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Article *in* Proceedings of the Romanian Academy - Series A: Mathematics, Physics, Technical Sciences, Information Science · September 2017

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Volume 18, Number 3/2017, pp. 256–264

EXPERIMENTAL INVESTIGATIONS OF COLD-FORMED JOINTS FOR MULTI-STOREY STEEL FRAMED STRUCTURES

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Abstract. This paper investigates the behaviour of back-to back cold formed C beams-to-SHS column joints under monotonic and cyclic loading. An experimental program was carried out in order to evaluate the structural performance of such joints in cold formed low rise multi-story steel framed structures. Two different configurations of joining a double C section beam with a square hollow section column have been tested experimentally. The first type consists of two flange diaphragms welded to the column which are fixed to the beam's flanges through 4 × M20 bolts each. The second joint configuration has T-stubs placed at the upper and lower flanges of the beam, each of them being fixed through 4 × M20 bolts by the column and 4 × M20 bolts by the beam's flange. The behaviour and failure mechanisms of joints have been observed experimentally, their stiffness, strength and ductility have been evaluated based on experimental studies. The test results provide essential data for the validation of detailed numerical and analytical studies which can be employed for further assessment of the response, with a view to the development of design-oriented procedures. Using the experimental results, finite element models were built and calibrated to evaluate mechanical responses of the analysed joints. Preliminary results shows good agreement between analytical and experimental results.

Key words: cold-formed joints, experimental investigations, hollo-bolts, diaphragm connection, finite element models.

1. INTRODUCTION

Cold-formed steel members are now widely used as construction materials worldwide, as they offer the following advantages: lightness, high strength and stiffness, ease of prefabrication and short erection time. The most common used cold-formed sections are C-sections, Z-sections and Σ-sections. For single story buildings test results are available for joints [1] and for full scale frames [2]. Generally, cold-formed profiles are used as secondary members but recently they have seen an increase in usage as structural frames for multi-storey buildings. Also hollow sections are very efficient structural solutions due to their geometric shape under compression and torsional loading. Because their closed shape, there are some issues that arise when establishing the structural connections. To be helpful in design practice, simple to apply and easy to use connection design procedures need to be available for design engineers.

The analysis results and design of a steel structure depends quite significantly on the behaviour of the connections. In the past, most designers considered beam-to-column connections either as pinned or as rigid. The global behaviour of cold-formed steel portal frames of bolted joints were studied experimentally by Lim [3], Dundu & Kemp [4], Kwon *et al*. [5] and Ahamed, Hazlan & Mahendran [6]. All these studies provided evidence of the crucial importance of joint performance on the global response of frames, which are semi-rigid and in almost all cases with partial strength [1, 3]. However, previous studies by Lim [7], Chung [8], [9, 10, 11] have shown that there are always rotational deformations in the beam-to-column connections. As cold-formed members are slender compared to hot-rolled members, the deformations are significant. As a result, bolted joints in cold formed steel portal frames have a semi-rigid behaviour, influencing the structural behaviour of cold-formed steel structures; these effects are amplified in multi-story framed structures. Also, these types of joints are partially resistant [1, 3, 8]. An important contribution to the global flexibility of the joints, besides the bearing effect (e.g. bolt hole elongation), is due to the deformation induced by the local buckling or distortion of the thin walled profiles. In an unwisely configured joint premature local buckling can cause the failure of the joint itself well below the expected load bearing capacity. It is therefore, important for the semi-rigid joints to develop sufficient rotation capacity for ductile mechanism to be formed before the failure of the joint. Having these considerations, a combination between SHS column and cold formed steel beam for a multi-story framed structure applied in seismic area could have the advantage of beam end “plastic mechanism” instead of “story mechanism” under seismic load combination – a preferred failure mode, combined with adequate bracing system developed in the framed structure, even the ductility of these connections is limited.

