Optimization of Design Procedure for Column-base Connections according to EN 1993-1-8:2006

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Abstract
The stiffness and the strength of a column-base connection have significant impacts on the behavior of a steel frame. The paper develops an interaction curve between moment and axial force for the column-base connections according to EN 1993-1-8:2006 with the variation of the base plate thickness and the bolt diameter. This is the fundamental base to determine the ultimate strength of a column-base plate that allows the designers to estimate the strength of column-base connections. This research investigates the relationship between the base plate thickness, the bolt diameter, and the moment strength of the column-base plate with a specific axial force to select an optimum solution. Also, the initial rotational stiffness is determined under a specific axial load and varied moments that satisfy the ultimate limit state. The relationship between moment-rotation is subsequently performed with the variation of the base plate thickness or the bolt diameter. The design procedure is proposed based on moment-axial force interaction curves and moment-rotation curves. This allows for optimizing the column-base connection from proper selections of base plate dimensions and bolt diameters.

Keywords
rotational stiffness, moment-rotations curves, interaction curves M-N, moment resistance

1 Introduction
The strength and stiffness of a column-base connection depend on the components in the connection including bolts, base plate, and base concrete. The stiffness and behavior of the connection have noticeable impacts on the strength and behavior of the whole structural system. The connections, however, are only considered as fixed, or pin connections, although their stiffness is "semi-rigid" in practice. Recently, many researchers studied to determine the stiffness of the connections (Wald and Sokol [1], Waynand et al. [2], Abubakar and Ahmad [3], Eröz et al. [4], Daniūnas and Urbonas [5], Shafeiefar and Khonsari [6], Kanvinde et al. [7, 8], Jayarajan [9]). The determination of column-base stiffness has been investigated and has been then introduced into Eurocode 3 using the component method [10]. The reliability and effectiveness of this method in design have been illustrated based on experimental and theoretical investigations (Jayarajan [9], Jaspart and Vandegans [11], Wald et al. [12], Latour et al. [13], Krystosik [14]). The component method helps determine the stiffnesses and ultimate limit states of the column-base connections more accurately. This method was presented in the studies about behaviors of the components: base plate in bending, anchor bolts in tension and concrete in compression of Wald et al. [15, 16], Steenhuis et al. [17].

The rotational stiffness of a column-base has significant impacts on the whole structural system and moment distributions but has negligible effects on the axial force at the column base [18]. The effects of rotational stiffness of connections on the behaviors of the whole structural systems have been studied by many researchers (Daniūnas and Urbonas [5], Krytosik [14], Ermopoulos et al. [19]), as well as the economy in this analysis (Waynand et al. [2]) to demonstrate the necessity of connection stiffness in analysis.

The rotational stiffness of a column base depends on the internal forces including moment and axial force. This relationship has been illustrated using the interaction curve $M-N$ of the column-base, and the moment rotation curve $M-\phi$ via the rotational stiffness $S_{\phi_{\text{ini}}}$ These curves have been studied by many researchers (DeWolf and Sarisley [20], Penserini and Colson [21], Thambiratnam and Paramasivam [22],...
Ermopoulos et al. [19]) using theory and experimental investigations. In terms of the interaction curve between moment and axial force (M–N), DeWolf and Sarisley [20] and Thambiratnam and Paramasivan [22], have built this interaction curve (M–N) for a column-base connection on the basis of test results. An analysis model proposed by Penserini and Colson [21], provided quite accurate this relationship. A column-base connection model is proposed by Colson and Penserini [23], to show the relationship between the initial rotational stiffness and the ultimate strength of this connection. These two parameters can be determined using the geometrical and material properties of the connections.

The capacities of column-base connections can be estimated using interaction curves between moment – axial force (M–N), as proposed by Thambiratnam and Paramasivam [22], Penserini and Colson [21]. Stamatopoulos and Ermopoulos [24] then have built a diagram to determine the dimensions of the equivalent rigid plate and the relationship between the moment and the axial force. The development of interaction diagrams M–N based on test results have required times and costs but is only applied for the investigated column-bases.

Wald et al. ([12, 18]) investigated the interaction curves (M–N) for H-section columns using the finite element analysis with the variation of parameters of the connections, and then compared these to the predicted curves from Eurocode 3 using the component method. There is a good agreement between the two methods.

Examples for determinations of the interaction curve M–N and rotational stiffness have been carried out by Wald et al. [12], Simões da Silva et al. [25], but the design procedure has not been reported. This research, therefore, investigates the effects of elements of the column-base connections on their capacities using H sections. These results are the basis to propose a designing procedure for a column-base connection in a specific frame that allows the designers to easily select a column-base connection. H section is commonly used under the combined actions of the moment and axial load (DeWolf and Sarisley [20], Hawkins [26]). Razzaghi and Khoshbakht [27] and Vayas et al. [28] studied the nonlinear moment-rotation curve M–ϕ for I-section base column connections. Razzaghi and Khoshbakht [27] and Vayas et al. [28] applied the regulations in Eurocode 3.

This research investigates the capacities and the stiffness of the column-base connections using the H section column. This procedure can be easily conducted for other built-up sections but checking for local buckling is required.

Under the actions of a load combination in structural analysis, moments are initially obtained at the column-base connections with the assumption that the column ends are fixed. The obtained moments are used to determine the initial rotational stiffness sjini which will be incorporated into the structural analysis to get the other moments. This procedure is iterated until the moment and rotational stiffness become nearly unchanged, which helps reflect the behaviors of the structural systems more accurately.

The interaction relationship between moment and axial load depends on the plate thickness, the bolt diameter, the length and locations of the anchor bolts, the material properties, and the axial load. Shaheen et al. [29] and Gomez et al. [30] have investigated the effects of grout properties on shear strengths of the column-base connections.

This paper investigates the influence of the base plate thickness and the bolt diameter on the capacities of H section column-base connections (via the interaction curves M–N) under the actions of moments and axial forces according to the Eurocode 3. This helps determine the column-base connections, and the design procedure is then made faster.

