Passing-through I-Beam-to-CHS Column Joints made by Laser Cutting Technology: Experimental tests and design model

Maël Couchaux[a], Vojtech Vyhlas[a,c], Alper Kanyilmaz[b], Mohammed Hjiaj[a]

[a] *Institut National des Sciences Appliquées de Rennes, 20 Avenue des Buttes de Coësmes, 35708, Rennes, France*
[b] *Department of Architecture, Built Environment and Construction Engineering, Politecnico di Milano, Italy*
[c] *Department of steel and timber structures, Czech Technical University in Prague, Thákurova 7, 166 29 Prague, Czech Republic*

Abstract

Connections of I-beam to CHS column in buildings are either simple (fin plate connection, directly welded beam connections) and flexible or rather complex and rather rigid (external diaphragm connections). The passing-through I-beam-to-CHS column connections made by laser cutting technology offer a solution that limit the amount of welding and enhance the stiffness of the joint. An experimental campaign has been carried out to investigate the behaviour of these type of moment resisting connections under monotonic equal bending moment caused by gravity loads. These experimental tests demonstrate that the uninterrupted continuity of the I-beam produces rigid-full strength connections. In addition, the same configuration has been tested under opposite bending moments applied either monotonically or in a cyclic manner to characterise the connection behaviour under seismic loading. The failure modes are quite different compared to those observed under equal bending moment and involve substantial force redistribution between the different joint components after cracks propagation has occurred. These tests highlighted the impact of the tube-wall thickness and the welding technique on the connection response. An analytical model is proposed to evaluate the bending resistance and the initial rotational stiffness considering the interaction between the tube and the segment of the beam section inside the tube. This beam is assumed to rest on two supports and restrained at the support by the tube. The design model predictions are in good agreement with experimental tests and numerical simulations.

Keywords: *tubular construction, joint fabrication, laser cutting technology, circular hollow sections, passing-though connection, bending resistance*

1. Introduction

Circular Hollow Section (CHS) are now widely used in construction such as truss girders, buildings, bridges, stadiums, car parks… due to their optimal mechanical properties and aesthetics interest for architects. In buildings, the combination of CHS for columns and I profile for beams is an attractive solution that has been increasingly adopted during the last decades. The CHS columns are generally filled with concrete in order to ensure the fire resistance and the I-profile is connected to a concrete slab. Depending on the bracing system adopted for the frame, two options exist for the connection between the I-profile and the CHS column. e. For non-sway frames with lateral bracings, simple connections can be used that are generally composed of a fin plate directly welded to the CHS column and bolted on the beam web (see Figure 1-a). The design method for these connections can be found in existing design guides ([1], [2]) and have been introduced in the draft of Eurocode 3 part 1-8 [3]. For sway frames, the design and fabrication of I-beam to CHS-column moment resisting connections is more involved. A direct welding of an I-beam to the CHS-column (see Figure 1-b) without any stiffener or diaphragm produces a connection with low bending resistance and rotational stiffness [4]. This design is not suitable for portal frames that sustain large loads.

a- Fin plate connection *b- Directly welded beam* *c- External diaphragm*

Figure 1 : *Classical connections of I-beam to CHS-columns*

Togo [5] proposed an analytical model to determine the axial resistance of CHS to branch plate connections based on a ring model. This model was extended by de Winkel [6] and Wardenier [7] to other configurations such as I beams to CHS, multiplanar joints.

To strengthen these beam-to-column joints, external diaphragm are generally welded to the CHS (see Figure 1-c) increasing the amount of welds and stiffeners. This type of connections have been intensively studied in Japan ([10], [11]). The CIDECT design guide 9 [12] provides rules based on these researches to calculate their resistance. During experimental tests, it was observed that failure was triggered by cracking at the re-entrant corners of the diaphragm. Schneider and Alostaz ([13], [14]) performed cyclic tests on various typologies of CHS to I-beam connections and concluded that the passing through I-beam connection is an interesting solution able to develop substantial amount of strain hardening. They also recommended to avoid direct I-beam to CHS-column connection in seismic area as they are sensitive to weld fracture and develop large distortion of the column panel. In the same way, the addition of external diaphragm did not solve the problem of the column panel distortion. The use of through-plate-to-hollow sections favour the increase of the resistance of gusset plates as the resistance of branch plate T connections is limited. Voth and Packer ([15], [16], [17]) showed that the axial resistance of through-plate-to CHS connection (see Figure 2-a) can be estimated by adding the resistance of branch plate-to-CHS in tension with that in compression. The draft of EN 1993-1-8 [3] also partially includes these results.

a- Through plate-to-CHS connection *b- Through I-beam-to-CHS connection*

Figure 2 : *Passing Through plate and beam to CHS connections*

Hoang et al [18] developed design rules for passing through plates to CHS loaded by axial force and bending moment. The experimental tests and numerical studies have shown that buckling of the passing through plate occurs. The passing through I-beam to CHS column (see Figure 2-b) is also an interesting solution that can potentially increase the bending moment resistance of the joint.

Elremaily and Azizinamini [19] performed tests on passing-through-I-beams to CFT column connections filled with concrete. The designed connections were able to develop the full bending moment resistance of the beam when the strong column weak beam criterion was fulfilled. However, in the case of a strong beam weak column, failure occurred by weld and tube-wall tearing. The Laser Cutting Technology (LCT) is a promising solution for cutting the hole to pass the I-profile in the CHS column [20] thanks to several advantages including reduction of the heat affected zone and weld quantity, high cut precision and quality... Bursi et al [21] highlighted that bolted connections made by laser cutting respect EN 1090-2 [22] requirements. Kanyilmaz and Castiglioni [23] investigated the applicability of LCT to the erection of passing through I-beam to CHS column. The laser cutting technology used in their research allowed an efficient cut of tubes up to 12.5 mm thickness. They suggested to automatically measure the dimensions of beam and column parts in order to optimize the slot. A 3D cutting allow to perform inclined angle cutting and thus full penetration welds for 8.8 mm tube-wall thickness and partial penetration for 10 mm. The conclusions of their work have been applied to erect four types of through I-beam to CHS column connections in the framework of the EU RFCS project LASTEICON [25]. Recently Das et al [27] presented an extensive parametric study on passing-through I-beam-to-CHS column using a finite element model calibrated based on preliminary test results. Their study demonstrated that both the through I-beam as well as the CHS column contribute significantly to the ultimate joint strength. However, this numerical study did not consider the welding technique that influence the ultimate resistance of these joints, particularly under cyclic loadings for which detailed experimental investigation is needed. Benedetto et al ([28], [29]) performed static and cyclic tests on a single joint configuration made with laser cutting technology with full penetration butt weld loaded by a bending moment applied on one side of the joint. Failure is triggered by crack propagation on the weld connecting the tube and the I-beam in the area transmitting a transverse tensile force. A finite element model was developed to model the facture with a good accuracy. This model was used to perform a parametric study and validate analytical models for rotational stiffness [28] and resistance [29] based on the component method.

Unfortunately the number of tests on passing through I-beam to CHS column is limited to fully validate a design method particularly for the resistance. In addition, the effect of welding technique; full penetration or fillet weld, should be investigated particularly under opposite bending moment, as it will influence the ultimate resistance. The analytical models for resistance developed by Das et al [27] and Di Benedetto [29] did not consider the interaction between the different components such as passing through I-beam and tube-wall in transverse tension and compression. This aspect should be investigated in details.

This paper presents in section 2 the results of experimental tests performed on 13 passing-through-I-beam-to-CHS-column connections made by laser cutting that have been designed based on the conclusions of Kanyilmaz and Castiglioni [23]. The objective of this campaign was to characterize the behaviour of steel specimens. Experimental tests on composite connections (CHS filled with concrete, concrete slab) have been performed by other partners involved in the LASTEICON project [25]. These connections have been tested for two practical load cases; the gravity and seismic loadings. Gravity loading generally produce equal (hogging) bending moments on both sides of the joint. Lateral loading produce opposite (hogging and sagging) bending moments on both sides of the joint. The objective of the tests, the experimental test set-up and the mechanical characteristics of materials are presented in section 2.1. The monotonic behaviour of LCT connections under equal bending moment is discussed in section 2.2 in terms of bending resistance and initial rotational stiffness. In addition, a conventional connection in which the I-beam is directly welded to the CHS column has been tested for comparison. In section 2.3, the behaviour of the specimens under monotonic and reverse bending moment is investigated and the influence of the tube-wall thickness and welding technique on the resistance of this type of connection is explored. The cyclic test performed under opposite bending moment are described in section 2.4. The results of these tests are compared against monotonic test results. In addition an analytical model is proposed for the evaluation of initial rotational stiffness and the bending resistance of connections loaded by equal or opposite bending moments in section 3.2 and 3.3, respectively. This model consider the interaction between the internal segment of the beam and the tube.

