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Grzegorz GREMZA[2]

ON THE DESIGN OF A STEEL END-PLATE BEAM-TO-COLUMN BOLTED JOINT ACCORDING TO PN-EN 1993-1-8

Considering joints with unstiffened columns, the load capacity of an inner bolt-row being a part of bolts group defined by a flange capacity is directly proportional to a distance between bolts. In turn, a flexibility of the column flange in the inner bolt-row area depends not only on that distance but also on a flexibility of other basic joint components. Hence, that situation may occur, when internal forces in inner bolt-row will be greater than its capacity estimated as an equivalent of T-stub. This possibility has been taken into account in the standard [3] – see the rule in the point 6.2.4.2 (3). In practice, this rule is not implemented in calculations of this kind of joints. In this work, a simplified algorithm of these joints calculation as well as an example, where the need for force reduction in the inner bolt-row to the value of bolt resistance has occurred, were presented. Moreover, the influence of the aforementioned reduction on the joint stiffness was estimated.

Keywords: component method, equivalent T-stub, joint capacity and stiffness

1. Introduction

In available publications in the field of bolted end-plate beam-to-column joints calculation, e.g. [1], [2], it is recognized that the load capacity of analysed joint is sufficient, if the condition $M_{j,Ed} < M_{j,Rd}$, introduced in point 6.2.7(1) of standard [3] is fulfilled, what may result from the general rule contained in the point 6.1.3 (4) of this standard. However, in specific provisions concerning the capacity of an equivalent of T-stub in tension zone – see point 6.2.4.2(3) – additional requirements regarding the values of forces in each bolt-rows and in groups of these rows are introduced.

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It is required that:

a) forces transferred by each bolt-row should not exceed the design resistance determined considering only that individual bolt-row, and,

b) the total force of each bolt-rows group, comprised two or more adjacent bolt-rows within the same bolt-group, should not exceed the design resistance of that group of bolt-rows.

These provisions indicate the necessity of estimation the values of forces in individual bolt rows and groups of rows, in order to compare them with the resistances of these rows and groups of rows. If the forces in some rows or group of rows would be greater than their load capacity, the load capacity of the joint should be reduced.

Such a case may occur in joint of a beam with an unstiffened column (Fig. 1a), when the load capacity of these joint is determined by the resistance of the column flange in tension zone.

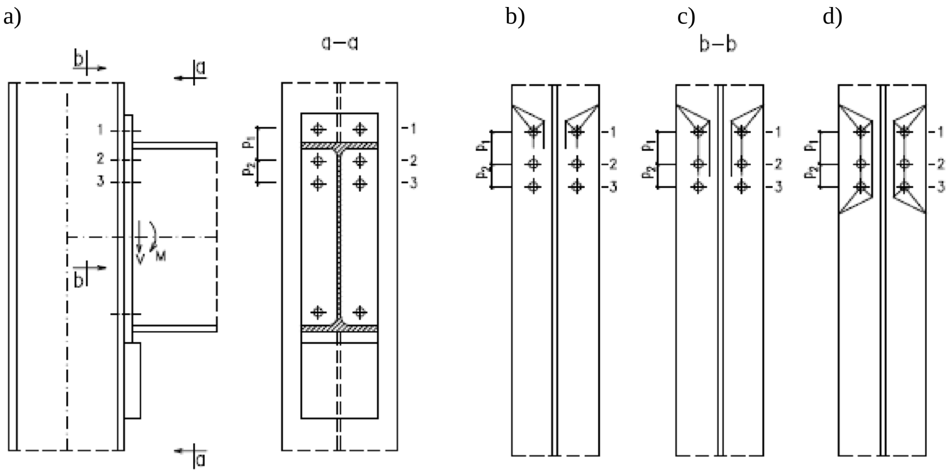


Fig. 1. Joint of a beam with an unstiffened column; a) side view and section a-a, b), c) and d) yield lines of a column flange

According to the standard [3], the model of the destruction of the unstiffened column flange is assumed analogously to the model of equivalent T-stub, considering the individual bolt rows and groups of these rows. Bolt rows are numbered starting from the most distant one from the centre of compression. In case of unstiffened column flange, only one group of bolts with the 1st and next rows may occur, while in case of the end plate – two groups, one group above a beam flange and the second one under that flange. Thus, in a column with an unstiffened web, group of rows 1-2, 1-2-3 or 1-2-3-4 may occur, if the fourth row is present, while group 2-3 or 2-3-4 does not occur. However, such groups appear for the modelling of end-plate with one row of bolts placed above a beam flange.

