LOAD BEARING CAPACITY OF SPLICED COLUMNS WITH SINGLE ROW BOLTED BUTT-PLATES

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ABSTRACT

A butt-plate splice makes part of the column and therefore must be designed for second order bending moments and shear forces in addition to the axial force. Building Codes may require or recommend minimum strength and/or stiffness for such splices. The presence of a splice can have influence on the bearing capacity of the column and on the force distribution in the overall structure. This paper suggests an expression for the calculation of the rotational stiffness of single row bolted butt-plate column splices and a procedure for obtaining the reduced bearing capacity of such a spliced column under compression. The methods are based on the assumption of a virtual moment arm in the splice which is a function of a linear stress gradient across the column section due to Eurocode 3 defined column imperfections.

INTRODUCTION

In general, a column splice for the transfer of axial loads can be manufactured in two ways: with butt-plates welded perpendicular to the cut section of the two columns or with cover plates on the flanges and web. The cover plates can create a gap between the two column ends or allow direct contact. The splices should preferably be located at floor level but for practical reasons this usually occurs at 0.5 m to 1.0 m above floor level, i.e. about one quarter up a story high column. The axial loads can be transferred in different ways through the column-to-column connection: by direct bearing through the butt-plates or through the flanges and web or by compression and tension in the cover plates. In all cases there will be shear forces in the splice.

Eurocode 3 Part 1.8 (2006) gives design rules for column splices requiring minimum capacities for bending moment and shear force in case of load transfer through the cover plates and a minimum normal compressive force to be accounted for in case of bearing.

The splice is part of the column and must be designed for 2^{nd} order bending moments and shear forces in addition to the axial force. A strength requirement for the column splice derived on this basis has been presented earlier. In addition, a stiffness requirement for column splices with small imperfections was suggested on the basis of allowing a maximum reduction of 5% in the Euler buckling load. A limited number of experimental tests on HE100A columns has shown that butt-plate splices can have a negative influence on the buckling load of columns. (Snijder & Hoenderkamp, 2008).

In this paper the theory on rotational stiffness of column splices will be extended to include larger imperfections such that butt-plate separation is allowed to occur. It will also give a method of analysis to quantify the influence of the splice on the load bearing capacity of the column.

LITERATURE

Code Requirements

In Eurocode 3 Part 1.8, a distinction is made between bearing and non-bearing column splices. Where the members are not prepared for full contact in bearing, the moment resistance should be not less than 25% of the moment capacity of the weaker section applied in both directions and the design shear force should be taken not less than 2.5% of the squash capacity also to be applied in two directions. Where the members are prepared for full contact in bearing, cover plates, bolts and welds should be able to transmit 25% of the maximum compressive force in the column. The background to these requirements could not be traced.

A study of design methods for column splices subject to concentric axial loading as suggested by the National Building Codes of Europe, Great Britain, Germany, The Netherlands, United States, Canada, Australia and Japan (Snijder & Hoenderkamp, 2005, 2008) yields a profusion of different empirical approaches which lead to a variety of design procedures with different load combinations which must be applied to the connection. The study was extended to requirements and rules on fabrication published in Great Britain, The Netherlands and Australia. The majority of the studied codes only address design requirements for strength with the exception of BS5950 and NEN6772 which also refer to splice stiffness. A BS5950 requirement states that for direct contact bearing splices, the stiffness in the connection must be maintained. In a BCSA-SCI publication on simple connections it is stated that an accurate elastic analysis of the connection should be used to verify that it is at least as stiff as the member. It further suggests that even where a splice connection is entirely in compression, it is advisable to maintain full continuity of stiffness through the connection. NEN 6772 requires that the stiffness of a splice must be included in the analysis of the building structure.

Only half of the building codes studied require second order effects be taken into account in the design of the column splice. A number of codes state that the connection materials such as plates, bolts and welds must secure that the two column sections remain in place.

Research

The influence on the stability of columns of specific column imperfections which are introduced by the application of column splices was further investigated (Lindner & Gietzelt, 1988; Lindner, 1998, 1999, 2002, 2008) based on earlier research (Popov & Stephen, 1977; Sheer et al., 1987). It was concluded, that in case slip is prevented in the splice, e.g. by pre-stressing the bolts, standard column stability checks would suffice to cover column splice imperfections. In case slip is not prevented, a less favourable buckling curve must be used. It has been advised to transfer at least 10% of the normal force by the connectors to secure both column parts in location. Full scale buckling tests on butt-spliced columns for weak axis buckling were carried out on HE240A (S235) sections. Results were compared with load bearing capacities confirming that spliced columns can be checked as normal columns for stability. Splice stiffness was not addressed.