2. Experimental tests

2.1. Tests presentation

2.1.1 Introduction

The objective of this campaign is to characterize the behaviour of I-beam to a CHS column joints made by laser cutting technology (LCT). The beams, which are hot-rolled I-profile, cross the CHS column and thus increase the rotational stiffness and the bending resistance of the joint comparatively to conventional detailing. Thirteen joints have been tested for two load cases relevant in practice:

- Gravity loads: the joint is subjected to equal (hogging) bending moments and limited shear force (see Figure 3-a). This loading condition will be identified using acronym LC1.
- Seismic loads: opposite (hogging and sagging) bending moments are applied to the joint either monotonically or in a cyclic manner (see Figure 3-b). This loading condition is labelled LC2.

For the two load cases, the same test set-up has been used and only the forces exerted by the load-jacks were different. For the gravity load, the two load-jacks exerted loads forces in the same direction at a distance of 2.5 m from the column axis. For the second load case, the load-jacks applied two forces equal in magnitude and opposite in direction. For this load case, large displacement were expected due to column panel shear deformation. Thus, the lever arm was reduced to ensure that the vertical displacement at failure do not exceed ± 200 mm. The load-jacks were placed at a distance of 1.7 m from the column axis on each side.

a- LC1 : Equal bending moment *b- LC2 : Opposite bending moment*

Figure 3 : *Loading conditions studied*

2.1.2 Tests set-up and geometry of specimens

The test set-up was designed to accommodate symmetric/asymmetric monotonic and asymmetric cyclic loading conditions. The specimen consist of an I-beam and a CHS column. Each extremity of the CHS column was pinned using hinges (see Figure 4-a). The hinge at the bottom is fixed by cleats bolted to the strong floor. The upper hinge is fixed by cleats bolted to a HEB 400 beam bolted to two HEA 300 columns braced by 4 HEA 200 diagonals (see Figure 4-b). Two load-jacks whose capacity is

1500 kN apply a vertical load to the beam at each extremity corresponding to a distance of 1700/2500 mm from the CHS column axis. Lateral-torsional bracings are placed at 2.2 m from the column axis to prevent lateral torsional buckling of the beam.

The two beams connected to the load-jack were bolted to the specimens by end-plate connections designed to remain elastic during each tests and used during all the test campaign. Only the cross-section made of a IPE400 beam of 2.4 m length and the column was replaced at each test (see Figure 5).

a- Side view

b- Front view

Figure 4 : *Test set-up (dimensions in mm)*

The characteristics of connections tested are given in Table 1. The beam is an IPE 400 and the tube diameter is 355.6 mm. Both elements are made of steel grade S355. The main difference between connections C3-1, C3-2a/b and C3-3 is the tube-wall thickness which is equal to 8.8 mm, 10 mm and 12.5 mm, respectively. For economic reasons [23], full penetration welds (see Figure 6-a) are used for connections C3-1. The cutting angle of the tube-wall was equal to 30° and the gap to 1 mm based on conclusions of Kanyilmaz and Castiglioni [23]. For specimens C3-2, both filled welds (see Figure 6-b) and full penetration welds are considered to compare the impact of these two welding solutions on the resistance and the ductility of the joint. For fillet welds, the throat thickness is equal to 8 mm for welds between flanges and tube and 6 mm for welds between the beam web and the tube. The throat thickness of the welds was designed in order to be able to carry the tensile capacity of the beam flange and web (full strength fillet welds). The gap was already equal to 1 mm for fillet welds. Piscini et al [24] demonstrated numerically that the resistance of fillet weld is not affected for this range of gap. For the last specimen (C3-3), only fillet welds are considered also for economic reasons.

Ultrasonic tests have been performed to detect potential discontinuities [23] and it was concluded that full penetration has been obtained for tube of 8.8 mm thickness. For tubes of 10 mm thickness, it was not possible to penetrate more than 90% of the complete penetration required by EN ISO 5817 [30] to qualify the weld as “full penetration”. An additional specimen C3-0 (with standard welded connection), in which the beam was welded to the tube, has been tested in order to compare the results of the laser cutting technology joints to a conventional connection. Each specimen, from C3-1 to C3-3 and also C3-0, was tested under equal bending moment (LC1) (see Figure 3-a). Connections C3-1, C3-2a and C3-3 were tested under monotonic opposite bending moment (loading condition LC2). The results of these tests and particularly the elastic displacement δ_y were very useful in devising the cyclic test. Moreover, these monotonic tests serve also to evaluate the impact of cyclic loadings on the ductility and resistance of the joints. For connection C3-1*, only one load-jack was used, this loading condition is labelled LC2*.

***Figure 5 :** Geometry of specimens tested (dimensions in mm)*

a- Full/partial penetration weld *b- Fillet welds*

***Figure 6 :** Details of welding's of the beam flange*

Connections C3-1, C3-2b, C3-2bN, and C3-2a were tested under cyclic opposite bending moment according to the ECCS-45 [31]. In specimen C3-2bN, gravity loads from upper storeys are simulated applying an initial compressive force to the column before applying cyclic loads. The objective of this test was to assess the effect of this load on low-cycle fatigue of the joint. An initial compression force of 800 kN was obtained by preloading the column with 8 anchor bolts positioned on a circular flange bolted to the tube. The value of the axial force corresponds to 20% of the compressive resistance of the tested tubes. Axial strain gauges pasted on the anchor bolts provide an estimate of the preloading force.

***Table 1 :** Characteristics of specimens tested*

Connection	Tube	Welding	Loading condition	Vertical loading
C3-1	CHS 355.6×8,8	Full penetration butt welding	LC1 – monotonic	-
C3-2a	CHS 355.6×10	Fillet welding	LC1 – monotonic	-
C3-2b	CHS 355.6×10	Partial penetration butt welding	LC1 – monotonic	-
C3-3	CHS 355.6×12,5	Fillet welding	LC1 – monotonic	-
C3-0	CHS 355.6×10	Fillet welding (no laser cutting)	LC1 – monotonic	-
C3-1*	CHS 355.6×8,8	Full penetration butt welding	LC2* – monotonic	-
C3-1	CHS 355.6×8,8	Full penetration butt welding	LC2 – monotonic	-
C3-1	CHS 355.6×8,8	Full penetration butt welding	LC2 - cyclic	-
C3-2a	CHS 355.6×10	Fillet welding	LC2 - monotonic	-
C3-2a	CHS 355.6×10	Fillet welding	LC2 - cyclic	-
C3-2b	CHS 355.6×10	Partial penetration butt welding	LC2 - cyclic	-
C3-2bN	CHS 355.6×10	Partial penetration butt welding	LC2 - cyclic	800 kN
C3-3	CHS 355.6×12,5	Fillet welding	LC2 - monotonic	-

2.1.3 Measurements

Two inclinometers I_d and I_g were placed on the upper beam flanges at 200 mm from the column wall (see Figure 7 and Figure 8). LVDT were positioned to measure the vertical displacements of the beam (V_{1g}, V_{1d}…) and the horizontal displacement of the column (U_{1g}, U_{1d}, U_{2g} and U_{2d}). Horizontal LVDT (U_{31} to U_{42}) have been placed between the two supports welded on the beam at 65/100 mm from the tube-wall (see Figure 7 and Figure 8) in order to estimate the displacements of the joint in both the tensile and the compressive area caused by the bending moment. For connections tested under opposite bending moment, diagonal LVDT (U_{51} to U_{62}) have been added to evaluate the distortion of the column web panel (see Figure 8).

Figure 7 – *Displacements transducers and inclinometers for LC1 (dimensions in mm)*

Figure 8 – *Displacements transducers and inclinometers for LC2 (dimensions in mm)*

Strain gauges have been positioned on some specimens in order to estimate the stresses on the tube-wall and on the beam (see Figure 9). For all instrumented specimens, 4 rosette strain gauges were placed on the tube-wall at 20 mm from beam flanges, 2 axial strain gauges at 25 mm from the tube-wall on the exterior face of the beam flanges. For connection C3-2b tested under loading condition LC1, 4 axial strain gauges have been pasted on the exterior face of the beam flanges inside the tube.

a- C3-0 *b- C3-2b under LC1*

c- C3-1/C3-2b under LC2 *d- C3-3 under LC2*

Figure 9 – *Strain gauges (dimensions in mm)*

In addition, Digital Image Correlation has been used to evaluate the rotation of the connection. This technic is based on the comparison between two images taken at different stages whilst the connection is deforming. One of these images is taken as reference image and the other as a deformed image. Targets consisting of black squares with a white fill and a black dot at the centre were fixed on the tube-wall and the beam as shown in Figure 10.

Figure 10 *– Speckle pattern and target used for image correlation*

2.1.4 Mechanical characteristics of steel

Coupons have been extracted from IPE 400 and tubes, and tested under tension with a machine INSTRON according to NF EN 10002-1 [32]. The average mechanical characteristics are listed in Table 2 and respect the minimum requirements of steel grade S355.

Table 2 : *Mechanical characteristics of IPE 400 and CHS*

Component	Yield strength	Tensile strength	Elongation	Necking
	N/mm^2	N/mm^2	%	%
IPE400-flange	365	527	18	60
IPE400-web	371	538	17	58
CHS 355,6×8,8	372	510	18	66
CHS 355,6×10	382	519	18	65
CHS 355,6×12,5	371	507	18	64

2.2. Monotonic tests under equal bending moment

2.2.1. Loading protocol

For the five specimens tested under equal bending moment (loading condition LC1) the monotonic loading is applied in a displacement-controlled manner considering three steps:

- In the first stage, 50% of the elastic bending resistance of the connection is applied. Next, the specimen is unloaded.
- In the second stage, the forces corresponding to 100% of the elastic resistance are applied and followed by unloading,
- Finally, loading until failure of the joint or the beam.

