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New Materials
New Codes
New Applications
Contact Splices in Compression Members

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Summary

Compression forces in a splice may be transferred by contact. Due to this method additional imperfections occur and must be taken into account. If a contact splice is situated within the length of a column these initial imperfections influence the buckling behaviour and therefore the load carrying capacity of the column. It is reported on theoretical investigations with respect to the member behaviour and the design of the splice and its fastenings. Furthermore 14 full scale tests were carried out. The test results are compared to the results of ultimate load calculations and to results using the method of DIN 18 800 part 2 and Eurocode 3 for compressed columns.

1. Introduction

The load transfer of compression forces within the splice of a column may take place by full contact bearing which is common especially for long columns. Due to this method the amount of welding can be reduced significantly. On the other hand additional efforts with regard to the accuracy must be done.

Splices are normally arranged on the same level as the floors. This system provides a solution for an easy erection of the columns and has the advantage that within the buckling length no splice occur. An additional splice within the column height is necessary if this solution in special cases is not appropriate or possible. In this case the splice must be developed in such a way that all internal forces can be transferred whereby bending moments subjected to dead loads and live loads must be taken into account as well as moments due to tolerances and initial imperfections. In the following end plate connections are dealt with, the results must be enlarged for a butt joint with splice plates and for all splices in scaffolding.
2. Design regulations

Due to the German code for buildings DIN 18 801 [1] the splice material and its fastenings may be designed for 50 percent of the column load if the splice is situated in the outer quarter of the column. If no bending moments have to be transferred, e.g. for base plates, 10 percent of the column load is sufficient to design the fasteners or welds.

In the German bridge code DIN 18 809 [2] it is stated that the fastenings must be designed for an axial force which depends on the slenderness of the column but which at least must be 25 percent of the axial force.

Eurocode 3 [3] says that the members which are prepared for full contact in bearing, the splice shall be designed to provide continuity of stiffness about both axes and to resist any tension where moments are present for any reason, including initial imperfections and second-order deformations. Furthermore the splice material and its fastenings shall be proportioned to carry a force acting in any direction perpendicular to the member of 2.5% of the compressive force.

Additionally prEN 1090 [4] give some indications for tolerances which should be met. The angle of inclination between both end plates must be taken as 1/500 and an eccentricity between the two parts of the column must be taken as e = 5 mm.

A more comprehensive review of the existing regulations up to 1987 is given in [6].

As it can be seen in these codes nothing is said about forces due to eccentricities or initial imperfections and therefore the designer must take a decision by himself. This was the cause for the following investigations.

3. Imperfections at a contact splice

In the case of contact splices additional imperfections occur and must be accounted for. Here end plate connections for the columns are taken into account only. Therefore two types of imperfections subjected to the splice may occur:

- a angle of inclination \( \psi \) between the two endplates, fig. 1 a),
- an eccentricity \( e \) between the two centroidal axis, fig. 1 b).

![Fig. 1. Additional imperfections at the joint](image-url)
Initial imperfections of the column itself consist of the initial bow of the two parts of the column. The value of the angle of inclination between the end plates may be taken as

$$\psi = 1/500$$

(1)

which is in accordance to [4] and is confirmed by measurements of built structures.

In [9] it was reported about measurements on columns with end plate connections of different kind of structures. A mean value of $e = 2.4 \text{ mm}$ was found from 50 measurements. Tolerances for rolled sections give in some cases greater values but it is unlikely that extreme values from two different sections will come together at one joint. Here again the tolerance given by [4] should be taken into account by

$$e = 5 \text{ mm}$$

(2)

For the equivalent initial bow imperfection of the column the results based on the European buckling curves should be accounted for

$$w_0 = L/j$$

(3)

where the value $j$ depends on the buckling curve. As an equivalent geometrical imperfection $j$ is given in [5] $j = 300$ for buckling curve a, $j = 250$ curve b and $j = 200$ curve c respectively.

Due to these three imperfections additional moments may occur which should be taken into account.

### 4. Theoretical investigations

#### 4.1 General

Theoretical investigations were carried out in [7] where some assumptions were taken in order to arrive at a solution which can be used easily in practical design.