The moment-rotation curve M–ϕ is also built and then incorporated into model analysis for iteration design procedure. This iteration design is applied by Jayarajan [9], or other authors ([21, 23, 31]). Their studies are only based on experiments or iteration procedures to build the moment-rotation curves M–ϕ that are cumbersome for the applications. This paper, therefore, proposes a design procedure for column-base connections in a specific steel frame according to Eurocode 3 with the support of innovative structural analysis programs. This procedure helps select the bolt diameters and base plate thicknesses using the interaction curves M–N of the column-base connections more properly. This is the optimization for the column-base connection design.

2 M–N interaction diagram

The interaction curve M–N of H-section column-base connection (Fig. 1) can be built based on the following points:

Point (–1) corresponds to pure tensile resistance of the column base with and Mj = 0 and Nc(M=0) > 0 Anchor bolts are in tension.

Point (0) is the pure bending, where Mn=0,max is the bending design resistance of the column base. Plastic bending is observed of the anchor plate

Point (1) under the actions of the moment Mj and compressive axial load Nc. The effective section illustrated in Fig. 1 has one flange in compression and a couple of the anchor bolts under tension at the opposite side.
Point (2) under the actions of the moment $M_2$ and compressive axial load $N_2$. The effective section includes a half under compression and another half under tension in the anchor bolts at the opposite side.

Point (3) under the actions of the moment $M_3$ and compressive axial load $N_3$. The effective section is the T-section including the web and one flange in compression at one side and the anchor bolts in tension at the opposite side.

Point (4) is the pure compression $N_4$ ($M=0$) and is the design compression resistance of the column base whereas anchor bolts do not work.

The capacities of the column section can be determined due to pure moment $M_{pl,Rd}$ pure axial force $N_{pl,Rd}$ or combined moment $M_{pl}$ and axial force $N_{id}$ according to Eurocode 3 [10].

The interaction curve can be constructed as follows:

+ The column section can be designed in a specific steel frame with the assumption that the column-base connections are pinned or fixed. The axial load is negligibly affected and nearly unchanged regardless of the variation of the stiffnesses of the column-base connections and is the base to determine the dimensions of the base plate. The anchor bolts are then arranged based on the selected base plate.

+ Based on the geometric and material properties, the ultimate points on the interaction curve ($M-N$) can be determined with the variation of the base plate thickness and the bolt diameter.

  - Step 1: Determine the tensile capacities of the connections from the component method:

    With prying force: $F_{rd} = \min\left(F_{rd,1}; F_{rd,2} \right)$, 
    \[ \text{(1)} \]

    Without prying force: $F_{rd} = \min\left(F_{rd,1}; F_{rd,3} \right)$, 
    \[ \text{(2)} \]

    where $F_{rd,1}$, $F_{rd,2}$ - are tensile capacities of base plate components corresponding to the yielding mechanism at the flanges (or the web) of the column with prying force; $F_{rd,3}$ is the tensile capacity of bolt connection; $F_{rd,1*}$ is the capacity of the base plate component corresponding to the collapse mechanism due to the formation of the contacts between the extremities of the T-stub plate and concrete block, as detailed in [10].

    - Step 2: Determine the width of the equivalent rigid plate ($A_{eff}$) [10, 17].

    \[ c = \frac{\sqrt{f_y}}{3f_y \gamma_{M0}}, \]  
    \[ \text{(3)} \]

    where $f_y$ is the yield stress of the base plate material; $f_y$ is the concrete design; $\gamma_{M0}$ is the safety factor.

    - Step 3: Determine the effective area of the compressive concrete area $A_{eff}$. 

    \[ N_i = A_{eff,i} f_y - F_{rd}, \quad i = -1,0,1,2,3,4, \]  
    \[ \text{(4)} \]

    where: $f_y$ is the concrete design strength: $f_y = \frac{2k_j f_{ck}}{3\gamma_c}$ with $k_j$ is the concentration factor; $f_{ck}$ is the compressive design strength of concrete for the foundation, as presented in [10, 17].

    - Step 4: Determine the moment capacities of column-base connections at ultimate points 

    \[ M_i = F_{i,c,Rd} r_i + A_{eff,i} r_i^* \]  
    \[ \text{(5)} \]

    where $F_{i,c,Rd}$ is the tensile capacity of base plate area and anchor bolts on the left side as presented in Eurocode 3 [10]; $r_i^*$ is the level arm of the centroid of the compressive area to the neutral axis; $r_i$ - is the level arm of the tensile area to the neutral axis.
• Step 5: Determine the moment capacity of the column section [10, 17].

\[
M_{N_j,rd} = M_{pl,rd} \frac{1 - N_{sd}}{N_{pl,rd}} \frac{1 - 0.5(A - 2b_f)}{A} \tag{6}
\]

As a specific axial load, the ultimate moment of the column-base connection \( M_{td} \) is the lesser value of the two intersection values of the axial load line \( N = N_{td} \) with the interaction curve \((M-N) (M_{td})\) or with the capacity line of the column-base section \((M_{Nj,rd})\).

\[
M_{rd} = \min\left( M_{td,rd} , M_{Nj,rd} \right) , \tag{7}
\]

where \( M_{td,rd} \) is the moment capacities of column-base connection under the axial load \( N = N_{rd} \) as presented in Eurocode 3 [10, 15].

The examples for the determination of these points in the relationship curve \((M-N)\) are illustrated in Table 1.

3 Moment resistance and rotational stiffness

Based on the results of investigated interaction curves in Section 2, a range of bolt diameters and base plate thicknesses are selected for the investigation in this section.

The axial load of a column is nearly unchanged assuming whether the column-based connection is fixed, pinned, or semi-rigid [19]. Therefore, the ultimate moment \( M_{rd} \) is determined under a specific axial load \( N \) with the variations of the bolt diameters \( d \) and the plate thicknesses \( t \).

The initial rotational stiffness \( S_{j,ini} \) of the connection can be determined based on the component stiffnesses including base plate component \( k_r \), concrete component \( k_c \) and bolt component \( k_b \) using case 1 to case 4 [10, 16].

\[
S_{j,ini} = \frac{e}{e + a} \frac{E_c z^2}{\mu \sum_1^1 \frac{1}{k_i}} , \tag{8}
\]

with \( e = \frac{M_{rd}}{F_{sd}} \), \( a = \frac{k_c z_c - k_c z_t}{k_c + k_t} \),

\[
\mu = (1.5)^{\gamma^2} , \quad \gamma = \frac{1 + \frac{z/2}{M_{rd} / N_{sd} + \frac{z/2}{M_{pl} / N_{pl}}} ,}
\]

where \( z_c, z_t \) are the distance from the neutral axis to the centroid of the compression and tension areas; \( z \) is the distance between the centroids of the compression and tension areas.