For laser-cut joints (specimens C3-1 to C3-3), the design resistance was equal to 150 kN, and 50 kN for the conventional connection (specimen C3-0). Connection C3-2b was loaded by two opposite forces whose magnitude is close to 30 kN and unloaded. Next, the loading protocol described above was applied. With the help of axial strain gauges placed inside the tube (see Figure 9-b), the bending moment that develop in the beam inside the tube under load case LC2 can be estimated.

2.2.2. Moment rotation curves and failure modes

The failure modes, initial rotational stiffness and maximum bending moment applied to the connections are given in Table 3 for specimens tested under load case LC1. The moment-rotation curves are depicted in Figure 12. The rotation of connection depicted in Figure 12 and for the calculation of the initial rotational stiffness is evaluated based on horizontal LVDT measurements and using the following relationship:

$$tg\phi_{c,U} = \frac{(\Delta_{U32} + \Delta_{U42}) - (\Delta_{U31} + \Delta_{U41})}{4h_U} \tag{1}$$

with:

h_U : Distance between horizontal LVDT U_{31}/U_{32} and U_{41}/U_{42}.

Δ_{Uij} : Displacement measured by LVDT U_{ij}.

***Table 3 :** Failure modes and maximum load applied under LC1*

Specimen	Laser-cutting	Weld	Tube	$S_{j,ini,u}$ (MNm/rad)	$M_{j,max}$ (kNm)	Failure mode
C3-0	No	Fillet weld	355,6×10	18,7	214,9	Tube-wall crushing
C3-1	Yes	Full penetration	355,6×8,8	183	538,2	Bottom Flange buckling
C3-2a	Yes	Fillet weld	355,6×10	171	552,4	Bottom Flange buckling
C3-2b	Yes	Full-penetration	355,6×10	165	536,3	Bottom Flange buckling
C3-3	Yes	Fillet weld	355,6×12,5	166	545,9	Bottom Flange buckling

Under equal bending moments, failure of laser-cutting joints is due to the development of plastic hinges in the beam on both side of the tube-wall. Subsequent significant strain hardening took place and buckling of the bottom beam flange was observed (see Figure 11-a). These joints can be classified as full strength joints. However, a part of the ductility come from the yielding of the beam section inside the tube (see section 2.2.4). The resistance and initial rotational stiffness of the laser-cutting joints obtained are close and as expected the tube-wall thickness as well as the type of welding do not have an obvious influence on the behaviour of the laser-cutting joints under symmetric loading. The initial rotational stiffness of the laser cutting solutions is 10 times greater than that of the conventional connection C3-0. The failure mode of connection C3-0 is obviously different and corresponds to local tube-wall crushing in the compressive area (see Figure 11-b). The maximal bending moment that can sustain a LCT connections is twice more than that resisted by a conventional connection (see Table 3). This clearly demonstrate the interest of the laser cutting solution for gravity loadings.

a- Beam flange buckling : laser cutting joints *b- Tube-wall crushing : conventional joint*

Figure 11 : *Failure under monotonic equal bending moments*

The passing-through beam behave as a simply supported beam whose rotation at the supports are restrained by the CHS column walls. The moment rotation curves are similar for the 4 laser-cutting connections with an important hardening branch due to the development of a plastic hinge in the segments of beam inside and outside the tube. For the conventional connection C3-0, the hardening branch is less substantial and followed by a slow decrease of the bending moment.

Figure 12 *– Moment-rotation curves under LC1*

a- C3-2b *b- C3-0*

Figure 13 – *Comparisons of rotations evaluation under LC1*

The moment-rotation curves obtained with image correlation and horizontal LVDT (see Eq (1)) are depicted in Figure 13. For connections C3-2b, the rotation obtained by image correlation is lower than that calculated with horizontal LVDT outputs. The latter devices provide also the rotation of the beam that develop outside the tube. Image correlation measurement enable to capture the joint rotation. For bending moments around 450 kNm, the non-linear branch of the connection behaviour can be clearly observed with Image Correlation confirming that yielding starts in the beam section inside the tube. A rotation of 10 mrad is measured with image correlation at the end of the test. Concerning specimen C3-0, both curves are similar during the test as the main source of rotation come from the joint. The beam segment outside the tube have a little influence on the total rotation measured by horizontal LVDTs.

2.2.3. Strain gauges measurement

Axial strains measured with rosette strain gauges R_s and R_i located on the external tube-wall surface at 20 mm from the beam flanges (see Figure 14 for connections C3-0 and C3-2b) are, respectively, positive and negative. Yielding of the tube-wall appears rather at an early stage for the conventional connection as it starts for a bending moment ranging between 100 and 135 kNm. The increase of the axial strains in connection C3-2b is less pronounced as the bending moment is mainly transferred by the segment of the beam inside the tube (see section 2.2.4). On the contrary, the bending moment is directly transferred to the tube-wall for connection C3-0 and cause local bending moment in the tube-wall that favor yielding of this component. For connection C3-2b, the strain increases significantly for bending moments greater than 450 kNm and tube-wall yielding is observed. For this value of the bending moment, the beam segment inside the tube yields and becomes more flexible comparatively to the tube and thus a greater amount of the bending moment is transmitted to the tube.

Figure 14 : *Axial strains on tube-wall under LC1*

2.2.4. Bending moment in the beam portion inside the tube

For connection C3-2b, the bending moment at mid-span of the beam section inside the tube M_I has been evaluated with strain gauges J_{S1} and J_{S2} (see Figure 9-b). The ratio between M_I and the externally applied bending moment M_E is depicted in Figure 15 as a function of M_E. The bending moment M_I is very close to the applied bending moment and equal to about 95% of M_E. This confirm that the tube-wall transfer limited amount of the bending moment at the beginning of loading. The yield strength was reached for a bending moment equal to about 400-450 kNm confirming yielding of the beam segment inside the tube.

Figure 15 : *Internal bending moment of the beam under LC1 for C3-2b*

2.3. *Monotonic tests under opposite bending moment*

2.3.1. Loading protocol

For the three specimens tested under monotonic opposite bending moment (load case LC2) the loading was applied in a displacement-controlled manner in four steps:

- applying downward displacements by two load-jacks to produce compressive forces of about 40 kN,
- applying upward displacements by two load-jacks to produce tensile forces of about 40 kN,
- applying, by two load-jacks, opposite displacements (one actuator upward, the other downward) corresponding to forces in the order of 20-30 kN,
- applying, by two load-jacks, displacements equal in magnitude and opposite in direction until failure occurred in the joint or in the beam.

The three first steps were performed in order to condition the specimen and close the different slots of the test set-up. For specimen C3-1*, the loading procedure was similar to that of connections tested under LC1 loading protocol.

2.3.2. Failure modes and moment-rotation curves

The failure modes along with the maximum total bending moment applied to the joints are given in Table 4.

Table 4 : *Failure modes and maximum load applied under LC2*

Specimen	Weld	Tube	$M_{j,tot,max}$ (kNm)	Failure mode
C3-1*	Full penetration	355.6×8,8	461.8	Weld/tube-wall tearing, tube-wall crushing
C3-1	Full penetration	355.6×8,8	478.0	Weld/tube-wall tearing, tube-wall crushing
C3-2a	Fillet weld	355.6×10	723.4	Weld/tube-wall tearing, tube-wall crushing
C3-3	Fillet weld	355.6×12,5	863.8	Weld/tube-wall tearing, tube-wall crushing

Under LC2, the resistance decreases and the failure mode is quite different to that observed under loading condition LC1: (weld and tube-wall tearing, tube-wall crushing). This provides valuable information on the type of failure modes expected to develop under cyclic loading. The use of full strength fillet weld and a tube-wall thickness of 10 mm (specimen C3-2a) increased the resistance by 50% compared to specimens of 8.8 mm tube-wall thickness and full penetration butt weld (specimens

C3-1 and C3-1*). The type of weld probably has an influence on the resistance of the connection. This fact will be confirmed during the cyclic tests. The bending resistance obtained experimentally for specimen C3-2b is equal to 723.4 kNm and for specimen C3-3 the same is equal to 863.8 kNm resulting in a resistance increase of 19%. The non-linear behaviour of the specimens under load case LC2 is characterized by redistribution of forces in the joint. This redistribution was clearly observed in specimen C3-1*. The first crack develops in the weld located in the tensile area (see Figure 16-a and b). For specimen C3-1*, the first crack appeared for a bending moment of 406 kNm corresponding to the first drop observed in the moment-displacement curve depicted in Figure 19. Cracks continue to develop until complete detachment between the upper face of the beam flange in tension and the tube-wall occurs (see Figure 16-c).

a- Localisation of failure on the joint *b- Weld cracking*

c- Weld/tube-wall tearing *d- Tube-wall crushing*

Figure 16 : *Sequence of failure of C3-1**

a- Localisation of failure on the joint

b- Full penetration weld cracking

c- Fillet weld cracking

d- Weld/tube-wall tearing

e- Fillet weld tearing

f- Tube-wall crushing

g- Shear panel deformation

Figure 17 : *Failure in specimens C3-1, C3-2a and C3-3*

At this stage, the transverse tensile force due to bending moment cannot be transferred anymore to this area and will be transmitted to the other side of the joint. The flange applies also a transverse compressive force on the tube-wall causing crushing of the tube-wall (see Figure 16-d). The local buckling of the beam flange occurred as a direct consequence of the development of a bending moment close to the plastic bending moment of the beam.

