

PAPER • OPEN ACCESS

Properties of Beam-to-Column Joint in Steel-Concrete Composite Frame Unpropped or Partly Propped During Slab Execution

To cite this article: Grzegorz Gremza 2019 *IOP Conf. Ser.: Mater. Sci. Eng.* **471** 052027

View the [article online](#) for updates and enhancements.

Properties of Beam-to-Column Joint in Steel-Concrete Composite Frame Unpropped or Partly Propped During Slab Execution

Grzegorz Gremza ¹

¹ Faculty of Civil Engineering, Silesian University of Technology, ul. Akademicka 5, 44-100 Gliwice, Poland

grzegorz.gremza@polsl.pl

Abstract. The issues of load-bearing capacity and stiffness of end-plate beam-to-column composite joints under monotonic or cyclic bending moment loading have been discussed in numerous publications. Publicly available descriptions of various computational models and reports from experimental research concern joints with structure not changing in their entire load range. However, it is difficult to find the results of experimental tests or theoretical analyses of the load-bearing capacity and rotational capacity of a beam-to-column joint in composite frame with a reinforced concrete slab or steel-concrete composite slab executed on not propped steel member. Meanwhile, there are structural solutions in which floor slabs are executed using permanent formwork and temporary formwork girders arranged between the girders of a steel frame before concreting, without vertical slab props or with a limited number of them. In such a structure with a variable static scheme, the results of the internal forces analysis in the ultimate limit state may be different from the results for a structure made with full support. These differences are related among other to different characteristics of seemingly the same joints. The aim of the work was therefore an attempt to assess the impact of technology of the structure execution on the load capacity and rotational capacity of the beam-column connection in the steel-concrete composite frame in the ultimate limit state. The examples of steel-concrete composite joints of frame made with full temporary support as well as without vertical slab props were analysed. A modified two-stage non-linear component method was used in the calculations. Load capacities of the basic components as well as their initial stiffness were adopted according to Eurocode 3 and Eurocode 4, while their ultimate deformation values in ULS were adopted according to the author's own analysis or other available results. The rotational capacity of composite joints made at several levels of the steel joint's effort during slab concreting was considered. It was assumed that during execution the steel beams are always protected against torsional buckling. Conclusions regarding the estimation of the load capacity and rigidity of the joint as well as its rotational capacity depending on the assembly method are presented.

1. Introduction

There have already been countless articles published as well as many textbooks and manuals concerning examinations and calculations of load-bearing capacity of beam-to-column joints in steel and steel-concrete composite frames. In these publications, the highest attention is paid to calculations with the use of component method, introduced to designing steel and steel-concrete composite constructions by assumptions [1] and [2]. Some authors attempt to analytically establish the beam's rotational capacity and limiting deformability of its components ([3-7]); however, the knowledge within this scope still has



to be complemented. The least numerous publications concern analysis of joints in un-propped construction [8].

It may be assumed that a composite joint, which previously worked under a significant load $M_{j,a}$, as a steel joint, shall have a different load-bearing capacity $M_{j,Rd}$ and rotational capacity Φ_{Cd} at ultimate load-bearing limit state (figure 1b). It results from a partial use of load-bearing capacity $F_{Rd,i}$ and deformation $\Delta_{u,i}$ of i -th components of the steel joint before connecting the joint to a concrete slab (figure 1a).

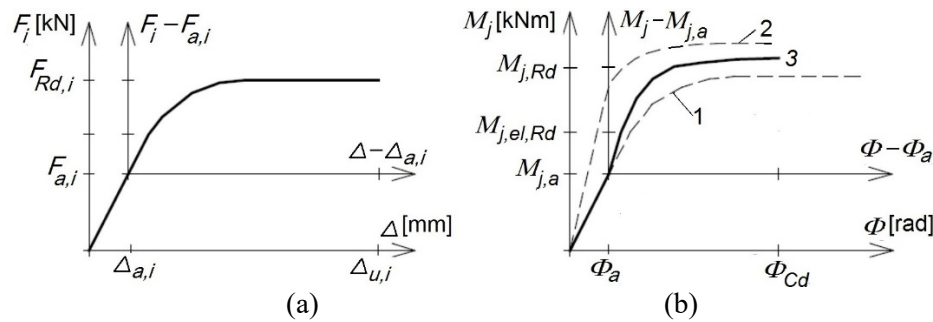


Figure 1. Relationships: a) force – deformation of i -th basic component, b) moment – rotation:
1 – steel joint, 2 – composite joint with slab propped during execution, 3 – real joint

