

## CLASSIFICATION OF COLUMN BASES

### Effect of rotational stiffness on the load effects of frames

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## INTRODUCTION

In traditional engineering the base connections of steel frames are considered as “pinned” or “rigid”. Respecting given detailing rules no restriction in rotation or no deformation is taken into consideration respectively. Completing the general analysis these assumptions serve as basis to calculations without any further confirmation [1].

Recently in engineering practice slightly different solutions occur. Column base with two bolts (See later in Fig. 7) is similar to the traditionally pinned solution, since the column base with four bolts (Fig. 8) usually can not be considered neither pinned nor rigid connection. The moment capacity and the rotational stiffness should be calculated applying the component method by EN 1993-1-8 [2]. The standard prescribes that the effects of the behaviour of the joints should be taken into account where such effects are significant (such as in cases semi-continuous joints). The theoretically correct method is rather laborious. Therefore many engineers adopt different approximations.

Stiffness classification of joints is based on the analysis of stability behaviours of portal frames [3]. Some column bases are however so flexible or rigid, that the structural frame response is not significantly different from that obtained by modelling as perfectly pinned or rigid.

This paper analyses the real effect of the joint deformations on the load effects. The validity of the stiffness classification in general analysis is also investigated. Only rolled HEA and IPE sections are investigated. Initial imperfections and relative significant eccentricity ( $e$ )  $z_C$  is taken into consideration.

## 1 DESIGN RESISTANCE OF COLUMN BASES

Calculation of the design resistance of column bases is carried out by the component method defined in EN 1993-1-8. The base plate and the column flange are considered as T-stubs (Fig. 1).

The design compression resistance the equivalent T-stub in compression should be determined as follows:

$$F_{c,Rd} = f_{jd} l_{eff} b_{eff} \quad (1)$$

where:  $f_{jd}$  is the design bearing strength of the joint

$l_{eff}, b_{eff}$  are the effective length and width of the T-stub flange given in Fig. 3

In case of greater eccentricity – like investigated cases – the effective area under the compressed column flange is calculated.

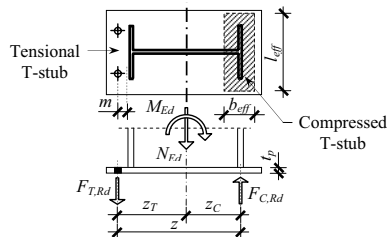


Fig. 1. Arrangement of column base

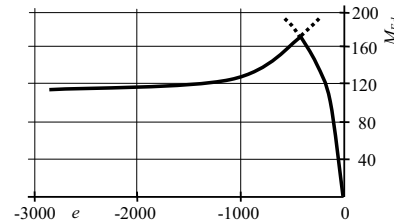


Fig. 2. Moment vs. eccentricity

Determination of the design tension resistance of the equivalent T-stub is similar to applied in cases of end-plated connections. In case of longer anchor bolts, when:

$$L_b \leq \frac{8,8m^3 A_s}{l_{eff}^3 t_p^3} \approx 5,8d \quad (2)$$

Contact of the T-stub and the concrete foundation does not happen, thus no prying force develops. Therefore prying force is not taken into consideration. The design tension resistance is:

$$F_{T,Rd} = \min \left\{ \begin{array}{l} \frac{l_{eff} t_p^2 f_y}{2m \gamma_{M1}} \\ \sum \frac{0,9 f_{ub} A_s}{\gamma_{M2}} \end{array} \right\} \quad (3)$$

In simplified form the moment resistance may be calculated as follows:

$$M_{j,Rd} = \min \left\{ \begin{array}{l} \frac{F_{T,Rd} z}{1 - |z_C|/e} \\ \frac{F_{C,Rd} z}{1 + |z_T|/e} \end{array} \right\} \quad (4)$$

In cases of base plates the moment resistance depends significantly on the value of eccentricity. In Fig. 2 the diagram has two different parts. In case of greater eccentricity the tension resistance governs, since in case of smaller eccentricity the compression resistance does. The eccentricity can be taken as great if:

$$e \geq \frac{F_{C,Rd} z_C + F_{T,Rd} z_T}{F_{C,Rd} - F_{T,Rd}} \quad (5)$$

The value of eccentricity in the 6.2.8 section of the EN 1993-1-8 is given as follows:

$$e = \frac{M_{Ed}}{N_{Ed}} = \frac{M_{Rd}}{N_{Rd}} \quad (6)$$

This expression in the given form is not clear, since  $N_{Rd}$  is not defined in the standard. Here  $N_{Rd}$  must be the value of compression force which exists simultaneously with  $M_{Rd}$ . If the calculated moment resistance is close to the design value:

$$N_{Rd} \approx N_{Ed} \quad (9)$$

$$\text{but: } N_{Ed} + F_{T,Rd} \leq F_{C,Rd} \quad (10)$$

In case of a given base arrangement the limit values of the moment and the normal force are shown in Fig. 3.

