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EXPERIMENTAL TESTS ON SPLICED COLUMNS FOR SPLICE STRENGTH AND STIFFNESS REQUIREMENTS

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Abstract. *A column 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. In this paper strength and stiffness requirements for column splices are derived on the basis of column stability considerations. A strength requirement for the column splice is suggested based on an imperfect column. A stiffness requirement is suggested by allowing a 5% reduction in the Euler buckling load of the column. In order to validate the suggested strength and stiffness requirements for column splices, full scale experimental tests on spliced pin-ended columns were carried out. Three different column splices were used. Columns without splices were tested for reference purposes. All tests were performed three times. The tests showed that the splices do not have a negative influence on the stability of the columns, even when tensile stresses are present at the splice location.*

1 INTRODUCTION

Column splices are required because of several reasons such as insufficient length of a standard section, transportation requirements or necessary changes in section. Columns should preferably be spliced 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 two parts of the column are connected by either end plates or cover plates (figure 1). The vertical load can be transferred by direct bearing or through the splice material. Splice type I in figure 1 has end plates and the compressive force is transferred by bearing. Splice type II has cover plates and the compressive force is either transferred by the splice material (type IIa) or by bearing (type IIb). Eurocode 3 [1] gives design rules for column splices requiring minimum capacities for bending moment and shear force in case of load transfer through the splice material (type IIa) and a minimum normal compressive force to be accounted for in case of bearing (type I and IIb). The background to these rules is unknown and seemingly not based on applied mechanics.

This paper identifies requirements for column splices with respect to stability. Because the splice is part of the column, it must be designed for 2nd order bending moments and shear forces in addition to the axial force. A strength requirement for the column splice is derived on this basis. A stiffness requirement for the column splice is suggested on the basis of allowing a maximum reduction of 5% in the Euler buckling load due to the splice.

In order to validate the suggested strength and stiffness requirements for column splices, a small series of experimental tests was carried out for splice type I, figure 1. Three different column splices were used: splice A with 12 mm thick end plates, splice B with 6 mm thick end plates and splice C only had a single web cover plate which allowed minimum initial stiffness in the connection. The column splices were positioned at quarter points along the longitudinal axis of HE100A S235 column sections.

Full scale tests were performed on 3.4 m long pin-ended columns including a column without splice for reference purposes. Two additional column lengths of 4.5 m and 8.5 m were tested with splice B. All tests were repeated twice, the programme thus consisted of 6×3=18 tests.

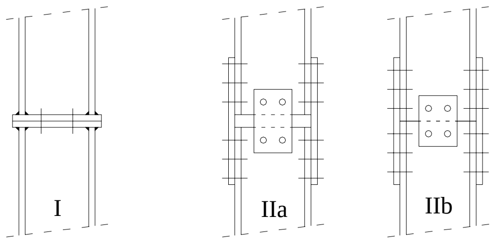


Figure 1: Column splice types.

2 LITERATURE

In Eurocode 3 [1], a distinction is made between bearing and non-bearing column splices. Where the members are not prepared for full contact in bearing, the internal moments should be taken not less than 25% of the moment capacity of the weaker section and the internal shear forces should be taken not less than 2.5% of the normal force capacity. Where the members are prepared for full contact in bearing, splice material should be provided to transmit 25% of the maximum compressive force in the column. The background to these rules could not be traced. Especially the rule of 25% moment capacity can be severe in many practical cases. An inventory of code requirements [2] shows that very different requirements are given by various countries. The Australian code AS 4100 – 2002 (Section 9 Connections) even provides more severe requirements than the Eurocode; also see [3].

Based on earlier research [4,5] the influence of specific column imperfections caused by the application of column splices on the stability of columns was investigated further [6-9]. It was concluded, that in case slip is prevented (e.g. by pre-stressing the bolts) in the splice, standard column stability checks suffice to cover column splice imperfections. In case slip is not prevented, a less favourable buckling curve must be used. It was advised to transfer at least 10% of the normal force by the connectors to secure both column parts in location. Fourteen full scale buckling tests on spliced columns (splice type I) for weak axis buckling were carried out on HE240A (S235) sections. Additional tests were performed on spliced columns with splice type IIb without web cover plates. Results were compared with load bearing capacities confirming that spliced columns can be checked as normal columns for stability. Splice stiffness was not addressed.