The welded flange diaphragm connections for tubular section columns attracted researchers’ interest as this type of connection do not require weld access holes. The weld access holes induce stress and strains concentration resulting in brittle failure. It was the case for numerous modern steel buildings that produced brittle fractures during the Northridge and Kobe Earthquakes in 1994, respectively 1995 as well during other earthquakes [12].

The beam-to-column connections without stiffeners have been rather avoided, as a way for determining the stiffness and strength of the column’s flange was missing. Zoetemeijer [13] introduced the bolted T-stubs connections, proposing a formula to determine the stiffness and strength of the connection, regarded as a fundamental approach for the idealization of the performance of the tension zone of such connections. More recent studies regarding T-stub connections [14] illustrate how the use of thinner steel plates and lower steel grades improve ductility, providing in the same time a comparison for the moment resistance, rotational stiffness and ductility.

Blind bolted T stub connections with hollow column section for low rise structures were studied by Lee *et al.*, [15]. They also found these connections semi-rigid, but channel side plate connection was found to be much stiffer than a typical face connection and has the potential to achieve a rigid connection status for braced frames according to the EC3 classifications [16].

Chung and Wong [9] tested 16 internal and external beam-column sub-frames with various connection configurations, aiming to predict the structural behaviour of bolted moment connections between cold-formed steel sections. The authors proposed a semi-empirical formula for the flexibility prediction of the bolted moment connections. Their research works encouraged engineers to build light-weight low to medium rise moment frames with cold-formed steel sections [10].

To promote the light-weight structures for medium rise applications, this paper propose and investigates two connection configurations of a double C cold-formed section beam with a square hollow section column using diaphragm type connection and T stub connection with blind bolts. The study focus on the effect of beam-to-column joints stiffness on global behaviour of the proposed structure. The proposed joint specimens were tested under both monotonic and cyclic loading, in order to observe the effects of the load introduction type on the response parameters, focusing on the stiffness, strength, failure mechanisms and ductility. The same failure mode occurred for all of the 4 experiments. The results reported below aim to characterize the capacity of such a typology of framed structure under specific design load combination.

2. TESTING PROGRAM AND SPECIMENS DETAILS

2.1. Joint specimens design

To define a realistic frame configuration and study the applicability of cold formed steel sections in a multi-story framed structure, a four story reference building was firstly designed (see Fig. 1). The building consists of three spans of 5.6 m, three bays of 4.5 m and a story height of 3.3 m. SHS200 × 8 sections resulted as being the most suitable for the columns whereas for the frame beams double C300 × 3,0 section were chosen to be adequate for structures having 2, 3 and 4 stories. The secondary beams, C200 × 2,0 were placed at 600 mm distance. This structure was subjected to loads common in the Romanian design practice: self-weight of the cold formed steel structure, weight of a dry floor 0.75 kN/m^2 (*P* with a partial safety factor of 1.35 for the ultimate limit state), technological load 0.15 kN/m^2 (γ_{ULS} = 1.35), live loads 3.0 kN/m^2 (*Q* with a partial safety factor of γ_{ULS} = 1.5 for the ultimate limit state), snow load 2.0 kN/m^2 (*S* with γ_{ULS} = 1.5), wind load for a region with the wind pressure of 0.6 kPa (W with γ_{ULS} = 1.5) and earthquake loading (E) for a Romanian region with the control period T_c = 0.7s and ground acceleration a_g = 0.10g. The structure was analysed and designed according to EN 1993-1-1, EN 1993-1-3 and EN 1993-1-8. Upon the analysis, the central part of the structure resulted as being the most sensitive, if braces are provided only on external perimeter. Furthermore, two additional structures were analysed, having the same configuration and being subjected to the same loadings but with two and three stories respectively. The analysis showed that four stories configuration is recommended to have an internal bracing system or staircases area should be combined with shear walls which should contribute to increase the horizontal stiffness of the global structure, especially when the building is placed in seismic areas where peak ground acceleration can achieve 0.20 g values.