The capacities of the column-base connections can be determined based on the interaction curves \( M-N \) and moment rotation curves \( M-s \). This means that the

| Table 1 Moment resistances and initial rotational stiffness |
|---------------------------------|-----------------|----------------|-----------------|------|-----------------|
| Column: HE300A; S235; \( e \) = 10 mm |
| Base plate: 540 mm × 500 mm; S235; \( e = 60 \) mm; \( p = 340 \) mm; \( e = 80 \) mm; \( e = 65 \) mm (Fig. 4) |
| Bolt: Class 4.8; quantity: 4 |
| Foundation C25/30; 1000 mm × 1000 mm × 600 mm |
| \( t = 20 \) mm |
| \( d \) [mm] | \( M_1 \) [kNm] | \( N_1 \) [kN] | \( S_{j,ini} \) [kN/mm] | \( M_t \) [kNm] | \( N_t \) [kN] | \( S_{j,ini} \) [kN/mm] |
|----------------|----------------|----------------|----------------|------|----------------|----------------|
| 12 | -1795.7 | 97.1 | 18.9 | 110.8 | -680.6 | 28904.9 |
| 16 | -1795.7 | 180.9 | 35.1 | 119.6 | -638.8 | 26246.2 |
| 20 | -1795.7 | 282.2 | 54.3 | 130.3 | -588.1 | 24330.4 |
| 24 | -1795.7 | 349.1 | 66.7 | 137.3 | -554.7 | 23592.1 |
| 27 | -1795.7 | 393.7 | 75.0 | 142.0 | -532.3 | 23253.5 |
| 30 | -1795.7 | 393.7 | 75.0 | 142.0 | -532.3 | 24885.8 |
| 33 | -1795.7 | 393.7 | 75.0 | 142.0 | -532.3 | 27181.4 |
| 36 | -1795.7 | 393.7 | 75.0 | 142.0 | -532.3 | 27969.7 |
| 39 | -1795.7 | 393.7 | 75.0 | 142.0 | -532.3 | 28905.2 |
| 42 | -1795.7 | 393.7 | 75.0 | 142.0 | -532.3 | 29508.4 |
| 12 | 118.6 | -849.3 | 51281.3 | 110.8 | -1018 | 43597.5 |
| 16 | 127.4 | -807.4 | 36098.4 | 119.6 | -976.0 | 43597.5 |
| 20 | 138.1 | -756.7 | 30300.8 | 130.3 | -925.4 | 51579.5 |
| 22 | 145.1 | -723.3 | 28362.1 | 137.3 | -891.9 | 41938.6 |
| 24 | 149.8 | -701.0 | 27469.5 | 142.0 | -869.6 | 38246.4 |
| 27 | 149.8 | -701.0 | 29126.0 | 142.0 | -869.6 | 39616.6 |
| 30 | 149.8 | -701.0 | 30199.2 | 142.0 | -869.6 | 40471.0 |
| 33 | 149.8 | -701.0 | 31404.5 | 142.0 | -869.6 | 41401.0 |
| 36 | 149.8 | -701.0 | 32173.4 | 142.0 | -869.6 | 41978.8 |
| 39 | 149.8 | -701.0 | 33077.3 | 142.0 | -869.6 | 42642.9 |
| 42 | 149.8 | -701.0 | 33655.1 | 142.0 | -869.6 | 43059.1 |
| 12 | -2139.4 | 97.1 | 19.3 | 132.6 | -838.6 | 33553.9 |
| 16 | -2139.4 | 180.9 | 35.8 | 141.4 | -796.7 | 31218.1 |
| 20 | -2139.4 | 282.2 | 55.4 | 152.1 | -746.0 | 29528.8 |
| 22 | -2139.4 | 349.1 | 68.2 | 159.1 | -712.6 | 28933.4 |
| 24 | -2139.4 | 406.7 | 79.1 | 165.1 | -683.8 | 28350.6 |
| 27 | -2139.4 | 528.8 | 101.8 | 177.9 | -622.8 | 27806.5 |
| 30 | -2139.4 | 567.0 | 108.9 | 182.0 | -603.7 | 28595.3 |
| 33 | -2139.4 | 567.0 | 108.9 | 182.0 | -603.7 | 30319.7 |
| 36 | -2139.4 | 567.0 | 108.9 | 182.0 | -603.7 | 31457.2 |
| 39 | -2139.4 | 567.0 | 108.9 | 182.0 | -603.7 | 32825.9 |
| 42 | -2139.4 | 567.0 | 108.9 | 182.0 | -603.7 | 33724.1 |
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interaction curve $M-N$ can be used to quickly check the capacities of the column-base connections without iteration of the procedure in design.

The internal forces of the column are in the ultimate areas of the column-base and the capacities of column sections, which helps to determine these proper values of the bolt diameters and base plate thicknesses.

Assuming that column-base connections are fixed, pinned, or semi-rigid, the approximate axial load and moment can be obtained $N = N_{sd}$ and $M = M_{sd}$. The ultimate moment $M_{rd}$ can be determined corresponding to the axial load $N = N_{sd}$. Therefore, the bolt diameters and base plate thicknesses can be properly selected on the basis of the investigated results as presented in Table 2. The optimum of the bolt diameter and the base plate thickness allows the designers to obtain the moment resistance $M_{rd}$ that is approximately equal to the moment $M_{sd}$.

The rotational stiffness $s_{ini}$ or $s_{j}$ (see Fig. 2) subsequently can be incorporated into the analysis model to get the moment at the column-base connection $M_{sd}$. The iteration is applied with the average moment of the obtained moment $M_{sd}^{i}$ and the initial moment $M_{sd}^{0}$. The iteration process is applied until the obtained moment and the initial rotational stiffness reach these constant values.