For the other specimens tested under load condition LC2, the failure mechanism is similar. At the beginning of the process, a crack appears in the weld located in the tensile area close to the tube-wall for specimen C3-1 (see Figure 17-b) and in the fillet weld for connections C3-2a and C3-3 (see Figure 17-c).

The first cracks initiate in specimen C3-1 for a force applied by the load-jacks equal to 140 kN, producing a drop in the load-displacement curve at this load level (see Figure 18). This crack initiation is followed by weld/tube-wall tearing for connection C3-1 (see Figure 17-d) or fillet weld tearing for connection C3-2b and C3-3 (see Figure 17-e). In the meantime, crushing of the tube-wall is observed in the compressive area (see Figure 17-f).

At failure, shear deformation of the column panel has developed for connections C3-1 and C3-2a (see Figure 17-g). For specimens with full penetration butt weld (connection C3-1* and C3-1), the decrease in resistance occurring after cracks initiation and weld/tube-wall tearing is very low (see Figure 18). In presence of fillet weld, the force continue to increase after cracks initiation, however a brutal decrease of the force is observed due to sudden weld tearing. For the three specimens, the force in load-jack 1 continue to increase even after tube-wall or weld tearing whereas the force in load-jack 2 decrease (connection C3-1) or increase at a slow rate (connections C3-2a and C3-3). This difference in term of the applied force is mainly due to force redistribution mechanism where forces acting on one side of joint are transmitted to the other side of the joint. While the tensile part of one side is tearing, the other side can transfer a force in compression. The total bending moment applied to the joint $M_{\mathrm{j,tot}}$ permit to evaluate the externally applied loads:

$$M_{\mathrm{j,tot}} = F_1 L + F_2 L \tag{2}$$

We can observe that this total bending moment is nearly constant (see Figure 19) after the first crack initiation for connection with full penetration welding (connections C3-1* and C3-1). In connections with fillet weld, the bending moment continues to increase even after cracks initiation and the sharp decrease of the bending moment is due to crack propagation. After this decrease, the total bending moment remains constant.

Figure 18 *– Force-displacement curves under LC2*

Figure 19 *– Total bending moment-displacement curves under LC2*

2.3.3. Strain gauges measurement

Axial strains measured with rosette strain gauges R_s and R_i placed on the tube on the side of load-jack 2 (see Figure 9) are shown in Figure 20 for connections C3-1, C3-2b and C3-3. The strains measured by R_s and R_i are respectively positive (tension) and negative (compression). At the start of the loading, the evolution of the axial strains is quite similar for the three specimens. However, the non-linear regime starts later for specimens with the thickest tube-wall. On the part of the tube transferring transverse tensile force, the curves are non-linear at a quite earlier stage. However, this is not the case for the part of the tube transferring transverse compressive force. For connections C3-1 and C3-2b, the evolution of the strain within the tube-wall is similar but crack appears sooner in specimen C3-1 made with full penetration butt weld. The fillet weld has a favourable effect by delaying cracks initiation in the tensile area. The evolution of the column distortion measured by rosette strain gauges $R_{m,2}$ (see Figure 9) is presented in Figure 21. During the elastic stage, the evolution of the distortion is similar for specimens C3-1 and C3-2 with tube-wall thickness equal to 8.8 and 10 mm, respectively. The non-linear regime starts, for each specimen, at a different value of the applied bending moment that are close to that observed with the moment-displacements curves. There is full agreement between strain gages and displacement measurements.

Figure 20 : *Axial strains on tube-wall under LC2*

Figure 21 : *Distortion on column panel under LC2*

2.4. Cyclic tests under opposite bending moment

2.4.1. Loading protocol

The cyclic tests were performed considering ECCS-45 [31] based on the elastic displacement δ_y deduced from the previous monotonic tests C3-1 and C3-2a under loading condition LC2. Cyclic loading was executed in displacement-controlled manner. The elastic displacement δ_y and the number of cycles performed outside the limit $\pm\delta_y$ are given in Table 5. Testing of specimen C3-2bN was stopped at an early stage of the loading protocol to avoid any deterioration of the preloading system (anchor bolts). Testing of the remaining specimens was pursued/conducted until the end of the loading protocol.

Table 5 : *Cyclic tests – Loading procedure*

Specimen	δ_y (mm)	Number of cycles		
		$\pm2\delta_y$	$\pm4\delta_y$	$\pm6\delta_y$
C3-1	20	3	3	3
C3-2a	24	3	3	-
C3-2b	24	3	3	2
C3-2bN	24	3	1	-

2.4.2. Failure modes

The failure modes along with the maximum bending moments obtained in monotonic and cyclic tests $M_{j,tot,u,mon}$ and $M_{j,tot,u,cycl}$ are listed in Table 6. The failure modes obtained during cyclic tests are similar to those observed in monotonic tests: weld/tube-wall tearing in the tensile area, crushing of the tube-wall in the compressive area and deformation of the column panel in shear. The maximum bending moment obtained during cyclic tests for specimen C3-2a is equal to 707/710 kNm and is close to that of monotonic test (723 kNm).

Table 6 : *Failure modes and maximum bending moment applied under LC2*

Specimen	Weld	Tube	$M_{j,tot,u,cycl}$ (kNm)	$M_{j,tot,u,mon}$ (kNm)	Failure mode	Vertical loading (kN)
C3-1	Full penetration	355,6×8,8	-445/524	478	Weld/Tube-wall tearing and tube-wall crushing	-
C3-2a	Fillet weld	355,6×10	-710/707	723	Weld tearing, tube-wall crushing	-
C3-2b	Partial penetration	355,6×10	-569/619	-	Weld tearing, tube-wall crushing	-
C3-2bN	Partial penetration	355,6×10	-564/580	-	Weld tearing, tube-wall crushing	800

Regarding connection C3-1, the maximum bending moment obtained is different for sagging and hogging moment. The maximum positive/negative bending moment attained during test is 524/-445 kNm whereas the maximum monotonic bending moment is equal to 478 kNm. The latter value is close to the average between the maximum positive and negative bending moment reached during cyclic loading being equal to 484 kNm. The single difference between specimen C3-2a and specimen C3-2b concerns the welding technique (full strength fillet weld or partial penetration butt weld). The resistance of a specimen with fillet weld is 15% higher than that of a specimen with partial penetration butt welds. It confirms the observations made during the monotonic tests: the type of welding strongly influence the ultimate resistance of the joint under opposite bending moment. Compared to connection C3-2b that was tested without vertical loading, the application of a vertical load of 800 kN to connection C3-2bN does not modify either the resistance nor the failure modes. For connection C3-2b, buckling of the beam web occurs also inside the tube as a result of the development of large shear force after major tearing of the tube-wall and the weld in the tensile area. This phenomena was not observed in the other specimens because the applied displacement magnitude was lower resulting in less degradation in the tensile area.

a- Localisation of failure on the joint

b- Cracking of weld during $\pm 2\delta_y$

c- Tearing of weld during $\pm 4\delta_y$

d- Tube-wall tearing during $\pm 6\delta_y$

e- Tube-wall crushing

f- Column panel deformation

Figure 22 : *Failure in specimens C3-1*

2.4.3. Force and moment-displacement curves

For each specimen, the force versus displacement curves for cyclic and monotonic loadings are compared in Figure 23. In addition, the total bending moment applied to the joint as a function of the displacement applied by load-jack 2 is depicted in Figure 24.

The monotonic force-displacement ~~curves~~ of connection C3-1 do not envelope the cyclic curve (see Figure 23-a), particularly for negative vertical force in load-jack 1. For positive vertical force, the cyclic and monotonic curves are quite close. The negative force in load jack 1 continue to increase even after weld/tube-wall tearing whereas the positive force of jack 2 decreases dramatically. This unbalanced evolution of load-jack forces is the result of the redistribution of the forces acting on the joint from one side of the joint to the other one. This phenomenon was also observed during monotonic tests (see section 2.3.2). However, the maximum total bending moment applied to the joint decreases after each cycle. In contrast with the force-displacement curve, the monotonic moment-displacement curve envelope the cyclic one's (see Figure 24-a). The first cracks appear on the weld during the cycles of amplitude $2\delta_y$ (see Figure 22-b) and propagate over the entire width of the beam flange during cycles of amplitude $4\delta_y$ (see Figure 22-c). During cycles of amplitude $6\delta_y$, cracks propagate in the tube-wall (see Figure 22-d). These crack propagations explain the decrease of the load applied to the joint with each cycle amplitude. However, tearing of the tube-wall does not cause a complete failure of the joint as the bending moment can be transmitted to the beam by shear and by the compressive part of the connection. In contrast with what happened with monotonic tests, the first crack is followed by a substantial decrease of the forces applied to the specimens.