As it is well known, such structural analysis had to be taken into account the flexibility of the beam-to-column joint that influences the whole structural behaviour. The problem was to determine the semi-rigid character and to model it in the structural analysis in order to obtain more realistic results. Fig. 2 presents the structural model that includes the semi-rigid behaviour of joints.

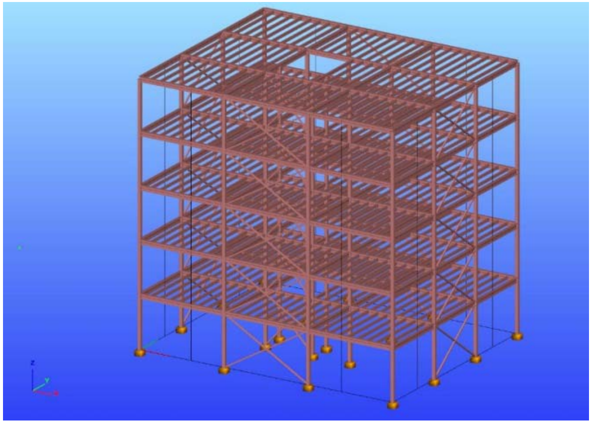

Fig. 1 – The reference structure – structural model.

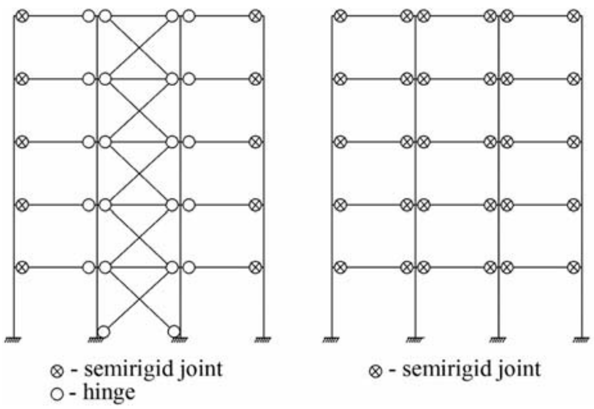

Fig. 2 – Modeling frames with and without bracings.

The estimated weight of steel in such a structure, located in Central Europe, is around 35 to 40 kg/m^2, and the necessary time for erection, after casting the foundations can be from 7 up to 10 days, depending by the grade of prefabrication. The mechanical performance of the proposed structure directly depends on the joints' behaviour. So, finding out a simplified method to control the design, the joint stiffness for the structural analysis is essential.

2.2. Experimental set-up for joint tests

The arrangement used for testing the connections is shown in Fig. 3. A hydraulic actuator was used to apply the horizontal displacement at the top of the cantilever. The cantilever was laterally supported at the level of load introduction point, to prevent out of plane movement and stability loss of the joint specimens (the force application keep the specimen vertical plane). The SHS column was restrained at both ends with pinned supports.

Extensive instrumentation layout was designed in order to assess the parameters used for interpreting the results. To develop equal criteria for the results comparison, the specimens were instrumented in the same manner. The actuator loads were recorded and monitored during the test. For the connections using welded flange diaphragm (hereinafter DCBC) the measured parameters are: slip and uplift of both supports; column web deformation; bolts slippage; gusset-beam displacement; top displacement. For the connections using T-stubs fixed with blind bolts (called hollo bolts-hereinafter HBBC – see Fig. 4) the measured parameters are: slip and uplift of both supports; gusset-column displacement; bolts slippage; gusset-beam displacement; top displacement. The joints instrumentation is presented in Figs. 5, 6.

The top displacement was measured using Celesco position transducer while TRS type displacement sensors were used for all other parameter measurements.