The yield lines for an equivalent of T-stub flange of the column for the non-circular mechanism of failure is presented in Fig. 1d. If a group of bolts, consisted of rows *1-2* in joint with three rows is considered (see Fig. 1c), the length of the equivalent of T-stub is bigger only by $(p_1 + p_2)/2$ from the effective length for row *1* of this group. Thus, there is a big difference of the effective length values for bolts in row *1* and *2* from a group of rows (compare Fig. 1b and 1c). Analysis of these lengths indicates that the design resistance capacity of the inner bolt rows of the bolt group is much smaller than design resistance of the end bolt-row and is directly proportional to the effective length of the inner row. In turn, the distribution of forces for individual bolt rows is dependent on the stiffness of all components of the joint - see Fig. 2, on which only the stiffness coefficients relevant to the rotational stiffness of the joint are shown.

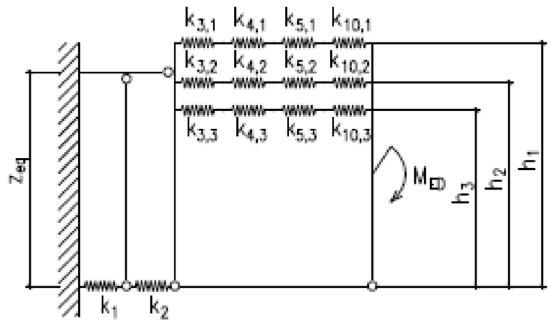


Fig. 2. Component model of joint with marking as in [N1]

For instance, decreasing of the effective length of an unstiffened column flange in bending in the area of row 2 by 50% will also decrease the load capacity of this flange by 50%, but decreasing of the connection stiffness in that area does not exceed a dozen or so percent. This is due to the fact that the flexibility of a joint in the area of the bolt-row 2 is the sum of the flexibility of the joint components: column web in tension, column flange in bending, end-plate in bending, bolts in tension and, in addition, a beam web flexibility in the tensile zone, which was omitted in the algorithm of the stiffness of joints calculation given in standard [3] as a negligible value. It can, therefore, be concluded that when the effective length of the column flange in bending in the area of the inner rows is changed, the redistribution of internal forces in the connection will be different than the variation of the load capacity of these rows, which means that the internal bolts may be overloaded.

The presented analysis indicates that checking the condition of the joint load capacity in accordance with 6.2.7.1(1) in [3]: $M_{j,Ed} < M_{j,Rd}$ where $M_{j,Rd}$ – load capacity according to 6.2.7.2 may not be sufficient. So, in order to ensure the load-bearing capacity of individual bolt rows, it is necessary to fulfill the conditions contained in point 6.2.4.2(3) of standards [3].

In the paper, the algorithm for calculating of an end-plate joint between a beam and unstiffened column with three rows of bolts, with an indication of what components should be considered for individual rows of bolts and groups of rows, will be presented. In that algorithm, the requirements listed in point 6.2.4.2 (3) of standards [3] will be taken into account. This algorithm will be supplemented with a numerical example, in which there will be a need to reduce the load capacity of the joint due to the need to ensure the safety of bolt group 1-2.

The influence of earlier plasticization of the T-stub flange (at lower values of the bending moment in the frame girder) on the rotational stiffness of the joint will also be assessed.