A research project at the Eindhoven University of Technology on the design of column splices for strength and stiffness was instigated by the Dutch steel fabricators who specifically objected to the rather severe requirement to supply a minimum of 25% of the moment capacity of the column section through the splice material.

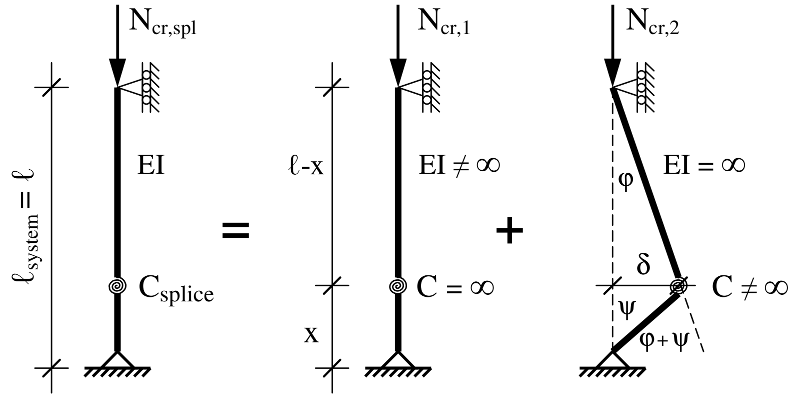

Figure 1: Column model for critical load of spliced column

CRITICAL LOAD OF SPLICED COLUMNS

The critical load of a spliced column $N_{cr,spl}$ can be estimated by combining the critical loads for the two subsystems as shown in Figure 1. In the subsystem 1, the splice rotational stiffness C is assumed to be infinite while the column has a finite bending stiffness EI. In subsystem 2 on the right, the splice rotational stiffness C is finite while the column is assumed to have infinite bending stiffness EI. The Euler buckling load for column 1, $N_{cr,1}$, where the splice stiffness is taken as $C = \infty$, is:

$$N_{cr,1} = \frac{\pi^2 EI}{\ell^2} \tag{1}$$

Equilibrium in the deformed state gives the critical load $N_{cr,2}$ of subsystem 2, where the bending stiffness is taken as $EI = \infty$

$$N_{cr,2} = \frac{C\ell}{x(\ell - x)} \tag{2}$$

Now the critical load for the spliced column with finite values for the splice rotational stiffness C and column bending stiffness EI can be obtained by using the Dunkerley formula (Dunkerley, 1894) as follows:

$$\frac{1}{N_{cr,spl}} = \frac{1}{N_{cr,1}} + \frac{1}{N_{cr,2}} \tag{3}$$

Substituting Eqs. (1) and (2) into Eq. (3) and simplifying yields the following equation for the critical load of the spliced column:

$$N_{cr,spl} = \frac{C\ell}{\dfrac{C\ell^3}{\pi^2 EI} + x(\ell - x)} \tag{4}$$

ROTATIONAL STIFFNESS OF COLUMN SPLICES

Full Contact between Butt-Plates

A generally accepted method for calculating the rotational stiffness of a column splice under compression is not available to the knowledge of the authors. In order to get an indication of the rotational stiffness of a column splice under compression the authors have adopted a simplified design procedure which employs an equation from Eurocode 3 Part 1.8 (EN1993-1-8:2004, 2006) for the initial rotational stiffness of beam-to-column connections subject to bending

$$C = \frac{Ez^2}{\mu \sum \frac{1}{k_i}} \tag{5}$$

in which z is a moment arm represented by the distance between a compression point and the center of the bolt group in the tension area; μ is given a unit value for initial stiffness; and k_i is a stiffness factor to be determined according to Eurocode 3. The suggested method requires adjustment to the values for z and k_i in order to take the column-column end plate splice configuration with a single central row of bolts as shown in figure 2a into account.