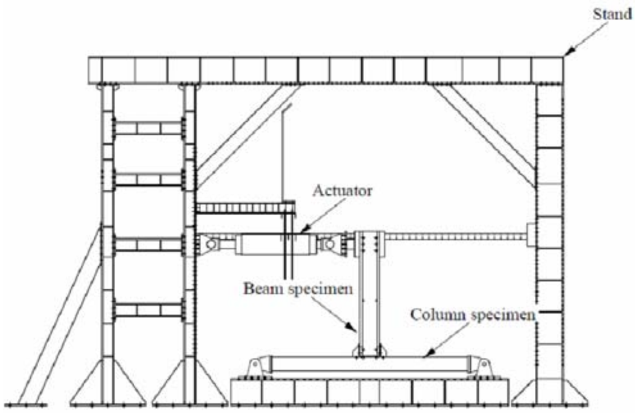

Fig. 3 – Joint test arrangement.

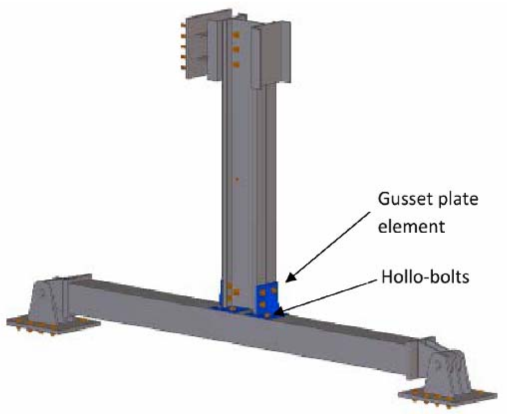

Fig. 4 – Hollo bolt (HBBC) connection.

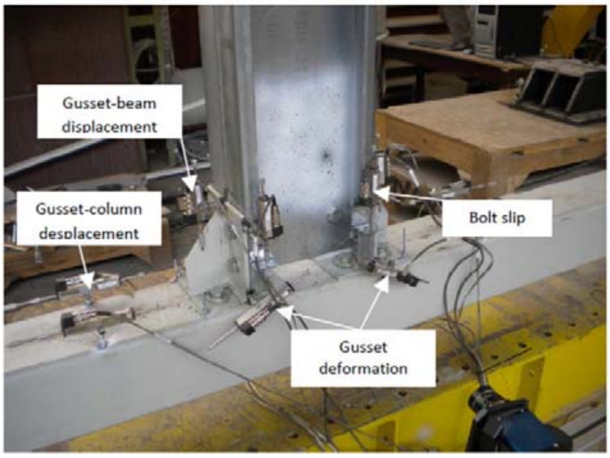

Fig. 5 – HBBC specimen instrumentation.

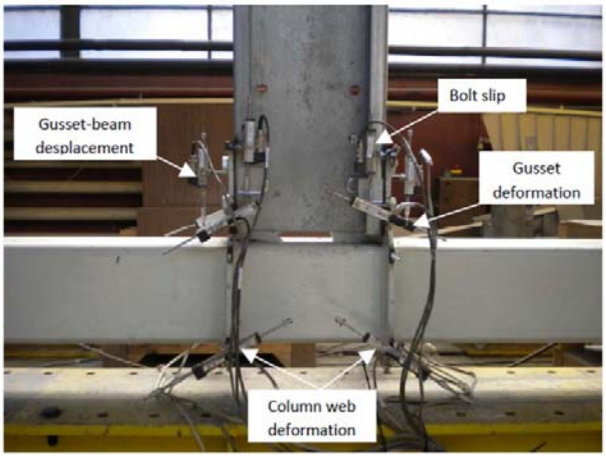

Fig. 6 – DCBC specimen instrumentation.

2.3. Tested joint specimens

The size of the beam and column specimens, and the testing setup were chosen to obtain in the connected members a similar bending moment as observed in the structure.