2. Algorithm of beam-to-column joint load capacity estimating

Analyzed joint consists of compression zone, shear zone and tension zone. The load-bearing capacity of the tension zone is not greater than the minimum load capacity of the compression and shearing zones. According to [3], in the compression zone are located: column web, web and flange of a frame girder (beam) or flange and web of a haunch, and the shear zone consists of a web of a column. In the tension zone is located: column web, column flange, end-plate with bolts and web of the frame girder. In general, the way of estimation the joint load capacity according to the standard involves determining the minimum design resistance of the joint parts in tension and in shear and then determining the minimum design resistance of each bolt-row as individual as well as a part of a group of them. Next, the sum of obtained design resistances in the rows from 1 to *r* is compared with the minimum load capacity of the joint parts in tension zone and in the compression zone. If this sum is greater than the minimum load capacity of the tension zone and compression zone, the load capacity of the row *r* is reduced. The second stage is to determine the load capacity of the joint according to the standard formula (6.23) [3]. In the third stage, the values of forces in each bolt rows are determined, e.g. by use of the elastic model as in Fig. 2. Then, the obtained values of forces are compared in individual bolt rows and in bolt-row groups. In the case when the forces in the rows or groups of rows are greater than their design resistances, the calculated load capacity of the joint is proportionally reduced.

Details of the algorithm are given in Table 1.

Table 1. Details of algorithm of beam-to-column joint calculation

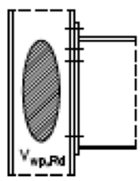
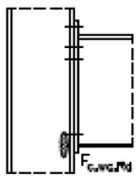
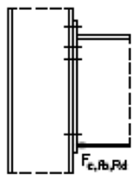
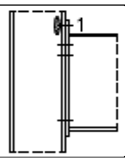
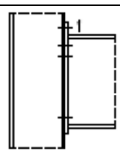
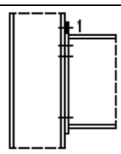
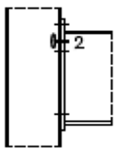
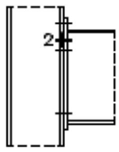
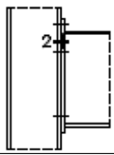
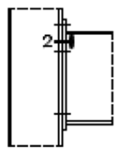
Design resistance of basic components in compression and shear			
$F_{c,v,\min,Rd} = \min\begin{cases} V_{wp,Rd}/\beta \\ F_{c,wc,Rd} \\ F_{c,fb,Rd} \end{cases}$	Column web panel in shear	Column web in transverse compression	Beam flange and web in compression
			
	$V_{wp,Rd}/\beta$ acc. 6.2.6.1	$F_{c,wc,Rd}$ acc. 6.2.6.2	$F_{c,fb,Rd}$ acc. 6.2.6.7

Table 1. (cont.) Details of algorithm of beam-to-column joint calculation

<table>
<tr><th colspan="4">Algorithm of the design resistance of the bolt-row 1 determination</th></tr>
<tr><td rowspan="3">$F_{t1,Rd} = \min\begin{cases} F_{t1,wc,Rd} \\ F_{t1,fc,Rd} \\ F_{t1,ep,Rd} \end{cases}$
acc. 6.2.7.2(6)</td><td>Column web in transverse tension</td><td>Column flange in transverse bending</td><td>End-plate in bending</td></tr>
<tr><td></td><td></td><td></td></tr>
<tr><td>$F_{t1,wc,Rd}$ acc. 6.2.6.3</td><td>$F_{t1,fc,Rd}$ acc. 6.2.6.4</td><td>$F_{t1,ep,Rd}$ acc. 6.2.6.5</td></tr>
<tr><td colspan="4">Limiting the design resistance due to the compression and shear of the joint parts, acc. 6.2.7.2(7)
$F_{t1,Rd} \le F_{c,v,\min,Rd}$</td></tr>
</table>