For the calculation of the internal forces different methods may be used. Most common is the application of the theory of elasticity in combination with the yield strength $f_y$ as a limiting stress. This method is called in DIN 18 800 and the Swiss code SIA 161 "elastic-elastic" and was used here with respect to an easy design. In this case the initial bow imperfection $w_0$ may be reduced by $2/3$ with respect to [5]. In the analysis second order effects must be taken into account.

The bending moments caused by the different imperfections are given in table 1. In addition to these values the moments due to other loads like wind loads must be added.

Two different limit states must be verified:

- verification of the column itself,
- verification of the contact splice
Table 1. Bending moments due to imperfections

<table>
<thead>
<tr>
<th></th>
<th>moment distribution</th>
<th>( M(\xi) )</th>
<th>eq.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td><img src="image1" alt="Image" /></td>
<td>( M_p(\xi) = N L \psi \left( \frac{\sin^2 \beta \sin^2 \xi}{\varepsilon \sin^2 \varepsilon} \right) )</td>
<td>(4)</td>
</tr>
<tr>
<td>2</td>
<td><img src="image2" alt="Image" /></td>
<td>( M_e(\xi) = Ne \left( \frac{\cos \beta \sin \xi}{\sin \varepsilon} \right) )</td>
<td>(5)</td>
</tr>
</tbody>
</table>
| 3 | ![Image](image3) | \[
M_{\psi}(\xi) = \frac{8NL}{\alpha \varepsilon^2 f} \left( \frac{\cos \beta \sin \xi + \sin(1 - \varepsilon) - 1}{\sin \varepsilon} \right) + \frac{8NL}{\beta \varepsilon^2 f} \left( \frac{\sin \xi (1 - \cos \beta)}{\sin \varepsilon} \right) - \frac{8NL}{\varepsilon f} \left( \frac{\sin \beta \sin \xi}{\sin \varepsilon} \right)
\] | (6) |

### 4.2 Design of the column

Normally all internal forces caused by different effects must be taken into account. But here it seems to be unlikely that the effects of angle of inclination \( \psi \), the eccentricity \( e \) of the two parts of the column and the initial bow \( w_0 \) occur with its extreme values at the same time. Therefore it is defensible to use a rule of propagation of errors in calculating the resulting moment by eq. (7).

\[
M(\xi) = \sqrt{M_p(\xi)^2 + M_e(\xi)^2 + M_{\psi}(\xi)^2}
\]  

(7)

It is of interest whether these bending moments can become greater than the moment of a member without splice with an initial bow \( w_0 \) over the whole length which is given by eq. (8).

\[
M_{max} = \frac{8}{\varepsilon^2 f} \left( \frac{1}{\cos \varepsilon / 2} - 1 \right)
\]

(8)

Intensive parameter studies were carried out, where different buckling curves, column lengths, stations of the splice are accounted for, see [7]. The main result of this investigations was that
columns with splices within the buckling length can be analysed in the same way as columns without splices.

4.3 Design of splice material and its fastenings

In [7] a method is described how to calculate the internal forces at the contact splice now allowing for linear adding up of the moments due to the different types of imperfections.

The result of a simplification of this analysis is given by eqs. (9) to (15) using fig. 2.

![Diagram showing factors for the design of resulting forces in the splice](image)

**Fig. 2. Factors for the design of resulting forces in the splice**

\[ Z \quad \text{resulting tension force in the splice} \]
\[ = N n_t \]

\[ D \quad \text{resulting compression force in the splice} \]
\[ = N n_c \]

\[ V \quad \text{resulting shear force in the splice} \]
\[ = N n_v \]

where \( n_t, n_c, n_v \) are given by fig. 2 and
\[ e_\varepsilon = e + 0.01 + \frac{4L}{f} (\xi - \xi^2) \]  \hspace{1cm} (12)

\[ e = \frac{M}{N} \]  \hspace{1cm} (13)

\[ e_\varepsilon = \frac{W}{A} \]  \hspace{1cm} (14)

\[ W = \text{elastic section modulus} \]

\[ e = L \sqrt{\frac{N}{(EI)}} \]  \hspace{1cm} (15)

5. Test programme and test specimen

Only little information on columns with contact splices by tests on very short columns [8] are available so far. Therefore at the Technical University of Berlin 14 full scale tests on pin-ended columns with wide flange cross sections of mild steel St 37 were carried out in order to see whether the assumptions in [7] are defensible or not [10].