The stiffness factors k_i in Eqn. (5) are to be determined for all individual components that are active in resisting the forces in the connection. In a column splice subject to compressive stresses only as shown in Figure 2 there exists just one stiffness component: axial compression. It is suggested that the stiffness factor becomes

$$k = \frac{A}{\ell} \tag{6}$$

in which A is the sectional area of the column and ℓ is its length. The equation for the initial rotational stiffness of the column splice can be simplified and rewritten as

$$C = \frac{EAz^2}{\ell} \tag{7}$$

The axial load on the column splice in Figure 2a with the associated second order bending moment due to the imperfection at the splice $e_{spl,I}$ will cause a typical linear stress distribution in the column section as shown in Figure 2b. It is suggested that the moment arm z be obtained from the linearly extended stress distribution across the steel section as shown in Figure 2b. A full compressive stress distribution holds true for small load eccentricities at the splice, i.e. no tensile stresses in the splice

$$\frac{N_{Ed}}{A} \geq \frac{N_{Ed} e_{spl,I} h/2}{I} \tag{8}$$

where N_{Ed} is the design axial compressive load, I is the second moment of area and h is the height of the section. Then

$$e_{spl,I} = e^* \sin \frac{\pi x}{\ell} \leq \frac{i^2}{h/2} \tag{9}$$

in which e^* is the maximum column imperfection at mid height defined by Eurocode 3 and can be expressed as follows (Snijder & Hoenderkamp, 2008)

$$e^* = \frac{1-\chi}{\chi} \frac{n-1}{n} \frac{M_{pl,Rd}}{N_{pl,Rd}} \tag{10}$$

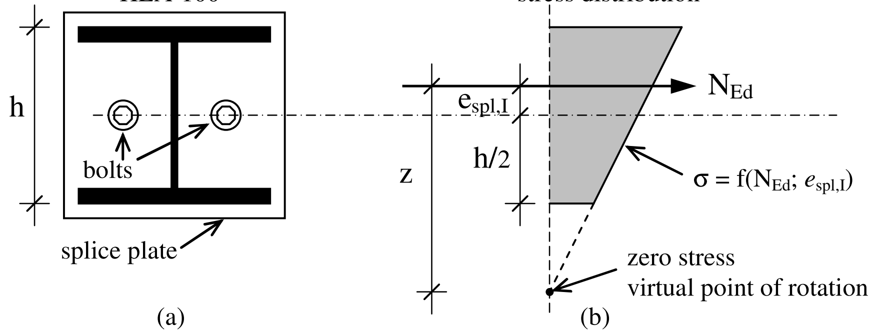

Figure 2: Alternative moment arm

where $N_{pl,Rd}$ is the design plastic resistance of the gross cross-section (squash load), $M_{pl,Rd}$ is the design plastic resistance for bending and the radius of gyration is

$$i = \sqrt{\frac{I}{A}} \tag{11}$$

Reduction factor χ is a function of Φ and the relative slenderness of the column $\bar{\lambda}$

$$\chi = \frac{1}{\Phi + \sqrt{\left(\Phi^2 - \bar{\lambda}^2\right)}} \tag{12}$$

$$\varphi = 0.5\left\{1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right\} \tag{13}$$

$$\bar{\lambda}^2 = \frac{N_{pl,Rd}}{N_{cr,spl}} \tag{14}$$

where α is an imperfection factor dependent on the pertaining instability curve.
The bearing capacity of the spliced column can be expressed as follows

$$N_{b,Rd,spl} = \chi N_{pl,Rd} \tag{15}$$

For this ultimate column load the factor n now becomes

$$n = \frac{N_{cr,spl}}{N_{b,Rd,spl}} \tag{16}$$

From the geometry in Figure 2b it can quite easily be shown that the moment arm is

$$z = \frac{i^2}{e_{spl,I}} + e_{spl,I} \tag{17}$$

This moment arm can now be used in Eq. (7) to obtain the initial rotational stiffness of the splice subject to axial force and bending moment. It should be noted here that the moment arm and thereby the rotational stiffness of the splice are independent of the size of the axial load.

