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R. Das, C.A. Castiglioni, Maël Couchaux, B. Hoffmeister, H. Degee. Design and analysis of laser-cut based moment resisting passing-through I-beam-to-CHS column joints. *Journal of Constructional Steel Research*, 2020, 169, pp.106015. 10.1016/j.jcsr.2020.106015 . hal-02865280

**HAL Id: hal-02865280**

**<https://univ-rennes.hal.science/hal-02865280>**

Submitted on 25 Jun 2020

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# Design and Analysis of Laser-Cut based Moment Resisting passing-through I-Beam-to-CHS Column Joints

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## Abstract

Conventional circular hollow section (CHS) connections are often prone to severe local distortion of the CHS column surface, premature flange fractures and demand excessive welding quantity due to much-needed local stiffeners, gusset plates or the direct welding technique. This results in an unavoidable complexity, which leads to a possible economic disadvantage. This paper proposes an innovative I-beam-to-CHS-column “passing-through” connection, which avoids the foretold drawbacks and increases the structural performance of these joints. The moment-resisting connection studied in this article was developed during the research project LASTEICON. It consists of primary beams connected to an I-beam passing through the CHS column through slots obtained via Laser Cutting Technology. The characterization of the ultimate resistance of the proposed connection under symmetrical and antisymmetrical loading conditions in bending is studied in detail. This is performed through a parametric study based on finite element (FE) models, which are primarily validated through an experimental campaign. Encouraging agreements obtained between the numerical and experimental results in terms of joint stiffness as well as joint resistance are presented. Moreover, different failure modes are identified and further characterized by

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28 developing comprehensive guidelines to design the proposed connection. Analytical  
 29 calculations for several case studies are performed following the proposed design procedure  
 30 and the results are compared with numerical as well as experimental results. Promising  
 31 agreement is achieved, therefore confirming its practical implementation. Ultimately, the  
 32 “passing-through” connections are compared with conventional (direct weld) joints to  
 33 highlight the former’s advantages from a structural perspective.

34 **Keywords:** Beam-to-CHS-column connection; Tubular structures; CHS joints; Hollow section  
 35 joints; Through Beam connections; Passing-through joints.

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#### List of symbols

$A_{vb}$	Through I-beam shear area
$b$	Through I-beam flange width
$C_f$	Compressive force in the through I-beam flange
$d_b$	Through I-beam depth
$d_c$	CHS column diameter
$f_b$	Punching shear stress
$f_y$	Material yield stress of steel
$f_{yb}$	Material yield stress of the through I-beam
$f_{yc}$	Material yield stress of the CHS column
$f_{yw}$	Material yield stress of the through I-beam web
$L_b$	Overall span of the beam
$L_c$	CHS column length
$LC1$	Load Case 1: Monotonic gravitation/symmetric loading
$LC2$	Load Case 2: Monotonic opposite bending loading
$M_{bp}$	Bending moment corresponding to the punching shear stress
$M_{b,opp}$	Moments developed at either face of the CHS under LC2
$M_{b,sym}$	Moments developed at either face of the CHS under LC1
$M_{b,Rd,opp}$	Flexural resistance of the joint under LC2
$M_{b,Rd,sym}$	Flexural resistance of the joint under LC1
$M_{ip,1,Rd}$	In-plane moment resistance of the CHS
$M_{pl,Rd,beam}$	Flexural resistance of the through I-beam
$M_{Rd,CHS}$	Flexural resistance of the CHS
$N_{1,Rd}$	Transverse tensile/compressive resistance of the CHS chord face
$P$	Vertical load at the free end of the main I-beam
$t_c$	CHS column thickness

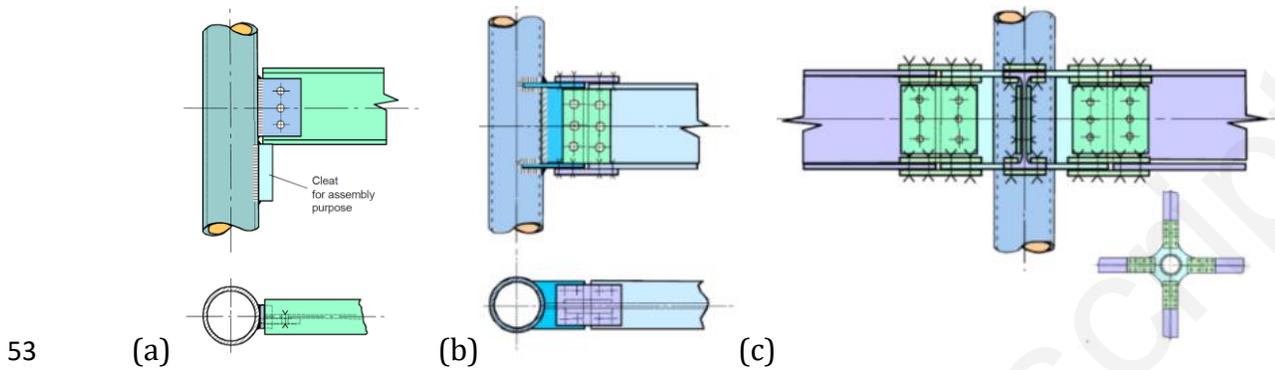
$t_f$	Through I-beam flange thickness
$t_w$	Through I-beam web thickness
$T_f$	Tensile force in the through I-beam flange
$V_{bb}$	Shear corresponding to $M_{pl,Rd,beam}$
$V_{bc}$	Shear corresponding to $M_{pl,Rd,CHS}$
$V_{bj}$	Shear corresponding to $M_{b,Rd,opp}$
$V_{bp}$	Shear corresponding to $M_{bp}$
$V_{bu,opp}$	Joint ultimate strength under LC2
$V_c$	Shear in the CHS column
$V_{joint}$	Shear strength of the joint
$V_n$	Total joint resistance
$V_{pl,Rd,beam}$	Shear resistance of the through I-beam
$V_u$	Effective horizontal shear in the joint panel
$V_{wn}$	Shear strength of the I-beam web
$W_{eff,CHS}$	Effective section modulus of the CHS
$W_{el,CHS}$	Elastic section modulus of the CHS
$W_{el,beam}$	Elastic section modulus of the through I-beam
$W_{pl}$	Plastic section modulus of the through I-beam

37

## 38 1. INTRODUCTION

39 A current area of interest in structural engineering is the search for ways to improve the  
40 connections between hollow and open sections by limiting the complexity inherent to this  
41 joint type. Over the last few decades, engineers worldwide have strived to introduce new  
42 concepts to these joints in order to exploit the outstanding structural and architectural  
43 properties of Hollow Section (HS) profiles. These beneficial properties range from excellent  
44 resistance in terms of compression, tension as well as bending in all directions [1], their  
45 simple application in lightweight structures, the reduced need for fire protection measures  
46 in comparison with equivalent H-section and the possible establishment of composite  
47 behaviour by simply filling the HS columns with concrete. The Committee for International  
48 Development and Education on Construction of Tubular structures (CIDECT) has provided  
49 the necessary design guides [2-5] to design different types of open-to-hollow section joint  
50 connections. Some examples of these are shown in Fig. 1. A number of research studies [6-8]

51 have further extended similar design concepts through a component-based approach, which  
52 has already provided a clearer understanding regarding their structural behaviour.



53 (a) (b) (c)  
54 **Fig. 1.** Examples of (a) one-way nominally pinned, (b) one-way moment resisting and (c) four-  
55 way moment resisting joints between open section and CHS investigated by CIDECT [1]

56  
57 Among several types of steel connections, the I-beam-to-CHS column connection has  
58 often proved to be complicated. In today's industry, I-beam-to-CHS connections are generally  
59 constructed by connecting I-beams to the CHS columns either by direct welding or by  
60 adopting local stiffeners and gusset plates. The first solution, i.e. the direct welding technique,  
61 causes a vulnerability towards severe local distortion on the CHS column and premature  
62 flange fractures. This was observed in a number of components- and full-scale tests done in  
63 a European Coal and Steel Community (ECSC) research program [9]. Schneider and Alostaz  
64 [10] also highlighted the similar issue by testing several directly welded and unstiffened  
65 beam-to-CHS connection prototypes. In order to get rid of such high amount of concentrated  
66 local stresses on the CHS and improve the connections, researches [11-12] adopted the  
67 second solution i.e. using local stiffeners and gusset plates. Wang et al. [11] used outer ring  
68 diaphragms to stiffen the I-beam-to-CHS connections with weak beams or weak columns. The  
69 weak beam joints (i.e. beam resistance - lower than the CHS connection) unexceptionally

70 exhibited final fracture at the link between the diaphragm and the beam flange while the  
71 weak column joints (i.e. CHS connection resistance - lower than the beam) demonstrated  
72 better seismic performance and ductility. Sabbagh et al. [12] investigated similar full strength  
73 I-beam-to-CHS moment-resisting joints for earthquake applications with external diaphragm  
74 plates bolted to the beam and welded to the circumference of the column. The partial  
75 contributions of the web panel and other connection components were highlighted in this  
76 study. The authors recommended avoiding excessive yielding and distortion of the web  
77 panel, as well as large stress concentration in the diaphragms since these can lead to weld  
78 fracture between the diaphragm plates and the column. However, as indicated by several  
79 researches in Japan [13-16], adoption of local stiffeners leads to an excessive welding  
80 quantity, which can cause both economic and practical difficulties during the joint fabrication  
81 and damages the aesthetics of the design. The cost of steel fabrication can account for 30-  
82 40% of the global project budget [17], with joint assemblage consuming the major share.  
83 Using local stiffeners or plates increases the joint complexity of the connection and can  
84 therefore increase this cost to an even higher extent. For this reason, their use in structural  
85 construction has not yet been prevalent even though several design guides and research  
86 studies had been published regarding the I-beam-to-CHS column connections.

87 In order to improve the I-beam-to-CHS column connections, a few types of  
88 connections were studied at a very preliminary stage with a “passing-through” approach in  
89 which steel elements were inserted or embedded through another element consisting of pure  
90 steel or concrete infilled composite hollow section column [18-21]. In 2010, Mirghaderi et al.  
91 [18] depicted the force transfer mechanism of connections constructed by a vertical plate  
92 passing through a Rectangular Hollow Section (RHS) column. In addition, these vertical

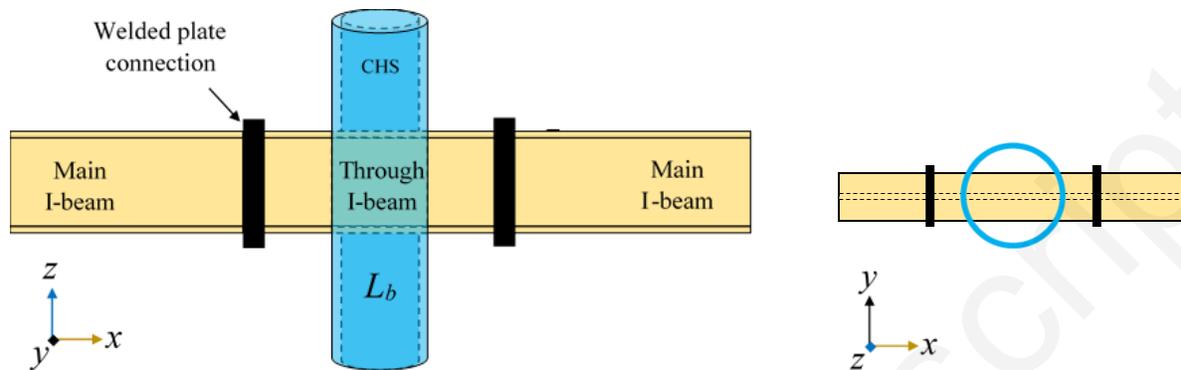
93 plates were welded to the column flanges and the main beams were connected to the through  
94 plate. The authors suggested a design approach to determine the dimensions of the through  
95 plate and other pertinent parts. Voth and Packer [19-20] conducted a comparison study  
96 between T-type conventional (Branch) and “passing-through” plate-to-CHS connections with  
97 experimental as well as numerical investigations and pointed out the advantages offered by  
98 the “passing-through” mechanism. Although, several studies provide substantial knowledge  
99 regarding the “passing-through” approach, most of them use plates as the “through” member.  
100 Therefore, a clear understanding cannot be gathered regarding a “through” I-beam section,  
101 which provides the necessary motivation behind this research study. An exception to this is  
102 found in the work of Alostaz and Schneider [21], which used a girder section as the “through”  
103 member inside a concrete filled CHS column. The moment-rotation behavior of this  
104 configuration was further compared with five different I-beam-to-CHS column connection  
105 types. The authors concluded that continuing the girder through the CHS column provides  
106 the most favorable inelastic connection behavior as it minimizes the local distortions  
107 occurring in the CHS column wall. However, a connection with a concrete filled composite HS  
108 column produces significant differences in the force-transfer mechanism compared to a pure  
109 steel HS column. This limits the available knowledge for the “passing-through” steel  
110 connections between an open and a hollow section. More details about the available  
111 literature was previously documented as a first study [22].

112 A novel moment resisting joint configuration is investigated in this research and is  
113 designated further as the “LASTEICON” connection (Fig. 2). This connection was initially  
114 proposed through a project [23] funded by the European Commission, where an I-beam  
115 passes through the CHS column via laser cut slots made on the column surface and the

116 primary beams (“*main*” I-beams) are connected to both ends of the passing through member  
117 (“*through*” member). The through member (“*Through I-beam*” in Fig. 2a) is welded to the  
118 outer face of the CHS column and connected to the “*main*” I beams by welded plate/beam  
119 splice connections. The applied moment is effectively transferred by the through member to  
120 the CHS column, whereas, the CHS column contributes significantly to the overall resistance  
121 of the connection against the transverse tensile/compressive forces. Although previous  
122 studies used such a “*passing-through*” concept, detailed results reflecting this behavior were  
123 not achieved due to practical difficulties regarding the traditional cutting process, fabrication  
124 of the connection as well as controlling the tolerance issues. However, thanks to the Laser  
125 Cutting Technology (LCT), advantages such as, significant reduction in welding quantities,  
126 swift fabrication process, controlled management of tolerance, better precision and  
127 minimization of human error through computer-programmed automation, facilitated this  
128 investigation in performing an in-depth understanding of the “*passing-through*” I-beam-to-  
129 CHS column connection. Further details regarding the complete fabrication process using  
130 LCT as well as a cost estimation was discussed in a previous article [24] in which the  
131 proposed LASTEICON connection is compared with the conventional connection from an  
132 economic perspective. A detailed description of the laser cutting procedure was also  
133 provided to show its potential in the steel construction sector.