The monotonic curve of connection C3-2a envelopes the cyclic one (see Figure 23-b and Figure 24-b). The first cracks were observed at the weld toe close to the tube during cycles of amplitude $2\delta_y$. These cracks did not cause any decrease of the forces applied to the joint. During cycles of amplitude $4\delta_y$, the cracks propagates along the width of the beam flange. As for connection C3-1, tearing of the tube-wall is observed during cycles of amplitude $6\delta_y$. The drop of the applied force due to sudden propagation of cracks appears earlier during the cyclic test. After this sudden fall, the applied force range between 140 and 150 kN during cycles of amplitude $4\delta_y$ and 110 kN during cycles of amplitude $6\delta_y$. The monotonic curves of connection C3-2a with fillet weld, envelopes quite well the cyclic curve of specimens C3-2b and C3-2bN (partial penetration butt weld) until crack initiation occurs. After crack initiation and propagation, the force applied by the load-jack decreases continuously and the cyclic curves are moving away from the monotonic curves. Similarly to specimen C3-1 made with full penetration butt weld, the cracks initiate during cycles of amplitude $2\delta_y$ and propagate over the width of the beam flange during cycles of amplitude $4\delta_y$. After this drop due to crack propagation, the applied force range between 130 and 150 kN during cycles $4\delta_y$ and 110 and 120 kN during cycles of amplitude $6\delta_y$. The cyclic curve specimen C3-2bN, tested with an additional axial force of 800 kN, is close to that of specimen C3-2b tested without axial force.

The moment-displacements curves obtained for connections C3-1, C3-2a and C3-2b are compared against each other in Figure 25 for the four cycle rates applied during tests.

a- C3-1 *b- C3-2a*

c- C3-2b *d- C3-2bN*

Figure 23 – *Force-displacement curves during cyclic and monotonic tests*

a- C3-1 *b- C3-2a*

c- C3-2b *d- C3-2b and C3-2bN*

Figure 24 – *Moment-displacement curves during cyclic and monotonic tests*

During the first cycles, connections C3-2 and C3-1 exhibit the same behaviour and their curves match quite well (see Figure 25-a). Specimen C3-1 starts to yield earlier as a result of a thinner tube-wall. At the beginning of the first cycle of amplitude $2\delta_y$, connection C3-2a and C3-2b exhibit the same behavior but the curves diverge after the first load drop occurs for specimen C3-2b caused by crack initiation in the weld. We can also observe that after crack propagation, the curves for specimen C3-1 and specimen C3-2b match quite well. During the three cycles of amplitude $2\delta_y$, the behavior of connection C3-2a is purely plastic without any degradation. During cycles of amplitude $4\delta_y$, the behaviour of connections C3-1 and C3-2b are very similar. After crack propagation and the corresponding drop of the bending moment, the cyclic curve of connection C3-2a is close to that of connection C3-1 and C3-2b. The same conclusion can be drawn during cycles of amplitude $6\delta_y$. After cracks propagation, the behaviour of the three specimens is described by similar curves. This can be explained by the fact that in this case only, the beam in shear inside the tube and the compressive area are able to transmit the applied bending moment. The beam is identical for the three specimens whereas the tube-wall capacity is larger for connections C3-2.

a- Cycles from $\pm\delta_y/4$ to $\pm\delta_y$ *b- Cycles $\pm 2\delta_y$*

c- Cycles $\pm 4\delta_y$ *d- Cycles $\pm 6\delta_y$*

Figure 25 – *Moment-displacement curves during the loading rates*

2.4.4. Energy dissipation

The cumulated energy dissipated during each cycle by the connections vs. the number of the cycle is presented in Figure 26-a. The energy dissipated during a cyclic loading corresponds to the area under the force-displacement curve of the two load-jacks. During the first three cycles of amplitude ranging from $\delta_y/4$ to $3\delta_y/4$, the energy dissipated is quasi null as the joints remain elastic. Next, a limited dissipation appears during cycle of amplitude δ_y for the three specimens whilst the dissipation becomes substantial during the cycles of amplitude $2\delta_y$. The two specimens C3-2 (tube-wall thickness of 10 mm) dissipate more energy than C3-1.

a- Effect of the cycle number

b- Effect of the cumulative displacement

Figure 26 : *Energy dissipation during cyclic tests*

During initial cycles including the first cycle of amplitude $2\delta_y$, connections C3-2a and C3-2b dissipate the same amount of energy. It is consistent since these two specimens have similar properties except the welding technique. After these cycles, connection C3-2a dissipate more energy than connection C3-2b mostly due to the loss of capacity of connection C3-2b after weld cracking. The increase of energy dissipation is more pronounced between two cycle rates mainly due to the increase of the maximal displacement applied. In order to avoid this discontinuity, the cumulative energy dissipated is presented as a function of the cumulative displacement (see Figure 26-b). It can be noted that the energy dissipated by the connections is thus more progressive and the tube-wall degradation do not results in a decrease of the rate of energy dissipation. The energy dissipated by connection C3-2b gradually tend to that of connection C3-1, which is consistent with the fact that the curves of the two assemblies are similar for cycles of amplitude greater than $2\delta_y$.

2.5. Conclusion on experimental tests

Under equal bending moment, laser cutting joints were able to develop the plastic bending resistance of the beam and can be classified as rigid/full strength joints. In fact a great part of the applied bending moment is sustained by the beam segment inside the tube. Bending deformations begin to develop in the tube once the beam section inside the tube yields. The mechanical characteristics of the laser cutting joints are far beyond those of conventional directly welded I-beam to CHS column connections.

Under monotonic opposite bending moment (loading condition LC2), failure modes are significantly different from those observed under symmetric loading and corresponds to the weld tearing in the tensile area and tube-wall crushing in the compressive zone. The tube-wall thickness and the type of welding had a clear influence on the resistance of the connection. Full/partial penetration butt welds developed lower resistance than full strength fillet welds. This difference can be somewhat explained by the fact that for tubes of 10 mm thickness, the penetration was not perfect. However, the difference with tubes of 8.8 mm thickness was not significant. Even after major weld tearing, the connections were able to develop non-negligible resistance, particularly for connections with full/partial penetration butt welds. For connections with fillet weld, an abrupt drop of the applied force occurred after important strain hardening took place. Connections tested under cyclic loading exhibit failure modes that are similar to those observed during monotonic tests (LC2). In addition, the maximum applied load during cyclic test is close to that obtained under monotonic loading (LC2). However, the ductility of the joints under cyclic loading is less substantial than that observed under monotonic loading (LC2). This is due to an earlier crack propagation in the tensile area during cyclic tests. The cyclic tests confirmed the impact of the type of welding on the resistance of the connection, full strength filled weld had a better performance than full/partial penetration butt weld.

3. Analytical model

3.1. Introduction

An analytical model is proposed to evaluate the bending resistance and initial rotational stiffness of passing through-I-beam-to-CHS column connections loaded under equal and opposite bending moments. For the two loading cases, the model assume a two-support beam resting on the tube-wall. The beam cross-section rotation at these supports is constrained by the tube. The interaction between the beam and the tube is modelled by a flexural spring placed at the support. The results of the proposed model are compared against the experimental test results presented in section 2 and numerical results of Das et al [27].

3.2. Connections under equal bending moment : load case LC1

3.2.1. General assumptions

Experimental tests pointed out an interaction between the tube and the beam. The redistribution of external loads depends on the relative stiffness of these two components. The objective of the proposed model is to consider this interaction. An accurate estimate of the components stiffness is then necessary. CIDECT design guide 9 [12] propose to evaluate the rotational stiffness of directly welded I-beam to CHS column connection. However, the suggested stiffness expressions were far below those obtained experimentally for connection C3-0. In a more recent research work carried out by Zhao et al [33], a more accurate expression for the stiffness of tube-wall was suggested (see Figure 27). This expression is adopted in the present design model:

$$k_{\mathrm{t}} = \frac{F}{\delta} = 0{,}73E \cdot D \cdot e^{[-0{,}64\beta - 0{,}015\gamma]} \times (\gamma - 0{,}5)^{(-2{,}81+1{,}46\beta)} \times \left(1 + 0{,}425\ln(\tau_{\mathrm{p}})\right) \tag{3}$$

with

$$\beta = \frac{b_{\mathrm{fc}}}{D},\ \gamma = \frac{D}{2t_{\mathrm{t}}},\ \tau_{\mathrm{p}} = \frac{t_{\mathrm{fc}}}{t_{\mathrm{t}}}$$

The domain of validity for this expression is:

$$0{,}3 \le \beta \le 0{,}9,\ 7 \le \gamma \le 30,\ 0{,}4 \le \tau_{\mathrm{p}} \le 1{,}2$$

Figure 27 : *Definition of the stiffness by Zhao [33]* ***Figure 28 :*** *Modelling of directly welded I-beam to CHS*

To account for the contribution of welds to stiffening of the tube wall, the weld leg has been added to the beam flange thickness and beam flange width respectively in the expression of τ_p and β. For directly welded I-beam to CHS column connection (see Figure 28), this transverse stiffness can be used (see Figure 27) to calculate the initial rotational stiffness:

$$S_{j,ini} = 0,5 k_t \left(h_b - t_{fb} \right)^2 \tag{4}$$

Tests performed on passing through I-beam-to-CHS column connections under equal bending moment show that the failure occurs in the beam where a plastic hinge develops and not in the connection itself. Hence, a substantial part of the bending moment is transferred to the internal segment of the beam and a limited part go to the tube as a result of the relative stiffness of these two elements.