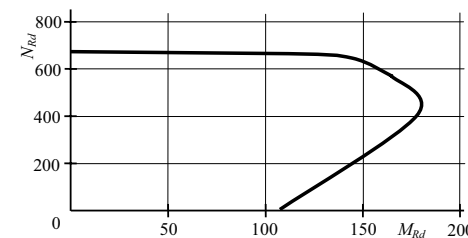


Fig. 3.  $N_{Rd}$  -  $M_{Rd}$  diagram

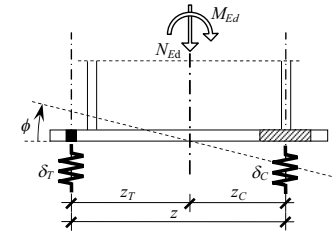


Fig. 4. Model of rotational stiffness

## 2 ROTATIONAL STIFFNESS

Calculation of rotational stiffness of column bases is established on the component method [4][5] (Fig. 4). The stiffness coefficients of components are given as follows:

Concrete in compression:  $k_{13} = \frac{E_c \sqrt{l_{eff} b_{eff}}}{1,275 E}$   $l_{eff}$  effective length of T-stub in compression  
 $b_{eff}$  effective width of T-stub in compression

Base plate in bending:  $k_{15} = \frac{0,425 l_{eff}^3}{m^3}$   $l_{eff}$  effective length of T-stub in tension

Anchor bolts in tension:  $k_{16} = 2 \frac{A_s}{L_b}$   $L_b \approx 10 - 11 d$

$$k_C = k_{13} \quad k_T = \frac{k_{15} k_{16}}{k_{15} + k_{16}} \quad (11)$$

The initial value of the rotational stiffness can be defined as follows:

$$S_{j,ini} = \frac{E z^2}{\sum \frac{1}{k_i}} \frac{e}{e + e_k} \quad (12)$$

where:  $e_k = \frac{z_C k_C - z_T k_T}{z_C + z_T}$  (13)

Following the demonstrated method, the initial value of the rotational stiffness of a given base connection can be defined. Although in cases of engineering activity the definition of the arrangement and measures of the base connection is the main task. Acceptable results can be achieved only by an iteration method.

### 3 PARAMETRIC STUDY

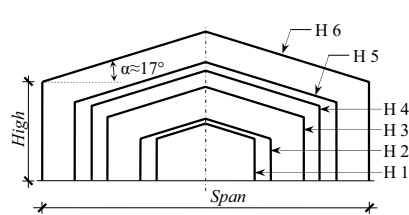


Fig. 5. Arrangement of portal frames

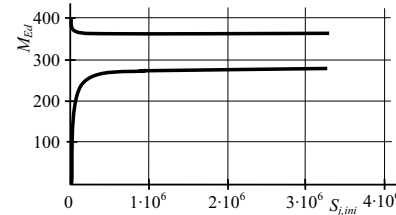


Fig. 6. Relation between moment and rigidity

To analyze the consequences of variation in the rotational stiffness series of portal frames were investigated. Only rolled section (HEA and IPE) were taken into consideration.

Table 1. Frame arrangements

	Span [mm]	High [mm]	Column	Beam
H 1	9 000	3 800	HEA 180	IPE 240
H 2	12 000	3 800	HEA 220	IPE 300
H 3	18 000	5 800	HEA 300	IPE 400
<b>H 4</b>	<b>21 000</b>	<b>6 800</b>	<b>HEA 360</b>	<b>IPE 500</b>
H 5	24 000	7 200	HEA 400	IPE 550
H 6	30 000	9 000	HEA 500	IPE 600
I 1	9 000	3 800	IPE 240	IPE 240
I 2	12 000	3 800	IPE 300	IPE 300
I 3	18 000	5 800	IPE 400	IPE 400
<b>I 4</b>	<b>21 000</b>	<b>6 800</b>	<b>IPE 500</b>	<b>IPE 500</b>
I 5	24 000	7 200	IPE 550	IPE 550
I 6	30 000	9 000	IPE 600	IPE 600

A summary of cross sectional arrangements are displayed in Fig. 5 and important parameters are demonstrated in Table 1. (In consequence of extension limit only 2 x 6 versions are demonstrated.) Repeated general analysis is done to define the relation between the rigidity of the base connection and the bending moment at the base and at the shoulder. A diagram – in case of frame signed H 4 – is illustrated in Fig. 6.