The procedure to calculate the capacities and rotational stiffnesses of the connections is illustrated in Fig. 3.

4 Numerical analysis
This section presents two numerical examples to illustrate the proposed designing procedure of the column-base connections.

| $d$ [mm] | $M_{sd}$ [kNm] | $N_{s}$ [kNm] | $S_{ini}$ [kNm/rad] | $M_{sd}$ [kNm] | $N_{s}$ [kNm] | $S_{ini}$ [kNm/rad] |
|---------|----------------|-------------|---------------------|----------------|-------------|---------------------|
| 12      | 140.4          | -1021.      | 46880.0             | 132.6          | -1204.      | 46880.0             |
| 16      | 149.2          | -979.3      | 42899.4             | 141.4          | -1162.      | 46880.0             |
| 20      | 159.8          | -928.6      | 36604.3             | 159.1          | -1078.      | 50192.2             |
| 22      | 166.8          | -895.2      | 34586.4             | 152.1          | -1017.      | 50192.2             |
| 24      | 172.9          | -866.4      | 33189.5             | 161.5          | -1049.      | 45042.2             |
| 27      | 185.7          | -805.3      | 31536.3             | 177.9          | -987.9      | 39166.2             |
| 30      | 189.7          | -786.2      | 30279.2             | 182.0          | -968.8      | 37533.8             |
| 33      | 189.7          | -786.2      | 33781.0             | 182.0          | -968.8      | 40360.4             |
| 36      | 189.7          | -786.2      | 34891.0             | 182.0          | -968.8      | 41333.8             |
| 39      | 189.7          | -786.2      | 36213.5             | 182.0          | -968.8      | 42472.2             |
| 42      | 189.7          | -786.2      | 37073.6             | 182.0          | -968.8      | 43200.5             |

Continuation of Table 1

Table 2 Moment resistance $M_{rd}$ and rotational stiffness $s_{j}$ of column base (for column section HEA300)

| $d$ [mm] | $M_{sd}$ [kNm] | $s_{j}$ [kNm/rad] | $M_{sd}$ [kNm] | $s_{j}$ [kNm/rad] | $M_{sd}$ [kNm] | $s_{j}$ [kNm/rad] |
|---------|----------------|-------------------|----------------|-------------------|----------------|-------------------|
| 12      | 51.9           | 14580.6           | 52.9           | 14913.5           | 54.0           | 15076.5           |
| 16      | 67.0           | 15952.8           | 68.2           | 16740.5           | 69.4           | 17291.6           |
| 20      | 84.9           | 16888.9           | 86.4           | 18034.9           | 87.8           | 18902.9           |
| 22      | 96.6           | 17466.7           | 98.2           | 18813.1           | 99.8           | 19861.1           |
| 24      | 104.3          | 17856.0           | 108.3          | 19240.4           | 110.0          | 20406.3           |
| 27      | 104.3          | 19340.5           | 120.3          | 20579.5           | 131.3          | 21734.0           |
| 30      | 104.3          | 20336.8           | 120.3          | 21785.7           | 137.9          | 22961.8           |
| 33      | 104.3          | 21489.9           | 120.3          | 23198.7           | 137.9          | 24620.5           |
| 36      | 104.3          | 22445.3           | 120.3          | 24136.9           | 137.9          | 25735.3           |
| 39      | 104.3          | 23153.5           | 120.3          | 25275.3           | 137.9          | 27099.1           |
| 42      | 104.3          | 23746.0           | 120.3          | 26206.0           | 137.9          | 28007.5           |

Table 2 continued
4.1 Example 1

Design the column-base connection in a steel structural frame as follows:

Sections HEA300 and IPE400 regulated in [10] are used as column and rafter sections.

Actions are applied to the investigated frame according to the EN 1991-1-1 [32], including: the dead load of self-weight of the rafter \( g \) = 3.315 kN/m, the dead load of the roof (purlins & roof sheet) \( g_f \) = 6.6225 kN/m; Live load \( q \) = 3 kN/m; Wind load for the roof: \( w_d \) = 0.8 kN/m, \( w_s \) = –3.9 kN/m and for the wall: \( h_w \) = 2.4575 kN/m, \( h_s \) = 3.9 kN/m; The point load on the top of the column \( H_w \) = 1.56 kN/m; The snow load: \( s \) = 5 kN/m.

Load combinations are listed as follows:

- **LC1**: \( 1.35 \times (g + g_f) \)
- **LC2**: \( 1.35 \times (g + g_f) + 1.5 \times s \)
- **LC3**: \( 1.35 \times (g + g_f) + 1.5 \times s + 1.5 \times 0.7 \times q \)
- **LC4**: \( 1.35 \times (g + g_f) + 1.5 \times s + 1.5 \times 0.6 \times (wd + h_w + H_w) + 1.5 \times 0.7 \times q \)
- **LC5**: \( 1.35 \times (g + g_f) + 1.5 \times 0.5 \times s + 1.5 \times (wd + h_w + H_w) + 1.5 \times 0.7 \times q \)
- **LC6**: \( 1.35 \times (g + g_f) + 1.5 \times s - 1.5 \times 0.6 \times (wd + h_w + H_w) + 1.5 \times 0.7 \times q \)
- **LC7**: \( 1.35 \times (g + g_f) + 1.5 \times 0.5 \times s - 1.5 \times (wd + h_w + H_w) + 1.5 \times 0.7 \times q \)
- **LC8**: \( 1.0 \times (g + g_f) + 1.5 \times (w_s + h_w) \)

### Table of moment resistance of base column \( \text{Mrd} \)

| \( d \) [mm] | \( M_{rd} \) [kNm] | \( s_j \) [kNm/rad] | \( M_{rd} \) [kNm] | \( s_j \) [kNm/rad] |
|---------------|-------------------|------------------|-------------------|------------------|
| 12            | 61.1              | 14274.3          | 62.1              | 14064.7          |
| 16            | 77.8              | 17965.5          | 79.0              | 17862.2          |
| 20            | 97.8              | 20869.9          | 99.2              | 20868.0          |
| 22            | 110.8             | 22572.6          | 112.4             | 22628.4          |
| 24            | 122.0             | 23635.6          | 123.7             | 23734.6          |
| 27            | 145.3             | 26078.6          | 147.3             | 26266.8          |
| 30            | 166.1             | 27901.9          | 168.3             | 28164.1          |
| 33            | 166.1             | 30724.9          | 168.3             | 31084.0          |
| 36            | 166.1             | 32718.3          | 168.3             | 33154.9          |
| 39            | 166.1             | 35225.3          | 168.3             | 35764.3          |
| 42            | 166.1             | 36967.7          | 168.3             | 37585.0          |
The column-base connections are initially assumed as fixed ends for model analysis, and the internal force diagrams are subsequently obtained as shown in Fig. 4.