Since the research carried out so far on spliced columns is limited and code requirements can be very severe, it was decided to start a research project on strength and stiffness requirements for splices in columns and to perform experimental tests on spliced columns. The results of this project [10] are summarised in this paper.

3 STRENGTH REQUIREMENT

To obtain a strength requirement, the column model as given in figure 2a was used. In the concentrically loaded column a splice is located at a distance x from the base. By setting the column design equation for concentrically loaded columns equal to the formula for imperfect columns subject to axial forces, the column imperfection e^* can be calculated using the following expression:

$$\frac{F}{\chi F_p} = \frac{F}{F_p} + \frac{n}{n-1}\frac{F \cdot e^*}{M_p} \tag{1}$$

In eqn. (1) F is the compressive force, F_p is the section squash load, χ is the reduction factor depending on slenderness and buckling curve, e^* is the imperfection at mid height, n is the ratio between the Euler buckling load F_E and F, and M_p is the plastic moment capacity of the section. Assuming a sinusoidal column imperfection, the imperfection at the splice location can be calculated. Then the 2nd order bending moment M_{spl} at the splice location x is given by:

$$M_{spl} = \frac{n}{n-1}\chi F_p \cdot e^* \sin\left(\frac{\pi x}{l}\right) \tag{2}$$

The shear force V_{spl} at the splice location can be calculated by differentiating the bending moment:

$$V_{spl} = \frac{n}{n-1}\chi F_p \cdot e^* \frac{\pi}{l}\cos\left(\frac{\pi x}{l}\right) \tag{3}$$

The splice can now to designed for strength subject to F, M_{spl} and V_{spl}.

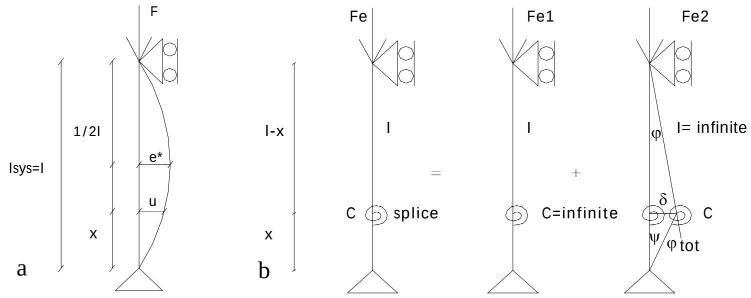


Figure 2: Strength (a) and stiffness (b) requirement model.

4 STIFFNESS REQUIREMENT

A splice may weaken the column thereby reducing its load carrying capacity. Sufficient stiffness at the splice location has to be present. The so-called '5% criterion' is used to obtain a stiffness requirement. This criterion states that a joint (in this case a splice) may be considered as rigid if the ultimate resistance of a frame (in this case a column) in which it is incorporated is not affected by more than 5% compared to the situation where a fully rigid joint is present [11]. Here, this criterion can be safely applied to the Euler buckling load of a spliced column instead of to the ultimate resistance of a spliced column. The Euler buckling load of a spliced column can be estimated by combining the Euler buckling loads for the two subsystems in figure 2b:

$$F_{E,spl} = \frac{C \cdot l}{\dfrac{C \cdot l^3}{\pi^2 EI} + x(l-x)} \tag{4}$$

In eqn. (4), C is the splice stiffness, l is the column length, EI is the column bending stiffness and x indicates the splice location. Acceptance of a 5% reduction in Euler buckling load due to the splice stiffness leads to the following requirement:

$$F_{E,spl} \geq 0.95\frac{\pi^2 EI}{l^2} \tag{5}$$

Substituting eqn. (4) into eqn. (5) leads to the following requirement for the rotational stiffness of the column splice:

$$C \geq 19\frac{\pi^2 EI}{l^2}\frac{x(l-x)}{l} \tag{6}$$

For $x = l/4$ this yields:

$$C \geq \frac{57\pi^2 EI}{16l} \tag{7}$$

5 EXPERIMENTAL TESTING

5.1 Test programme

A test programme was designed on the basis of relatively light sections (HE100A) to keep failure loads limited. The steel grade was S235 but the actual yield stress was measured to be 298 N/mm^2. The columns were designed with at first a relative slenderness of 1.0 resulting in a length of 3390 mm. Three different type I splices were designed: splice A with 12 mm end plates, splice B with 6 mm end plates and splice C with a loosely connected cover plate for the web only, thus providing a connection with minimum initial rotational stiffness. See figure 3. Additional columns without splices were tested for reference.