134 This present study investigates the proposed LASTEICON I-beam-to-CHS column  
135 connection using a comprehensive parametric study. Results obtained from detailed  
136 analytical calculations are further validated by FE numerical simulations and preliminary  
137 experimental endorsements. The primary objective is to identify and characterize the

138 behavioral influences caused by each parameter and propose a constructive design approach  
139 for the future designers.



140 *Fig. 2. Schematic diagrams of the proposed LASTEICON connection*

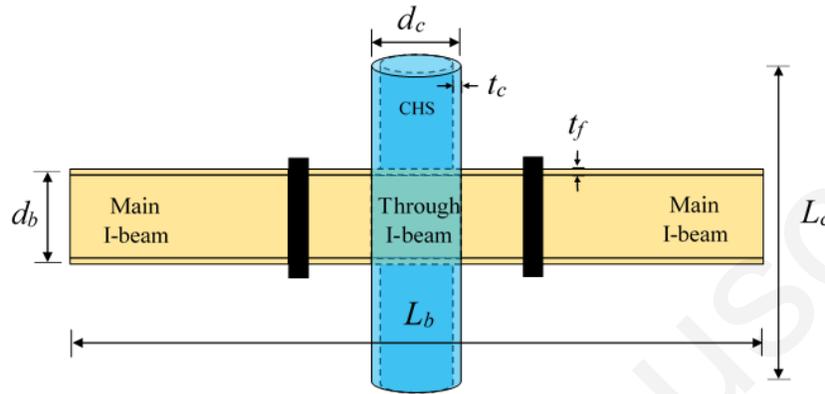
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142 **2. DESIGN APPROACH FOR THE LASTEICON TWO-WAY MOMENT RESISTING JOINTS**  
143 **WITH I-PROFILE PASSING THROUGH A CHS COLUMN**

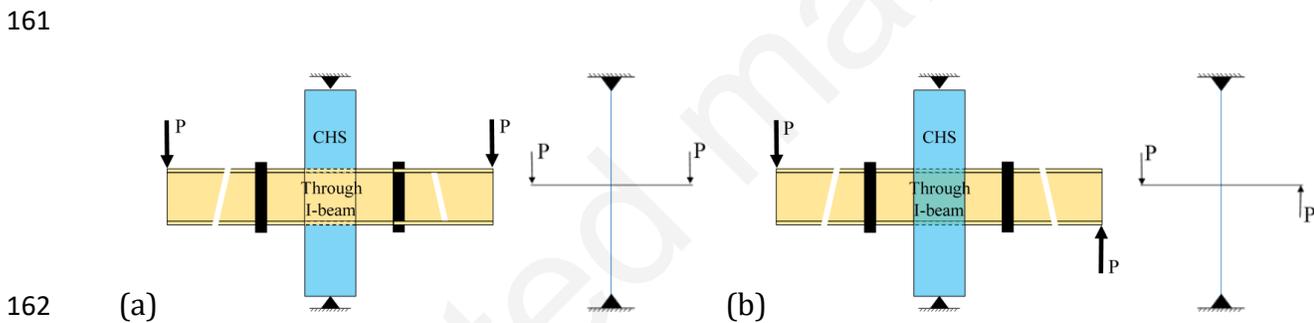
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145 A new design approach was developed for the proposed I-beam-to-CHS column connection  
146 based on detailed FE parametric studies, preliminary experimental validations and  
147 conceptual understandings obtained from the available literature [8, 18, and 25]. Relevant  
148 geometric notations necessary for the design procedure are shown in Fig. 3. Two different  
149 load cases were considered to gather a detailed understanding of the moment connection  
150 behaviour. Load Case 1 (LC1) defines a monotonic loading case with two unidirectional  
151 vertical loads, each acting at the end points of the main beam as shown in Fig. 4a, whereas  
152 Load Case 2 (LC2) designates a monotonic opposite bending load case, where both the loads  
153 are applied in an opposite direction, as shown in Fig. 4b. Design procedures were developed  
154 for each loading scenario. This study primarily focuses on deriving the joint strength of the  
155 “passing-through” zone. Hence, the connections between the “through” member and “main”

156 I-beams were assumed adequately strong in all cases. These connections can however be  
 157 designed according to well-known classical approaches (according to Eurocodes) and are  
 158 therefore not discussed in this study.



159  
 160 **Fig. 3.** LASTEICON I-beam-to-CHS column connection with relevant geometric notations



162 (a) (b)  
 163 **Fig. 4.** (a) LC1-Monotonic gravitational loading, (b) LC2-Monotonic opposite bending loading

164  
 165 **2.1. Design flexural strength of the LASTEICON connection**

166 As the through members contribute significantly to the strength of the proposed LASTEICON  
 167 “passing-through” connections, a different force-transfer mechanism was identified for each  
 168 loading scenario, LC1 and LC2, in comparison with unstiffened and conventionally welded I-  
 169 beam-to-CHS connections.

170

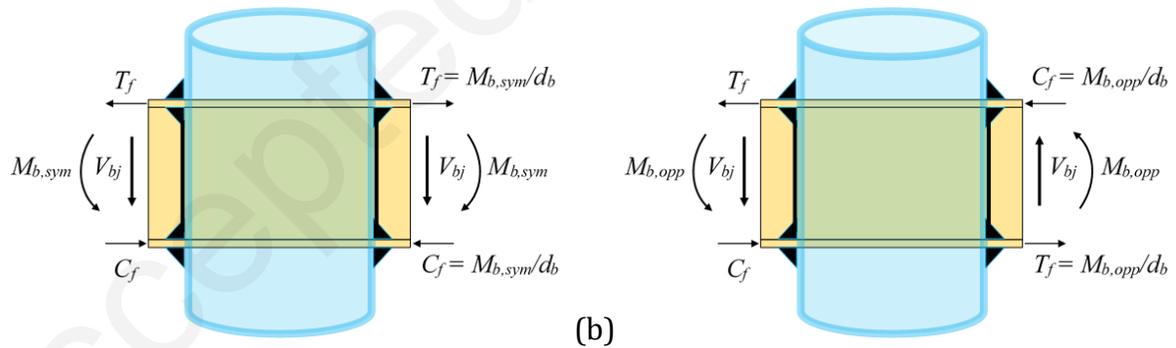
171 **2.1.1 LC1: Monotonic gravitational loading**

172 Under a gravitational or symmetric loading, the moments developed at either face of the CHS  
 173 column connection ( $M_{b,sym}$ ) cancel each other out (Fig. 5a) leading to a ‘rigid body’ like  
 174 behavior of the joint. As a result, the resistance of the joint depends solely on the flexural  
 175 resistance of the through I-beam just outside the CHS column. Therefore,

176 
$$M_{b,Rd,sym} = M_{pl,Rd,beam} \quad (1)$$

177 Where,  $M_{b,Rd,sym}$  is the joint flexural strength under symmetric loading,  $M_{pl,Rd,Beam} = \frac{W_{pl} f_{yb}}{\gamma_{M0}}$ ,  
 178 is the flexural resistance of the I-beam section obtained from EN 1993-1-1 [26],  $W_{pl}$  is the  
 179 plastic section modulus of the I-beam,  $f_{yb}$  is the material yield stress of the I-beam, and  $\gamma_{M0}$  is  
 180 the partial safety factor for cross section resistance. In order to reflect the nominal predicted  
 181 strength and allow a direct comparison between the numerical and experimental outcomes,  
 182  $\gamma_{M0}$  was taken as 1 for the analytical calculations.

183



184 (a) (b)  
 185 **Fig. 5.** (a) Schematic diagram of forces acting at the joint panel under symmetric loading (LC1)  
 186 (b) Schematic diagram of forces acting at the joint panel under antisymmetric loading (LC2)

187  
 188  
 189

190 **2.1.2 LC2: Monotonic opposite bending loading**

191 Under an opposite or antisymmetric loading condition (Fig. 4b), the moment transfer  
192 mechanism could be visualized from the free body diagram illustrated in Fig. 5b. The beam  
193 moment demand can be resolved into flange forces, tensile force in the beam flange,  $T_f$  and  
194 compressive force in the beam flanges,  $C_f$ . Assuming that the bending moment is carried  
195 entirely by the furthest fibers of the flanges, the tensile and compressive forces in the beam  
196 flange,  $T_f$  and  $C_f$ , can be estimated as:

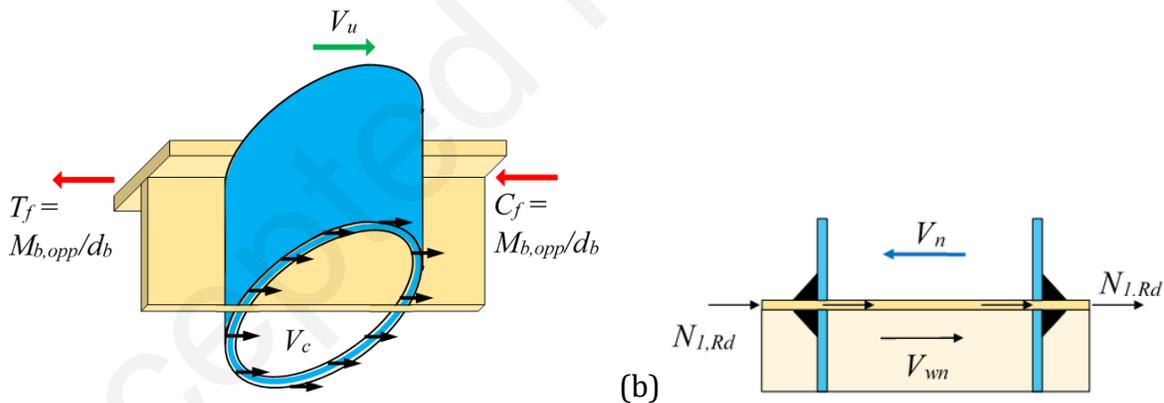
197 
$$T_f = C_f = \frac{M_{b,opp}}{d_b} \quad (2)$$

198 Where,  $M_{b,opp}$  is the moment demand at either side of the connection due to opposite bending  
199 loading and  $d_b$  is the total depth of the beam (Fig. 3a). The column shear ( $V_c$ ) transferred  
200 through the joint increases the joint shear strength by reducing the beam flange forces  
201 transferred to the joint. As a result, the effective horizontal shear force acting on the joint  
202 panel,  $V_u$ , can be written as,

203 
$$V_u = \frac{2M_{b,opp}}{d_b} - V_c \quad (3)$$

204 The numerical studies (discussed in Sections 3 and 4) showed that the beam flange forces  
205 and column shear transferred through the joint produce large shear forces in the through I-  
206 beam web as well as a substantial amount of transverse tensile and compressive forces on  
207 the CHS chord face at the flange connection zones. The newly proposed design procedure was  
208 therefore developed based on the shear resistance of the through I-beam web and the  
209 transverse tensile/compressive resistance offered by the CHS chord face. Fig. 6 shows an  
210 isolated portion of the top flange within the connection panel where the axial flange forces,  
211  $T_f$  and  $C_f$ , tend to push the beam flange through the column. From the equilibrium of the

212 horizontal forces shown in Fig. 6, the horizontal shear force in the joint is resisted by the  
 213 shear strength of the I-beam web,  $V_{wn}$ , and the in-plane design moment resistance of the CHS  
 214 column wall,  $M_{ip,1,Rd}$ , which is defined according to EN 1993-1-8, Table 7.4 for X-type joints  
 215 [27] and is derived from the transverse tensile/compressive resistance of the CHS chord face,  
 216  $N_{1,Rd}$ . However, as  $M_{ip,1,Rd}$  was recommended for the Branch-type (conventional) connections,  
 217 the calculated values were doubled to associate the increased resistance provided by the  
 218 passing through elements as suggested by the latest draft of EN 1993-1-8, Table 9.4 [28] for  
 219 passing through connections. Compared to the conventional connections where only the  
 220 outer wall of the CHS column provides the resistance against transverse tensile or  
 221 compressive forces, it can be observed through the LASTEICON connections that, the inner  
 222 wall also offers resistance through the passing through elements therefore doubling the  
 223 resistance of this connection.



224 (a) (b)  
 225 **Fig. 6.** (a) Horizontal forces acting at the joint under opposite bending loading (LC2), (b) 2D  
 226 view of the passing through flange plates inside the CHS column illustrating the active  
 227 resistances

228

229 The complete joint shear capacity is reached when all the contributing mechanisms have  
230 reached their individual shear strengths. Thus, the total joint resistance,  $V_n$ , is calculated as  
231 the sum of the individual nominal shear strengths of the contributing mechanisms (Eq. 4).

$$232 \quad V_n = (V_{wn} + V_{cn}) \quad (4)$$

233 Where,

$$234 \quad V_{wn} = 0.6f_{yw}d_c t_w \quad (5)$$

$$235 \quad V_{cn} = \frac{M_{ip,1,Rd}}{d_b} \quad (6)$$

236  $f_{yw}$  is the material yield stress for the I-beam web and  $t_w$  is the thickness of the I-beam web.  
237 The web shear yield is calculated based on an average yield shear stress of  $0.6f_{yw}$  acting over  
238 the web area within the joint panel. The corresponding safety factors ( $\gamma_{M0}$  and  $\gamma_{M5}$ ) in Eqs. 5  
239 and 6 respectively, was taken as 1 for the analytical calculations to reflect on the nominal  
240 predicted strength. Therefore, the flexural resistance of the LASTEICON joint can be  
241 calculated as,

$$242 \quad M_{b,Rd,opp} = (V_n + V_c) \frac{d_b}{2} \quad (7)$$

243

### 244 **2.1.3 Checks for additional failure modes**

245 Aside than the flexural failure of the joint, three additional failure modes can occur as local  
246 distortions due to the bending forces. In order to avoid such undesired failure at the joint,  
247 three checks are needed which are described below. However, as the joint shows a rigid like  
248 behaviour under the gravitational loading, LC1, these checks are only necessary for LC2.