The obtained internal forces \( (M_{\text{max}}, N_{\text{max}}) \) are shown to be less than the design capacities of the column and girder \( (M_{\text{pl,Rd}}, M_{\text{pl,Rd}}) \).

The base plate dimensions are illustrated in Fig. 5 including \( a \times b = 540 \text{ mm} \times 500 \text{ mm}, e_a = 60 \text{ mm}, e_b = 80 \text{ mm}, e_c = 65 \text{ mm} \). The steel grade S235 is used for this investigation as regulated in EN 1993-1-1:2005 [33]. The material properties are \( f_y = 235 \text{ MPa}, E_s = 210000 \text{ MPa} \). Bolt class 4.8 is used with the ultimate strength \( f_{ub} = 400 \text{ MPa} \).

Material properties of concrete grade C25/30 include \( f_{ck} = 25 \text{ MPa}, E_c = 31476 \text{ MPa} \). The footing dimensions \( (a_1 \times b_1 \times h) \) are equal to \( 1000 \text{ mm} \times 1000 \text{ mm} \times 600 \text{ mm} \).

The internal forces at the base are \( M_{\text{max}} = 136.6 \text{ kNm} \) and \( N_{\text{max}} = 214.6 \text{ kNm} \) corresponding to the load combinations of 3, 4 and 6. The internal forces of the column and rafter with the assumption of rigid connections are checked as demonstrated in Table 3.

The capacities of column sections HEA300 include ultimate pure axial force \( N_{\text{pl,Rd}} = 2664.4 \text{ kN} \) and pure ultimate moment \( M_{\text{pl,Rd}} = 325.07 \text{ kN} \).

The investigated bolt diameters vary from 12–42 mm; and the base plate thicknesses vary from 20–40 mm.

The results are listed in Table 1 when \( t = 20 \text{ mm} \) and \( t = 24 \text{ mm} \). The whole results are then used to develop the interaction curves in Fig. 6 and Fig. 7.

![Fig. 4 Bending moment and axial force distribution of frame F1 (LC3)](image)

The bolt diameter \( d = 27 \text{ mm} \) and the plate thickness \( t > 24 \text{ mm} \) can be selected for the base connections corresponding to the moment \( M_{\text{sd}} = 136.6 \text{ kNm} \) and axial load \( N_{\text{sd}} = 214.6 \text{ kNm} \) that are lower than the ultimate capacities based on the interaction curves in Fig. 6 and Fig. 7.

The axial loads for investigation should be less than the design compression load of the column section \( N_{\text{pl,Rd}} = 2644.4 \text{ kN} \) and the base plate \( N_{\text{4(0)}} \). The investigated axial loads are from 200 kN to 1200 kN.

The ultimate moment \( M_{\text{pl,Rd}} \) and rotational stiffness \( s_j \) under the axial load of 200 kN are determined and presented in Table 3.

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**Table 3 Checking sections of F1 with rigid joint**

| Validation | Column HEA300 | Rafter IPE400 |
|------------|---------------|---------------|
| \( M_{\text{max}} \) [kNm] | -136.6 + 259.93 | -260.4 + 303.3 |
| \( N_{\text{max}} \) [kN] | 214.6 | 16.1 |
| \( M_{\text{pl,Rd}} \) [kNm] | 325.07 | 307.18 |
| \( N_{\text{pl,Rd}} \) [kN] | 2664.4 | 1984.89 |
| \( M_{\text{sd,Rd}} \) [kNm] | 295.99 | 271.76 |
Fig. 6 M-N Interaction diagrams for different bolt diameter (Column HEA300, $f_y = 235$ MPa; Bolt $d = 4.8$, $n = 4$, $f_{ub} = 400$ MPa; Concrete: C25/30; Base plate, $e_a = 60$ mm, $e_b = 80$ mm, $e_c = 65$ mm)
Fig. 7 M-N Interaction diagram for different base plate thickness (Column HEA300, $f_y = 235$ Mpa; Bolt 4.8, $n = 4$, $f_{ub} = 400$ MPa; Concrete: C25/30; Base plate, $e_a = 60$ mm, $c_b = 80$ mm, $c_c = 65$ mm)
The moment capacity $M_{rd}$ of the base connection is determined, as presented in Table 2. The moment $M_{rd} = 136.6 \text{kNm}$ and the axial load of $214.6 \text{kN}$ (Table 3) are used to select the bolt diameter ($d$) and the plate thickness ($t$). The first selection $d = 27$ and $t = 32 \text{mm}$ ($M_{rd} = 139.4 \text{kNm}$); the second selection $d = 30$ and $t = 26 \text{mm}$ ($M_{rd} = 152.5 \text{kNm}$).

With a specific axial load, the initial rotational stiffness can be calculated corresponding to the variations of moment values from 10 kNm to 310 kNm. The relationships $M-s$ can be determined, as presented in Table 2. The moment capacity of the base connection corresponding to the axial load $N = 214.6 \text{kN}$ (Table 3) is used for this investigation as regulated in EN 1993-1-1:2005 [10]. The material properties are $f_{yd} = 235 \text{MPa}$; $f_{cu} = 400 \text{MPa}$. Bolt class 4.8 is used with the ultimate strength $f_{ub} = 400 \text{MPa}$. Material properties of concrete grade C25/30 include $f_{ck} = 25 \text{MPa}$; $E_c = 31476 \text{MPa}$. The footing dimensions ($a \times b \times h$) are equal to 1000 m $\times$ 1000 m $\times$ 600 mm.

