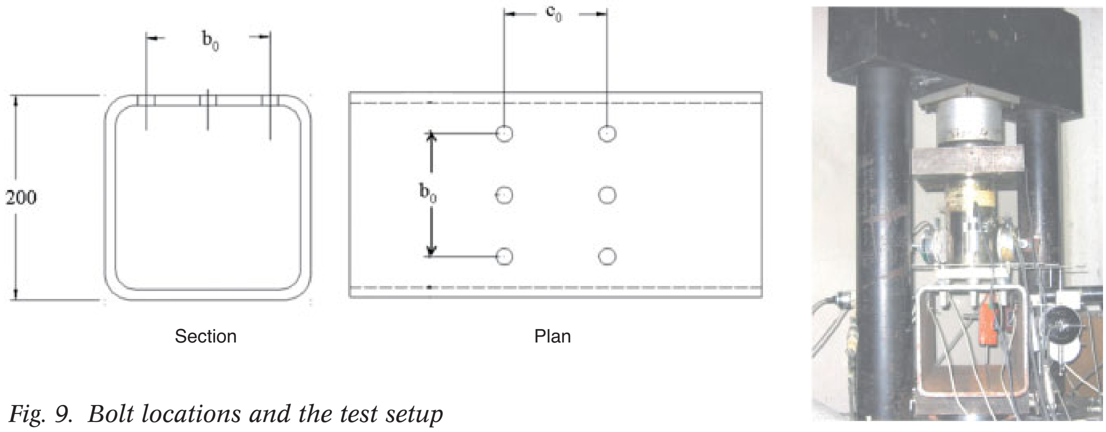

Fig. 9. Bolt locations and the test setup

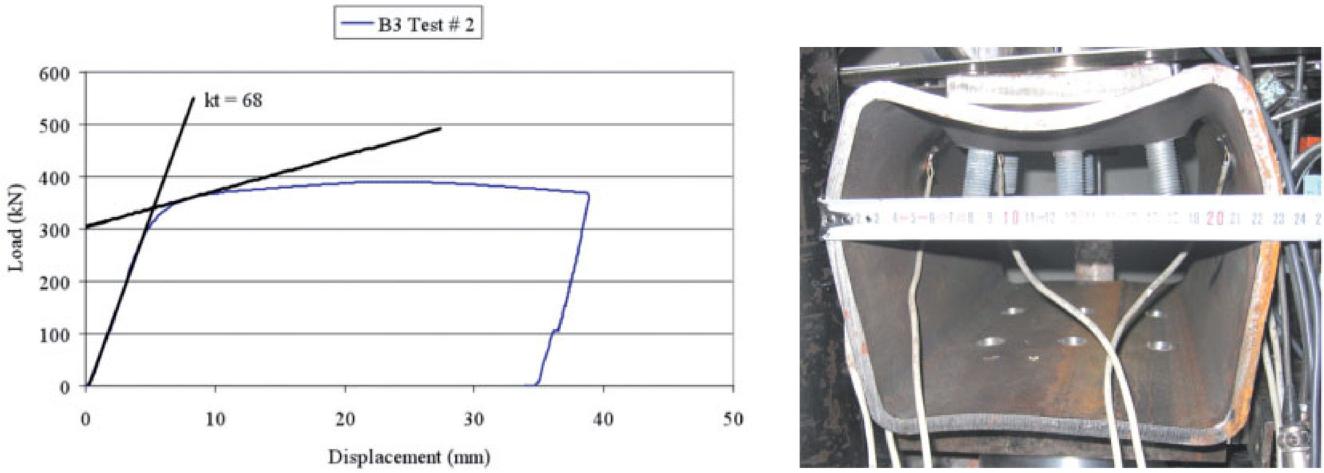

Fig. 10. Typical load v. displacement curve and deformation pattern

hand, increasing the bolt row spacing did not influence the resistance primarily within the scope of this study.

3.1 Resistance of hollow section column face in bending component

The model for square hollow section face in bending used here is based on the method described in Eurocode 3 with proposed supplementary dimensional definitions. Considering the deformation similarity, the equivalent T-stub is assumed to be as shown in Fig. 11. Since the corners are not as rigid as a T-stub stem, it is more reasonable to take m as $b_1 - 0.6$ r (where b_1 is the distance from bolt hole centre to sidewall and r is the external corner radius of the square hollow section). The multiplier 0.6 might be related to $1\text{-}t_c/r$ or $1.5t_c$, where t_c is the wall thickness of the hollow section. These assumptions should be verified in further analytical or experimental studies.