249

250

251

252 **Check 1: Check for flexural resistance of main beams:**

253 For smaller sections, the through member might prove to be weaker than the joint panel and  
254 can thus lead to failure of the whole system due to flexural plasticity just outside the CHS  
255 column. To avoid such kind of a failure,

256 
$$M_{b,Rd,opp} < M_{pl,Rd,Beam} \quad (8)$$

257 Where,  $M_{pl,Rd,Beam} = \frac{W_{pl}f_{yb}}{\gamma_{m0}}$ , is the flexural resistance of the I-beam section [26].

258

259 **Check 2: Check for local buckling of CHS column:**

260 Conditions to avoid premature local buckling are described below. Firstly, the CHS column  
261 sections should be classified according to Table 5.2 of EN 1993-1-1 based on the diameter-  
262 to-thickness ratio of the CHS. Now, as Class 3 and Class 4 hollow sections are deemed  
263 susceptible to local buckling [29], their flexure resistance,  $M_{Rd,CHS}$ , is derived from the  
264 following equations.

265 For Class 3 sections: 
$$M_{Rd,CHS} = \frac{W_{el,CHS}f_{yc}}{\gamma_{m0}} \quad (9)$$

266 For Class 4 sections: 
$$M_{Rd,CHS} = \frac{W_{eff,CHS}f_{yc}}{\gamma_{m0}} \quad (10)$$

267 Where,  $f_{yc}$  is the material yield stress for the CHS column,  $W_{el,CHS}$  and  $W_{eff,CHS}$  are respectively  
268 defined as the elastic section modulus and effective section modulus of the CHS according to  
269 EN 1993-1-1. To avoid a failure due to local buckling of the CHS,

270 
$$M_{b,Rd,opp} < M_{Rd,CHS} \frac{(L_b - d_c)L_c}{(L_c - d_b)L_b} \quad (11)$$

271 Where,  $L_b$  is the total length of the beam,  $d_c$  is the external diameter of the CHS column, and  
272  $L_c$  is the total length of the CHS column as shown in Fig. 3a. It is recommended to avoid slender  
273 CHS columns (Class 3 and Class 4 hollow sections) in the LASTEICON joints. This can be done

274 simply by using the first step of this design check i.e. classification according to the EN 1993-  
275 1-1.

276

277 **Check 3: Check for punching shear failure:**

278 A further check is also suggested for the conventional I-beam-to-CHS joints as specified in EN  
279 1993-1-8 [27] and CIDECT guidelines [8] to avoid punching shear failure. This check is also  
280 included for these LASTEICON joints to offer additional safety. According to the available  
281 design guidelines, the check is only needed if,

282 
$$b \leq d_c - 2t_c \quad (12)$$

283 and if required, the following restriction should be respected.

284 
$$f_b t_f \leq 1.16 f_{yc} t_c \quad (13)$$

285 Where,  $b$  is the through I-beam flange width,  $t_c$  is the CHS thickness,  $t_f$  is the I-beam flange  
286 thickness and  $f_b$  is the stress at which punching shear occurs on the CHS column wall.

287 Therefore, as  $f_b$  is induced due to the bending moment,  $M_{bp}$ ,

288 
$$M_{bp} = f_b W_{el,beam} \quad (14)$$

289 Where  $W_{el,beam}$  is the elastic section modulus of the through I-beam. Therefore, to avoid failure  
290 due to punching shear in the CHS column wall,

291 
$$M_{b,Rd,opp} < M_{bp} \quad (15)$$

292

293 **2.1.4 Correlation to the global configuration**

294 The above-mentioned design procedure determines the resistance of the passing through  
295 joint from a local perspective. However, in order to correlate the design procedure to the  
296 numerical and experimental prototypes and further compare the analytical results with the

297 numerical simulations, the joint strengths should be calculated in terms of the shear force  
 298 developed due to the vertical loads acting at the extremities of the main I-beams. Therefore,  
 299 if  $P$  is the vertical load at the free end of the main I-beam and  $V_{bj}$  is the corresponding shear  
 300 developed on the beam at the location of the CHS column face (Fig. 5),

$$301 \quad P = V_{bj} = \frac{M_b}{(L_b - d_c)/2} \quad (16)$$

302 Where,  $M_b$  is equal to  $M_{b,sym}$  under LC1 and  $M_{b,opp}$  under LC2. In the through beam connection  
 303 detail, it is reasonable to consider that the entire column shear force is effectively reducing  
 304 joint shear forces since the column is continuous through the joint and is directly attached to  
 305 the beam through proper welds. This makes the joint strength dependent on the global  
 306 configuration and the column shear ( $V_c$ ), which can be calculated from,

$$307 \quad V_c = P \frac{L_b}{L_c} = \frac{M_b}{(L_b - d_c)} \frac{2L_b}{L_c} \quad (17)$$

308 Therefore, the joint strength derived in Eq. 7 can be rewritten as,

$$309 \quad M_{b,Rd,opp} = \frac{(V_{wn} + V_{cn})}{\left[ \frac{2}{d_b} - \frac{2L_b}{L_c} \left( \frac{1}{L_b - d_c} \right) \right]} \quad (18)$$

310 Furthermore, it can be derived in terms of  $V_{bj}$  following Eq. 16. Similar expressions were also  
 311 derived in terms of shear corresponding to the moments for all three checks. Therefore, Eqs.  
 312 8, 11 and 15 can be rewritten as,

$$313 \quad V_{bj} \left( = \frac{M_{b,Rd,opp}}{(L_b - d_c)/2} \right) < V_{bb} \left( = \frac{M_{pl,Rd,Beam}}{(L_b - d_c)/2} \right) \quad (19)$$

$$314 \quad V_{bj} \left( = \frac{M_{b,Rd,opp}}{(L_b - d_c)/2} \right) < V_{bc} \left( = \frac{M_{Rd,CHS}}{(L_c - d_b)/2} \left( \frac{L_c}{L_b} \right) \right) \quad (20)$$

$$315 \quad V_{bj} \left( = \frac{M_{b,Rd,opp}}{(L_b - d_c)/2} \right) < V_{bp} \left( = \frac{M_{bp}}{(L_b - d_c)/2} \right) \quad (21)$$

316 As a detailed parametric study is presented in this investigation, the minimum value of all  
317 four  $V_b$  values is considered as the shear force corresponding to the ultimate strength of the  
318 LASTEICON joint. Therefore, to identify the probable failure mode,  $V_{bu,opp}$  is taken as,

$$319 \quad V_{bu,opp} = \min(V_{bj}, V_{bb}, V_{bc}, V_{bp}) \quad (22)$$

320

321

## 322 **2.2 Design shear strength of the LASTEICON connection**

323 The shear strength of the “passing-through” joint can be determined from the shear strength  
324 of the through I-beam. According to EN 1993-1-1, Clause 6.2.6, it can be calculated as,

$$325 \quad V_{joint} = V_{pl,Rd,beam} = \frac{A_{vb}f_{yb}}{\sqrt{3}\gamma_{m0}} \quad (23)$$

326 Where,  $A_{vb}$  is the shear area of the through I-beam.

327 Table 1 lists all the parametric variations along with their ultimate flexural and shear  
328 strength of the joint calculated according to this design procedure. Values corresponding to  
329 all three checks are also provided to show the failure predictions made by the proposed  
330 design procedure.

331 **Table 1:** Analytical values corresponding to the joint strength,  $M_{b,Rd,opp}$  and  $V_{bj}$ , all three checks,  $V_{bb}$ ,  $V_{bc}$  and  $V_{bp}$  and ultimate joint  
 332 strength,  $V_{bu,opp}$ <sup>2</sup>

Varying Parameters	Joint Flexural Strength		Joint Shear Strength	Check 1: Beam plasticity	Check 2: Local buckling	Check 3: Punching shear			Joint Ultimate Strength	Failure Modes
	$M_{b,Rd,opp}$ (kNm)	$V_{bj}$ (kN)	$V_{joint}$ (kN)	$V_{bb}$ (kN)	$V_{bc}$ (kN)	$f_b$ (MPa)	$M_{bp}$ (kNm)	$V_{bp}$ (kN)	$V_{bu,opp}$ (kN)	
For Beam Section variation (IPE)										
IPE 220	116.3	50.1	325.9	43.6	198.9	475.3	119.8	51.6	43.6	Beam Flexure
IPE 270	161.0	69.3	453.0	74.0	203.7	428.7	183.9	79.2	69.3	Joint Panel Shear
IPE 330	225.9	97.3	631.3	122.9	209.7	380.3	271.1	116.8	97.3	Joint Panel Shear
IPE 400	316.3	136.2	875.2	199.8	217.3	323.9	375.8	161.8	136.2	Joint Panel Shear
IPE 500	475.7	204.8	1227.7	335.4	229.1	273.3	527.5	227.2	204.8	Joint Panel Shear
For CHS Thickness variation ( $t_c$ )										
4.0	184.7	79.5	875.2	199.8	69.9	129.6	150.3	64.7	64.7	CHS Punching
6.0	216.0	93.0	875.2	199.8	103.0	194.4	225.5	97.1	93.0	Joint Panel Shear
8.0	259.9	111.9	875.2	199.8	135.1	259.2	300.6	129.5	111.9	Joint Panel Shear
10.0	316.3	136.2	875.2	199.8	217.3	323.9	375.8	161.8	136.2	Joint Panel Shear
12.5	404.4	174.1	875.2	199.8	267.8	404.9	469.7	202.3	174.1	Joint Panel Shear
For CHS Diameter variation ( $d_c$ )										
273.0	319.6	135.2	875.2	196.3	125.9	323.9	375.8	159.0	125.9	Local Buckling
323.9	313.0	133.9	875.2	198.4	179.3	323.9	375.8	160.7	133.9	Joint Panel Shear
355.6	316.3	136.2	875.2	199.8	217.3	323.9	375.8	161.8	136.2	Joint Panel Shear
406.4	327.4	142.5	875.2	202.0	285.9	323.9	375.8	163.6	142.5	Joint Panel Shear
457.0	342.6	150.8	875.2	204.3	279.3	323.9	375.8	165.4	150.8	Joint Panel Shear

333

<sup>2</sup> Reference configuration chosen for parametric studies: IPE400 section passing through a CHS column with diameter,  $d_c = 355.6$  mm and thickness,  $t_c = 10.0$  mm, total column length,  $L_c = 2340.0$  mm, total beam length,  $L_b = 5000$  mm, material yield strength for the beams,  $f_{yb} = 355$  Mpa, and CHS columns,  $f_{yc} = 377$  Mpa, and IPE 400 as the "main" I-beam sections.

334 **Table 1:** Analytical values corresponding to the joint strength,  $M_{b,Rd,opp}$  and  $V_{bj}$ , all three checks,  $V_{bb}$ ,  $V_{bc}$  and  $V_{bp}$  and ultimate joint  
 335 strength,  $V_{bu,opp}$ <sup>3</sup> (continued..)

Varying Parameters	Joint Flexural Strength		Joint Shear Strength	Check 1: Beam plasticity	Check 2: Local buckling	Check 3: Punching shear			Joint Ultimate Strength	Failure Modes
	$M_{b,Rd,opp}$ (kNm)	$V_{bj}$ (kN)	$V_{joint}$ (kN)	$V_{bb}$ (kN)	$V_{bc}$ (kN)	$f_b$ (MPa)	$M_{bp}$ (kNm)	$V_{bp}$ (kN)	$V_{bu,opp}$ (kN)	
For CHS and Beam Material variation ( $f_{yc}$ & $f_{yb}$ )										
275.0 & 275.0	237.9	102.5	678.0	154.8	158.5	236.3	274.1	118.0	102.5	Joint Panel Shear
355.0 & 355.0	307.1	132.3	875.2	199.8	204.6	305.0	353.8	152.4	132.3	Joint Panel Shear
440.0 & 440.0	380.7	163.9	1084.7	247.6	253.6	378.1	438.6	188.9	163.9	Joint Panel Shear
355.0 & 377.0	316.3	136.2	875.2	199.8	217.3	323.9	375.8	161.8	136.2	Joint Panel Shear
355.0 & 440.0	342.5	147.5	875.2	199.8	253.6	378.1	438.6	188.9	147.5	Joint Panel Shear
440.0 & 355.0	345.4	148.7	1084.7	247.6	204.6	305.0	353.8	152.4	148.7	Joint Panel Shear
For varying Moment-to-shear (M/V) ratio by varying beam length ( $L_b$ )										
2500.0	322.3	300.6	875.2	432.7	434.6	323.9	375.8	350.5	300.6	Joint Panel Shear
3400.0	319.0	209.5	875.2	304.8	319.6	323.9	375.8	246.9	209.5	Joint Panel Shear
5000.0	316.3	136.2	875.2	199.8	217.3	323.9	375.8	161.8	136.2	Joint Panel Shear
6600.0	315.0	100.9	875.2	148.6	164.6	323.9	375.8	120.4	100.9	Joint Panel Shear
7500.0	314.5	88.0	875.2	129.9	144.9	323.9	375.8	105.2	88.0	Joint Panel Shear

336

<sup>3</sup> Reference configuration chosen for parametric studies: IPE400 section passing through a CHS column with diameter,  $d_c = 355.6$  mm and thickness,  $t_c = 10.0$  mm, total column length,  $L_c = 2340.0$  mm, total beam length,  $L_b = 5000$  mm, material yield strength for the beams,  $f_{yb} = 355$  Mpa, and CHS columns,  $f_{yc} = 377$  Mpa, and IPE 400 as the "main" I-beam sections.