<table>
<tr><th colspan="5">Design resistance of individual bolt-row 2</th></tr>
<tr><td rowspan="3">$F_{t2,Rd,ind} = \min\begin{cases} F_{t2,wc,Rd} \\ F_{t2,fc,Rd} \\ F_{t2,ep,Rd} \\ F_{t2,wb,Rd} \end{cases}$
acc. 6.2.7.2(6)</td><td>Column web in transverse tension</td><td>Column flange in transverse bending</td><td>End-plate in bending</td><td>Beam web in tension</td></tr>
<tr><td></td><td></td><td></td><td></td></tr>
<tr><td>$F_{t2,wc,Rd}$
acc. 6.2.6.3</td><td>$F_{t2,fc,Rd}$
acc. 6.2.6.4</td><td>$F_{t2,ep,Rd}$
acc. 6.2.6.5</td><td>$F_{t2,wb,Rd}$
acc. 6.2.6.8</td></tr>
<tr><td colspan="5">Limiting the design resistance due to compression and shear of the joint parts
$F_{t2,Rd,ind} \le F_{c,v,\min,Rd} - F_{t1,Rd}$.</td></tr>
</table>

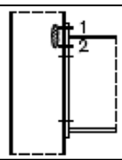
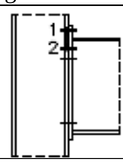
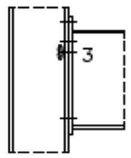
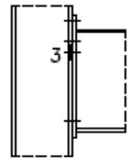
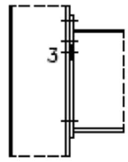
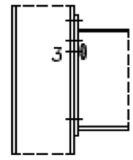
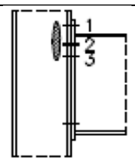
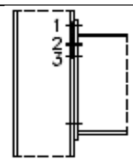
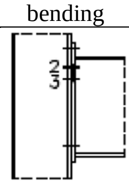
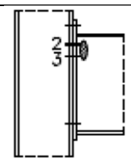
<table>
<tr><th colspan="3">Design resistance of row 2 as a part of group of bolt rows 1-2</th></tr>
<tr><td rowspan="3">$F_{t1-2,Rd} = \min\begin{cases} F_{t1-2,wc,Rd} \\ F_{t1-2,fc,Rd} \end{cases}$</td><td>Column web in transverse tension</td><td>Column flange in transverse bending</td></tr>
<tr><td></td><td></td></tr>
<tr><td>$F_{t1-2,wc,Rd}$ acc. 6.2.6.3</td><td>$F_{t1-2,fc,Rd}$ acc. 6.2.6.4</td></tr>
<tr><td colspan="3">Limiting the design resistance of the bolt-row due to design resistance of the group of bolt-rows according to 6.2.7.2(8): $F_{t2,Rd} = \min(F_{t2,Rd,ind}, F_{t2,Rd,group})$, where $F_{t2,Rd,group} = F_{t1-2,Rd} - F_{t1,Rd}$.</td></tr>
<tr><td colspan="3">If in connections exposed to dynamic actions and vibrations $F_{t1,Rd} > 1.9F_{t2,Rd}$
then $F_{t2,Rd} \le F_{t1,Rd} \cdot h_2 / h_1$ - acc. 6.2.7.2 (9) +NA.5.</td></tr>
</table>