2. Analysed joint and its simplified mechanical model

In the analysis described herein, a two-sided joint of the HE220A column and I300PE beams connected to a concrete slab was examined. Thickness of the end-plate is 16 mm. Bending moment shall be transmitted by 1 row of HV bolts class 10.9 M20 (variant A) or M16 (variant B) in an assembly stage. Assembly load shall be transmitted to steel girders via reinforced-concrete prefabricated slabs or folded sheets. There was assumed a longitudinal reinforcement in the form of 8 pieces of $\phi 12$ bars. The joint is equally loaded on both sides ($\beta = 0$ according to [1]). Calculated load-bearing capacity of a steel joint (without a concrete slab) established according to EC3 at $\gamma_{M0} = 1.0$ and $\gamma_{M2} = 1.25$ shall be $M_{j,a,Rd} = 41.60$ kNm (variant A) and 37.43 kNm (variant B), and of a composite joint in a propped construction shall be $M_{j,Rd} = 128.3$ kNm, with a partial coefficient for reinforcement $\gamma_s = 1.15$. Influence of the beam-to-slab connection's deformability was neglected.

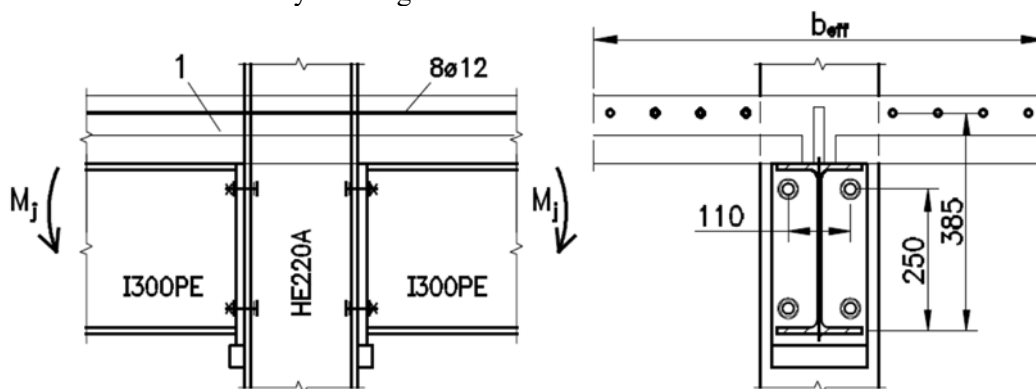


Figure 2. Analysed joint; 1 – composite concrete or steel-concrete slab

Simplified mechanical models of a steel joint as well as composite steel-concrete joint were analysed (figure 3). Primary components of both joints were modelled as springs with non-linear force-displacement characteristic. Stiffnesses k_{t1} , k_{t2} , k_{c1} , k_{c2} , and k_{ac} are output stiffnesses resulting from stiffness of particular parts of joints and they shall change together with the value of forces $F_t = F_{t1} + F_{t2}$, F_{ac} and $F_c = F_{c1} + F_{c2}$, as well as corresponding displacements.

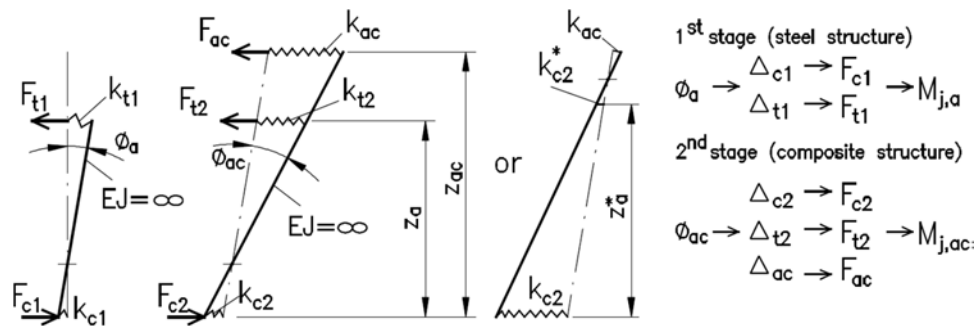


Figure 3. Model including phases of the joint assembly in steel-concrete composite frame (one side)