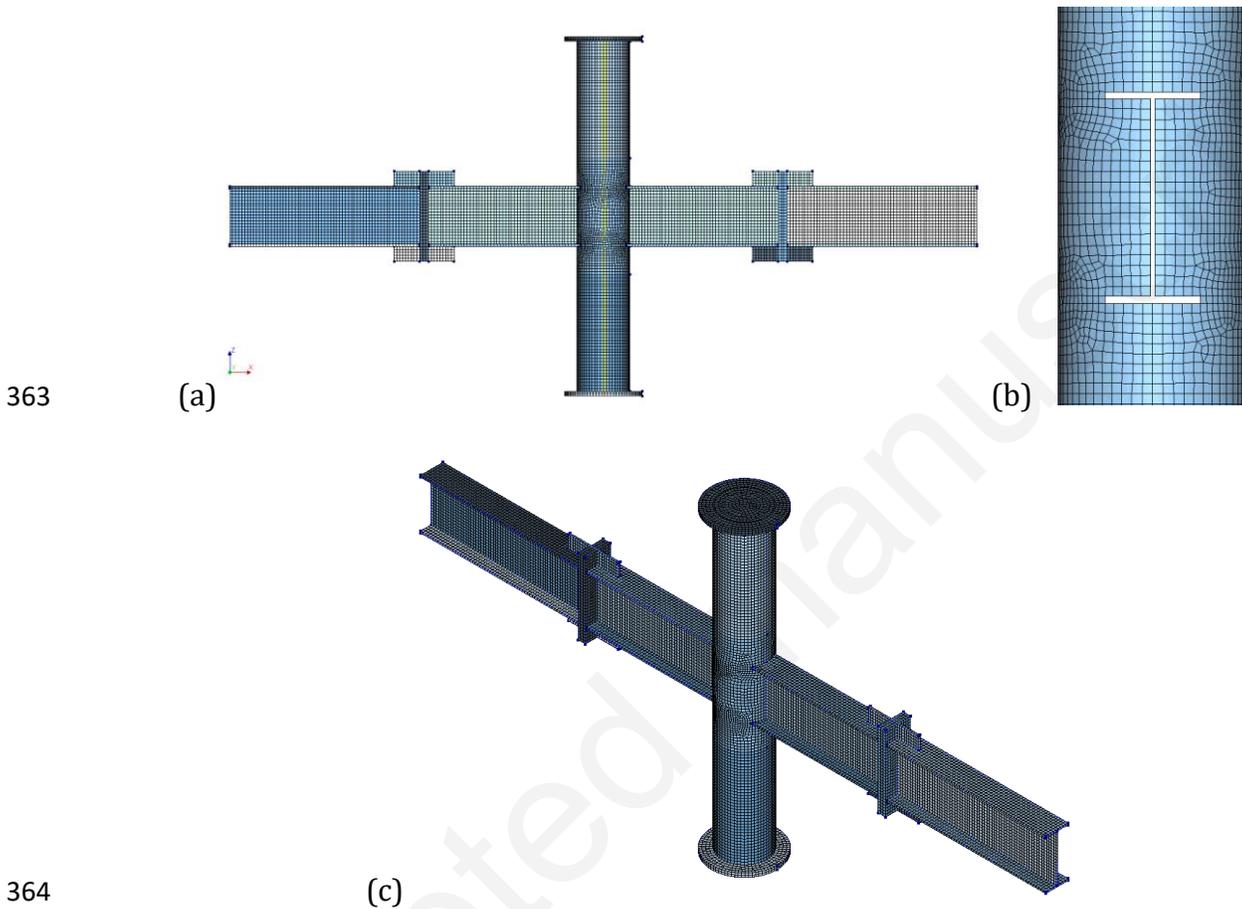
### 337 3. MODELLING APPROACH AND EXPERIMENTAL VALIDATION OF THE NUMERICAL 338 MODELS

339  
340 This section presents the modelling techniques adopted to build the numerical prototypes in  
341 the finite element software DIANA 10.2 [30] and discusses their validation with respect to  
342 the relevant experimental investigations obtained in the LASTEICON project.

#### 343 344 *Modelling assumptions and Finite Element (FE) models*

345 The configurations were modelled using 3D geometries and solid elements such as CHX60,  
346 CTP45 and CTE30 [31]. The laser cut slots on the CHS column surface were taken into account  
347 to allocate the through I-beam and allow for the necessary reduction in CHS stiffness. As  
348 mentioned in Section 2, to avoid any secondary connection failure and rather focus on the I-  
349 beam-to-CHS “passing-through” zone, the slots in the FE models were defined assuming a  
350 zero spacing tolerance, thus connecting the CHS column and the “through” beam with a  
351 perfectly welded connection. To avoid heavy and complicated numerical models and save  
352 computation time, welds were not modelled explicitly and the members were connected  
353 through common nodes. Fig. 7 shows some illustrations of the numerical model. Two  
354 different load cases were considered where both vertical loads were incremented  
355 simultaneously in each analysis step. Geometric nonlinearity was considered in the  
356 numerical simulations. Material nonlinearity was introduced in the models through actual  
357 stress strain relationships obtained from the coupon tests on the experimental prototypes.  
358 The material yield strength for the I-beam,  $f_{yb}$ , and for the CHS column,  $f_{yc}$ , was found to be  
359 355MPa and 377MPa respectively. Furthermore, in-built material models in DIANA 10.2

360 were used according to Table 3.1 (EN 10025-2), EN 1993-1-1, to compare the variations in  
361 the connection behaviour due to different steel grades, when used for either the CHS column  
362 or the through beams.



365 **Fig. 7.** Examples of the numerical model meshed in DIANA 10.2, (a) frontal view of the complete  
366 connection configuration, (b) slots on the CHS column to accommodate the through beam, (c)  
367 isometric view of the configuration.

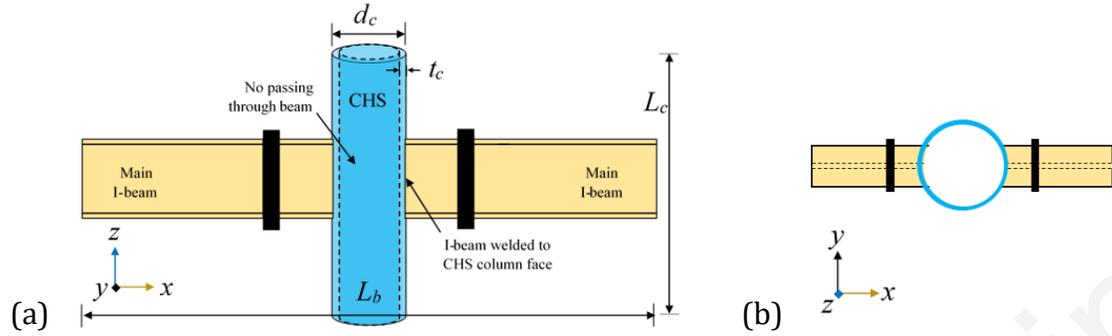
### 368 **Experimental Validation of the Numerical FE Models**

369 A preliminary experimental campaign was conducted by INSA, Rennes [32] to validate the  
370 numerical model for both load cases. Two additional solid circular plates, with 30 mm  
371 thickness and 520 mm diameter, were connected to each extremity of the CHS column and  
372

373 was pinned by rollers following the boundary conditions shown in Fig. 4. Bracings were  
374 placed to limit the lateral torsional buckling of the beam. These additional plates and bracings  
375 were also considered in the numerical models to have an exact replica of the experimental  
376 specimens and thus provide an appropriate validation. Further details about the test set-up  
377 can be found in the LASTEICON experimental report provided by INSA, Rennes [32]. Two  
378 load-jacks of 1500 KN capacity, were applied at the extremities of the main beams at a  
379 distance of 2500 mm from the axis of the CHS column for LC1. This distance was reduced to  
380 1700mm for LC2 to allow for a larger rotation of the node (joint panel) at failure. The  
381 monotonic loadings were applied in three steps: (i) Application of 50% of the theoretical  
382 design resistance evaluated with nominal mechanical characteristics, and unloading, (ii)  
383 Application of 100% of the design resistance and unloading, and finally (iii) Loading until  
384 failure of the joint or the beam. Inclinometers and LVDTs were placed in necessary locations  
385 to measure the vertical and horizontal displacements at specific positions of the joint  
386 configuration. Three specific connection configurations (see Table 2) were investigated in  
387 the experimental campaign: two different passing-through LASTEICON connection  
388 configurations (one for each load case, LC1 and LC2) and one conventional I-beam-to-CHS  
389 connection configuration (Fig. 8) under LC1 without any “passing-through” mechanism. The  
390 beams were directly welded to the CHS column surface for the third test (i.e. the conventional  
391 configuration). Experimental and numerical results were compared through force-  
392 displacement curves (for an extremity of the beam, where the vertical force is increased  
393 systematically) and failure modes. The material stress-strain curves obtained from the  
394 coupon tests (and adopted for the numerical models) are plotted in Fig. 9a. Very good  
395 agreement was found between the experimental (\_Exp.) and numerical (\_Num.) results in

396 terms of initial stiffness and ultimate resistance of the LASTEICON as well as the conventional  
397 joints as shown in Fig. 9b.

398         The experiments regarding LASTEICON\_LC1 and Conventional\_LC1, characterized by  
399 a very ductile behaviour, were stopped at an arbitrary point before reaching the  
400 displacement capacity of the loading system. A similar approach was followed for the  
401 numerical models for these two cases, i.e. the numerical simulations were stopped when the  
402 displacement was found as large enough to get relevant and useful information on the  
403 behaviour of the joint. On the other hand, the experiment regarding LASTEICON\_LC2 failed  
404 in a less ductile manner due to damage in the connection zone (tearing of the CHS column  
405 surface) between the through I-beam flange and the CHS column. To identify “failure” in the  
406 numerical FE-models, the accumulated plastic strains were compared to a limit value, which  
407 were calibrated by the test results. A similar approach has been used in different research  
408 studies [18, 33] to validate the numerical models against experimental results. For each  
409 tested specimen and its corresponding FE-model, the location and the values of the  
410 accumulated plastic strains were determined from the simulations at the deformation stage  
411 of the tested specimen corresponding to the first visually detected failure (i.e. I-beam flange  
412 plasticity in case of LASTEICON\_LC1, CHS wall tearing in case of LASTEICON\_LC2 and CHS  
413 wall crushing in case of Conventional\_LC1). For this particular study, the limit value for  
414 equivalent plastic strain was considered equal to 6.5%.

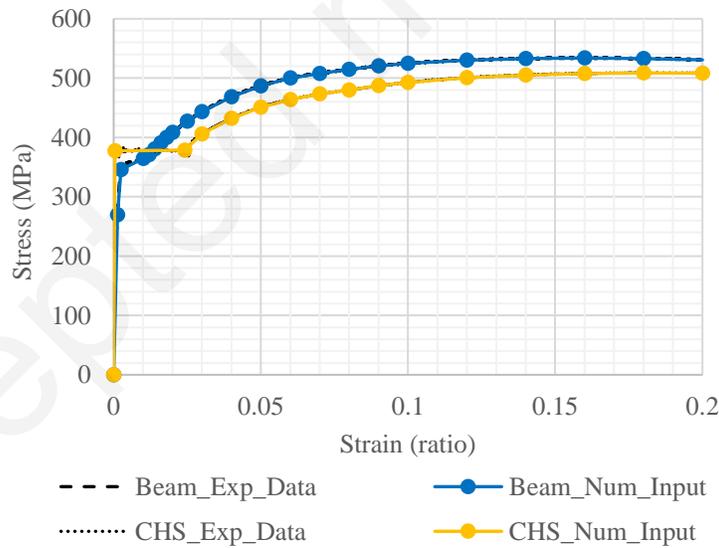


415 **Fig. 8.** Schematic diagram of a conventional open-to-CHS connection (a) frontal view and (b)  
 416 top view

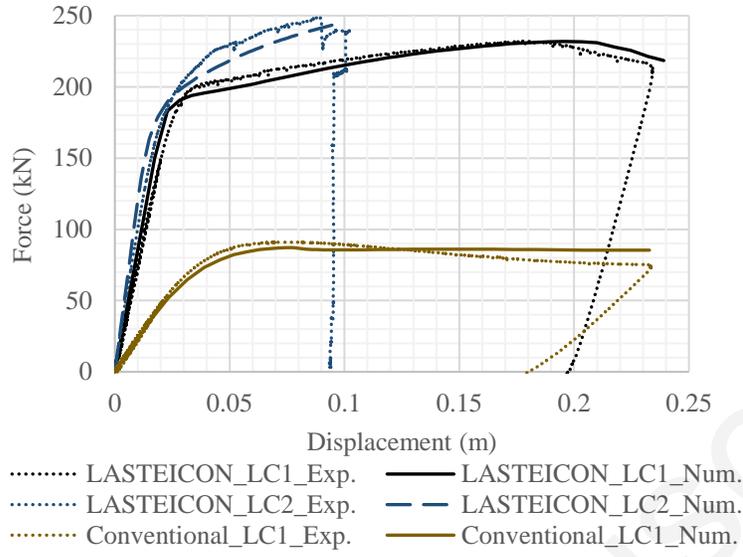
417 **Table 2:** Specimens tested in the Preliminary Experimental Campaign

Configuration Type	Loading Scenario	Specimen Name	Beam (IPE)	$L_b$ (mm)	$d_c$ (mm)	$t_c$ (mm)	$L_c$ (mm)
LASTEICON	LC1	LASTEICON_LC1	IPE 400	5000.0	355.6	8.8	2340.0
LASTEICON	LC2	LASTEICON_LC2	IPE 400	3400.0	355.6	10.0	2340.0
Conventional	LC1	Conventional_LC1	IPE 400	5000.0	355.6	10.0	2340.0

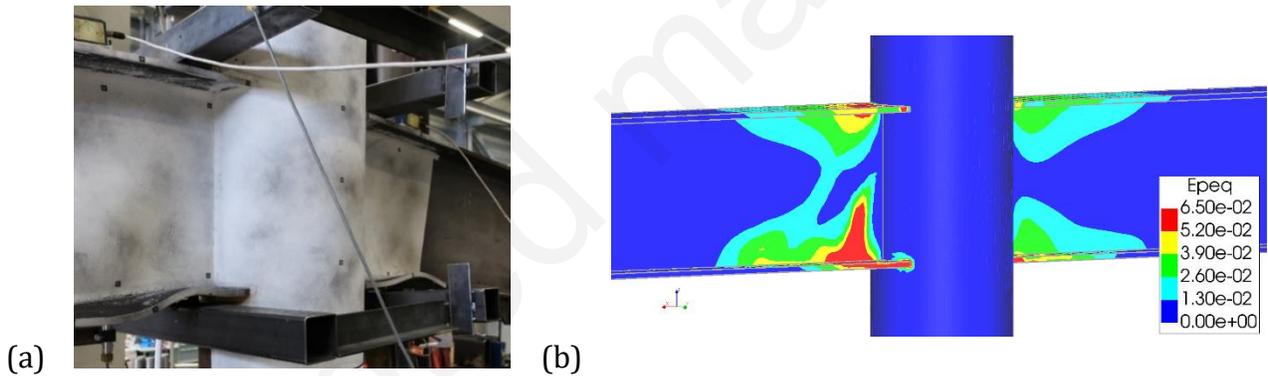
418



419



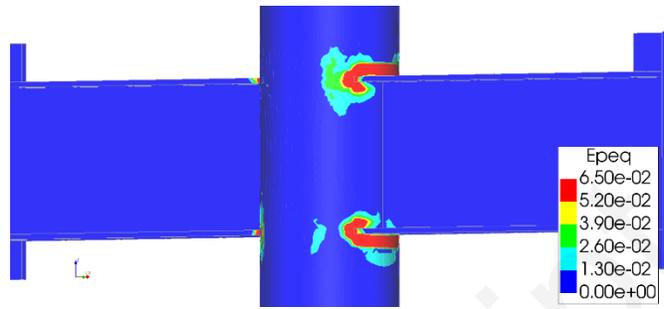
421 **Fig. 9.** (a) Actual stress-strain relationship obtained for S355 from the experimental tests, (b)  
 422 comparison of force-displacement curves between numerical and experimental results