Table 1. (cont.) Details of algorithm of beam-to-column joint calculation

<table>
<tr><td colspan="5">Design resistance of individual bolt-row *3*</td></tr>
<tr><td rowspan="2">$F_{t3,Rd,ind} = \min\begin{cases} F_{t3,wc,Rd} \\ F_{t3,fc,Rd} \\ F_{t3,ep,Rd} \\ F_{t3,wb,Rd} \end{cases}$</td><td>Column web in transverse tension</td><td>Column flange in transverse bending</td><td>End-plate in bending</td><td>Beam web in tension</td></tr>
<tr><td>$F_{t3,wc,Rd}$
acc. 6.2.6.3</td><td>$F_{t3,fc,Rd}$
acc. 6.2.6.4</td><td>$F_{t3,ep,Rd}$
acc. 6.2.6.5</td><td>$F_{t3,wb,Rd}$
acc. 6.2.6.8</td></tr>
<tr><td colspan="5">Limiting of the bolt-row resistance due to components in bending and shear
$F_{t3,Rd} \le F_{c,v,\min,Rd} - (F_{t1,Rd} + F_{t2,Rd})$ - acc. 6.2.7.2(7).</td></tr>
<tr><td colspan="5">Design resistance of row *3* as a part of group of rows *1 – 3* and *2 – 3*</td></tr>
<tr><td rowspan="2">$F_{t1-3,Rd} =$
$\min\begin{cases} F_{t1-3,wc,Rd} \\ F_{t1-3,fc,Rd} \end{cases}$
$F_{t2-3,Rd} =$
$\min\begin{cases} F_{t2-3,ep,Rd} \\ F_{t2-3,wb,Rd} \end{cases}$</td><td>Column web in transverse tension</td><td>Column flange in transverse bending</td><td>End-plate in bending</td><td>Beam web in tension</td></tr>
<tr><td>$F_{t1-3,wc,Rd}$
acc. 6.2.6.3</td><td>$F_{t1-3,fc,Rd}$
acc. 6.2.6.4</td><td>$F_{t2-3,ep,Rd}$
acc. 6.2.6.5</td><td>$F_{t2-3,wb,Rd}$
acc. 6.2.6.8</td></tr>
<tr><td colspan="5">Limiting of resistance due to resistance of the group of rows – acc. 6.2.7.2(8)
$F_{t3,Rd} = \min(F_{t3,Rd,ind}, F_{t3,Rd,group,c}, F_{t3,Rd,group,b})$,
where $F_{t3,Rd,group,c} = F_{t1-3,Rd} - (F_{t1,Rd} + F_{t2,Rd})$, $F_{t3,Rd,group,b} = F_{t2-3,Rd} - F_{t2,Rd}$.</td></tr>
<tr><td colspan="5">If in connections exposed to dynamic actions and vibrations
$F_{t3,Rd} > 1.9F_{tx,Rd}$, where x = 1.2 to $F_{t3,Rd} \le F_{tx,Rd} \cdot h_3 / h_x$ - acc. 6.2.7.2 (9) +NA.5.</td></tr>
<tr><td colspan="5">Design bending resistance $M_{j,Rd}$ acc. to formula (6.25)</td></tr>
<tr><td colspan="5">Stiffness coefficients of the joint basic components – acc. to point 6.3.</td></tr>
<tr><td colspan="5">Values of forces in each bolt-row ($N_{i,Ed}$)
may be calculated e.g. using the model shown in Fig. 2.</td></tr>
<tr><td colspan="5">The requirements contained in point 6.2.4.2(3) a), omitted in algorithms:
If $N_{i,Ed} > F_{ti,Rd,ind}$, then $w_i = F_{ti,Rd,ind} / N_{i,Ed}$ for $i = 1, 2, 3$</td></tr>
<tr><td colspan="5">The requirements contained in point. 6.2.4.2(3) b), omitted in algorithms
If $\sum_{i=1}^{r} N_{i,Rd} > F_{t1-r,Rd}$, then $w_{r+2} = F_{t1-r} \Big/ \sum_{i=1}^{r} N_{i,Rd}$ for $r = 2, 3$ - for group of column flange.
If $\sum_{i=2}^{3} N_{i,Rd} > F_{t2-3,Rd}$, then $w_6 = F_{t2-3} \Big/ \sum_{i=2}^{3} N_{i,Rd}$ - for end-plate</td></tr>
<tr><td colspan="5">If $w_i < 1.0$, then $w_{\min} = \min(w_i)$ for $i =$ 1, by 1, to 6 and
$M_{j,Rd,red} = w_{\min} \cdot M_{j,Rd}$.</td></tr>
</table>

3. Calculation example

As an example, a joint in frame that was made of S235 steel, with end-plate bolted connection category E between IPE 500 and column HE 300 B with geometrical characteristics as in Fig. 3, was selected.