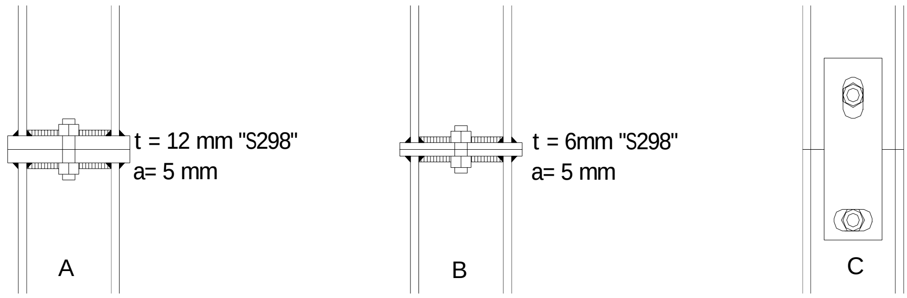


Figure 3: Type I column splices used.

Table 1: Column specimens with constant length 3390mm.

HE100A Length 3390 mm	No splice	Splice A	Splice B	Splice C
Specimen number	1a,b,c	2a,b,c	3a,b,c	4a,b,c
Location splice ¼ l [mm]	n.a.	848	848	848
Splice stiffness C [kNm/rad]	∞	6220	372	minimum
$F_{buc}^{without} = \chi\ A\ f_y$ [kN] without splice	424	424	424	424
$F_{buc}^{with} = \chi\ A\ f_y$ [kN] with splice	424	408	252	minimum
Reduction percentage [%]	0	4	41	unknown

Table 1 gives structural properties of the 3390 mm long specimens. Each test was done three times (a,b,c). The splice stiffness for splice A was relatively large, for splice B it was intermediate and splice C had a minimum stiffness. Eqn. (7) gives for this column a minimum required splice stiffness of 7606 kNm/rad. Theoretically obtained splice stiffnesses are tabulated for types A and B; for splice C the rotational stiffness could not be quantified but it is likely to be lower than that of splice B. The splice stiffness for the three connections do not satisfy the stiffness requirement and therefore it may be expected that the column load bearing capacities will be affected by the presence of a splice. Table 1 gives the estimated failure loads F_{buc} for a column without and with splice in addition to a reduction percentage in failure load compared to the unspliced column.

A generally accepted method for calculating the rotational stiffness of a column splice under compression is not available to the knowledge of the authors. A procedure adopted by the authors to take

Table 2: Column specimens with splice B.

HE100A	Splice B	Splice B	Splice B
Specimen number	3a,b,c	5a,b,c	6a,b,c
Length [mm]	3390	4530	8460
Location splice ¼ l [mm]	848	1133	2115
Splice stiffness C [kNm/rad]	372	120	44
$F_{buc} = \chi\ A\ f_y$ [kN] without splice	424	285	95
$F_{buc} = \chi\ A\ f_y$ [kN] with splice	252	103	24
Reduction percentage [%]	41	64	75

compression into account is based upon extending the lever arm from the position of the normal force F to a virtual point of zero compression beyond the cross-section. This requires the assumption of extended linear strain distribution. However, it could be agued that a normal force suppressing all tensile stresses at the splice location provides full stiffness to the splice. For the specimens with length 3390 mm the normal stresses at the splice location were calculated using F and M_{spl} according to eqn. (2). It can be shown that at the splice location all stresses are compressive and tensile stresses do not occur. On this basis it may be expected that the column load bearing capacities will not be affected by the presence of a splice.

Furthermore, the shear force calculated with eqn. (3) was relatively small and could be taken by the bolts in case of splices A and B and by friction in case of splice C (though friction is not allowed in many codes [2]).

Splice B has been used in 9 tests with three different column lengths, see table 2. Eqn. (7) gives for a column length of 4530 mm a required splice stiffness of 5692 kNm/rad and for a column length of 8460 mm this value becomes 3048 kNm/rad. The calculated splice stiffnesses C are well below this value and therefore it may be expected that the column load bearing capacities will be affected by the presence of the splice: especially for the longer specimens the reduction percentages in failure load increase considerably. Here, using F and M_{spl} according to eqn. (2), it turned out that zero stress was present at one side of the splice and compression at the other side for the 4530 mm long specimens. For the 8460 mm long specimens however, tension occurred at the splice location. On the basis of stresses present at the splice location, at least for the 8460 mm long specimens it may be expected that the column load bearing capacities will be affected by the presence of the splice.