424 **Fig. 10.** LASTEICON\_LC1 Failure: (a) flange buckling of beam flanges in compression observed  
 425 from experiments, (b) von Mises equivalent plastic strains in the numerical model

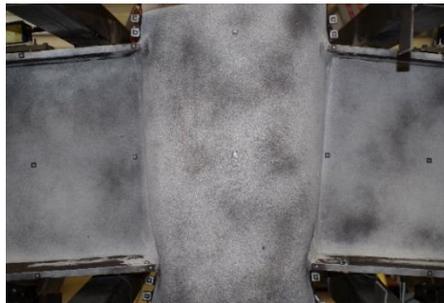


(a)

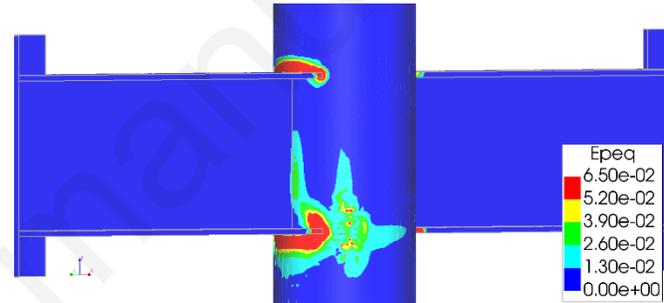


(b)

426  
427 **Fig. 11.** LASTEICON\_LC2 Failure: (a) beam-to-column connection tearing observed from  
428 experiments, (b) von Mises equivalent plastic strains in the numerical model



(a)



(b)

429  
430 **Fig. 12.** Conventional\_LC1 Failure: (a) CHS column surface in compression observed from  
431 experiments, (b) von Mises equivalent plastic strains in the numerical model

432  
433 Similar failure modes were obtained from the numerical and experimental studies for  
434 all three configurations (Fig. 10-12). The failure due to beam flange plasticity is clearly visible  
435 for the LASTEICON\_LC1 specimen, where the buckled I-beam flanges of the experimental  
436 prototype (Fig. 10a) validates the strain concentration (Fig. 10b) on the I-beam flanges just  
437 outside the CHS column. Failure in the LASTEICON\_LC2 specimen occurred due to a tearing  
438 of the CHS column surface at the connection zone between the CHS column and the through  
439 flanges (Fig. 11a). Similar failure behaviour was obtained in the numerical models as the

440 limiting plastic strains developed around the connection zone (Fig. 11b). This phenomenon  
441 however occurred due to a shear failure of the through I-beam web and transverse tensile  
442 failure of the CHS column wall. It is explained in Section 4 with more details. A good  
443 agreement was also obtained regarding the failure mode in the conventional joint  
444 configuration under LC1 (Fig. 12), as the CHS wall crushing under compression was emulated  
445 by the high plastic strain concentration on the CHS column wall at the I-beam-to-CHS  
446 connection zones in the numerical models.

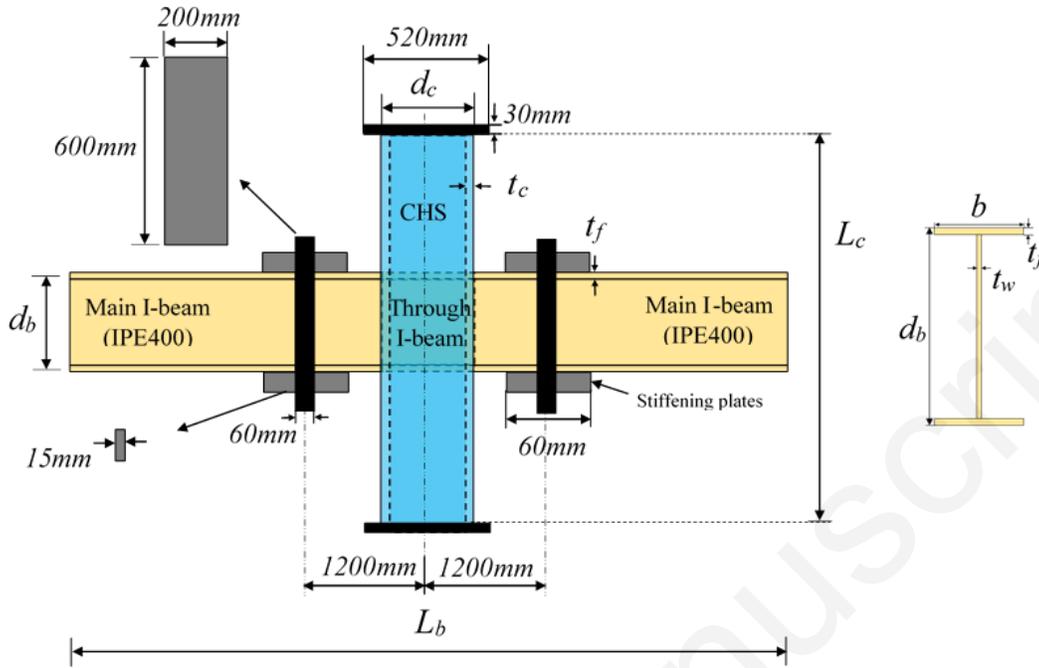
#### 447

#### 448 **4. PARAMETRIC STUDY ON THE LASTEICON CONFIGURATION BASED ON NONLINEAR**

#### 449 **STATIC ANALYSIS**

450

451 Five parameters were principally identified to have a significant influence on the ultimate  
452 joint strength and were thus varied in this study. The chosen parameters are: (1) through I-  
453 beam section (*IPE*), (2) CHS column thickness ( $t_c$ ), (3) CHS column diameter ( $d_c$ ), (4) Material  
454 properties for both CHS and through I-beam ( $f_{yb}$  and  $f_{yc}$ ), and (5) Moment-to-shear ( $M/V$ )  
455 ratio. The reference configuration, chosen as a starting point for the parametric study,  
456 consisted of an IPE400 section passing through a CHS column with 355.6mm diameter ( $d_c$ )  
457 and 10mm thickness ( $t_c$ ) with a total beam length ( $L_b$ ) of 5000mm (see Fig. 13). Results for  
458 this particular configuration is presented using a solid blue line in all the force-displacement  
459 plots. All the parametric studies, except the one related to material variation, were done with  
460 the experimental material properties (Fig. 9a).

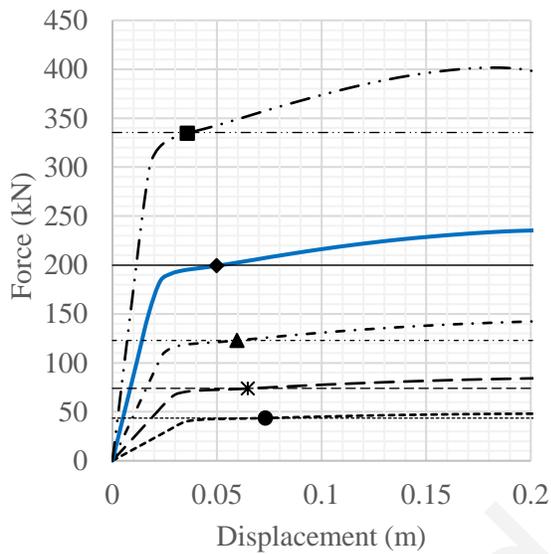


**Fig. 13.** All parametric dimensions for the LASTEICON configuration

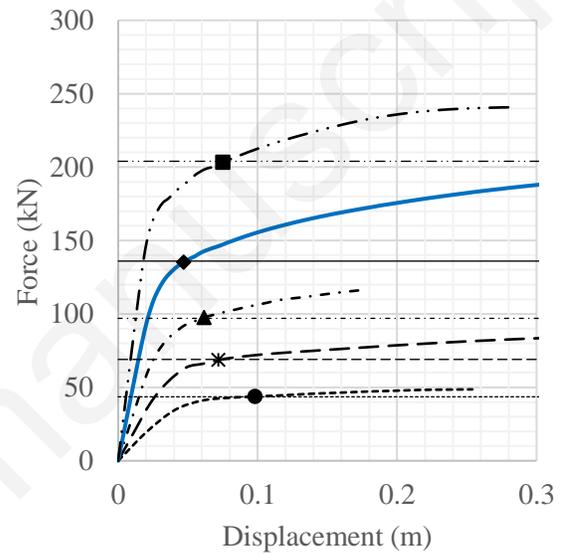
Both the aforementioned load cases, LC1 (Fig. 4a) and LC2 (Fig. 4b), were considered for each parametric variation. The loads were applied to the extremities of the main I-beams and were incremented to obtain the force-displacement behaviour. The monitored displacement is measured at the loading point. The investigated variations for each parameter are listed in the left most column of Table 1 (in grey colour) with their ultimate joint strengths derived from the proposed design approach in Section 2. The resulting analytical values were compared with the numerical results. The dotted horizontal lines in the force-displacement curves under LC1 and LC2 corresponds to the  $V_{bb}$  and  $V_{bu,opp}$  values listed in Table 1.

476 **4.1 Through I-beam section (IPE)**

477 The most important parameter influencing the ultimate joint strength was identified to be  
 478 the through I-beam section. Five different IPE sections were chosen as listed in Table 1.  
 479 Vertical force-displacement curves for the gravitational loading, LC1, and the opposite  
 480 bending loading, LC2, are plotted in Fig. 14 and 15, respectively.



481 ----- IPE220    - - - IPE270    - · - · - IPE330  
 ——— IPE400    - · · · - IPE500



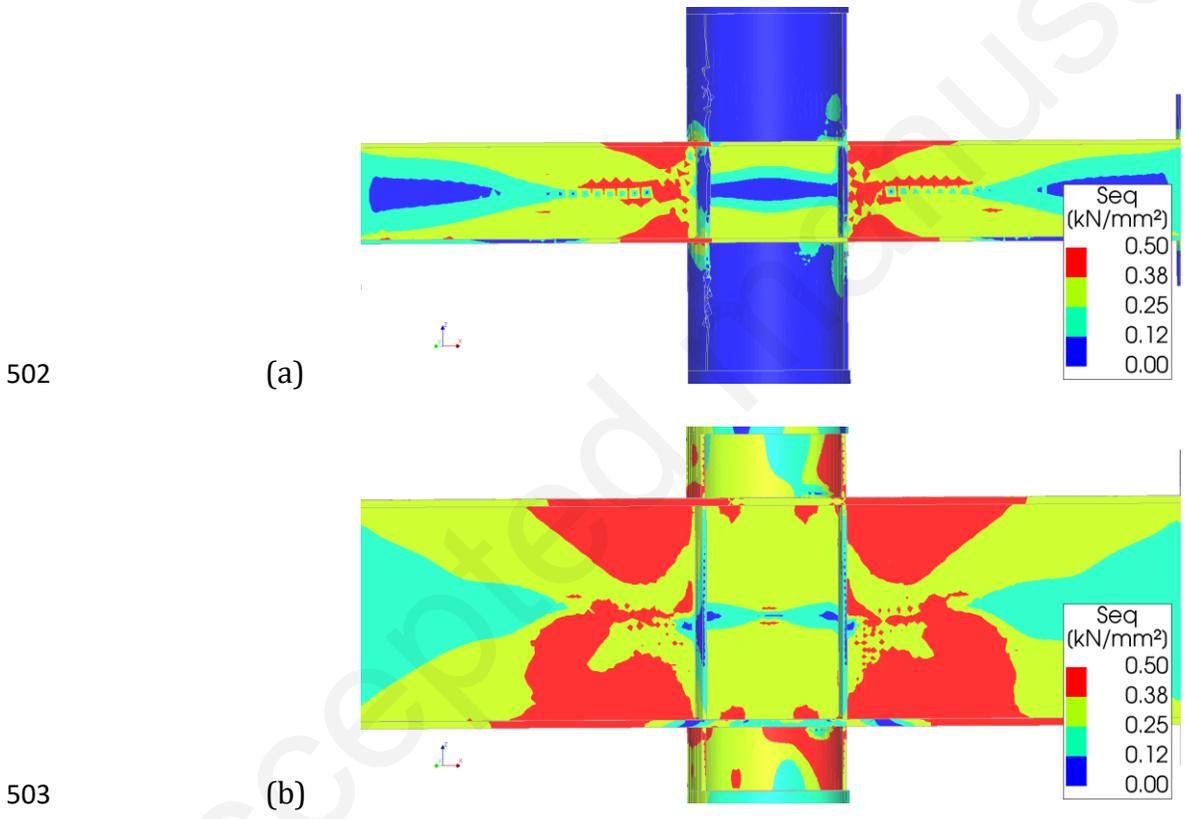
485 ----- IPE220    - - - IPE270    - · - · - IPE330  
 ——— IPE400    - · · · - IPE500

482 **Fig. 14.** Vertical force-displacement curve  
 483 comparisons for varying through IPE  
 484 sections under LC1

486 **Fig. 15.** Vertical force-displacement curve  
 487 comparisons for varying through IPE  
 488 sections under LC2

489  
 490 In the numerical models, failure was identified when the element strain reached the limiting  
 491 value for the equivalent plastic strain obtained from the experiments (mentioned in section  
 492 3). For LC1, the failure was solely dominated by the flange plasticity of the through I-beam  
 493 just outside the CHS column as shown in Fig. 16 in terms of von Mises stresses. The analytical  
 494 values of the plastic flexural resistance of the through I-beams ( $V_{bb}$  corresponding to

495  $M_{pl,Rd,beam}$ ) were compared with the force-displacement curves obtained from the FE models.  
 496 Larger sections provided a greater resistance beyond the analytical values compared to  
 497 smaller sections. This occurred due to the higher moment-curvature offered by the larger  
 498 sections through increasing the lever arm distance between their flanges. Von Mises stresses  
 499 shown in Figures 16a and 16b illustrate the failure obtained in the LASTEICON configurations  
 500 consisting of the weakest “through” I-beam - IPE220, and the strongest “through” I-beam -  
 501 IPE500.