The internal forces at the column base are 71.09 kNm and 214.6 kN (Table 3). The iteration calculations are applied to get the final results as follows:

In the first selection $d = 27$ and $t = 32 \text{mm}$, the internal forces at the column base are $M_{sd} = 108 \text{kNm}$ and axial load $N_4 = 214.6 \text{kN}$ with the rotational stiffness $s_j$ of 41046 kNm/ rad. The initial rotational stiffnesses, therefore, are determined corresponding to these above selections using Table 4 with the axial load of 214.6 kN.

In the second selection $d = 30$ and $t = 26 \text{mm}$, the internal forces at the column base are $M_{sd} = 113.8 \text{kNm}$ and axial load $N_4 = 214.6 \text{kN}$ with the rotational stiffness $s_j$ of 54069 kNm/ rad.

After applying the component method in Eurocode 3, it is found that the moment at the column base is reduced and axial load $N_4 = 214.6 \text{kN}$ (Table 3).

## 4.2 Example 2

The design of the column-base connections for frame F2 has similar dimensions and load actions to those of frame F1, but sections HEA400 and IPE450 are replaced for the columns and rafter (Table 6).

The design procedure is similar to Example 1.

The column base plate dimensions are also similar to this in Example 1, as follows: $a \times b = 540 \text{mm} \times 500 \text{mm}$, $e_a = 60 \text{mm}$, $e_b = 80 \text{mm}$, $e_c = 65 \text{mm}$. The steel grade S235 is used for this investigation as regulated in EN 1993-1-1:2005 [10]. The material properties are $f_{yd} = 235 \text{MPa}$; $f_{cu} = 21000 \text{MPa}$. Bolt class 4.8 is used with the ultimate strength $f_{ub} = 400 \text{MPa}$. Material properties of concrete grade C25/30 include $f_{ck} = 25 \text{MPa}$; $E_c = 31476 \text{MPa}$. The footing dimensions ($a_1 \times b_1 \times h$) are equal to 1000 m $\times$ 1000 m $\times$ 600 mm.

| $d$ [mm] | 12 | 16 | 20 | 22 | 24 |
|----------|----|----|----|----|----|
| 12 | fail | fail | fail | fail | fail |
| 16 | fail | fail | fail | fail | fail |
| 20 | fail | fail | fail | fail | fail |
| 22 | fail | fail | fail | fail | fail |
| 24 | fail | fail | fail | fail | fail |
| 27 | 19184.3 | 19939.1 | 20520.3 | 20965.4 | 21303.9 |
| 29 | 21194.8 | 22134.7 | 22873.4 | 23452.3 | 23904.6 |
| 30 | 33602.8 | 35200.0 | 36471.0 | 37480.6 | 38281.5 |
| 32 | 71827.2 | 75640.4 | 78771.6 | 81310.7 | 83370.2 |

Note: “Fail” stands for the moment is larger than the moment resistance of the connection and/or the column section.
Based on the interaction curves $M-N$ (similar to Example 1) for the section HEA400, bolt diameter of 16 mm and the thickness of larger than 20 mm are selected for the column-base connections. The $M_{rd} = 71.09 \text{kN}$ and $N_{rd} = 221.7 \text{kN}$ are in the interaction curve ($M-N$).

The moment capacity $M_{rd}$ of the base column connection is then determined using the database as presented in Table 7.

Therefore, the moment $M_{rd} = 71.09 \text{kN}$ and the axial load of 221.7 kN are used to select the bolt diameter ($d$) and the plate thickness ($t$) with the following values: the first selection $d = 20$ and $t = 20$ mm.

The iteration calculations are similar to those in Example 1, and the results are subsequently listed in the Table 8.

With $d = 27$ and $t = 32$ mm, the internal forces at the column base are $M_{rd} = 108kNm$ and axial load $N_{rd} = 221.7 \text{kN}$ with the rotational stiffness $s_j$ of 46226 kN/rad.

### Table 5 Iterative results and comparisons between two selected solutions of column-base connections in the Frame F1

| Validation | $d = 27$ mm, $t = 32$ mm | $d = 30$ mm, $t = 26$ mm |
|------------|--------------------------|--------------------------|
| $M$ [kNm]  | $s$ [kNm/rad]            | $M$ [kNm]                | $s$ [kNm/rad]            |
| From Table 4 | 90                      | 83370                    | 90                      | 80543                        |
| From Table 4 | 110                     | 38189                    | 110                     | 61687                        |
| From Table 4 | 130                     | 25648                    | 130                     | 32781                        |
| Modeling $M_{sd}$ (Table 1) | 136.6                   | 25425                    | 136.6                   | 27372                        |
| 1st iterative result ($M_{sd}$) | 95.7                     | 97.8                     | 95.7                    | 97.8                          |
| $M_{sd} = 0.5(M_{sd} + M_{rd})$ | 116.1                   | 30985                    | 116.2                   | 48300                        |
| 2nd iterative result ($M_{sd}$) | 101.1                   | 111.5                    | 101.1                   | 111.5                         |
| $M_{sd} = 0.5(M_{sd} + M_{rd})$ | 106.8                   | 40159                    | 114.8                   | 52279                        |
| 3rd iterative result ($M_{sd}$) | 107.5                   | 113.1                    | 107.5                   | 113.1                         |
| $M_{sd} = 0.5(M_{sd} + M_{rd})$ | 108                     | 41046                    | 114                     | 53704                        |
| 4th iterative result ($M_{sd}$) | 107.99                  | 113.6                    | 107.99                  | 113.6                         |
| $M_{sd} = 0.5(M_{sd} + M_{rd})$ | 113.8                   | 54069                    | 113.8                   | 54069                        |
| 5th iterative result ($M_{sd}$) | 113.73                  |                         | 113.73                  |                               |

### Table 6 Checking sections of F2 with rigid joint

| Validation | Column HEA400 | Rafter IPE450 |
|------------|---------------|---------------|
| $M_{mm}$ [kNm] | -71.09 + 300.26 | 221.71        |
| $N_{mm}$ [kN]  | -277.15 + 295.4  | 43.43         |
| $M_{fmm}$ [kNm] | 602.02        | 399.92        |
| $N_{fmm}$ [kN]  | 3735.98       | 2322.29       |
| $M_{ffmm}$ [kNm] | 543.14        | 352.43        |