While modelling the composite joint, the loading of particular components of the steel joint was taken into account while executing a concrete slab. In the analysis, there was assumed a gradual increase of joint rotations Φ_a in steel construction and Φ_{ac} in composite construction. For each increase of joint rotations there was assumed an adequate value of extension or contraction of the spring Δ_{t1} , Δ_{t2} , Δ_{c1} , Δ_{c2} or Δ_{ac} , for which corresponding forces F_{t1} , F_{t2} , F_{c1} , F_{c2} and F_{ac} were specified, while preserving the condition of equilibrium of those forces. The sums of forces in both stages of joint's operation was compared to the load-bearing capacity of the weakest basic components that were taken into account at spring modelling. The total moment M_j as well as the total rotation Φ_j at loading M_j were specified with the use of the following formulas (see figure 3):

$$M_j = M_{j,a} + M_{j,ac} \text{ and } \Phi_j = \Phi_a + \Phi_{ac}. \quad (1a), (1b)$$

Bending moments transmitted correspondingly by a steel joint $M_{j,a}$ and composite joint $M_{j,ac}$ were specified in the following manner (see figure 3):

$$M_{j,a} = F_{t1} \cdot z_a \text{ and } M_{j,ac} = F_{ac} \cdot z_{ac} + F_{t2} \cdot z_a. \quad (2a), (2b)$$

If there occurred compression of the top zone of steel joint, then a spring with stiffness k_{c2} placed at the height of the bolt axis was replaced by a spring with stiffness k_{c2}^* placed in the axis of the top flange of I-girder (see figure 3).

3. Material model

While specifying the deformability, there was adopted an elastic-plastic material model of steel (figure 4). The limiting value of steel deformation of sections as well as end-plates was assumed as $\epsilon_{ua} = 18\%$, the calculated yield point $f_{ya} = 235$ MPa and the tensile strength $f_{ua} = 360$ MPa. The calculated yield strength for reinforcing steel was assumed as $f_{ys} = 434$ MPa and the limiting extensions as $\epsilon_{us} = 7\%$.

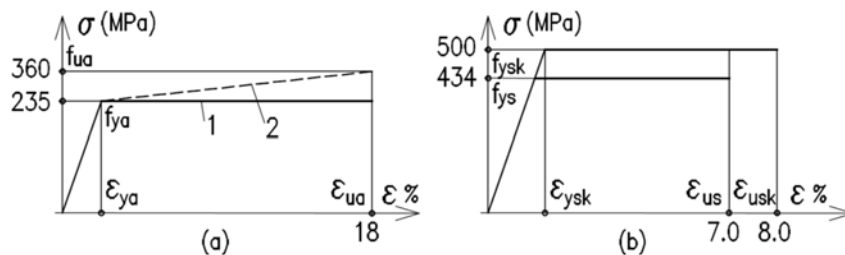


Figure 4. Material characteristics to calculate deformation (bilinear stress-strain relation):
a) structural steel, b) reinforcing steel (1, 2 on figure 4a - see point 5.3)

Values of extension for bolts were assumed in a semi-empirical manner (see point 5.1), without establishing a specified value of deformation ε_{ub} .

4. Characteristics of basic components of the joint under Eurocodes

Initial stiffnesses of basic components k (column 4 in table 1) as well as F_{Rd} (column 2) were adopted in accordance with the formulas provided in [1] and [2]. Values of the elastic load-bearing capacity provided in column 3 were assumed for the web and bolts based on the relationship between f_{ya} and f_{ua} as well as f_{yb} and f_{ub} , and for end-plate and column flanges such value was 2/3 of the bearing-capacity thereof. The force-displacement characteristic until the load-bearing capacity F_{Rd} of basic components was achieved for the first time, it was assumed as follows (comp. [4], [6]):

$$\Delta_{pl,EC} = 4,5\Delta_{el,EC}(F_{el,Rd}), \quad (3)$$

where $\Delta_{pl,EC}$ – “plastic” deformation of basic component, $\Delta_{el,EC}$ – elastic deformation upon achieving the loading $F_{el,Rd}$ equal to 2/3 of loading F_{Rd}

Adoption of the relationship (3) is an inexact analogy to the formula (see figure 5a and 5b):

$$S_j = S_{j,ini} (1,5M_{j,Ed}/M_{j,Rd})^\Psi, \quad (4)$$

where S_j , $S_{j,ini}$ – secant and initial stiffness, $M_{j,Ed}$ – joint’s moment load.