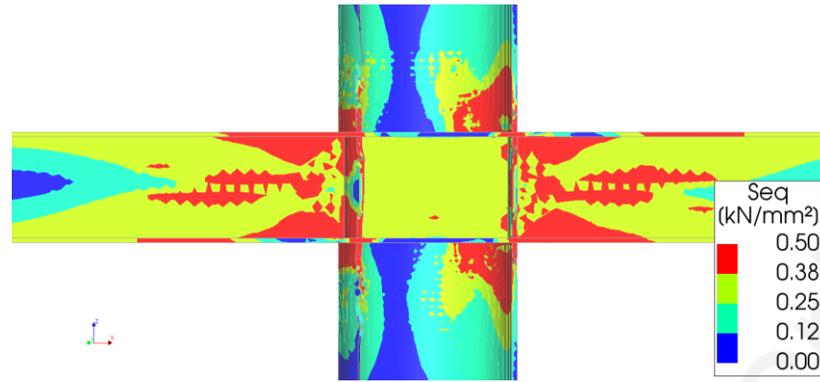
504 **Fig. 16.** Von Mises equivalent stresses ( $kN/mm^2$ ) at failure under LC1 for configurations with  
 505 (a) IPE220 through beam, (b) IPE500 through beam

506  
 507 No variation was however noticed in the failure pattern. The flanges of the through IPE beams  
 508 started to yield just outside the CHS column wall prior to all other components of the

509 connection. This occurred due to a rigid-body like behaviour of the main joint panel. Under  
510 LC1, both the moments nullify each other at the joint panel and do not contribute to the  
511 failure sequence. Therefore, failure is solely caused by flange plasticity of the through I-beam  
512 just outside the CHS column. The front half of the CHS column is removed from the figures to  
513 have a clear view of what is happening inside the “passing-through” connection zone.

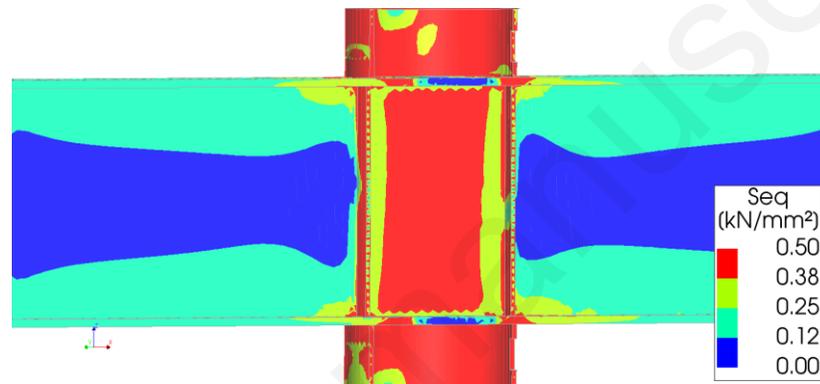
514 A different force-transfer mechanism was observed under LC2 as both the through I-  
515 beam web and the CHS column surface contributed to the ultimate joint strength. Failure  
516 occurred simultaneously in both the through I-beam web and the CHS column surface. This  
517 strongly validated the proposed design approach, since an effective transmission of bending  
518 moments occurred through a combination of shear resistance provided by the through  
519 section and the transverse tensile/compressive resistance of the CHS chord face. The  
520 ultimate joint strengths,  $V_{bu,opp}$  (see Table 1), are compared in Fig. 15 to highlight the  
521 agreement shared between the analytical and numerical results under LC2. For this load  
522 condition, the moment applied on the connection configurations is primarily resisted by the  
523 shear capacity of the through I-beam and is then transferred to the CHS column to utilize its  
524 transverse tensile/compressive resistance. Hence, if the through I-beam is not strong  
525 enough, it will be impossible to fully activate the CHS resistance. This phenomenon was  
526 identified using the LASTEICON configuration with a through IPE220, where failure occurred  
527 due to a beam flange plasticity outside the CHS column (Fig. 17a) prior to reaching the full  
528 capacity of the joint panel. No significant failure stresses were observed in the CHS column  
529 surface and the passing through I-beam web. A similar failure pattern was predicted by the  
530 analytical calculations presented in Table 1 ( $V_{bb} < V_{bj}$ ). The necessity of the proposed Design  
531 Check 1 can therefore be substantiated.

532



533

(a)



534

(b)

535 **Fig. 17.** Von Mises equivalent stresses ( $\text{kN/mm}^2$ ) at failure under LC2 for configurations with  
536 (a) IPE220 through beam, (b) IPE500 through beam

537

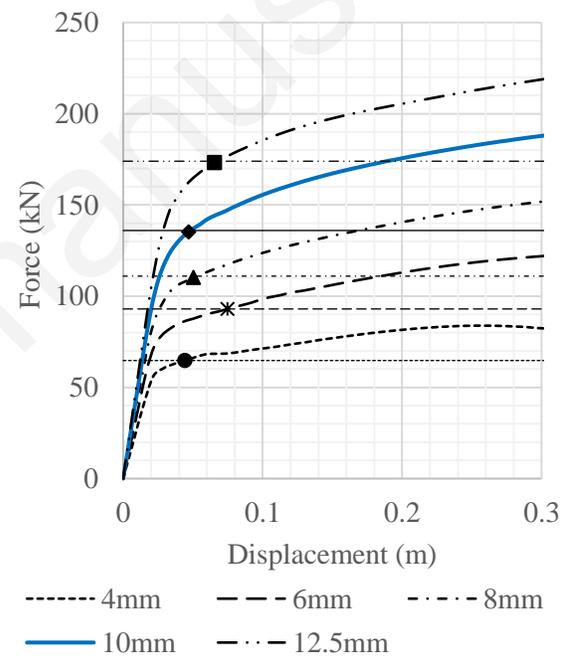
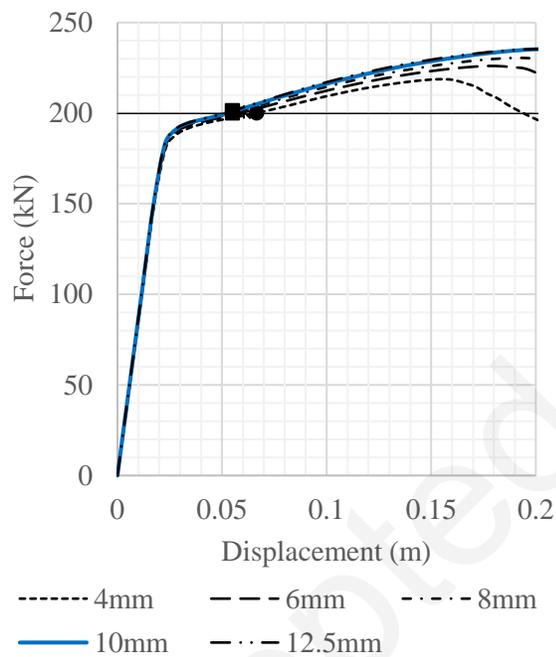
538 On the contrary, failure in the other configurations occurred due to a failure of the complete  
539 joint panel as predicted by the analytical design calculations. Maximum failure stresses were  
540 obtained at the faces of the CHS column at the flange connection zones as well as the through  
541 I-beam web as shown in Fig. 17b. The failure stresses developing at the connection zone of  
542 the CHS column clearly explained the tearing observed in the experimental prototype (Fig.  
543 11) and confirmed that it occurred due to an ultimate joint failure rather than any localized  
544 distortion of the CHS column wall, hence validating the benefits anticipated for the “passing-  
545 through” LASTEICON connections.

546

#### 547 4.2 CHS column thickness ( $t_c$ )

548 The CHS column thickness was varied from a smallest of 4 mm to a largest of 12.5 mm (Table  
549 1) to understand all possible failure sequences occurring due to any localized failure in the  
550 CHS column. Force-displacement curves, for LC1 and LC2, are shown in Fig. 18 and 19  
551 respectively for the CHS thickness variation.

552



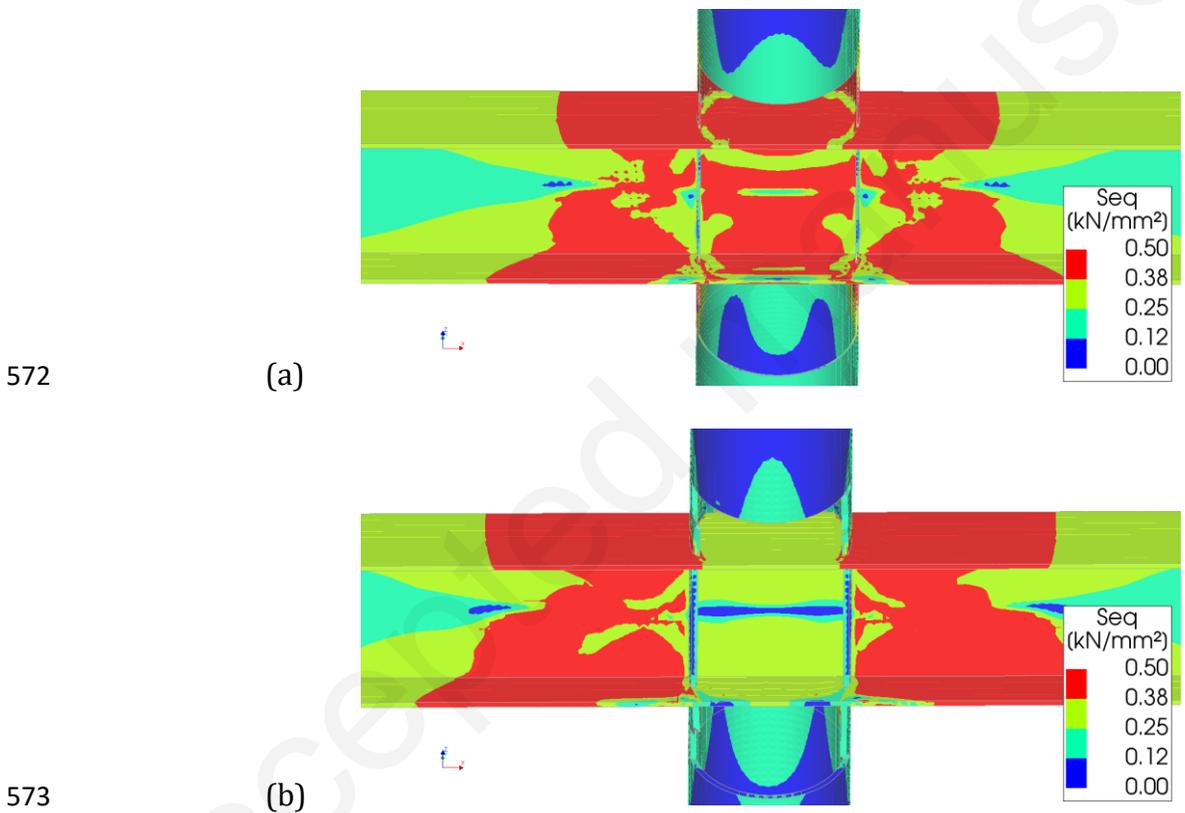
554 **Fig. 18.** Vertical force-displacement curve  
555 comparisons for varying CHS column  
556 thickness under LC1

558 **Fig. 19.** Vertical force-displacement curve  
559 comparisons for varying CHS column  
560 thickness under LC2

561

562 As the failure under LC1 is solely dominated by the flange buckling of the through I-beam and  
563 the I-beam section was kept constant in this subsection, varying CHS column thickness did  
564 not have any effect on the numerical models (Table 1, Fig. 18 and Fig. 20). Therefore, the

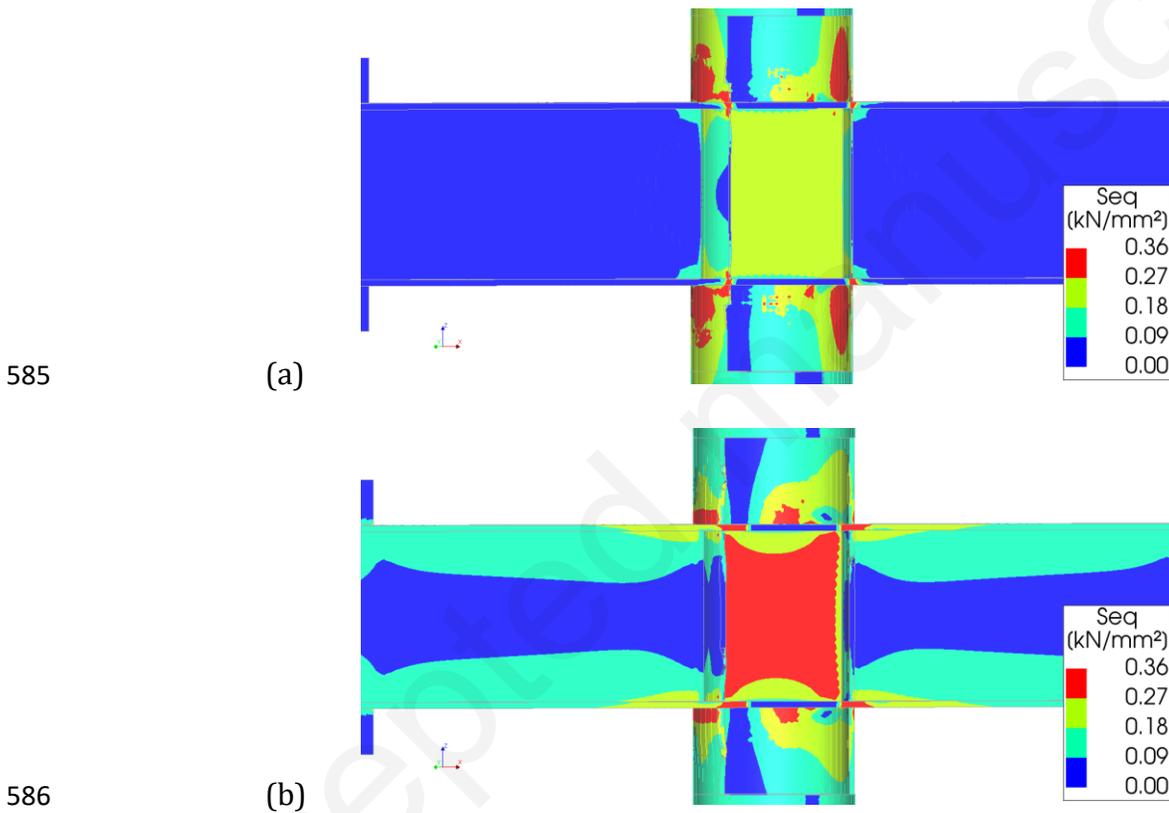
565 analytical value ( $V_{bb}$  corresponding to  $M_{pl,Rd,beam}$ ) of IPE 400 was compared with the force-  
 566 displacement curves. A thicker CHS was however observed to provide a slightly larger  
 567 resistance and ductility due to an increased overlapping and thus overstrengthening of the  
 568 “passing-through” connection zone. When the thickness was reduced to 4 mm, the CHS  
 569 column could not provide such amount of rigidity and therefore, some stress concentrations  
 570 were observed in the through flanges inside the CHS column as shown in Fig. 20a.  
 571 Nevertheless, the failure pattern remained unaffected for all configurations.