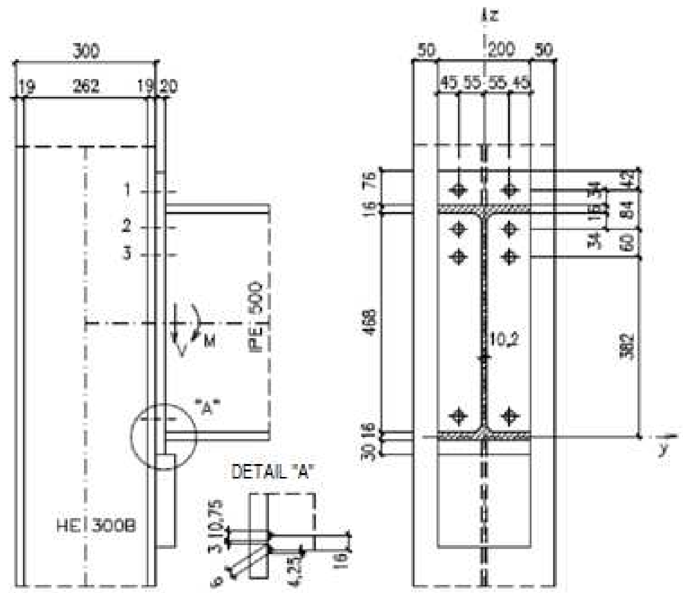


Fig. 3. Analyzed beam-to column joint with bolts M20-10.9

The obtained results of calculations are presented in Table 2.

Table 2. Design resistance of a joint without reduction resulting from the rule in point 6.2.4.2 (3)

<table>
<tr><td colspan="4">Design resistance of joint basic components in compression and in shear</td></tr>
<tr><td rowspan="3">$F_{c,v,\min,Rd} = \min\begin{cases} V_{wp,Rd}/\beta \\ F_{c,wc,Rd} \\ F_{c,fb,Rd} \end{cases}$
= 579.4 kN</td><td>Column web panel in shear</td><td>Column web in transverse compression</td><td>Beam flange and web in compression</td></tr>
<tr><td>579.4 kN</td><td>588.1 kN</td><td>1 068 kN</td></tr>
<tr><td>$V_{wp,Rd}/\beta$</td><td>$F_{c,wc,Rd}$</td><td>$F_{c,fb,Rd}$</td></tr>
<tr><td colspan="4">Algorithm of the design resistance of the bolt-row *1* determination</td></tr>
<tr><td rowspan="3">$F_{t1,Rd} = \min\begin{cases} F_{t1,wc,Rd} \\ F_{t1,fc,Rd} \\ F_{t1,ep,Rd} \end{cases}$
= 264.3 kN</td><td>Column web in transverse tension</td><td>Column flange in transverse bending</td><td>End-plate in bending</td></tr>
<tr><td>411.0 kN</td><td>266.5 kN</td><td>264.3 kN</td></tr>
<tr><td>$F_{t1,wc,Rd}$</td><td>$F_{t1,fc,Rd}$</td><td>$F_{t1,ep,Rd}$</td></tr>
<tr><td colspan="4">There is no need to limit the resistance due to joint components in bending and shear
$F_{t1,Rd} = 264.3\,\text{kN} < F_{c,v,\min,Rd} = 579.4\,\text{kN}$</td></tr>
</table>

Table 2. (cont.) Resistance of a joint without reduction resulting from the rule in point 6.2.4.2 (3)

Design resistance of individual bolt-row *2*

	Column web in transverse tension	Column flange in transverse bending	End-plate in bending	Beam web in tension
$F_{t2,Rd,ind} = \min\begin{cases}F_{t2,wc,Rd}\\F_{t2,fc,Rd}\\F_{t2,ep,Rd}\\F_{t2,wb,Rd}\end{cases}$ =266.5 kN	411.0 kN	266.5 kN	323.1 kN	624.2 kN
	$F_{t2,wc,Rd}$	$F_{t2,fc,Rd}$	$F_{t2,ep,Rd}$	$F_{t2,wb,Rd}$