Separation of Butt-Plates

If the eccentricity of the axial load is increased beyond a distance $2i^2/h$, the combined axial and bending stress on one side of the connection (the tension flange) will result in a tensile stress if the butt-plates were welded together. Since they are bolted together in the center of the splice, the plates will separate at the tension flange.
The coming apart of the plates will change the stressed I-shaped cross-section of the spliced column into a T-shape as the intended tensile flange is not participating anymore in resisting the eccentric axial load. Upon gradual separation of the end plates, the location of the neutral axis of the T-section will move towards the compression flange. When the neutral axis is at a point on a T-shaped cross-section where the applied eccentric load will cause a triangular stress distribution in that section, the separation of the plates will stop, see Figure 3. Here the tensile stress

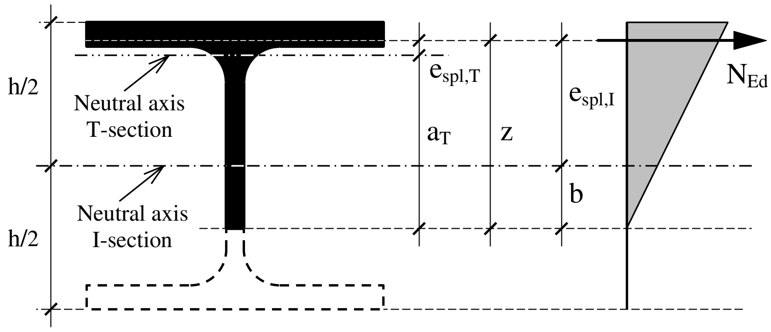

Figure 3: Stress distribution after butt-plate separation

due to bending is equal to the axial compression from the axial load. Increasing the load eccentricity $e_{spl,I}$ will cause a further shift of the neutral axis of the T-section towards the compression flange until again the tensile stress due to bending is equal to the axial compression from the axial load . For the point of zero stress

$$\frac{N_{Ed}}{A_T} - \frac{N_{Ed}\, e_{spl,T}\, a_T}{I_T} = 0 \tag{18}$$

from which follows

$$e_{spl,T} = \frac{I_T}{A_T a_T} = \frac{i_T^2}{a_T} \tag{19}$$

where A_T is the sectional area of the T-section, I_T is its second moment of area, i_T is the radius of gyration and a_T is the location of the neutral axis measured from the point of zero stress. The moment arm can be expressed as follows

$$z = e_{spl,T} + a_T = \frac{i_T^2}{a_T} + a_T \tag{20}$$

The eccentricity of the axial load measured from the neutral axis of the I-section is

$$e_{spl,I} = \frac{i_T^2}{a_T} + a_T - b = z - b \tag{21}$$

in which b is the distance between the neutral axis of the I-section and the point of zero stress in the T-section, see Figure 3. The rotational stiffness of the splice now is

$$C = \frac{EA_T z^2}{\ell} \tag{22}$$

It should be noted here that using the simple looking equations (19-21) can be quite cumbersome. It is suggested to start the calculation procedure by giving distance b specific values, e.g. $(h/2)$-t_f or 0.0. For a column splice with end plates and a single row of bolts in the center the minimum value for b is 0.0 mm. For this case A_T, I_T, a_T, $e_{spl,T}$ and $e_{spl,I}$ can quite simply be determined as the geometric properties of T-sections obtained from half I-sections given in the literature.