As the performed design of the structure shown, the beams of the multi-story frame resulted back-to-back built up sections C300/3.0 and the columns SHS200/12.5 (adequate for building structures up to 5 stories and cover the section loss in case of blind bolts). The steel grade used for the beams was S350 GD+Z275 and for the columns S355J0. In accordance to these cross sectional dimensions two alternative joint configurations were proposed: one using T-stubs fixed with hollo bolts M20 grade 10.9 (HBBC) and the other one using welded flange diaphragm (DCBC). The design of the connection was made with FEM assistance, using the developed methodology and calibrated models from the previously analysed joints [17, 18].

Totally, four connections were tested: two of each type subjected to monotonic and cyclic loading. The summary of the tested specimens are presented in Table 1.

Table 1

Connection details

Element type	**Code**	**Loading Type**	**No. of Specimens**
T joint with hollo bolt	HBBC-M	Monotonic	1
T joint with hollo bolt	HBBC-C	Cyclic	1
Welded flange diaphragm	DCBC-M	Monotonic	1
Welded flange diaphragm	DCBC-C	Cyclic	1

DCBC – **D**iaphragm **C**onnection for **B**eam to **C**olumn joints
HBBC – **H**ollo **B**olts for **B**eam to **C**olumn joints

The T-stub connection, (Fig. 4) using hollo bolts is composed of two 10mm T-stubs (S235) placed at the upper and lower flanges of the C beam. Each of them are fixed through 4 × M20 bolts on the column and 4 × M20 bolts on the beam's flange. The T-stubs have 8 mm stiffeners. The welded flange diaphragm connection consists of two 10 mm plates (S235) surrounding the column, which are fixed to the beam's flanges through 4 × M20 bolts each. The flange diaphragm is welded to the column with 5 mm fillet weld on each side.

3. EXPERIMENTAL RESULTS

3.1. HBBC and DCBC specimen monotonic test

The monotonic tests have identified the failure modes of the different tested joint configurations. The stresses concentrate in the vicinity of the first bolt on the flange. The elongation of the flange bolt holes was observed; the buckling was firstly initiated in the flange, and then was extended into the web. The failure produced similarly for both specimens as in case of previous joint tests [1]. Load-displacement curves were plotted for both tested joints.

Comparative experimental curves for DCBC and HBBC joints are presented in Figs. 11, 12. There are no significant differences between the tested specimens. The reason for similar behaviour is the higher connection stiffness and capacity compared to the other components of the joint. Because there is important gain in terms of load bearing capacity when bolts are provided on the flanges, this solution was adopted and studied.

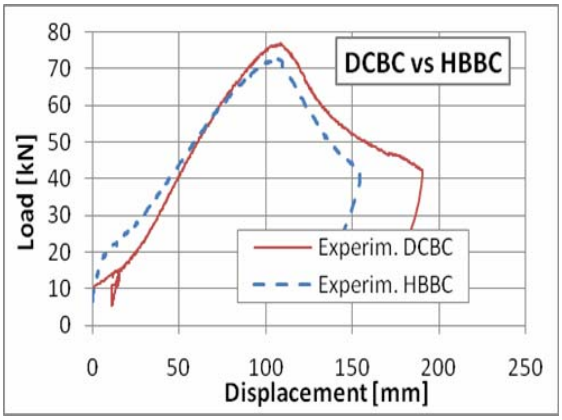

Fig. 11 – Load-displacement curves for tested specimens.

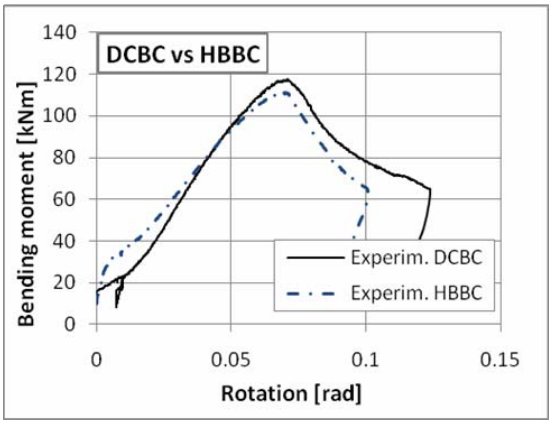

Fig. 12 – Moment-rotation curves for tested specimens.