Assuming in the formula (4) the value $\Psi = 2.7$ acc. [1] and [2] it is obtained that upon achieving for the first time the value $M_{j,Rd}$ (or $M_{j,a,Rd}$), the calculated value of rotation for the steel joint or the joint connected to the end-plate is 4.5 times larger than upon achieving the value $M_{j,Ed} = (2/3)M_{j,Rd}$ (or $M_{j,a,Ed} = (2/3)M_{j,a,Rd}$) which is recognized as the value corresponding to the end of joint’s elastic work.

In column 7 of table 2 there was provided the value of deformation corresponding to rotation Φ_{xd} equal to $4.5\Phi_{eld}$, where Φ_{eld} – rotation corresponding to the load of $(2/3) M_{j,a,Rd}$.

Table 1. Initial stiffness and elongation of basic components modelled as springs acc. to [1] and [2].

Part of joint	Load capacity F_{Rd}	Elastic capacity $F_{el,Rd}$	k	elongation $\Delta_{el,EC}$ and $\Delta_{pl,EC}$ of spring			Joint variant
				“Elastic” by $F_{el,Rd}$	“Plastic” by F_{Rd} (eq. 3)	Used at Φ_{xd} in steel joint	
-	[kN]	[kN]	[mm]	[mm]	[mm]	[mm]	-
2 bolts in tension (row)	352.8	317.5	8.25	0.136	0.611	0.096	A
	231.8	208.6	5.79	0.172	0.772	0.123	B
Column flange in transverse bending	166.4	110.9	5.09	0.104	0.467	1.313*	A
	149.7	99.8	5.09	0.093	0.420	1.205*	B
End-plate in bending	235.9	157.3	8.98	0.083	0.375	0.117	A
	188.8	125.9	8.98	0.067	0.300	0.155	B
Column web in compression	313.5	204.6	5.65	0.172	0.776	0.140	A
	313.5	204.6	5.65	0.172	0.776	0.126	B
Column web in tension	357.2	233.2	7.00	0.159	0.714	0.113	A
	357.2	233.2	7.00	0.159	0.714	0.102	B
Reinforcing steel (longitudinal bars)	393.4	393.4	8.62	0.228	1.027	-	A
	393.4	393.4	8.62	0.228	1.027	-	B

* increased in order to obtain $\Phi_{xd} = 4.5\Phi_{eld}$ in the entire steel joint

For pure steel joint $\Phi_{xd} = 7.12$ mrad (variant A) and $\Phi_{xd} = 6.85$ mrad (variant B)

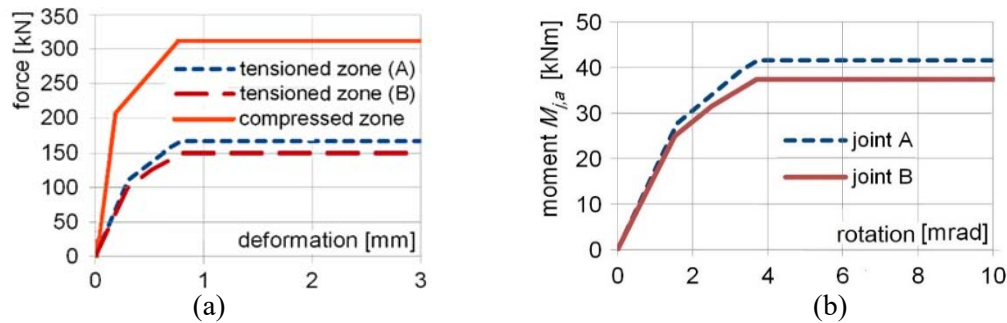


Figure 5. Adopted relationships: a) force-deformation of tension and compression zones of the steel joint (absolute values), b) moment-rotation of the entire joint

5. Deformability of basic joint components in ULS

In a current version of Eurocode 3 there is no information allowing to calculate deformations Δ_u of particular basic components of the joint at the limit state. Also, it does not provide relationships allowing to calculate the rotation Φ_{Cd} , which is achievable at the load equal to the load-bearing capacity of joint $M_{j,Rd}$ (comp. to figure 1). Therefore, the value of deformations Δ_u were assumed based on suggestion from the literature and own analyses. As limiting value Δ_u , it is meant the highest deformation achievable without the loss of load-bearing capacity, and not the moment of material tearing.