Again, the shear force calculated with eqn. (3) was low.

5.2 Test set-up and testing

In figure 4 the test set-up is shown. The columns were tested in a horizontal position. Hinge connections were made for both end supports; see figure 5 (left). The specimen was supported against weak axis buckling as indicated in figure 5 (right). After the specimen was placed in the frame, column imperfections $e^* = 6.3$ mm for column length 3390 mm, $e^* = 9.1$ mm for column length 4530 mm and $e^* = 18.3$ mm for column length 8460 mm at mid length of the specimen, were applied by an additional horizontal jack at mid span perpendicular to the column axis. The imperfections were obtained from eqn. (1). The axial load was applied by the jack shown in figure 5 (left). The horizontal deflections of the specimen during loading were recorded. The axial load was measured by a load cell.

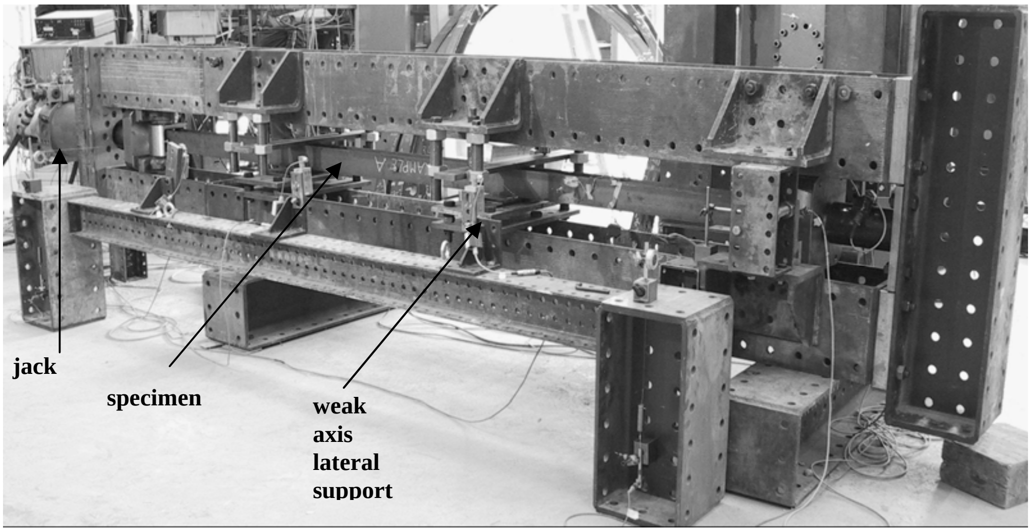
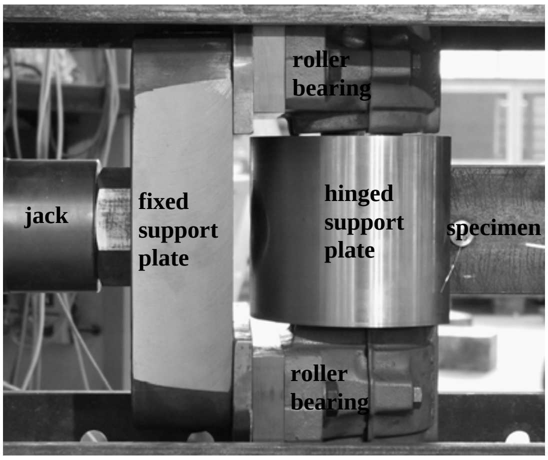


Figure 4: Test set-up.



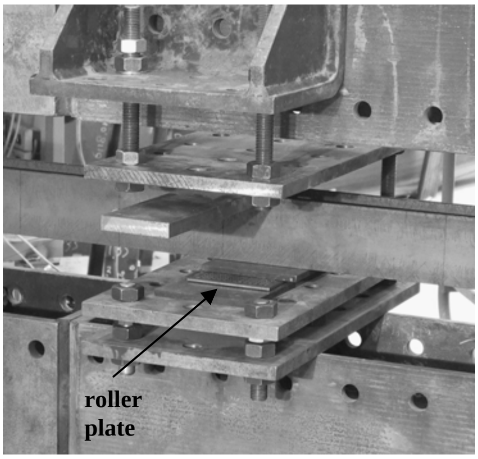


Figure 5: End support (left) and weak axis buckling prevention support (right).