Based on instrumentation in Figs. 5 and 6, the yield displacement (v_y), and corresponding force (P_y), were determined for both monotonically tested specimens. These values are summarized together with the values of maximum force (P_{max}), and the corresponding displacement (v_{max}) in Table 2.

Table 2

Monotonic test results: joint parameters derived from load-displacement curves

Specimen	$P_{e,\,exp}$ [kN]	v_e [mm]	P_{max} [kN]	v_{max} [mm]	v_u [mm]	$P_{Rd,\,th}$ [kN]	$P_{e,exp}$ / $P_{Rd,th}$
DCBC-M	75,3	90,8	76,8	108,6	111.2	68,58	1,10
HBBC-M	70,2	79,9	72,6	107,4	109,7	68,58	1,02

Table 3

Monotonic test results: joint parameters derived from moment-rotation curves

Specimen	$S_{j,\,ini}$ [kNm/rad]	θ_y [mrad]	θ_{ult} [mrad]	μ [θ_{ult}/θ_y]	M_{max} [kNm/rad]	$\theta_{M,max}$ [mrad]
DCBC-M	2107	54,23	68,60	1,27	117,60	62,88
HBBC-M	1838	47,56	58,54	1,23	111,19	55,37

The load bearing capacity of the specimens with diaphragm connection resulted a little bit higher, but no important differences between the tested specimens. The capacity of the joint did not reached the shear capacity of the connected *C* profiles. It can also be observed that all specimens have limited ductility. In Table 2, $P_{\mathrm{Rd,th}}$ represents the calculated capacity of the cold formed beam, while maximum bending moment achieved, using the calculated effective cross section and measured characteristics of the cold-formed steel ($f_y = 507\mathrm{N/mm}^2$, $f_u = 577\mathrm{N/mm}^2$). Moment-rotation relationships characterizing the joint response were derived for both analysed joints. Bending moments were computed at the intersection of center line of the profiles and borderline between cold formed C profiles and joining elements (diaphragm plate end or T stub). The corresponding relative rotation between the joining element and the connected double C sections was determined from acquired data, so as to represent both flexibility of the connection (due to bolt bearing) and post-buckling deformations in the element. For the simplified joint representation (as in Fig. 2), both moments and rotations were considered at the intersection of the element center lines.