The profiles used in the tests were chosen as HE 240 B which takes into account that the column length \( L \) should be in a range of 3.5 to 5 m, the nondimensional slenderness \( \lambda_x \) should be in the most sensitive range of 0.6 to 1.0 for the influence of the imperfections. The actual yield strength was measured with a mean value of 250 MPa as well as the actual geometrical dimensions. Calculations of the cross section properties show that they were close to the nominal values from tables.

With regard to the contact splices 3 types of columns were tested, which can be seen from table 2:
- columns without splice,
- columns with a splice in midspan,
- columns with a splice in the quarterpoint.

In some tests additionally moments about the strong axis at both ends were introduced with a value of \( M_{yr} = 0.025 \text{ N} \), where \( N \) ist the axial force.

The initial bow imperfections were also measured, both for the single parts of the columns with a maximum value of \( L/1430 \) and the columns which wer mounted from two parts with a maximum value of \( L/820 \). In the later case an additional eccentricity between the two column axis of around 1 mm could also be seen.
### Table 2

<table>
<thead>
<tr>
<th>test no</th>
<th>L [m]</th>
<th>splice</th>
<th>eccentricity [cm]</th>
<th>Ntey [kN]</th>
<th>Nety [kN]</th>
<th>Ncal [kN]</th>
<th>Ncode [kN]</th>
<th>Ncode Nety</th>
<th>we</th>
<th>1/ψy</th>
<th>mm</th>
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<td>10</td>
<td>4.84</td>
<td>w</td>
<td>0</td>
<td>1945</td>
<td>1908</td>
<td>0.981</td>
<td>1633</td>
<td>0.840</td>
<td>3.0</td>
<td>-</td>
<td>460</td>
</tr>
<tr>
<td>20</td>
<td>4.84</td>
<td>w</td>
<td>0</td>
<td>2072</td>
<td>1997</td>
<td>0.964</td>
<td>1633</td>
<td>0.788</td>
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<td>m</td>
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<td>(1731)*</td>
<td>1.33</td>
<td>1.70</td>
<td>180</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
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<td>m</td>
<td>0</td>
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<td>1683</td>
<td>1.011</td>
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<td>0.981</td>
<td>1.70</td>
<td>180</td>
<td></td>
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<tr>
<td>50</td>
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<td>q</td>
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<td>1978</td>
<td>1.005</td>
<td>1668</td>
<td>0.847</td>
<td>2.79</td>
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<td>q</td>
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<td>1809</td>
<td>1866</td>
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<td>2.66</td>
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<tr>
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<td>0</td>
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<tr>
<td>90</td>
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<td>m</td>
<td>0</td>
<td>2251</td>
<td>2208</td>
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<td>0.892</td>
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<td>100</td>
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<td>2204</td>
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<td>2007</td>
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<td>1.20</td>
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<tr>
<td>110</td>
<td>4.50</td>
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<td>2.5</td>
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<td>1650</td>
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<tr>
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<tr>
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<td>1809</td>
<td>1819</td>
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<td>1702</td>
<td>0.941</td>
<td>1.46</td>
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<td></td>
</tr>
<tr>
<td>140</td>
<td>3.70</td>
<td>q</td>
<td>2.5</td>
<td>1920</td>
<td>1873</td>
<td>0.976</td>
<td>1702</td>
<td>0.886</td>
<td>1.86</td>
<td>132</td>
<td></td>
</tr>
</tbody>
</table>

\[ m = 0.995 \]

<table>
<thead>
<tr>
<th>splice:</th>
<th>w: without, m: midspan, q: quarterpoint</th>
</tr>
</thead>
</table>

- screws broken

Great effort was taken to measure the angle of deviation of the end plates. A special measurement set up was determined allowing to measure geometrical values at 36 up to 100 point per plate. In order to avoid measurement errors all measurements were taken at least three times. For this measurements the end plates were used which are a part of the columns tested in this programme and end plates of columns which were produced in 3 different constructual steel companies in Germany. The different types of end plates can be seen from fig. 3. It should be mentioned especially that no special methods were used regarding the accuracy of the end plates such as to mill the plates.