574 **Fig. 20.** Von Mises equivalent stresses ( $kN/mm^2$ ) at failure under LC1 for configurations with  
 575 (a) 4mm thick CHS, (b) 12.5mm thick CHS

576  
 577 The CHS column thickness has, however, a substantial effect in the force-displacement  
 578 behaviour as well as the failure mode of the joint configuration under LC2. As listed in Table

579 1, the failure mode changed from a punching shear failure for the thinner columns (Class 4  
580 sections) to an ultimate joint panel failure for the thicker columns. Although the 6 mm thick  
581 CHS column was calculated as a Class 4 section, it was close to the Class 3 limit and therefore,  
582 offered a punching shear resistance marginally higher than the actual joint strength (also see  
583 Table 1). The ultimate joint strengths calculated as per Table 1 are further compared in Fig.  
584 19 to highlight the agreement shared between the analytical and numerical resistances.



587 **Fig. 21.** Von Mises equivalent stresses ( $kN/mm^2$ ) at yield under LC2 for configurations with (a)  
588 4mm thick CHS, (b) 12.5mm thick CHS

589  
590 As suggested by the analytical calculations in Section 2, slender CHS columns, specifically  
591 belonging to Class 4, are susceptible to local buckling as well as punching shear failure.  
592 Although local buckling of the CHS columns was not observed in any cases, the slender

593 column with 4 mm thickness, failed due to localised punching shear and highlighted the  
594 importance of the aforementioned design checks. In the configuration with 4mm thick CHS,  
595 yielding occurred on the CHS column surface prior to any yielding in the through I-beam web  
596 (Fig. 21a). Failure in such slender column connections was thus observed to be dominated  
597 by the local stress concentrations occurring in the CHS column surfaces near the I-beam  
598 flanges. In such cases, the through I-beam webs did not develop the failure stresses yet. On  
599 the other hand, for the configurations with a thicker CHS column (8, 10, 12.5 mm), the I-beam  
600 web yielded first (Fig. 21b) and failure occurred in the joint panel - combined failure of both  
601 the through I-beam web and the CHS column surface.

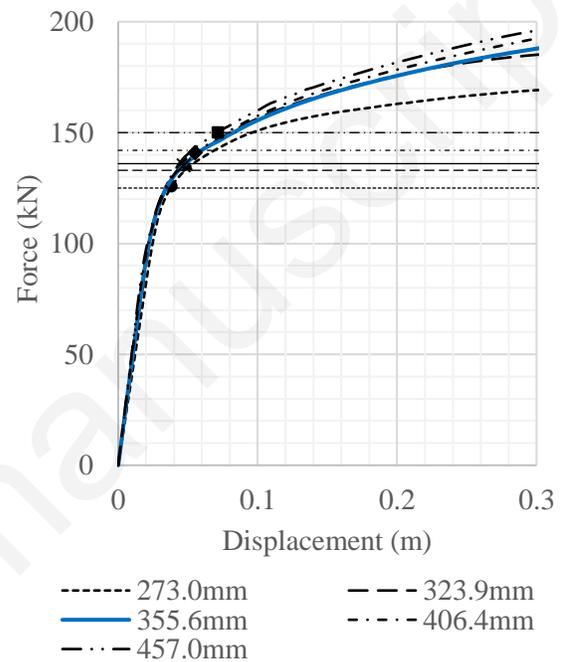
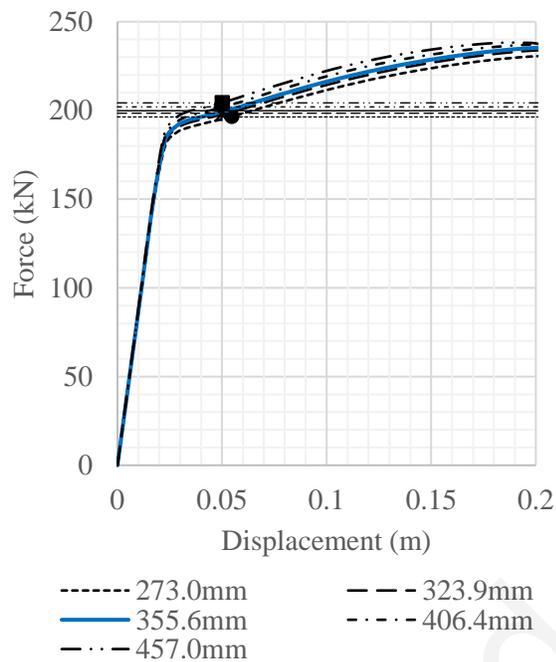
602

#### 603 **4.3 CHS column diameter ( $d_c$ )**

604 The CHS column diameter was varied from 273 mm to 457 mm (Table 1). The diameters were  
605 chosen based on their availability in the steel construction industry. Force-displacement  
606 curves in Fig. 22 and 23 describes the effect of diameter variation for LC1 and LC2,  
607 respectively.

608 As the through I-beam was kept constant, the CHS column diameter did not have any  
609 significant effect on the vertical force-displacement curves for LC1. However, small  
610 differences were noticed (Fig. 22) due to the fact that  $V_{bb}$  is compared instead of  $M_{pl,Rd,beam}$ . As  
611  $V_{bb}$  is derived from  $M_{pl,Rd,beam}$  according to Eq. 19, it incorporates a small deviation due to  $d_c$ .  
612 Therefore, although small differences were noticed in the analytical values and force-  
613 displacement curves, the CHS column diameter failed to show any substantial effect in the  
614 joint configuration under LC1 from a behavioural perspective. In LC2, however,  $d_c$  played a  
615 noticeable role in influencing the joint strength as shown in Table 1 and Fig. 23. Successful

616 arguments were again observed in the FE models compared to the analytical calculations as  
 617 the design approach was able to produce a more or less correct prediction regarding the  
 618 failure mode and resistance for the LASTEICON configurations with different CHS diameters.  
 619 von Mises equivalent stresses are not shown due to qualitative similarity.



620  
 621 **Fig. 22.** Vertical force-displacement curve  
 622 comparisons for varying CHS column  
 623 diameter under LC1

624  
 625 **Fig. 23.** Vertical force-displacement curve  
 626 comparisons for varying CHS column  
 627 diameter under LC2

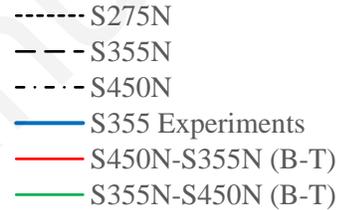
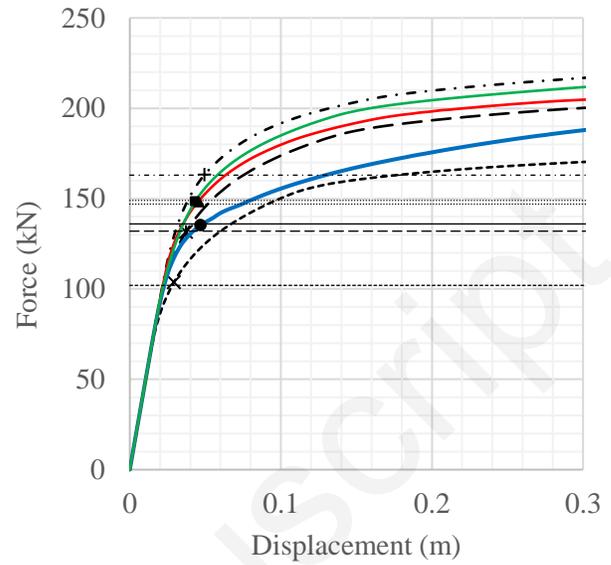
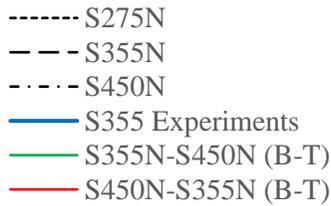
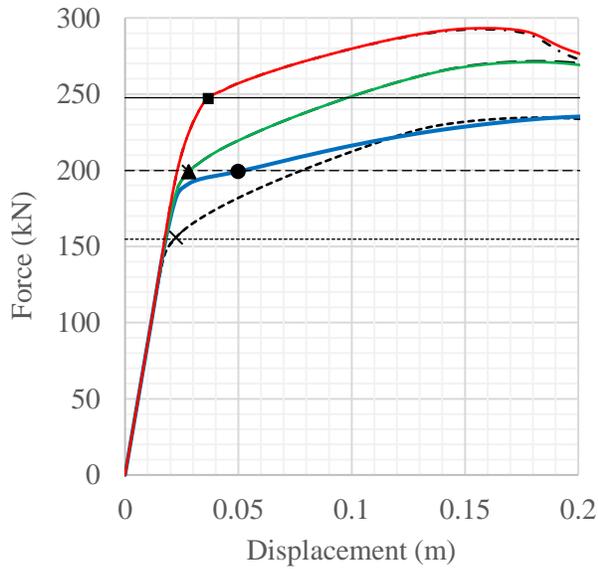
628

629 **4.4 Material properties for both CHS and through I-beam ( $f_{yb}$  and  $f_{yc}$ )**

630 Different steel grades were also chosen to identify the failure sequences in the joint panel.  
 631 Primarily, nominal material properties were chosen for three different steel grades, S275 ( $f_y$   
 632 = 275 MPa), S355 ( $f_y = 355$  MPa), and S450 ( $f_y = 440$  MPa), to model all the members in the  
 633 joint configuration ( $f_{yb} = f_{yc}$ ) as shown in Table 1.  $f_y$  stands for the yield strength of a steel  
 634 grade. In Fig. 24 and 25; “S275N”, “S355N” and “S450N” refer to the numerical models with

635 the corresponding nominal material properties with strain hardening, adopted according to  
636 Table 3.1 (EN 10025-2), EN 1993-1-1. “S355 Experiments” refers to the model with the  
637 experimental material stress-strain properties i.e. a combination of yield strengths for the  
638 beams and the CHS column ( $f_{yb} = 355$  MPa and  $f_{yc} = 377$  MPa). Furthermore, “S355N-S450N  
639 (B-T)” defines a material combination where the beam was modelled with S355N and the  
640 CHS column is constructed with S450N, whereas, “S450N-S355N (B-T)” denotes the opposite  
641 combination.

642 As shown in Fig. 24, the force-displacement curves for “S355N” and “S450N”  
643 overlapped with “S355N-S450N (B-T)” and “S450N-S355N (B-T)” respectively, for LC1.  
644 Similar results were noticed from the  $V_{bb}$  values calculated in Table 1. This validated the  
645 aforementioned conclusion that the failure under LC1 is solely dominated by flange buckling  
646 of the through I-beam. A significant difference was noticed between the force-displacement  
647 curves of “S355N” and “S355 Experiments” due to the difference in the ultimate stress and  
648 strain of the corresponding material curves. However, no exact overlap was noticed in the  
649 force-displacement curves under LC2 (Fig. 25). This justified the previous interpretation that  
650 both the through I-beam and the CHS contributes to the ultimate joint resistance.  
651 Additionally, the analytical value and the force-displacement curve for “S355N-S450N (B-T)”  
652 were closer to those for “S450N” compared to “S355N” and similarly, “S450N-S355N (B-T)”  
653 was closer to “S355N” than “S450N”. This observation possibly highlights a slightly larger  
654 contribution offered by the CHS compared to the through I-beam under LC2.