### Table 7 Moment resistance $M_{rd}$ corresponding to normal force

| $t$ [mm] | 20  | 22  | 24  | 26  | 28  | 30  |
|----------|-----|-----|-----|-----|-----|-----|
| $d$ [mm] | $M_{rd}$ [kNm] | $M_{rd}$ [kNm] | $M_{rd}$ [kNm] | $M_{rd}$ [kNm] | $M_{rd}$ [kNm] | $M_{rd}$ [kNm] |
| 12       | 69.7 | 70.9 | 72.0 | 73.1 | 74.3 | 75.4 |
| 16       | 87.6 | 88.9 | 90.3 | 91.6 | 92.9 | 94.2 |
| 20       | 108.9| 110.5| 112.1| 113.6| 115.2| 116.8|
| 22       | 122.8| 124.5| 126.3| 128.0| 129.7| 131.4|
| 24       | 134.6| 136.5| 138.4| 140.3| 142.1| 144.0|
| 27       | 159.4| 161.6| 163.8| 166.0| 168.1| 170.3|
| 30       | 171.7| 185.3| 187.8| 190.2| 192.7| 195.2|
| 33       | 171.7| 198.8| 212.4| 215.2| 218.0| 220.7|
| 36       | 171.7| 198.8| 212.4| 215.2| 218.0| 220.7|
| 39       | 171.7| 198.8| 212.4| 215.2| 218.0| 220.7|
| 42       | 171.7| 198.8| 212.4| 215.2| 218.0| 220.7|

### 5 Discussions

As the bolt diameter increases to 30 mm, the tensile capacity of the connection is unchanged due to the occurrence of the yield stress. This means that the increase of bolt diameter larger than 30 mm becomes ineffective, as the interaction curves ($M-N$) showed in Fig. 7.

The development of interaction curves $M-N$ for a variety of base plate thicknesses and bolt diameters is the basis to determine the capacities of the column-base connections and to select the optimum solutions of these connections.

The proposed designing procedure has been illustrated to be simple and effective thanks to the optimum selections of bolt diameters and base plate thicknesses based on capacities of column-base connections.

The capacities of column-base connections are determined using the rotational stiffness of the connections, which helps to utilize the maximum capacities of the connections and create more economical benefits.
Iteration calculations provide more accurate results of the rotational stiffness, which reflects the actual working of the structural systems in reality.

6 Conclusions

With specific column-base connection dimensions and the location of the anchor bolts, the interaction curves $M-N$ can be constructed to estimate the capacities of these connections with the variations of the bolt diameter and the plate thickness.

Under the actions of applied loads, the bolt diameters and the plate thicknesses can be fundamentally selected on the basis of the interaction curves. These interaction curves can also be used to optimize the bolt diameter and plate thickness to utilize the ultimate capacities of the column-base connections.

The initial rotational stiffnesses are determined and incorporated into the analysis models to reflect the actual behaviors of the connections.

Iteration calculations for semi-rigid connections can be carried out by using interaction curves as discussed. Therefore, the designing procedure proposed in this paper allows this procedure to become faster, simpler and more economical.

| Table 8 Iterative results of the solution of Frame F2 |
|-----------------------------------------------|
| Validation | $d = 20$ mm, $t = 20$ mm | $M$ [kN] | $s$ [kN/rad] | $N$ [kN] |
|---|---|---|---|---|
| Modeling $M_{01}^\mathrm{it}$ (Table 6) | 71.09 | 127365 |
| 1st iterative result ($M_{1}^\mathrm{it}$) | 127.3 |
| $M_{01}^\mathrm{it} = 0.5(M_{01}^\mathrm{it} + M_{1}^\mathrm{it})$ | 99.2 | 36209 |
| 2nd iterative result ($M_{2}^\mathrm{it}$) | 85.1 |
| $M_{01}^\mathrm{it} = 0.5(M_{01}^\mathrm{it} + M_{2}^\mathrm{it})$ | 92.1 | 53278 |
| 3rd iterative result ($M_{3}^\mathrm{it}$) | 99.9 | 221.7 |
| $M_{01}^\mathrm{it} = 0.5(M_{01}^\mathrm{it} + M_{3}^\mathrm{it})$ | 96 | 42818 |
| 4th iterative result ($M_{4}^\mathrm{it}$) | 91.6 |
| $M_{01}^\mathrm{it} = 0.5(M_{01}^\mathrm{it} + M_{4}^\mathrm{it})$ | 94 | 47797 |
| 5th iterative result ($M_{5}^\mathrm{it}$) | 95.8 |
| $M_{01}^\mathrm{it} = 0.5(M_{01}^\mathrm{it} + M_{5}^\mathrm{it})$ | 94.6 | 46226 |
| 6th iterative result ($M_{6}^\mathrm{it}$) | 94.56 |

References

[1] Wald, F. "Column Base Modelling", In: Ivanyi, M., Baniotopoulos, C. C. (eds) Semi-Rigid Joints in Structural Steelwork, International Centre for Mechanical Sciences, Springer, 2000, pp. 227–288. ISSN 0254-1971. https://doi.org/10.1007/978-3-7091-2478-9_4

[2] Waynand, K., Jaspart, J.-P., Steenhuis, M. "Economy studies of steel building frames with semi-rigid joints", Journal of Constructional Steel Research, 46(1–3), pp. 85–94, 1998. https://doi.org/10.1016/S0143-974X(98)00045-5

[3] Abubakar, I., Ahmad, I. U. "Reliability Analysis of Steel Column Base Plates", Journal of Applied Sciences Research, 3(3), pp. 189–194, 2007.

[4] Eröz, M., White, D. W., Des Roches, R. "Direct analysis and design of steel frames accounting for partially restrained column base conditions", Journal of Structural Engineering, 134(9), pp. 1508–1517, 2008. https://doi.org/10.1061/(ASCE)0733-9445(2008)134:9(1508)

[5] Daniūnas, A., Urbonas, K. "Influence of the semi-rigid bolted steel joints on the frame behaviour", Journal of Civil Engineering and Management, 16(2), pp. 237–241, 2010. https://doi.org/10.3846/jcem.2010.027

[6] Shafieifar, M. R., Khonsari, S. V. "Studying the behaviour of base plates with high degree of rigidity", In: 15th World Conference on Earthquake Engineering, Lisbon, Portugal, 2012, pp. 19801–19808.