Figure 29 : *Load distribution under equal bending moment*

Figure 30 : *Model under equal bending moment*

In fact, under equal bending moment, the joint can be modelled as a beam resting on two supports. The rotation at the support is constrained by the tube wall. The beam-tube interaction is modelled using flexural springs placed at the supports (see Figure 30). The rotation of the tube-wall at both beam-tube intersections produced by the bending moment M_t transferred directly to the tube is computed using:

$$\theta_t = \frac{M_t}{S_{j,t}} \tag{5}$$

where $S_{j,t}$ is the rotational stiffness of the tube corresponding to a conventional connection that can be estimated according to Equation (4).

The rotation at the support of the beam section inside the tube subjected to the internal bending moment M_I is:

$$\theta_t = \frac{M_I D}{2EI_b} = \frac{M_I}{S_b} \tag{6}$$

where D is the diameter of the tube and I_b is the moment of inertia of the beam.

The compatibility in rotation between the tube-wall and the beam gives:

$$M_t = M_I \frac{S_{j,t}}{S_b} \tag{7}$$

Finally, bending moment equilibrium at the support provides:

$$M_E = M_I + M_t \tag{8}$$

From which we deduce the relation between the internal bending moment in the beam and the external one's:

$$M_I = M_E \frac{1}{1 + \frac{S_{j,t}}{S_b}} \tag{9}$$

Connections C3-2b and C3-0 have a tube-wall thickness equal to 10 mm but different welds. It is expected that the flexural stiffness of the tube for both connections are similar. Then inserting in Equation (9) the initial rotational stiffness $S_{j,t}$ of C3-0 estimated experimentally equal to 18.7 MNm/rad (see Table 3), we obtain that the internal bending moment is close to 95% of the external bending moment applied to the connection. This ratio is similar to that measured during the test of connection C3-2b (see section 2.2.4).

3.2.2. Initial rotational stiffness

The initial rotational stiffness of the connection can be determined according to the previous model. Inserting Eq (9) into Eq (6), one obtain the connection rotation:

$$\theta_{\mathrm{t}} = \frac{M_{\mathrm{E}}}{S_{\mathrm{b}} + S_{\mathrm{j,t}}} \tag{10}$$

The initial rotational stiffness of the connection is:

$$S_{\mathrm{j,ini}} = S_{\mathrm{b}} + S_{\mathrm{j,t}} \tag{11}$$

The stiffness of the connection is larger than that of a regular simply supported beam ignoring its interaction with the tube. The initial rotational stiffness calculated analytically and experimentally are given in Table 7. The results are in quite good agreement. For directly welded I-beam to CHS column connection (connection C3-0), the rotational stiffness predicted by the model is below its experimental value as a consequence of the web contribution being neglected.

Table 7 : *Initial rotational stiffness under equal bending moment: comparison to tests*

Specimen	Laser-cutting	Weld	Tube	$S_{\mathrm{j,ini,test}}$	$S_{\mathrm{j,ini,ana}}$
	-	-	mm×mm	MNm/rad	MNm/rad
C3-0	No	Fillet weld	355,6×8,8	18,7	14,6
C3-1	yes	Full penetration	355,6×8,8	183	180,2
C3-2a	yes	Fillet weld	355,6×10	171	189,4
C3-2b	yes	Full penetration	355,6×10	165	181,8
C3-3	yes	Fillet weld	355,6×12,5	166	197,2

3.2.3. Bending resistance

From Equation (9), it is clear that the bending moment in the beam segment inside the tube will be lower than the external bending moment. Therefore, the plastic bending resistance of the beam $M_{\mathrm{pl,b}}$ will be reached outside the tube where the first hinge appears. In addition, the amount of the external bending moment transmitted to the tube is obtained by combining Equations (7) and (9) to produce:

$$M_{\mathrm{t}} = M_{\mathrm{E}} \frac{S_{\mathrm{j,t}}}{S_{\mathrm{b}} + S_{\mathrm{j,t}}} \tag{12}$$

The bending resistance of the tube $M_{t,u}$ can be calculated according to CIDECT design guide 9 [12] or prEN 1993-1-8 [3].

- **CIDECT design guide 9:**

$$M_{\mathrm{t,u,CIDECT}} = \frac{5 f_{\mathrm{y,t}} t_{\mathrm{t}}^2 (1 + 0,25\eta)}{1 - 0,81\beta} h_{\mathrm{b}} \tag{13}$$

- **prEN 1993-1-8:**

$$M_{\mathrm{t,u,EC3}} = 2,1 f_{\mathrm{y,t}} t_{\mathrm{t}}^2 (1 + 3\beta^2) \gamma^{0,25} h_{\mathrm{b}} \tag{14}$$

with:

f_{yt}: yield strength of the tube,

$\eta = \frac{h_{\mathrm{b}}}{D}$

Finally, the bending resistance of the connection is:

$$M_{\mathrm{j,u}} = M_{\mathrm{pl,b}} \le M_{\mathrm{t,u}} \frac{S_{\mathrm{j,t}} + S_{\mathrm{b}}}{S_{\mathrm{j,t}}} \tag{15}$$

One can note that elastic stiffnesses are used in Equation (15) instead of a secant stiffness. Without numerical simulations, the secant stiffness cannot be accurately estimated and we decided to use the elastic stiffness. This proposed formulation underestimate the capacity of the tube in bending and is conservative. The resistance evaluated according to Equation (15) considering either the resistance of the tube proposed by CIDECT or the expression suggested in Eurocode 3 are given in Table 8. The results are in quite good agreement. The model underestimate the resistance of laser-cutting joints mainly because strain hardening is not considered. Concerning conventional connection, Eurocode 3 approach provide a sharp underestimate of the bending resistance.

Table 8 : *Resistance under equal bending moment: comparison to tests*

Specimen	Weld	Tube	$M_{j,u,test}$	$M_{j,u,CIDECT}$	$M_{j,u,EC3}$
	-	mm×mm	kNm	kNm	kNm
C3-0	Fillet weld	355,6×8,8	214,9	198	143
C3-1	Full penetration	355,6×8,8	538,2	477	477
C3-2a	Fillet weld	355,6×10	552,4	477	477

C3-2b	Full penetration	355,6×10	536,3	477	477
C3-3	Fillet weld	355,6×12,5	545,9	477	477

The connection bending resistance predicted by the proposed analytical model and the numerical results obtained by Das et al [27] for different connection configurations are given in Table 9. The reference specimen C3-1-4 studied in Das et al [27] is similar to the tested connection C3-2b. In the FE model, full continuity between the I-beam and the tube was assumed. Based on the numerical results given in in Das et al [27], the plastic bending moment is estimated according to ECCS-45 [31]. The latter draw the frontier between the elastic and plastic domain. It can be seen that the results are very close as the failure mode corresponds to the development of a plastic hinge in the beam and not failure of the tube in bending.

Table 9 : *Resistance under equal bending moment: comparison to parametric study of Das et al [27]*

Variation	Specimen	D	t_t	Beam	$M_{j,pl,num}$	$M_{j,pl,ana}$
		mm	mm	-	kNm	kNm
Beam profile	C3-1-1	355,6	10	IPE220	95	97
	C3-1-2	355,6	10	IPE270	160	163
	C3-1-3	355,6	10	IPE330	265	271
	C3-1-4	355,6	10	IPE400	427	440
	C3-1-5	355,6	10	IPE500	722	748
Tube thickness	C3-1-6	355,6	4	IPE400	420	440
	C3-1-7	355,6	6	IPE400	423	440
	C3-1-8	355,6	8	IPE400	425	440
	C3-1-4	355,6	10	IPE400	427	440
	C3-1-9	355,6	12,5	IPE400	427	440
Tube diameter	C3-1-10	273	10	IPE400	424	440
	C3-1-11	323,9	10	IPE400	426	440
	C3-1-4	355,6	10	IPE400	427	440
	C3-1-12	406,4	10	IPE400	428	440
	C3-1-13	457	10	IPE400	429	440

*3.3. Connections under load cases LC2 and LC2**

Experimental tests performed on passing-through-I-beam-to-CHS column connections revealed that joints can be seen as a beam resting on two support and constrained elastically in bending by the tube. This behaviour was clearly observed under equal bending moment (see section 3.2). The external bending moment applied to one side of the connection M_E is decomposed into a bending moment acting on a cross-section of the beam segment inside the tube M_I, and the bending moment acting on the tube M_t:

$$M_E = M_I + M_t \qquad (16)$$

Figure 31 : *Load distribution under opposite bending moment*

Figure 32 : *Model under opposite bending moment: load case LC2*

Inside the tube, the beam is resting on two supports and is loaded by two opposite bending moments, the shear force is thus:

$$V_{j,I} = Q = \frac{2M_I}{D} \qquad (17)$$

The shear resistance of the beam is:

$$V_{wb,u} = \frac{A_{vb} f_{y,wb}}{\sqrt{3}} \qquad (18)$$

with

A_{vb}: shear area of the beam calculated according to Eurocode 3,
$f_{y,wb}$: Yield strength of the beam web.