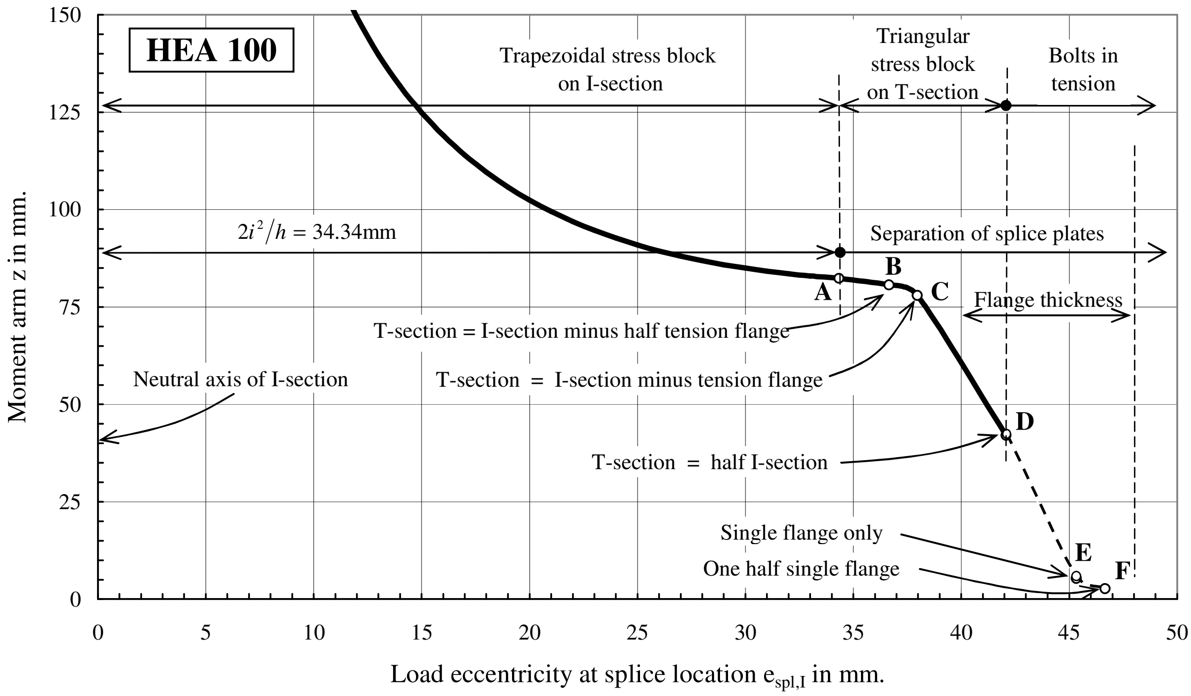

Figure 4: Moment arm versus load eccentricity

Table 1: Influence of imposed imperfections on load capacity of 3.39 m long HE100A spliced column

	Eq. nr.	Full HE100A I-section	Full section minus half tension flange	Full section minus full tension flange	One half HE100A section	Com-pression flange only	Half com-pression flange
Location on graph	-	**A**	**B**	**C**	**D**	**E**	**F**
$b_{imposed}$, mm	-	*48*	*44*	*40*	*0*	*-40*	*-44*
i^2_T, mm^2	-	1644.1	1420.4	757.6	116.9	5.333	1.333
a_T, mm	-	48.00	54.67	66.59	39.09	4.00	2.00
$e_{spl,T}$, mm	19	-	*25.98*	11.38	2.991	1.333	0.6667
$e_{spl,I}$, mm	(9)*, 21	**34.34**	**36.65**	**37.97**	**42.08**	**45.33**	**46.67**
z, mm	(17), 20	**82.34**	**80.65**	**77.97**	**42.08**	**5.33**	**2.67**
C, kNm	(7), 22	892.1	694.7	498.5	116.5	1.44	0.18
$N_{cr,2}$, kN	2	1403.9	1092.9	784.3	183.3	2.265	0.283
$N_{cr,spl}$, kN	3	434.7	399.6	349.3	142.0	2.257	0.283
% of $N_{cr,1}$ (no splice)	-	*69.0*	*63.4*	*55.5*	*22.5*	*0.40*	*0.04*
$\bar{\lambda}$	14	1.071	1.118	1.195	1.875	14.868	41.987
χ	12	0.553	0.525	0.481	0.235	0.004	0.001
$N_{b,Rd,spl}$, kN	15	**275.7**	**261.9**	**239.9**	**117.2**	**2.207**	**0.281**
% of $N_{b,Rd}$ (no splice)	-	*82.8*	*78.6*	*72.0*	*35.2*	*0.66*	*0.08*

* equation numbers in brackets refer to Full HE100A I-section (point A) only.

The two stages with trapezoidal and triangular stress distributions across the section of an HEA100 are shown in Figure 4. Up until a load eccentricity of $2i^2/h = 34.34$ mm (point A) the cross-section is subject to compressive stresses only as is shown in figure 2b. For a load eccentricity $0.0 \leq e_{spl,I} \leq 34.34$ mm there will be a trapezoidal stress distribution across the I-section. The length of moment arm z is given by Eq. (17) and changes from infinity to 82.34 mm.

For load eccentricities larger than 34.34 mm the splice plates will separate and the axial load and bending moment must be resisted by a reduced cross section. The bending moment that can be resisted by the reduced section due to the axial load is smaller than the bending moment on the full I-section as the neutral axis of this T-section is now closer to the axial load, i.e. $e_{spl,T} < e_{spl,I}$ as shown in Figure 3.