[7] Kanvinde, A. M., Grilli, D., Zareian, F. "Rotational Stiffness of Exposed Column Base Connections - Experiments and Analytical Models", Journal of Structural Engineering, 137(5), pp. 549–560, 2012. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000495

[8] Kanvinde, A. M., Jordan, S. J., Cooke, R. J. "Exposed column base plate connections in moment frames-Simulations and behavioural insights", Journal of Constructional Steel Research, 84, pp. 82–93, 2013. https://doi.org/10.1016/j.jcsr.2013.02.015

[9] Jayajaran, P. "Characterization of steel column bases- Eurocode 3 approach", Journal of Structural Engineering (Madras), 42(5), pp. 521–530, 2016.

[10] CEN "EN 1993-1-8:2006 Design of steel structures - Part 1-8: Design of joints", European Committee for Standardization, Brussels, Belgium, 2006.

[11] Jaspart, J. P., Vandegans, D. "Application of the component method to column bases", Journal of Constructional Steel Research, 48, pp. 89–106, 1998. https://doi.org/10.1016/S0143-974X(98)00196-1

[12] Wald, F., Šabatka, L., Bajer, M., Jehlička, P., Kabeláč, J., Kožich, M., Kufíková, M., Vild, M. "Component-based finite element design of steel connections", Czech Technical University, 2020. ISBN 978–80–01–06702–4
[13] Latour, M., Piluso, V., Rizzano, G. "Rotational behaviour of column base plate connections: Experimental analysis and modelling", Engineering Structures, 68, pp. 14–23, 2014. https://doi.org/10.1016/j.engstruct.2014.02.037

[14] Krytosik, P. "Influence of supporting joints flexibility on statics and stability of steel frames", International Journal of Steel Structures, 18, pp. 433–442, 2018. https://doi.org/10.1007/s13296-018-0008-z

[15] Wald, F., Sokol, Z., Steenhuis, M., Jaspart, J. P. "Component method for steel column bases", Heron, 53(1/2), pp. 3–20, 2008.

[16] Wald, F., Sokol, Z., Jaspart, J. P. "Base plate in bending and anchor bolts in tension", Heron, 53(1/2), pp. 21–50, 2008.

[17] Steenhuis, M., Wald, F., Stark, J. "Concrete in compression and base plate in bending", Heron, 53(1/2), pp. 51–67, 2008.

[18] Wald, F., Hofmann, J., Kuhlmann, U., Bečková, Š., Gentili, F., ..., van Kann, J. "Design of steel-to concrete joints. Design manual", European Convention for Constructional Steelwork, Brussels, 2014.

[19] Ermopoulos, J., Stammatopoulos, G., Owens, G. N. "Influence of support conditions on the behaviour of steel frames", In: Nordic Steel Construction Conference, Malmo, Sweden, 1995, pp. 819–826.

[20] DeWolf, J. T., Sarisley, E. F. "Column base plates with axial loads and moments", Journal of the Structural Division, 106(11), pp. 2167–2184, 1980. https://doi.org/10.1061/jsdeag.0005569

[21] Penserini, P., Colson, A. "Ultimate limit strength of column-base connections", Journal of Constructional Steel Research, 14, pp. 301–320, 1988.

[22] Thambiratnam, D. P., Paramasivam, P. "Base plates under axial load and moment", Journal of Structural Engineering, 112(5), pp. 1166–1181, 1986. https://doi.org/10.1061/(ASCE)0733-9445(1986)112:5(1166)

[23] Colson, A., Penserini, P. "Three-dimensional physical and mathematical modelling of the column base connections", In: Structural Stability Research Council Meeting Proceedings, Minneapolis, USA, 1988, pp. 301–308.

[24] Stammatopoulos, G., Ermopoulos, J. "Interaction curves for column base-plate connections", Journal of Constructional Steel Research, 44(1–2), pp. 69–89, 1997. https://doi.org/10.1016/S0143-974X(97)00038-2

[25] Simões da Silva, L., Weynand, K., Nikolova, B., Dimova, S., Veljkovic, M., ..., Simões, R. (eds.) "Eurocodes: Background and Applications Design of Steel Building", Publications Office of the European Union, Luxembourg, 2015. https://doi.org/10.2788/605700

[26] Hawkins, N. M. "The bearing strength of concrete loaded through flexible plates", Magazine of Concrete Research, 20(63), pp. 95–102, 1968. https://doi.org/10.1680/macr.1968.20.63.95

[27] Razaghi, J., Khoshbakht, A. "Numerical Evaluation of Column Base Rigidity", In: Topping, B. H. V. (ed.) Proceedings of the Eleventh International Conference on Computational Structures Technology, Civil-Comp Press, Stirlingshire, UK, Paper 5, 2012. https://doi.org/10.4203/ccp.99.5

[28] Vayas, I., Ermopoulos, J., Ioannidis, G. "Design of steel structures to Eurocodes", Springer, Cham, Switzerland, 2019.

[29] Shaheen, M. A., Tsavdaridis, K. D., Salem, E. "Effect of grout properties on shear strength of column base connections: FEA and analytical approach", Engineering Structures, 152, pp. 307–319, 2017. https://doi.org/10.1016/j.engstruct.2017.08.065

[30] Gomez, I. R. "Behavior and Design of Column Base Connections", University of California, 2010. ISBN 1124315713

[31] Ermopoulos, J. C., Stammatopoulos, G. N. "Mathematical modelling of column base plate connections", Journal of Constructional Steel Research, 36(2), pp. 79–106, 1996. https://doi.org/10.1016/0143-974X(95)00011-J

[32] CEN "EN 1991-1-1:2002 Actions on structures - Part 1-1: General actions Densities, self-weight, imposed loads for buildings", European Committee for Standardization, Brussels, Belgium, 2002.

[33] CEN "EN 1993-1-1:2005 Design of steel structures - Part 1-1: General rules and rules for buildings", European Committee for Standardization, Brussels, Belgium, 2005.