There is no need to limit the load capacity due to joint components in bending and shear

$$F_{t2,Rd,ind} = 266.5\,\text{kN} \le 579.4 - 264.3 = 315.1\,\text{kN}$$

Resistance of row *2* as a part of a the group of rows *1-2*

	Column web in transverse tension	Column flange in transverse bending
$F_{t1-2,Rd} = \min\begin{cases}F_{t1-2,wc,Rd}\\F_{t1-2,fc,Rd}\end{cases}$ = 348.5 kN	506.6 kN	348.5 kN
	$F_{t1-2,wc,Rd}$	$F_{t1-2,fc,Rd}$

Limiting of the row resistance due to the resistance of the group of rows:

$$F_{t2,Rd,group} = 348.5 - 264.3 = 84.2\,\text{kN},\ F_{t2,Rd} = \min(266.5, 84.2) = 84.2\,\text{kN}$$

Connection is not exposed to dynamic actions and vibrations.

Design resistance of individual row *3*

	Column web in transverse tension	Column flange in transverse bending	End-plate in bending	Beam web in tension
$F_{t3,Rd,ind} = \min\begin{cases}F_{t3,wc,Rd}\\F_{t3,fc,Rd}\\F_{t3,ep,Rd}\\F_{t3,wb,Rd}\end{cases}$ = 266.5 kN	411.0 kN	266.5 kN	303.7 kN	537.6 kN
	$F_{t3,wc,Rd}$	$F_{t3,fc,Rd}$	$F_{t3,ep,Rd}$	$F_{t3,wb,Rd}$

Limiting of the resistance due to components in bending and shear:

$F_{t3,Rd} \le$ 579.4-(264.3+84.2)= 230.9 kN, hence $F_{t3,Rd}$ = 230.9 kN.

Design resistance of row *3* as a part of group *1 – 3 and 2 – 3*

	Column web in transverse tension	Column flange in transverse bending	End-plate in bending	Beam web in tension
$F_{t1-3,Rd} = \min\begin{cases}F_{t1-3,wc,Rd}\\F_{t1-3,fc,Rd}\end{cases}$ = 569.2 kN	688.1 kN	569.2	538.0 kN	768.0 kN
$F_{t2-3,Rd} = \min\begin{cases}F_{t2-3,ep,Rd}\\F_{t2-3,wb,Rd}\end{cases}$ = 538.0 kN	$F_{t1-3,wc,Rd}$	$F_{t1-3,fc,Rd}$	$F_{t2-3,ep,Rd}$	$F_{t2-3,wb,Rd}$

Limiting of the resistance due to the resistance of group of row – acc. 6.2.7.2(8)

$$F_{t3,Rd,group,\text{c}} = 569.2 - (364.3 + 84.2) = 220.7\,\text{kN},\ F_{t3,Rd,group,\text{b}} = 538.0 - 84.2 = 453.8\,\text{kN}$$

$$F_{t3,Rd} = \min(230.9, 220.7, 453.8) = 220.7\,\text{kN}$$

Design resistance of a joint in bending

$$M_{j,Rd} = \sum_r F_{tr,Rd} \cdot h_r = 264.3 \cdot 0.526 + 84.2 \cdot 0.442 + 220.7 \cdot 0.382 = 260.54\,\text{kNm}$$

Table 2. (cont.) Resistance of a joint without reduction resulting from the rule in point 6.2.4.2 (3)

Stiffness coefficient of the basic joint components				
Component	Stiffness coefficient	Bolt-row		
		1	*2*	*3*
Column web in transverse tension	$k_3 \cdot 10^3$ [m]	4.620	2,116	4.267
Column flange in transverse bending	$k_4 \cdot 10^3$ [m]	44.683	20.466	41.272
Beam web in tension	$k_5 \cdot 10^3$ [m]	25.129	17.328	13.810
Bolts in tension	$k_{10} \cdot 10^3$ [m]	6.374	6.374	6.374
The effective stiffness coefficient	$k_{eff,r} \cdot 10^3$ [m]	2.296	1.359	2.050
The equivalent lever arm	z_{eq} [m]	0.463		
Column web panel in shear	$k_1 \cdot 10^3$ [m]	3.894		
Column web in compression	$k_2 \cdot 10^3$ [m]	8.655		