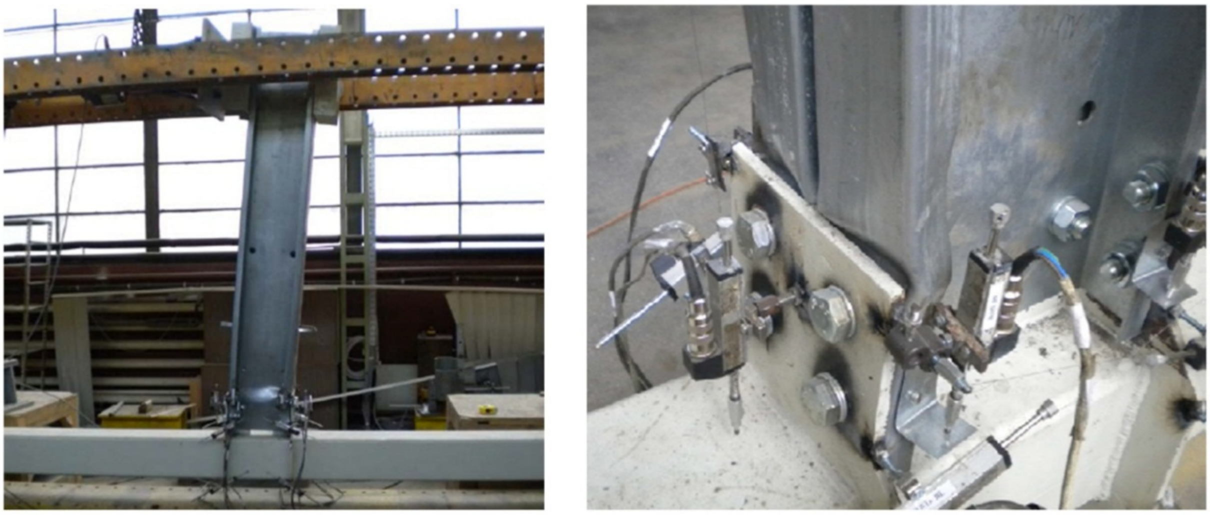

Fig. 13 – DCBC connection – monotonic test.

Fig. 14 – HBBC connection – monotonic test.

In Table 3 the yield and ultimate rotation capacity ($\Theta_y;\Theta_u$), the initial stiffness ($S_{\mathrm{j,ini}}$), and the maximum bending moment ($M_{\max}$) are presented for all monotonically tested specimens. All strength values were calculated using measured characteristics of the cold-formed steel ($f_y = 507\mathrm{N/mm}^2$, $f_u = 577\mathrm{N/mm}^2$). Both specimens have limited ductility. Table 3 associated with Fig. 12 shows the characteristic values of moment rotation curves for the tested specimens (HBBC and DCBC).

3.2. HBBC and DCBC specimen cyclic test

In case of the cyclic loading, the degradation of the specimens initiated with elongations in the bolt holes caused by bearing. For the cyclically tested specimens a modified ECCS procedure was used [1]. Compared to monotonic loading, in this case the phenomenon was amplified due to the repeated and reversal loading. However, the failure occurred also by local buckling, as in case of monotonic tests, but at the repeated reversals, the buckling occurred alternatively on both sides. There was no important slip or uplift of the supports – maximum measured value was 0.5 mm, the column's web did not suffer any local deformations; the gusset plates had small deformations – up to 2 mm. The bolt slip was 8mm. The experimental results in terms of load-displacement and moment-rotation curves are presented in Figs. 15–18.

Table 4

Cyclic test results: joint parameters derived from moment-rotation curves

Specimen	$S_{j,\,ini}$ [kNm/rad]	θ_y [mrad]	θ_{ult} [mrad]	μ [θ_{ult}/θ_y]	M_{max} [kNm/rad]	$\theta_{M,max}$ [mrad]
DCBC-C	1963	56.17	70.07	1,25	112.15	65.92
	-2045	-44.46	-67.27	1.51	-101.44	-60.71
HBBC-C	2038	52.75	75.67	1.43	100.52	73.82
	-1708	-62.92	-77.62	1.23	-94.24	-73.4

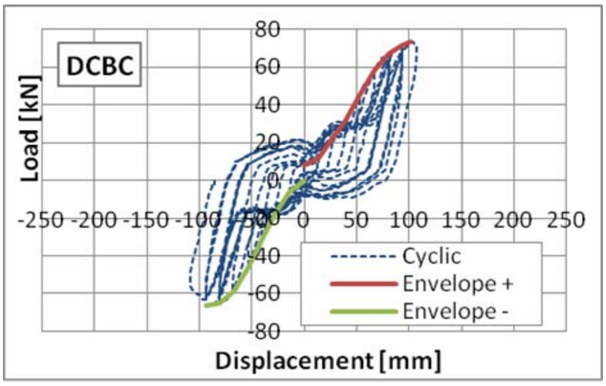

Fig. 15 – Load-displacement curves for DCBC specimens.