5.1. Bolts in tension and longitudinal reinforcement

Significant influence on bolt elongation in ULS has the threaded part of shaft within the area between the head and half of the nut height. Elongation of the bolt working in an end-plate connection upon achieving the tensile strength f_{ub} was estimated in accordance with the suggestion [3]:

$$\Delta_{b,u} = \frac{F_u}{E} \left(\frac{0.4d + l_s}{A} + \frac{l_g - l_s}{0.5(A + A_s)} \right) + \frac{F_u l_t + 0.6m}{E A_s} + \frac{F_u - F_y}{\alpha E} \frac{l_t + 0.6m}{A_s}, \quad (5)$$

where: A and A_s – gross and net area of the bolt cross-section, d and m – height of head and nut, l_g – distance from head to thread origin, l_s – length of bolt with cross-section A_s , l_t – length of thread to nut (from the head side), α – coefficient of 0.013 for bolts class 10.9, E – Young's modulus.

Limit value of elongation of reinforcement rods within the area of the joint was assumed to be

$$\Delta_{s,u} = 0.5h_c \varepsilon_{us}, \quad (6)$$

where h_c – height of column cross-section.

In accordance with the formula (6), limit value of reinforcement elongation shall be 7.30 mm.

5.2. Web in transverse compression and web in transverse tension

The maximum deformation of web in transverse tension was initially estimated in accordance with authors' own proposal (figure 6a):

$$\Delta_{c,wt,u} = \Delta_u(F_{c,wt,Rd}) = 0.25\varepsilon_u b_{eff,c,wt}(l - f_y/f_u), \quad (7)$$

where $b_{eff,c,wt}$ – effective width established in accordance with the Eurocode 3.

In accordance with the formula (7) compliant with the scheme in figure 6a, the value of deformation $\Delta_{c,wt,u}$ shall be 3.39 mm, and the value integrated in accordance with the scheme in figure 6b shall be 2.98 mm. Finally, calculations included $\Delta_{c,wt,u} = 2.98$ mm. The value of deformation of the transversely tensioned web HE220A in accordance with simple formulas provided in the literature may be 5.25 mm per side ($0.025h_c$, where h_c – height of column cross-section).

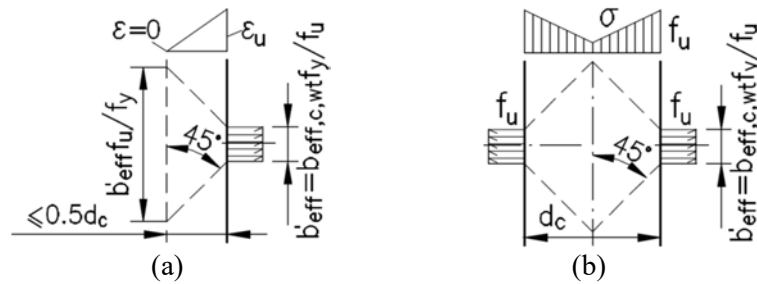


Figure 6. Simplified scheme for calculating of web transverse elongation (without axial forces)

The maximum value of web deformation in transverse compression was assumed under the following formula:

$$\Delta_{c,wc,u} = \left(4,8 - 0,11 \frac{d_c}{t_{wc}} \frac{1}{\varepsilon} \right) d_c, \quad (8)$$

where d_c – height of the column web, t_{wc} – web thickness, ε – value depending on the steel grade, corresponding to the value ε in EC3 [1].

The value of deformation in accordance with formula (8) shall be $\Delta_{c,wc,u} = 3.66$ mm. The author intends to present critical analysis of the means of specifying the deformability of the web transversely compressed or tensioned in ULS together with own proposal of analytical relationships in a separate publication.