For very large eccentricities beyond 42.08 mm (point D) tensile stresses will be introduced into the bolts and the rotational stiffness of the splice mainly becomes a function of individual stiffness factors k_i of the various components resisting the axial force and bending moment which must be used in Eq. (5). This is beyond the scope of this study.

The curve in Figure 4 shows that the moment arm z reduces rapidly with increasing load eccentricity. The curve has been further extended with a dotted line for the condition without bolts.

Table 1 shows intermediate calculation steps for six selected points, A-F, on the $e_{spl,I}$ versus z curve in Figure 4. The table is enlarged for a specific column to show the influence of the imposed column imperfections and thereby the rotational stiffness of the splice on its load carrying capacity. For a 3.39 m long HE100A, S235 column without splice $\alpha = 0.34$, $N_{cr,1} = 629.8$ kN and $N_{pl,Rd} = 499.1$ kN which yields an axial load capacity $N_{b,Rd\,(\text{no splice})} = 333.1$ kN.

DESIGN OF BUTT-PLATE SPLICED COLUMNS

In order to obtain the load carrying capacity of a butt-plate spliced column the rotational stiffness of the splice C must be known. This stiffness is a function of the eccentricity at the splice $e_{spl}\times n/n\text{-}1$ which is to be obtained from a code defined maximum imperfection at column mid height e^*, the critical load of the spliced column $N_{cr,spl}$ and load on the structure N_{Ed}. Both the imperfection and critical load of the spliced column are functions of the rotational stiffness of the butt-plate connection. Therefore an iterative procedure is necessary to obtain the load carrying capacity of the spliced column $N_{b,Rd,spl}$:

1. The column without splice is first designed for a maximum imperfection at mid height. This requires a load check for the column with a maximum second order eccentricity $e^*\times n/n\text{-}1$ defined by Eurocode 3. Obtain $N_{cr,1}$, $N_{pl,Rd}$, λ, χ, $N_{b,Rd}$. This yields e^* and then $(e_{spl}\times n/n\text{-}1)_i$ at splice location.
2. For the spliced column the rotational stiffness of the connection C_i is a function of the load eccentricity $(e_{spl}\times n/n\text{-}1)_i$ at the splice. For $(e_{spl}\times n/n\text{-}1)_i \leq 2i^2/h$ the moment arm z must be obtained for an I-section. For $(e_{spl}\times n/n\text{-}1)_i > 2i^2/h$ the moment arm z must be obtained for a T-section. $N_{cr,2,i}$ and the reduced critical load for the spliced column $N_{cr,spl,i}$ can now be calculated and will yield values for $\lambda_{,i}$, χ_i and $N_{b,Rd,spl,i}$. This will lead to an increased eccentricity $(e_{spl}\ n/n\text{-}1)_{i+1}$ at splice location.
3. If $(e_{spl}\times n/n\text{-}1)_i/(e_{spl}\times n/n\text{-}1)_{i+1} \geq 0.99$ the iteration procedure can stop. At this point the second order imperfection as defined by Eurocode 3 has now been applied to the spliced column. The percentage reduction in $N_{b,Rd,spl}$ will always be smaller than for $N_{cr,spl}$.
4. If $(e_{spl}\times n/n\text{-}1)_i/(e_{spl}\times n/n\text{-}1)_{i+1} < 0.99$, a new reduced rotational stiffness for the splice C_{i+1} must be obtained from eccentricity $(e_{spl}\times n/n\text{-}1)_{i+1}$ and calculate $N_{cr,spl,i+1}$ to continue the iterative procedure.