### 4 COLUMN BASES WITH FOUR BOLTS

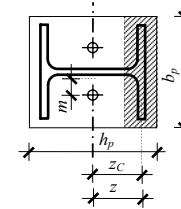


Fig. 7. Base connection with two bolts

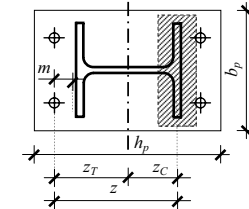


Fig. 8. Column base with four bolts

Generally used arrangement in case of four bolts is shown in Fig. 8. Calculation of the moment resistance is done by section 1. The value of the effective length of the tensioned T-stub [6]:

$$l_{eff} = \frac{b_p}{2} \quad (14)$$

The rotational stiffness is defined by the method displayed in section 2. Flow chart of the iteration process is illustrated in Fig. 9.

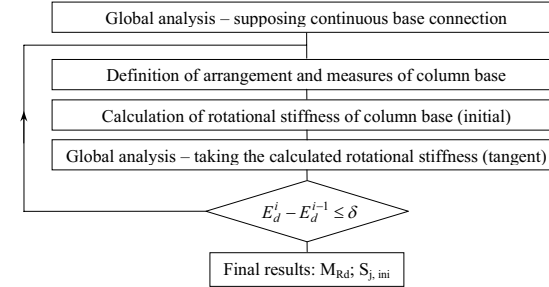


Fig. 9. Flow chart of the iteration process

The results of repeated iterations are demonstrated in Table 2. (See in next page.) In the first step continuous (rigid) connection was supposed at the base. The arrangement and measures of the base connection has been defined so that the moment resistance ( $M_{Rd}^{(0)}$ ) just exceeded the designed value.

Conclusions:

- The calculated initial stiffness in all investigated cases is smaller than the limit value – given by the standard – thus all connections can be classified as semi-continuous.
- After the iteration process the final value of the moment at base and the rotational stiffness are proven definitely smaller than in the first step. (47 – 64 %, 66 – 84 % respectively.)
- Taking into account the effect of the semi-continuous character, the moment at the shoulder becomes greater. (101 – 108 %.)

## 5 COLUMN BASES WITH STIFFENER

Generally used arrangement is similar to the connection with four bolts (see former section). A triangular stiffener is built in between the bolts (in continuation of the web of the column).

Table 2. Initial and final load effects of column bases with 4 bolts

Frame	$M_{Rd}^{00}$	$M_{Ed}^{10}$	$S_{j,ini}^0$	$S_{j,ini}^{limit}$	$M_{Rd}^{01}$	Fraction	$M_{Ed}^{11}$	Fraction	$S_{j,ini}^1$	Fraction
H 1	44,2	63,1	14829	41613	20,7	47 %	63,5	101 %	9780	66 %
H 2	84,6	110	26856	89692	40,0	47 %	114	104 %	19791	74 %
H 3	203	257	67896	198341	99	49 %	268	104 %	46071	68 %
<b>H 4</b>	<b>268</b>	<b>349</b>	<b>96039</b>	<b>306569</b>	<b>132</b>	<b>49 %</b>	<b>361</b>	<b>103 %</b>	<b>72960</b>	<b>76 %</b>
H 5	357	453	150072	394363	179	50 %	473	104 %	96021	64 %
H 6	619	739	277374	446040	398	64 %	801	108 %	232167	84 %
I 1	49,0	66,3	23988	64525	25,9	53 %	67,7	102 %	18607	78 %
I 2	93,6	113	45044	138534	46,7	50 %	119	105 %	32639	72 %
I 3	214	260	110532	251240	124	58 %	272	105 %	82657	75 %
<b>I 4</b>	<b>293</b>	<b>357</b>	<b>170096</b>	<b>446559</b>	<b>161</b>	<b>55 %</b>	<b>373</b>	<b>104 %</b>	<b>130492</b>	<b>77 %</b>
I 5	391	462	223025	587300	224	57 %	486	105 %	182377	82 %
I 6	628	740	338084	644560	403	64 %	772	104 %	269665	80 %
$M_{Rd}^{00}$ design moment resistance at the base calculated as continuous base connection $M_{Ed}^{10}$ design moment at the shoulder calculated as continuous base connection $S_{j,ini}^0$ initial rotational stiffness calculated as continuous connection $S_{j,ini}^{limit}$ limit value of initial rotational stiffness $M_{Rd}^{01}$ design moment resistance calculated as semi-continuous connection at the base $M_{Ed}^{11}$ design moment at the shoulder calculated as semi-continuous base connection $S_{j,ini}^1$ initial rotational stiffness calculated as continuous semi-connection										