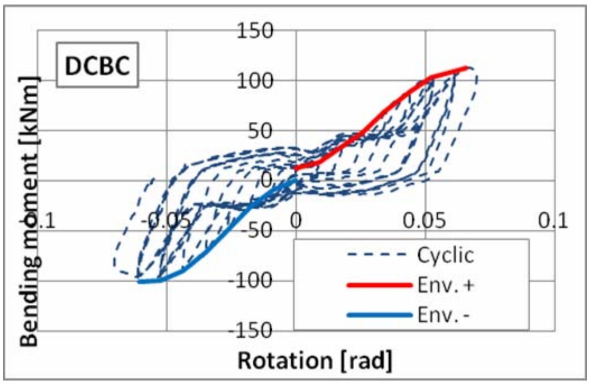

Fig. 16 – Moment-rotation curves for DCBC specimens.

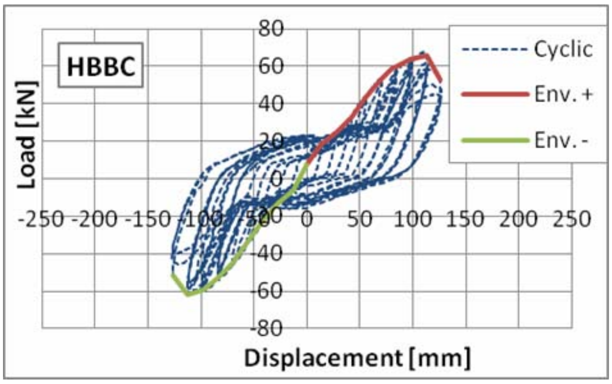

Fig. 17 – Load-displacement curves for HBBC specimens.

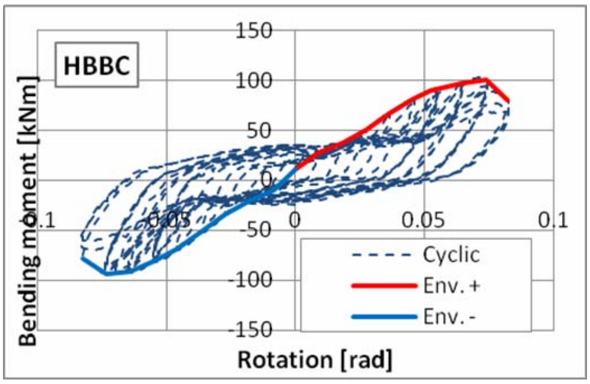

Fig. 18 – Moment-rotation curves for HBBC specimens.

4. CONCLUSIONS

Four specimen of bolted beam to column moment connections were tested to study their behaviour in terms of strength, stiffness and rotation capacity under monotonic and cyclic loading. According to EN 1993-1–8 joint classification, both type of joint was found with full strength and semi-rigid behaviour. In terms of rotation capacity both tested joint typology have higher rotation capacity than the required 35 mrad for high ductility steel structures in seismic areas, prescribed in EN 1998. The failure modes were in all cases due to flange compression by bolt hole bearing and web crippling.

The analysed joints in a typical framed structural application would produce excessive rotation. For elastic analysis the rotation capacity is outside of the recommended rotational capacity of 20 to 30 mrad. Excessive rotation (associated with excessive structural deformation) is a common problem when considering connection between cold-formed sections. Due to this reason, for such a structure is recommended to always use braced structural configuration.

The experimental results were doubled with FEM simulations using Abaqus software package. Preliminary results shows good agreement with test results. The calibrated numerical results will provide a better understanding of the presented joints behaviours, as more details are captured, improving and extending the experimental results. Based on experimental and numerical data further research undergoes to develop an analytical design approach of the considered joints.

AKNOWLEDGEMENTS

The research was supported by the project "Development and support of multidisciplinary postdoctoral programmes in major technical areas of national strategy of Research – Development – Innovation" 4D-POSTDOC, contract no. POSDRU/89/1.5/S/52603, project co-funded by the European Social Fund through Sectoral Operational Programme Human Resources Development 2007–2013.

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Received October 19, 2015