Another problem was the evaluation of the measurements because the angle of deviation of the end plate must be defined. In fig. 4 a picture of the spatial initial deformation of one of the end plates is given. An intersection in 2 rows show the results of Fig. 4 b.

From a combination of the angles of inclination in both axis results for each end plate can be calculated. This leads to values of 1/ψ between 110 and 856 and is therefore in most cases greater as the assumend value of 500 due to chapter 3.

It is known that residual stresses normally have a remarkable influence on the ultimate load of compressed members. Therefore residual stresses were measured here by using the Lehigh cut-off-method. The values of the strains measured were brought in a satisfying form by using a balance calculation. The extreme values are only 0.12 f, and are therefore in this case of less
importance to the load carrying capacity.

Fig. 3. Different types of end plates measured

Fig. 4. Spatial initial deformation of an end plate and measured values in 2 intersections

6. Test set up and test implementation

For the tests a 5000 kN-machine was used. At the end of the columns special supports were used, which were described earlier in detail [11]. Lateral deflections and twist are prevented but no resistance is provided against lateral bending whereas full resistance against torsion is given.
It is difficult to adjust the columns in such a way that they are exactly loaded centrically at both ends. A small unintentional eccentricity between 0 and 5 mm cannot be ruled out.

The tests were carried out with a speed of $\Delta \sigma = 1$ kN/(cm² min) which was proposed by TC 8 of ECCS. Deflections at midspan, the quarterpoints and the ends were measured every 100 kN up to a load of 1000 kN and constantly when approaching the ultimate load. The load-deflection-curves show the typical behaviour of compressed columns. The maximum of the load-deflection-curve was defined as the ultimate load. The ultimate loads reached in the tests are given in table 2.

7. Results of the column tests and comparisons

The test results are given in table 2. It can be seen that in tests no 70, 90, 100, which have splices at the quarterpoint and midspan respectively greater ultimate loads appeared as in tests no 10, 20 without splices. Therefore comparisons by ultimate load calculations were carried out in order to look for the reasons. For this comparisons the computer-programme LIDUR [12] was used which takes into account the plastic behaviour of the steel, spread of plastic zones in the longitudinal direction of the column, change of centred and shear center due to partly plasticisation of the cross section, residual stresses and initial bow imperfections. The angle of inclination between the two parts of the columns in the case of contact splices were accounted for by additional transverse forces $P = N \psi$. Furthermore the influence of an unintentional end eccentricity was investigated. Using the values measured for each test specimen theoretical ultimate loads are calculated which correspond with a mean value of 0.995 sufficiently with the test result, see table 2.

Additionally the ultimate design loads are calculated using the regulations of DIN 18 800 part 2 [5] and of Eurocode 3 assuming the column as an element without splices. Hereby the actual measured yield stresses were taken into account. The results are also given in table 2. It can be seen, that in all cases the design load due to the code is smaller or nearly the same as the test results with a mean value of 0.902. Therefore it is justified, as proposed in chapter 4, to design a column with a contact splice in the same way as an usual column. Besides that the contact splice itself and its fastenings must be treated in each individual case which can be done due to chapter 5.

8. Acknowledgements

The author would like to express his gratitude to the "Deutsche Forschungsgemeinschaft (DFG)" for providing financial assistance for the tests, to Dr.-Ing. R. Gietzelt/Berlin for carrying out the theoretical investigations and for Dipl.-Ing. Viest/Berlin for conducting the tests. Furthermore the German steel fabricating companies Krupp Stahlbau/Berlin, Stahlbau Lavis/Offenbach and Stahlbauwerk Müller/Offenburg offered the possibility the take measurements of steel components during production which was particularly appreciated.

9. Conclusions

If a contact splice with base plates is situated within the column length this column may be
designed in the same way as a column without splice. Besides that the contact splice itself and its fastenings must be treated in each individual case due to the internal forces including effects of imperfections.

References