5.3. Flange of column and end-plate in transverse bending, plastic hinge

In figure 7a, there is presented a modelled function of moment-curvature for the flange of steel T-stub, starting from the elastic state to shaping a full plastic hinge. The loss of stiffness of the plastic hinge until exhausting its bearing-capacity is illustrated in figure 7b. The ultimate lowest value of stiffness of the T-stub flange with effective length l_{eff} and thickness of the flange t (for a model without strain-hardening with resistance M_{pl} , or with strain-hardening with resistance M_{ult}) shall be:

$$B_u = (EI)_u \frac{l_{eff} t^3 f_y}{8 \varepsilon_u} \quad \text{or} \quad B_u = (EI)_u \frac{l_{eff} t^3 f_y}{8 \varepsilon_u} \frac{M_{ult}}{M_{pl}}. \quad (9)$$

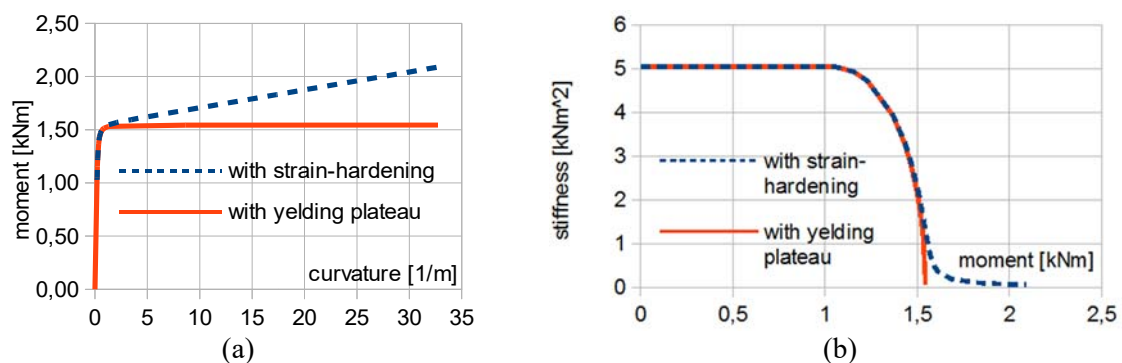


Figure 7. Relationship for a plastic hinge of T-stub: a) moment-curvature, b) moment-stiffness

While analysing the value of deformation of the system of substitute T-stubs and bolts, it was assumed that in the limit state of bearing-capacity there shall occur joint separation. In case of the joint with prestressed bolts in variant B, it shall certainly occur at adoption of the curve (1) in figure 4, and in case of the joint in variant B, at adoption of the curve (2). In [1], there is assumed an occurrence of

the prying effect in end-plate joints, which corresponds to necessity of occurrence of connection separation (which may be verified in a simple experiment). At the moment of exhausting the bearing-capacity of the steel joint, the estimated limit deformation of the system T-stub + web + bolts shall have few millimetres. In the composite joints in question, limit values of deformation of the tension zone shall not be achieved. At the load below 2/3 of the load-bearing capacity of the joint, they shall be elastic deformations, and at the load of 5/6 of the load-bearing capacity of the joint, they shall not exceed ~1 mm. More precise presentation of the analysis of limit deformations of the tension zones in accordance with own algorithm shall be contained in a separate publication.

6. Results and discussions

Values of load-bearing capacity and rotation capacity of the analysed joints in relation to the assembly load is listed in table 2. Graphs of path static equilibrium moment-rotation is presented in figure 8 and figure 9.

Table 2. Elastic load-bearing capacity and rotational capacity of analysed composite joints.

Effort of a steel joint	Joint with bolts M20 (variant A)				Joint with bolts M16 (variant B)			
	Assembly load	Moment capacity		Rotation at composite stage Φ_{cd}	Assembly load	Moment capacity		Rotation at composite stage Φ_{cd}
		composite stage	whole load range			composite stage	whole load range	
[%]	[kNm]	[kNm]	[kNm]	[mrad]	[kNm]	[kNm]	[kNm]	[mrad]
0	0.00	78.1	78.1	11.9	0.00	78.1	78.1	11.8
16.7	6.93	67.5	74.4	11.5	6.24	68.6	74.8	11.4
33.3	13.86	56.9	70.8	11.1	12.48	59.1	71.5	11.0
66.7	27.73	35.7	63.5	10.3	24.95	40.0	65.0	10.3
83.3	34.67	24.9	59.5	9.7	31.19	30.7	61.9	9.7