Bolt forces calculated according to the model as in Fig. 2 for $M_{Ed} = M_{j,Rd} = 260{,}5$ kN and design resistances of bolts			
Bolt-rows	Force [kN]	Design resistance [kN]	w_i
1	173.9	264.3	
2	247.7	266.5	
3	155.9	230.9	
1-2	173.9+247.7 = 421.6	348.5	0.827
1-3	421.6+155.9 = 577.5	569.2	0.986
2-3	247.7+155.9 = 403.6	538.0	
$w_{\min} = 0.827$			
$M_{j,Rd,red} = 0.827 \cdot 260.5 = 215.5$ kNm			

Using the standard formula (6.27) in [3], the effect of the joint load capacity reduction on its stiffness S_j was evaluated. The stiffness ratio μ given in formulas (6.28a) and (6.28b) was taken into account. Obtained results in the form of the M - $ø$ relationship, where $ø = M/S_j$ is shown in Fig. 4.

After load capacity reduction, a secant stiffness S_j of analysed joint under load $M = 2/3M_{j,Rd}$ is equal to 60% of its stiffness calculated without that reduction.

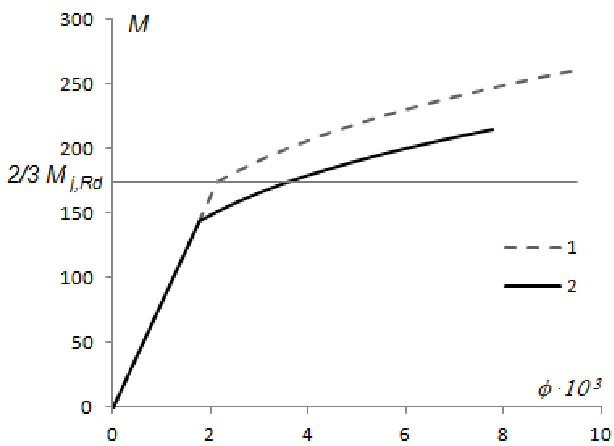


Fig. 4. Relationship M - $Ø$: 1 – for $M_{j,Rd}$, 2 – for $M_{j,Rd,red}$

4. Concluding remarks

In this work, the algorithm for calculating of a joint between the beam and unstiffened column with end-plate and three rows of bolts, with an indication on the components for each row of bolts and groups of bolt rows which should be considered, was described. In this regard, many available examples are not in compliance with the requirements contained in [3]. In such a joint, other groups of bolt rows for the column flange than for the end-plate should be considered. According to the standard [3], groups of rows starting from the row most distant from the compression zone are taken into account. Therefore, in a flange of an unstiffened column the groups of rows 1-2, 1-2-3, etc. may occur, whereas in extended end-plate it would be the row of bolt placed above upper flange of a beam and group of rows 2-3. So, in the case of an unstiffened column flange, the group of bolt rows 2-3 cannot be considered. Such a group will occur only in a joint with a stiffened column.

A calculation example of a beam-to-column joint was also presented. It was demonstrated that in joint under the load equal to load capacity $M_{j,Rd}$ derived from the formula (6.25) in [3] it may happen, that the load capacity of the column flange is exceeded in the area of the group of bolt rows 1-2. For this reason, the reduction of this capacity is necessary as follows from the point 6.2.4.2 (3) in the standard [3].

In the presented example, forces in each bolt rows were estimated on the basis of a linear model using stiffness coefficients adopted in the standard [3]. This way of calculation may be easily applied by a designer. In order to better understanding of the state of forces and deformations in a joint of a beam with an unstiffened column, it would be advisable to make a model of a joint by using the finite element method and to determine the values of these forces using elastic, elastically plastic and plastic models, e.g. as in [4]. These analyses would allow estimating the accuracy of simplified models and their suitability for design purposes.

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