5.3 Test results

The test results for the 3390 mm long specimens are presented in table 3. In the first column, the specimen number is given. Next, the splice is indicated. The length of the specimens is measured and represented in the subsequent column. The actually applied imperfections e^* are then indicated in the next column, followed by their average value for each set of 3 specimens. The experimental column buckling load per test is reported as well as the mean value per set of three tests. Then the calculated buckling load for a column without splice is given belonging to the average value for e^*. In the next column, the calculated buckling load for a column with splice is given. Finally, the percentage of the average experimentally determined buckling load compared to the calculated buckling load for a column without splice is given.

In table 4 the results are given for three different column lengths with splice B.

Table 3: Results for column specimens with constant length 3390 mm.

Spec.	Splice	Column length [mm]	e^* [mm]	e^*_{mean} [mm]	$F_{\exp}$ [kN]	$F_{\exp}^{mean}$ [kN]	$F_{buc}^{without}$ without splice [kN]	F_{buc}^{with} with splice [kN]	$\frac{F_{\exp}^{mean} - F_{buc}^{without}}{F_{buc}^{without}} 100$ [%]
1a		3392	7.0		559				
1b	–	3390	6.4	6.8	561	553	415	415	33
1c		3390	7.0		539				
2a		3390	6.3		531				
2b	A	3389	6.4	6.4	552	544	422	408	29
2c		3389	6.5		548				
3a		3389	6.2		559				
3b	B	3389	6.0	6.1	588	574	427	252	34
3c		3388	6.2		575				
4a		3389	4.6[1]	n.a.	593	n.a.	n.a.		
4b	C	3389	6.3	6.2	574	570	425	min	34
4c		3390	6.1		566				

[1]The imperfection was too small to cause buckling; the test was stopped at 593 kN.

6 DISCUSSION

For the unspliced reference specimens "1" it can be seen in table 3 that the experimental failure load is substantially larger (about 33%) than the calculated failure load on the basis of the buckling curves. This may be due to unintended friction in the test set-up or due to the fact that the European buckling curves which were used to calculate $F_{buc}^{without}$ are conservative. Looking at all experimental failure loads in table 3, all specimens seem to belong to the same population and the mean experimental failure loads are approximately 30% larger than the failure loads using the buckling curves, calculated as if the columns were unspliced. Therefore, it is concluded that the influence of end plate splices on the load bearing capacity of columns seems to be negligible.

The same holds for the long specimens shown in table 4: the mean experimental failure loads being approximately 30% larger than the calculated failure loads for unspliced columns. This is even true for the 8460 mm long specimens where tensile stresses are present in theory at the splice location.

7 CONCLUSIONS

Strength and stiffness criteria for column splices with respect to column stability have been derived. Column splices shall be designed for strength to normal force and 2nd order bending moments and accompanying shear forces. A stiffness requirement was derived on the basis of allowing a maximum reduction of 5% in the Euler buckling load due to the presence of a splice.

Tests on spliced columns showed that the end plate splices do not seem to have a negative influence on the stability of the columns, even for those tests where tensile stresses are theoretically present at the splice location. However, the number of tests was limited and further research is recommended.

Table 4: Results for column specimens with splice B.

Spec.	Splice	Column length [mm]	e^* [mm]	e^*_{mean} [mm]	$F_{\exp}$ [kN]	$F_{\exp}^{mean}$ [kN]	$F_{buc}^{without}$ without splice [kN]	F_{buc}^{with} with splice [kN]	$\frac{F_{\exp}^{mean} - F_{buc}^{without}}{F_{buc}^{without}}100$ [%]
3a	B	3389	6.2	6.1	559	574	427	252	34
3b		3389	6.0		588				
3c		3388	6.2		575				
5a	B	4530	8.9	9.3	389	370	284	103	30
5b		4528	9.3		362				
5c		4529	9.6		361				
6a	B	8462	18.3	18.3	104	123	95	24	29
6b		8460	18.3		149				
6c		8460	18.3		115				

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