Table 2: Iterative design procedure for two butt-plate spliced columns

Column	**HE100A, 3.39 m long**						**HE100A, 4.53 m long**			
Iteration	Eq. nr.	**No splice**	**1**	**2**	**3**	**4**	**No splice**	**1**	**2**	**3**
$e_{applied}$, mm	-	0.0	13.77	16.92	18.11	18.55	0.0	29.46	**38.68**	**41.97**
z, mm	17	∞	133.2	114.1	108.9	107.2	∞	85.27	73.38	43.03
C, kNm	7	∞	2333	1713	1560	**1511**	∞	956	709	243.6
$N_{cr,2}$, kN	2	∞	3671	2695	2455	2378	∞	1505	1115	383.3
$N_{cr,spl}$, kN	3	629.8	537.6	510.5	501.2	**497.9**	352.8	285.8	268.0	183.7
% $N_{cr,1}$ (no splice)	-	100	*85.4*	*81.1*	*79.6*	***79.1***	-	*19.0*	*24.0*	*47.9*
$\bar{\lambda}$	14	0.890	0.964	0.989	0.998	1.001	1.189	1.321	1.365	1.648
χ	12	0.667	0.620	0.604	0.598	0.596	0.484	0.417	0.397	0.293
$N_{b,Rd,spl}$, kN	15	333.1	309.6	301.5	298.6	**297.6**	241.5	207.9	198.1	146.2
% $N_{b,Rd}$ (no splice)	-	100	*92.9*	*90.5*	*89.6*	***89.3***	-	*13.9*	*18.0*	*39.46*
e^*, mm	10	9.17	10.15	10.48	10.60	10.65	13.14	14.9	15.47	19.24
n	16	1.89	1.74	1.69	1.68	1.67	1.46	1.37	1.35	1.26
$e_{spl}\times n/n\text{-}1$, mm	-	13.77	16.92	18.11	18.55	18.71	29.46	38.68	41.97	**66.67**
M_{spl}, kNm	-	4.59	5.24	5.46	5.54	5.57	7.11	8.04	8.31	9.75
% $M_{pl,Rd}$ (no splice)	-	*23.5*	*26.9*	*28.0*	*28.4*	***28.6***	*36.5*	*41.2*	*42.6*	*50.0*

The iterative procedure of the calculations is shown in Table 2 for two different lengths of HEA100 columns. The input eccentricity, e = 13.77 mm, for the first iteration is obtained from the first order imperfection defined by Eurocade 3 at mid height, e^* = 9.17, dividing by √2 and multiplying by $n/(n-1)$ to make it a second order imperfection at $x = \ell/4$. $M_{pl,Rd}$ = 19.5 kNm.
The procedure for the shorter column converges to a design load of 297.6 kN which is still 89.3% of the same column without splice. The design of the 4.53 m long column quickly diverges as the eccentricity rapidly increases at an increasing rate.
For second order eccentricities larger than 42.08 mm the bolts will be subjected to tensile forces and the rotational stiffness of the splice then is a function mainly of the bending stiffness of the butt plates. It should be noted here that the eccentricity of the first iteration, e = 29.46 mm still causes only compressive stresses in the splice. The adjusted rotational stiffness of the splice reduces the critical and ultimate loads thereby increasing the second order eccentricity to 38.68 mm that must be applied in the next iteration. The calculation of the second iteration moment arm of the remaining T-section is laborious but can also be read from the graph in Figure 4.
At the end of the third iteration the axial load must be located outside the section. The rotational stiffness of the bolted connection cannot be obtained with the suggested method of analysis anymore.

DISCUSSION

It could be argued that for situations where the entire cross-section at the column splice is subject to compressive stresses, there will be no reduction in stiffness, i.e. for $(e_{spl} \times n/n-1)_{\text{final}} < 2i^2/h$ the rotational stiffness of the splice has an infinite value and the axial load capacity need not be reduced from the no-splice condition. The condition of a complete compressive stress distribution across the full I-section at the splice would also allow the column to be designed without any minimum requirements for the connection. However, if friction is not allowed to be taken into account, a minimum splice would only require to be designed for the shear force at splice location.
It was found that none of the eight building codes studied has adopted such a design approach for column splices with butt-plates. (Snijder & Hoenderkamp, 2005, 2008)

CONCLUSIONS

A method of analysis for spliced columns with single row bolted butt-plates has been presented. Imperfections defined by Eurocode 3 allowed the rotational stiffness of the splice in addition to reduced values for critical and ultimate loads to be obtained with an iterative procedure. The method does not allow tensile forces in the bolts to be included.
The study of the behavior of butt-plate splices has shown that there are boundaries to the eccentricities that can be applied to the connection.
If the design procedure converges to a reduced design load, no additional materials need be applied to the column splice.
Columns with butt-plate splices always need to have their ultimate axial load reduced because Eurocode 3 requires the columns to be designed with specifically defined imperfections.

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