In the present model, the shear area of the beam A_{vb} is restricted to the contribution of the web because the value given in EN 1993-1-1 assume the development of strain hardening in the web.

The bending resistance of the internal section of the beam is thus:

$$M_{I,u} = \frac{V_{wb,u} D}{2} \tag{19}$$

The bending resistance of the tube $M_{t,u}$ can be estimated with the relationship proposed by Voth and Packer [15] for passing through plate to CHS connections or the new proposal of prEN 1993-1-8 [3]. The resistance of tube-wall to transverse force according to prEN 1993-1-8 [3] is:

$$F_{fc,u,EC3} = 2,3 f_{y,t} t_t^2 \left(1+3\beta^2\right) \gamma^{0,35} Q_f \tag{20}$$

With

$$Q_f = \left(1-|n|\right)^{C_1}$$
$$C_1 = \begin{cases} 0,25, & n<0 \text{ (compression)} \\ 0,2, & n \geq 0 \text{ (tension)} \end{cases}$$

n : ratio between the stress acting on the tube-wall close to the flange plate and the yield stress, in the present case:

$$n = \frac{M_j}{M_{c,u}}$$

$M_{c,u}$: Bending resistance of the tube.

Recently Voth and Packer [15] demonstrated that the resistance of passing-through plate to CHS column can be obtained by adding tension and compression resistances of branch-plate-to-CHS connection, the resistance of one side of the passing-through plate is :

$$F_{fc,u,Voth} = F_{fc,c,u} + F_{fc,t,u} = 0,85 f_{y,t} t_t^2 \left[1,45\left(1+3\beta^2\right)\gamma^{0,35} + 1,3\left(1+2,5\beta^2\right)\gamma^{0,55}\right] Q_f \tag{21}$$

The force transmitted by the flange plates to the column generate a shear force on the column panel and will be limited by its resistance estimated using the following Eurocode 3 expression:

$$V_{\mathrm{wp,u}} = 0{,}9\frac{A_{\mathrm{vc}} f_{\mathrm{y,t}}}{\sqrt{3}\beta} \tag{22}$$

where:

A_{vc}: shear area of the column:

$$A_{\mathrm{vc}} = \pi D t_{\mathrm{t}} / 2$$

β: transformation parameter equal to 2 in the present case according to EN 1993-1-8.

The bending resistance of the tube can be calculated according to

$$M_{\mathrm{t,u}} = F_{\mathrm{fc,u}}(h_{\mathrm{b}} - t_{\mathrm{fb}}) \leq V_{\mathrm{wp,u}}(h_{\mathrm{b}} - t_{\mathrm{fb}}) \tag{23}$$

The bending resistance of the joint is given by

$$M_{\mathrm{j,u}} = M_{\mathrm{I,u}} + M_{\mathrm{t,u}} \leq M_{\mathrm{pl,b}} \tag{24}$$

When only one side of the joint is loaded (see Figure 33), the bending resistance of the joint becomes:

$$M_{\mathrm{j,u}} = M_{\mathrm{I,u}} + 2M_{\mathrm{t,u}} \leq M_{\mathrm{pl,b}} \tag{25}$$

where β is equal to 1 and the bending resistance of internal portion of the beam is:

$$M_{\mathrm{I,u}} = V_{\mathrm{wb,u}} D$$

Figure 33 : *Model under opposite bending moment: load case LC2**

The calculation has been done considering the two proposal for the evaluation of $F_{fc,u}$ given by Eq. (20) and Eq. (21), the corresponding bending resistance are $M_{j,u,EC3}$ and $M_{j,u,Voth,}$ respectively. The bending resistance calculated with the two models are compared against the experimental results presented in this paper and the one obtained by Di Benedetto et al [28] in Table 10. The Eurocode 3 approach is quite conservative for the six specimens. It is not the case for the model that use the formulation of Voth, in particular for connection C3-1. The force redistribution in the different components is probably not adequate in the presence of full penetration butt weld. For full strength fillet weld and the specimen tested by Di Benedetto [28], this model is conservative.

Table 10 : *Resistance under LC2 and LC2*: comparison to tests*

Specimen	Weld	Tube	$M_{j,u,test}$	$M_{j,u,Voth}$	$M_{j,u,Voth}/M_{j,u,test}$	$M_{j,u,EC3}$	$M_{j,u,EC3}/M_{j,u,test}$
	-	mm×mm	kNm	kNm	-	kNm	-
Test 1 [28]	Full Penetration	219,1×6	150	129	0,86	115	0,77
C3-1*	Full Penetration	355,6×8,8	461,8	526	1,14	468	1,01
C3-1	Full Penetration	355,6×8,8	239	263	1,10	234	0,98
C3-2a	Fillet weld	355,6×10	362	331	0,91	294	0,81
C3-2b	Partial Penetration	355,6×10	297	300	1,01	265	0,89
C3-3	Fillet weld	355,6×12,5	432	407	0,94	362	0,84

The predictions of the models are compared against the numerical results of Das et al [27] in Table 11. The bending resistance evaluated numerically corresponds to the plastic bending moment determined according to ECCS-45 [31]. The mean values of the ratio obtained with Voth and Eurocode 3 approaches are respectively equal to 1.05 and 0.94. The first model is more accurate but overestimate the plastic bending resistance. The Eurocode 3 model generally underestimate the plastic bending moment.

Table 11 : *Resistance under LC2 and LC2*: parametric study of Das et al [27]*

Variation	Specimen	D	t_t	Beam	$M_{j,pl,num}$	$M_{j,u,Voth}$	$M_{j,u,Voth}/M_{j,pl,num}$	$M_{j,u,EC3}$	$M_{j,u,EC3}/M_{j,pl,num}$
		mm	mm	-	kNm	kNm	-	kNm	-
Beam profile	C3-1-1	355,6	10	IPE220	91	97	1,07	97	1,07
	C3-1-2	355,6	10	IPE270	145	163	1,13	145	1,01
	C3-1-3	355,6	10	IPE330	205	227	1,11	197	0,96
	C3-1-4	355,6	10	IPE400	292	295	1,01	260	0,89
	C3-1-5	355,6	10	IPE500	389	387	0,99	351	0,90
Tube thickness	C3-1-6	355,6	4	IPE400	135	155	1,15	145	1,07
	C3-1-7	355,6	6	IPE400	182	195	1,07	177	0,97
	C3-1-8	355,6	8	IPE400	231	242	1,05	215	0,93
	C3-1-4	355,6	10	IPE400	292	295	1,01	260	0,89
	C3-1-9	355,6	12,5	IPE400	357	367	1,03	324	0,91
Tube diameter	C3-1-10	273	10	IPE400	299	277	0,93	232	0,78
	C3-1-11	323,9	10	IPE400	298	281	0,94	251	0,84
	C3-1-4	355,6	10	IPE400	292	295	1,01	260	0,89
	C3-1-12	406,4	10	IPE400	289	314	1,09	274	0,95
	C3-1-13	457	10	IPE400	289	333	1,15	289	1,00

4. Conclusion

This paper presents the results of an experimental campaign performed on passing-through-I-beam-to CHS joints subjected to monotonic and cyclic loadings. Three loading cases were considered: equal bending moment (monotonic loading), opposite bending moment (monotonic and cyclic loadings). In addition, a design model that provide the bending resistance and the initial rotational stiffness has been proposed. The main conclusions of experimental tests are the following:

- Under equal bending moment, the LCT joints were able to ensure a full continuity and thus failure was triggered by the development of a plastic hinge in the beam. The resistance of the LCT connections is more than twice (2.5) the resistance of a conventional connection. The initial rotational stiffness is 10 times higher. It was observed that yielding of the beam occurs also inside the tube. These results from the beam flexural stiffness being significantly larger than that of the tube. Nevertheless, yielding of the beam section inside the tube results in a modification of the relative stiffness of these two elements, which produces an increase of the stresses in the tube. The tube-wall thickness and the type of welding do not affect the behaviour of this joint under equal bending moment.
- Under monotonic opposite bending moment, the joint behaves in a completely different manner. Failure occurs at the joint. The first stage consists in crack initiation of the weld in the tensile area whatever the type of welding. Next, the crack propagates through the weld and finally on the tube-wall. This tearing was accompanied by tube-wall crushing in the compressive area and the yielding of the column panel in shear. The resistance of the joint depends on the thickness of the tube and also on the type of welding. The yielding of the beam in bending close to the connection was observed only for tube of 12.5 mm thickness and full strength fillet welds. It was observed that the resistance of the joints with full strength fillet welds was higher than that of joints with full/partial penetration butt weld. One can note that for tubes of 10 mm thickness, the penetration was not complete and it should be considered as partial penetration butt welds.
- The specimens tested under cyclic loading exhibit similar failure modes as those observed during equivalent monotonic opposite bending moments and the maximum loading applied

was quite close. However, under cyclic loading, the joint ductility decreases. Indeed, crack propagation in the tensile area appears at an early stage in cyclic tests compared to monotonic one. Once cracks propagation in the tube-wall occurred, the cyclic curves of the different specimens match quite well. This outcome is the result of the bending moment being transferred by shear of the beam portion inside the tube as well as the compressive area. The cyclic tests confirmed the impact of the type of welding on the resistance of the connection: full strength fillet welds had a better performance than full/partial penetration butt welds tested.