Making the repeated calculations, the conclusions are made as follows:

- As an effect of the application of the stiffener, the thickness of the base plate can be reduced significantly (70 – 80 %).
- Since in consequence of the arrangement, the effective length of the T-stub becomes greater but the thickness of the base plate is reduced, the rotational stiffness of the base connection remains similar to the arrangement without stiffener.
- Thus the semi-continuous character and the behaviour of the frame does not differ significantly compared to the base connection with four bolts.

## 6 COLUMN BASES WITH TWO BOLTS

Generally used arrangement in case of two bolts is shown in Fig.7. Calculation of the moment resistance is done by section 2, but:

$$z = z_c \quad \text{and} \quad z_T = 0 \quad (15)$$

The value of the effective length of the tensioned T-stub [6]:

$$l_{eff} = 2\pi m \quad (16)$$

The results of repeated iterations are demonstrated in Table 3. (See in next page.) In the first step simple (hinged) connection was supposed at the base. In the same time an initial moment ( $M_{Ed}^{00}$ ) was taken into consideration in consequence of the imperfections.

Conclusions:

- The calculated initial stiffness in all investigated cases were greater than the limit value – given in the standard for hinged connections – thus all connections can be classified as semi-continuous.
- After the iteration process the final value of the design moment at base was smaller than the calculated moment resistance. Thus the consideration of the semi-continuous character seemed to be safe.
- Taking into account the effect of the semi-continuous character the moment at the shoulder became smaller. (97,1 – 100 %.) The differences are small.

Table 3. Initial and final load effects of column bases with 2 bolts

Frame	$M_{j,Ed}^{00}$	$M_{j,Ed}^{10}$	$S_{j,ini}$	$S_{j,ini}^{limit}$	$M_{j,Ed}^{01}$	$M_{j,Rd}^{01}$	$M_{j,Ed}^{11}$	Fraction
H 1	9,00	64,2	2689	694	8,68	9,28	63,8	99,4 %
H 2	16,00	118	4564	1495	14,6	16,1	116	98,3 %
H 3	37,50	280	11043	3306	37,8	37,8	275	98,2 %
<b>H 4</b>	<b>58,00</b>	<b>375</b>	<b>18397</b>	<b>5109</b>	<b>52,6</b>	<b>58,3</b>	<b>369</b>	<b>98,4 %</b>
H 5	78,00	494	23916	6573	71,8	79,2	485	98,2 %
H 6	163	826	50968	7435	163	163	802	97,1 %
$M_{Ed}^{00}$ initially supposed design moment at the base $M_{Ed}^{01}$ design moment at the base calculated as semi-continuous connection Other notations are the same as in Table 2.								

## 7 SUMMARY

Series of repeated calculations were completed. Portal frames with four bolted and two bolted base connections were investigated. Effects of rotations at the base connection were considered.

In case of four bolted connection, initially the base connection was supposed to be continuous. Taking it semi-continuous the design moment decreased significantly (47 – 68 %). Thus the change was on the safe side. Contrary the design moment at the shoulder increases. The value is not too great (101 – 108 %), but in most cases was not negligible. (Greater than 2,5 %.)

In case of two bolted connection, initially the base connection was supposed to be simple. Taking it semi-continuous the design moment slightly decreased. The design moment at the base calculated taking consideration the semi-continuous character of the connection was proven smaller than the calculated resistance. The design value of the moment at the shoulder became smaller (97,1 – 100 %). The changes look negligible.

Investigations proved that in the case of four bolted connections, consideration of the rotation at bases is unavoidable. Contrary, the two bolted connections may be calculated as simple ones without significant risk.

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