655

59

656 **Fig. 24.** Vertical force-displacement curve  
 657 comparisons for varying steel grades under  
 658 LC1

660 **Fig. 25.** Vertical force-displacement curve  
 661 comparisons for varying steel grades under  
 662 LC2

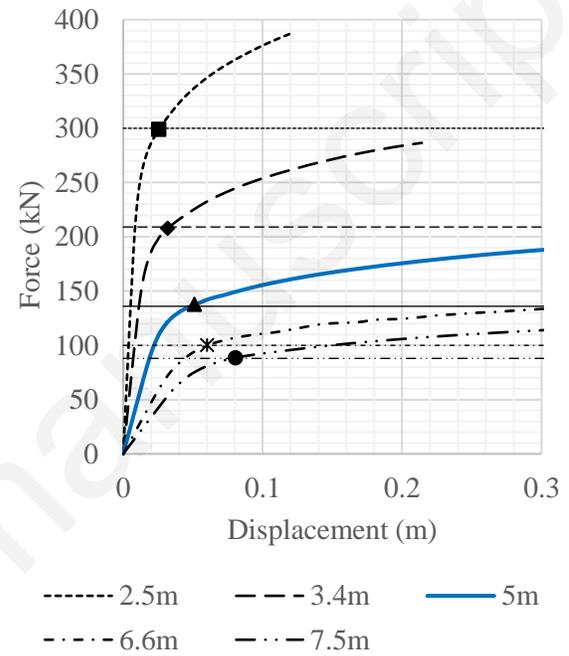
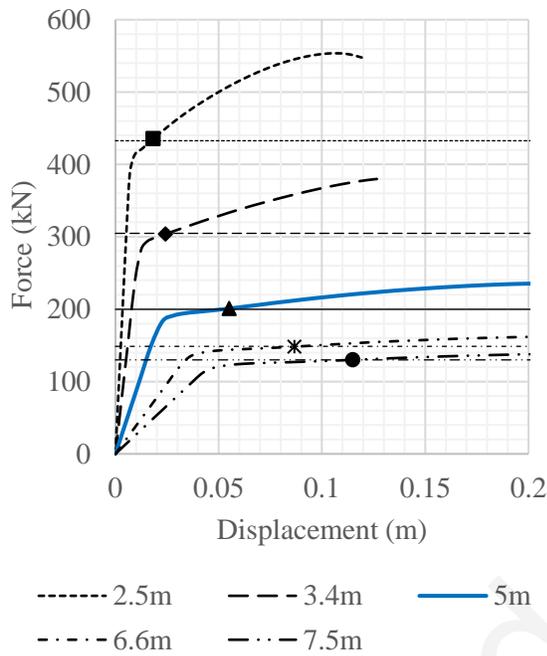
663

#### 664 4.5 Moment-to-Shear ( $M/V$ ) Ratio

665 In order to check the consistency of the proposed design approach, the joint configurations  
 666 were investigated for different  $M/V$  ratios. The  $M/V$  ratio was varied by varying the total  
 667 length of the beam,  $L_b$ , thus changing the lever arm between the CHS column wall face and  
 668 the extremity at which the vertical load is applied. However, the through I-beam length was  
 669 kept constant as shown in Fig. 13. Decreasing  $L_b$  decreased the  $M/V$  ratio and therefore, for a  
 670 certain vertical load (shear force) the joint configuration experienced a smaller moment  
 671 compared to the reference configuration. The contrary happened for an increased  $L_b$ . The

672 force-displacement curves are compared with relevant analytical calculations in Fig. 26 and  
 673 27 for LC1 and LC2, respectively. Good agreements were achieved between the analytical and  
 674 numerical results thus confirming the consistency of the proposed design approach.

675



676

680

677 **Fig. 26.** Vertical force-displacement curve  
 678 comparisons for varying beam length under  
 679 LC1

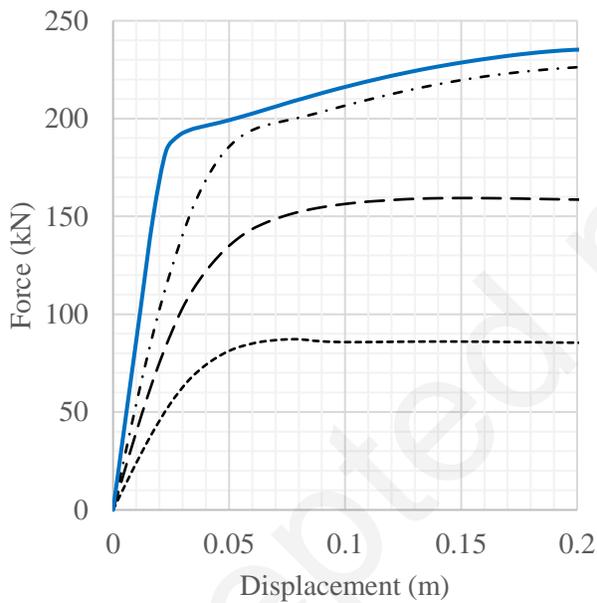
681 **Fig. 27.** Vertical force-displacement curve  
 682 comparisons for varying beam length under  
 683 LC2

684

685 **4.6 Comparison with Conventional I-beam-to-CHS joint configurations**

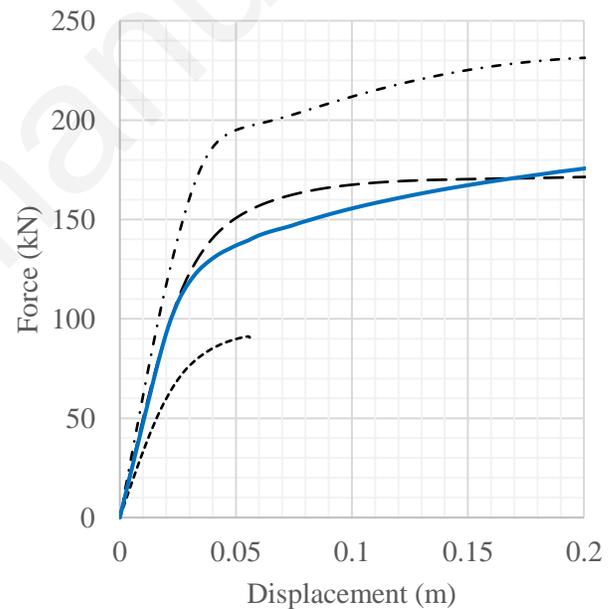
686 As discussed in Section 1 and 3, the conventional joint configurations involving an I-beam  
 687 and a CHS column are not completely capable of utilizing the advantages provided by the  
 688 hollow sections. Therefore, a short comparison study was done in order to see the potential  
 689 advantages of the proposed LASTEICON “passing-through” I-beam-to-CHS column  
 690 connection. As the CHS column governs the failure modes of such conventional connections,

691 the CHS column thickness alone was varied to check the minimum thickness required for  
 692 these conventional connections to reach the strength of the reference LASTEICON  
 693 configuration, which was kept constant throughout the parametric study. This reference  
 694 configuration constituted of an IPE400 section passing through a CHS column with 355.6 mm  
 695 diameter ( $d_c$ ) and 10 mm thickness ( $t_c$ ) with a total beam length ( $L_b$ ) of 5000 mm. However,  
 696 the conventional joints were modelled by simply removing the “passing-through” part of the  
 697 inserted IPE beam as well as the slots in the CHS columns. The force displacement curves are  
 698 compared in Fig. 28 and 29 for LC1 and LC2, respectively.



699  
 700  
 701  
 702  
 703

- Conventional\_10mm
- Conventional\_16mm
- Conventional\_22mm
- LASTEICON\_10mm



704  
 705  
 706  
 707  
 708

- Conventional\_10mm
- Conventional\_16mm
- Conventional\_22mm
- LASTEICON\_10mm

700 **Fig. 28.** Vertical force-displacement curve  
 701 comparisons between LASTEICON and  
 702 conventional plate-to-CHS connection  
 703 under LC1

705 **Fig. 29.** Vertical force-displacement curve  
 706 comparisons between LASTEICON and  
 707 conventional plate-to-CHS connection  
 708 under LC2

709 As the through I-beam solely dominates the failure under LC1, significant advantages were  
710 observed in the force-displacement curve comparisons as shown in Fig. 28. A conventional  
711 configuration with 22mm thick CHS column only proved to be sufficient to provide as much  
712 resistance as the LASTEICON configuration with a 10 mm thick CHS column under LC1. A  
713 significant decrease in the stiffness was also observed due to the removal of the “passing-  
714 through” part. Under LC2, a 16 mm thick column in the conventional configuration sufficed  
715 to be enough resisting as the LASTEICON joint with 10mm thickness as shown in Fig. 29. This  
716 proves a significant contribution of the “passing-through” I-beam towards strengthening the  
717 joint panel.

718

## 719 **5. DISCUSSIONS AND REMARKS**

720

721 A parametric study was done based on a series of nonlinear static analyses in accordance  
722 with the EN 1993-1-1 and EN 1993-1-8 prescriptions in order to construct a conservative  
723 design method for the proposed “passing-through” I-beam-to-CHS connection. The design  
724 hypothesis was developed based on a successful identification of the force-transfer  
725 mechanism and was further validated through numerical simulations and experimental  
726 results. Based on encouraging results, this newly proposed LASTEICON connection as well as  
727 its design procedure can be concluded as an efficient upgrade to the conventional I-beam-to-  
728 CHS column connections with direct welds. The following points highlights the noteworthy  
729 findings of this research study.

- 730 • In the monotonic gravitational loading LC1, both vertical forces were applied in the  
731 same direction, thus generating equal and opposite moments on either side of the CHS

732 column surface. These moments nullified each other, which led to a rigid body like  
733 behaviour of the actual “passing-through” joint panel. As a result, the ultimate joint  
734 strength (and failure) was solely governed by the plastic flexural resistance of the  
735 through I-beam just outside the CHS.

736 • In the opposite bending loading LC2, both the through I-beam as well as the CHS  
737 column contributed significantly towards developing the ultimate joint strength. For  
738 properly designed joints (i.e. safe from the three checks mentioned in the design  
739 procedure), failure occurred simultaneously in the through I-beam web (due to  
740 transverse shear) and the CHS column surface (due to transverse tensile/compressive  
741 forces), thus validating the anticipated force-transfer mechanism and the  
742 corresponding design procedure.

743 • Under LC2, while increasing the through I-beam section offered greater strength and  
744 stiffness, reducing it showed substantial vulnerability towards flexural failure of the  
745 through beam prior to full activation of the joint panel strength, justifying the first  
746 check suggested in the design procedure.

747 • To validate the second and third check, regarding the local buckling and punching  
748 shear failure respectively, few class 3 and 4 type CHS columns were deliberately  
749 chosen in the proposed configuration to examine the prediction accuracy of the design  
750 calculations. According to the design calculations, the 4 mm thick CHS column  
751 connection failed due to localized punching shear on the CHS column surface, prior to  
752 activation of the complete joint strength. The 6 mm thick CHS connection marginally  
753 avoided such a punching shear failure. The failure mode for Class 1 and 2 CHS columns

754 was observed to be the joint panel failure. These observations justified the  
755 requirement of Design check 3. However, to avoid such irregular failures, it is better  
756 to avoid the class 3 and 4 type CHS while constructing such “passing-through”  
757 connections. Similar encouraging agreements were found between the analytical and  
758 numerical models for CHS diameter variation.

- 759 • In real life structures, as the CHS column would be axially loaded rather than being an  
760 unloaded part of the joint configuration, significant compressive forces might make it  
761 vulnerable towards local buckling failure. Even though only one local buckling failure  
762 was observed in the present range of investigations, design check 2 is therefore  
763 recommended to avoid such a failure.
- 764 • Parametric studies with varying material properties and different material  
765 combinations made it evident that failure under LC1 was solely dominated by the  
766 through I-beam and failure under LC2 depended on both the through I-beam web as  
767 well as the CHS column for the proposed configuration safe from all the checks.  
768 Furthermore, these studies also hinted a slightly larger contribution from the CHS  
769 column compared to the through I-beam in developing the ultimate joint strength  
770 under LC2. Encouraging results supported the consistency of the suggested design  
771 approach for varying M/V (Moment-to-shear) ratios.
- 772 • Furthermore, a comparative study between the conventional and the LASTEICON  
773 configurations showed the advantages provided by the passing-through elements in  
774 terms of strength and stiffness of the whole joint. In a conventional configuration  
775 under LC1, the CHS column had to be made at least 2.2 times thicker to acquire an

776 equal resistance as a LASTEICON connection with similar geometric/sectional  
777 properties. However, under LC2, a 1.6 times thicker CHS column proved to be  
778 adequate.

- 779 • As mentioned in Section 1, the laser cutting technique surpasses the other cutting  
780 methods. Much lower amounts of slag are released during the fabrication of the joint  
781 assembly due to a simpler and reduced welding, thanks to the higher precision offered  
782 by the laser [34-36]. Additionally, the heat affected zones (HAZ) of laser cutting is  
783 much smaller, when compared to other methods [36]. Nevertheless, to refine the  
784 knowledge regarding the joint capacity, additional attention should be paid to the HAZ  
785 due to LCT and on the welding operation between the passing through elements and  
786 the slotted CHS column. Although the LCT reduces the importance of the HAZ  
787 significantly compared to the classical cutting procedures [24], additional  
788 experiments are still required to better quantify the LCT/welding interaction. The  
789 feasibility of cutting inclined angles was reported in a first study [24] for joints with  
790 thin CHS columns ( $\leq 10\text{mm}$ ), discussing the possibility of full penetration welds in the  
791 proposed connection, which resulted in a shorter fabrication time as well as an  
792 increased ductility compared to the fillet welds. Additional studies are currently  
793 ongoing to obtain full penetration welds above 10 mm thickness by controlling the  
794 chamfer angle and the gap size. The results achieved so far and presented in [32]  
795 however show that, in this range of thicknesses, the connections with fillet welds  
796 perform better in terms of global strength than connections with full penetration  
797 welds.

798

799 **6. CONCLUSION**

800

801 A new configuration for an open-to-hollow section joint with a “passing through” concept is  
802 proposed in this paper through the “LASTEICON” project funded by the European  
803 Commission. This type of an I-beam-to-CHS column joint is recommended for using in  
804 structures with predominant gravitational as well as opposite bending loading. The unique  
805 component of this joint is a through I-beam supporting the primary beams. Knowing the  
806 applied loads, the geometries and the materials, a design procedure is further proposed to  
807 calculate the resistance offered by such connections. The procedure is verified through a  
808 detailed numerical parametric analysis which is validated by an experimental campaign.  
809 Emphasis is put onto the generation of global models that are suitable to predict the ultimate  
810 resistance of the proposed joint configurations. These models can be used to assess the load  
811 transfer, stress concentrations and possible failure modes correctly with respect to the  
812 experimental investigations. Strong agreements are obtained between the design analytical  
813 calculations and the experimentally calibrated numerical simulations. Additional  
814 experimental results are however required to extend the validated range of application of the  
815 design procedure and are planned for the near future considering different loading  
816 conditions with axial compression/tension, different welding types and real-life  
817 uncertainties.

818

819

820

821

822 **ACKNOWLEDGEMENTS**

823  
824 This research study was possible thanks to the research fund provided by the European  
825 Commission with the contract LASTEICON EU-RFCS GA-709807 ([www.LASTEICON.eu](http://www.LASTEICON.eu)). The  
826 project consortium includes Fincon Consulting Italia srl (coordinator), RWTH Aachen,  
827 University of Pisa, Hasselt University, Instituto Superior Tecnico of Lisbon and INSA Rennes,  
828 ADIGESYS, OCAM and VALLOUREC. The cooperation of all of them is hereby gratefully  
829 acknowledged.

830  
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