An analytical model has been developed to determine the initial rotational stiffness and bending resistance of the connections under equal bending moment considering the stiffness of the different components of the connections such as the beam and the tube. The model consider that a two-support beam rest on the tube-wall and restrained by the tube at the supports. The rotational stiffness of the tube was estimated based on the axial stiffness model developed by Zhao et al [33]. The predictions of the analytical model are in quite good agreement with experimental and numerical results. In addition, a similar model is proposed for joints under opposite bending moment. The bending resistance is related to yielding of the internal segment of the beam in shear and the CHS in transverse compression and tension. For the latter, the formulations of Voth and Packer [15] and prEN 1993-1-8 [3] have been compared. The Eurocode 3 formulation always give conservative results particularly in presence of full strength fillet welds. The accuracy is increased with full/partial penetration butt weld. The results based on the proposal of Voth and Packer are more accurate but can be unsafe, particularly with full penetration butt weld probably due to a limited force redistribution between the beam and the tube.

The passing through-I-beam to CHS column joints made by laser cutting are very interesting solutions to transmit gravity loadings in sway frames thanks to the continuity created. Under opposite bending moment (seismic or wind loadings), a particular attention should be given to the tube-wall tearing in the tensile area particularly when the bending resistance of the column is lower than that of the beam. The design model proposed in this paper permit to estimate, with a quite good accuracy, the corresponding resistance.

Acknowledgments

The authors gratefully acknowledge financial support by the European Commission (Research Fund for Coal and Steel) through the project LASTEICON under EU-RFCS GA-709807 (www.LASTEICON.eu).

5. References

[1] J. Wardenier, Y. Kurobane, J.A. Packer, G.J. van der Vegte, X.L. Zhao, Design guide for circular hollow section (CHS) joints under predominantly static loading, CIDECT Design Guide 1, Geneva, Switzerland, 2008.

[2] Joints in steel construction: moment resisting joints to Eurocode 3, SCI Publication P398, 2013.

[3] Draft of EN 1993-1-8, Eurocode 3: Design of steel structures. Part 1-8: Design of joints, 2018.

[4] J. Wardenier, Semi-rigid connections between I-beams and tubular columns, Final Report, ECSC-EC-EAEC, Brussels. Luxembourg, 1995.

[5] T. Togo, Experimental study on mechanical behavior of tubular joints, D. Eng. Thesis, Osaka University, Osaka, Japan, 1967.

[6] G.D. De Winkel, The Static Strength of I-Beam to Circular Hollow Section Column Connections, Doctoral thesis, Delft University of Technology, 1998.

[7] J. Wardenier, A uniform effective width approach for the design of CHS overlap joints II Research Publishing Services Singapore, p.155-165, 2007.

[8] W. Wang, Y. Chen, W. Li, R. Leon, Bidirectional seismic performance of steel beam to circular tubular column connections with outer diaphragm, Earthquake Engineering and Structural Dynamics, 40 (2010) 1063–1081.

[9] A. Bagheri Sabbagh, T.M. Chan, J.T. Mottram, Detailing of I-beam-to-CHS column joints with external diaphragm plates for seismic actions, J. Constr. Steel Res. 88 (2013) 21–33.

[10] T. Kamba, H. Kanatani, Y. Fujiwara, M. Tabuchi, Empirical formulae for strength of steel tubular column to H-beam connections: Part 2 – A study on the tubular column to beam connections. Transactions of Architectural Institute of Japan, 325 (1983) 67-73. (in Japanese)

[11] T. Kamba, Study on deformation behaviour of beam-to-CHS column connections with external diaphragms. Research Report, Kinki Branch of Architectural Institute of Japan, Osaka, Japan, 41 (2001) 201-204 (in Japanese).

[12] Y. Kurobane, J.A. Packer, J. Wardenier, N. Yeomans, Design Guide for structural hollow section column connections, CIDECT Design Guide 9, 2004.

[13] S.P. Schneider, Y.M. Alostaz, Experimental Behavior of Connections to Concrete-Filled Steel Tubes, J. Constr. Steel Res. 45(3) (1998) 321-352.

[14] Y.M. Alostaz, S.P. Schneider, Analytical behavior of connections to concrete-filled steel tubes, J. Constr. Steel Res. 40 (1996) 95–127.

[15] A.P. Voth, J.A. Packer, Circular hollow through plate connections, Steel Constr. 9 (2016) 16-23.

[16] A.P. Voth, J.A. Packer, Branch Plate-to-Circular Hollow Structural Section Connections. I: Experimental Investigation and Finite-element Modelling, J. Struc. Eng. 138(8) (2012) 1007–1018.

[17] A.P. Voth, Branch Plate-to-Circular Hollow Structural Section Connections, Doctoral Thesis, University of Toronto, Toronto, Canada, 2010.

[18] V.L. Hoang, J.F. Demonceau, J.P. Jaspart, Resistance of through-plate component in beam-to-column joints with circular hollow columns, J. Constr. Steel Res. 92 (2014) 79–89.

[19] A. Elremaily, A. Azizinamini, Experimental behaviour of steel beam to CFT column connections, J. Constr. Steel Res. 57 (2001) 1099-1119.

[20] A. Kanyilmaz, The problematic nature of steel hollow section joint fabrication, and a remedy using laser cutting technology: A review of research, applications, opportunities, Eng. Struc. 183 (2019) 1027-1048.

[21] O.S. Bursi, M. D'Incau, G. Zanon, S. Raso, P. Scardi, Laser and mechanical cutting effects on the cut-edge properties of steel S355N, J. Constr. Steel Res. 133 (2017) 81-191.

[22] NF EN 1090: Execution of steel structures and aluminium structures, Part 2: Technical requirements for steel structures, CEN, February 2009.

[23] A. Kanyilmaz, C.A. Castiglioni, Fabrication of laser cut I-beam-to CHS-column steel joints with minimized welding, J. Constr. Steel Res. 145 (2018) 16-32.

[24] A. Piscini, F. Morelli, A. Kanyilmaz, C.A. Castiglioni, W. Salvatore, Studies on the behaviour of steel beam-to-column joints realized by using laser cutting technology, 16th European Conference on Earthquake Engineering, Thessaloniki, Greece, 18-21 June, 2018.

[25] C.A. Castiglioni, A. Kanyilmaz, W. Salvatore, F. Morelli, A. Piscini, M. Hjiaj, M. Couchaux, L. Calado, J. Proenca, J. Sio, S. Raso, A. Valli, M. Brugnolli, B. Hoffmeister

, J. Korndorfer, H. Degee, R. Das, R. Hojda, C. Remde, A. Galazzi, A. Mazzanti, EU-RFCS Project LASTEICON 709807, 2016–2020, www.lasteicon.eu.

[26] EN 1993-1-8, Eurocode 3: Design of steel structures. Part 1-8: Design of joints, 2005.

[27] R. Das, C.-A. Castiglioni, M. Couchaux, Hoffmeister B., Degée H., Design and analysis of laser-cut based moment resisting passing-through I-beam to CHS column joints, Jour. of Const. Steel Res. 169 (2020) 106015.

[28] S. Di Benedetto, M. Latour, G. Rizzano G., Assessment of the stiffness of 3D cut welded connections with CHS columns and through I-beams, Structures. 27 (2020) 247-258.

[29] S. Di Benedetto, M. Latour, G. Rizzano, Chord failure of 3D cut welded connections with CHS columns and thought I-beams, Thin-walled Struc.. 154 (2020) 106821.

[30] EN ISO 5817:2014, Welding — Fusion-welded Joints in Steel, Nickel, Titanium and Their Alloys (Beam Welding Excluded) — Quality Levels for Imperfections, 38, 2014.

[31] ECCS, Recommended testing procedures for assessing the behavior of structural elements under cyclic loads, European Convention for Constructional Steelwork, Technical Committee 1, TWG 13 – Seismic Design, No45, 1986.

[32] EN 10002: Tensile testing of metallic materials–Part 1: Method of test at ambient temperature, 2001

[33] B. D. Zhao, Y. Chen, Q.L. Cheng, D.W. Han, W. Tao, D.W. Xiao, An axial semi-rigid connection model for cross-type transverse branch plate-to-CHS joints, Eng. Struc. 181 (2019) 413-426.