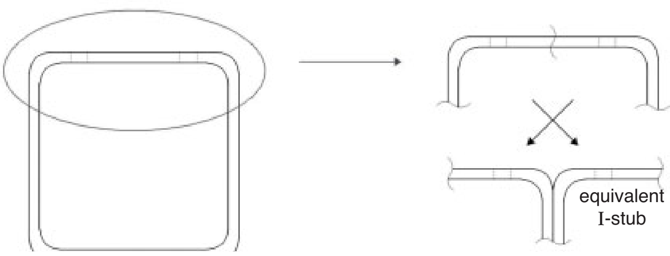

Fig. 11. Equivalent T-stub description of hollow section face in bending component

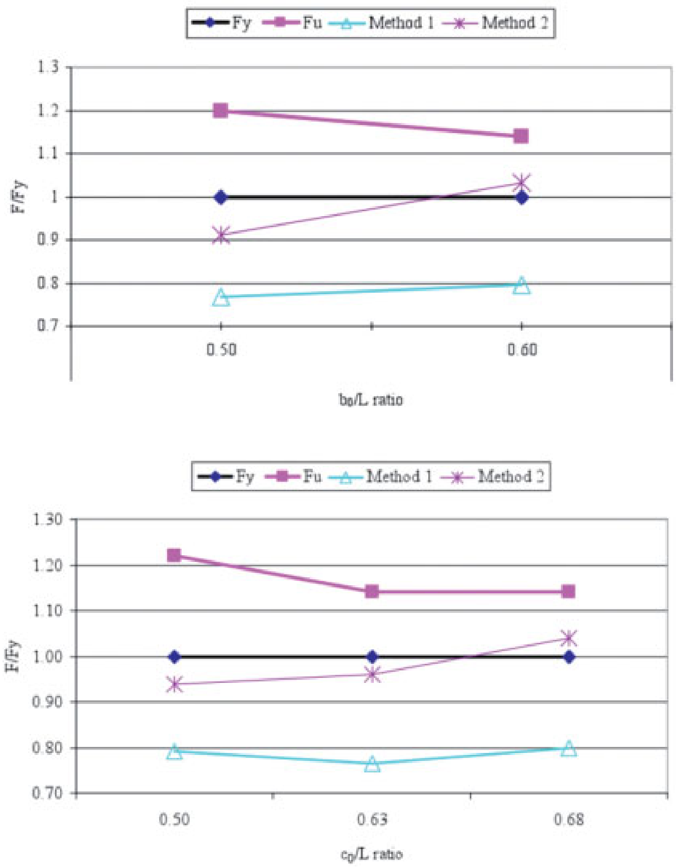

Fig. 12. Modelling results v. test results

The resistance of the square hollow section face in bending for the tests performed was calculated based on single bolt rows, and comparisons of the average modelling results with average test results in relation to b_0/L ratio and c_0/L ratio are given in Fig. 12. The predictions according to Method 2 are very close to average test results in the level of 1–2 % of the yield load. Method 1 estimates the yield load to be about 23 % lower on average. Taking into account the perfectly plastic behaviour of the specimens under this type of loading, all yield resistance predictions are below the load deflection curve of the tests.

As a result, the resistance of the square hollow section face in bending can be predicted according to the method described in Eurocode 3 with supplementary dimensional definitions. Taking into account the present study, the model proposed gives good agreement with the test results in terms of resistance. Further, it is applied to predict the moment resistance of the joints tested.

3.2 Joint moment resistance prediction of the tested specimens

After evaluating all joint components one by one, the weakest links appeared to be the hollow section face in bending with corresponding joint plastic moment resistance values of 100 and 113.6 kNm for the test specimens T1M and T2M respectively, and both values obviously remain below the moment–rotation curves obtained.

4 Conclusions

Considering the overall cyclic performances, both joints maintained high plastic rotations and acceptable energy dissipation capacities without experiencing a sudden loss in strength; consequently, they are suitable for use as semi-rigid partial strength joints in simply designed braced steel frames in seismic regions or in unbraced steel frames in regions with less seismic activity.

Reinforcing the hollow section column rear face by attaching a backing plate brought about an increase in strength and initial stiffness but a reduction in the energy dissipation capacity of the joint.

The energy dissipation capacity of the joints was mainly governed by deformability of the T-stub. The column face deformation was not recovered during the subsequent cycles. Therefore, energy dissipation due to column face deformability was only observed during the first cycles of the group cycles. Considering the cyclic actions, the desired behaviour should be such that the hollow section column face would participate more in energy dissipation through the deformation of the face both inwards and outwards during the cycles. Further studies could be carried out to improve this cyclic performance.

Increasing the wall thickness of the hollow section column or filling the column with concrete, which reduces column face deformation, can improve the strength of the joint but does not help the energy dissipation capacity very much because, again, energy would be dissipated mainly at the T-stub.

Mechanical modelling of the joints tested can be developed by using the component method described in Eurocode 3 part 1–8 with the proposed design approach. There was good agreement between the test results and the proposed models in terms of resistance.

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