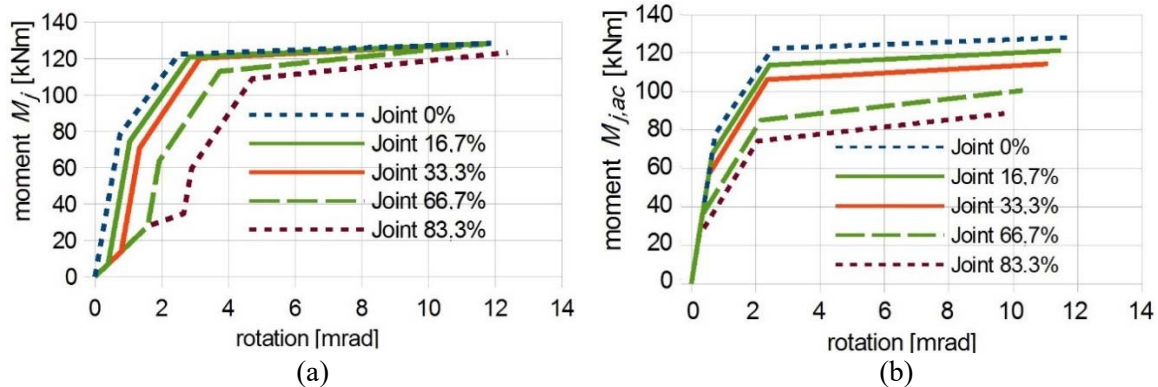


Figure 8. Moment-rotation path for joint in variant A: a) whole load range, b) composite joint

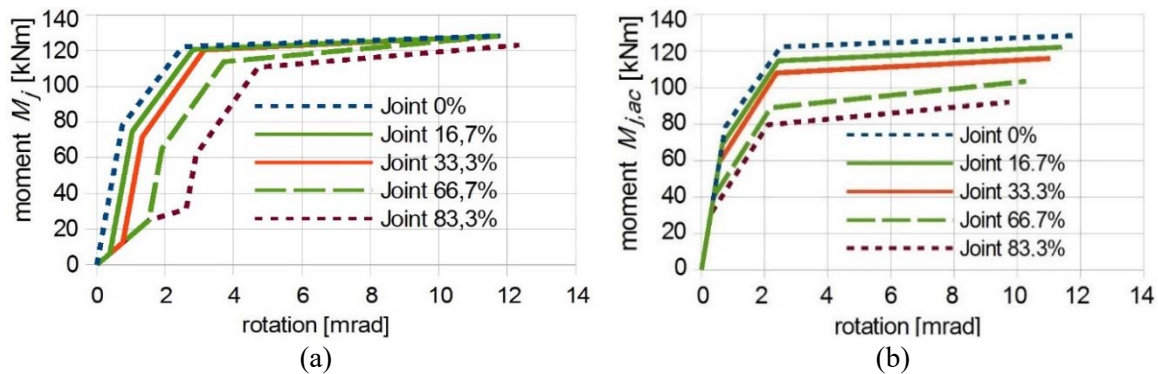


Figure 9. Moment-rotation path for joint in variant B: a) whole load range, b) composite joint

At the assembly load below $\frac{2}{3}$ of the load-bearing of the steel joint, the total load-bearing capacity of a composite joint and its total rotation capacity in ULS shall not differ from the load-bearing of the joint in a propped construction, however there shall decrease the value of load giving rise to plastification and flow of the compression zone (table 2 and figure 10). This means that at significantly smaller load than in the propped construction there shall begin a reduction of stiffness of the composite joint, which shall significantly influence the deformation of entire frame. There shall also significantly decrease the value of load, which may be safely imposed after hardening of the concrete slab. At the assembly load of $\frac{5}{6}$ of the load-bearing capacity of steel joint (the compression zone of joint in a plastic stage of work), the total bearing-capacity drops by about 5% in comparison to the propped joint, but the value of rotation slightly increases. Decrease of the elastic bearing-capacity from the load imposed after hardening of the concrete slab is a linear function of the steel joint stress at assembly (figure 10).

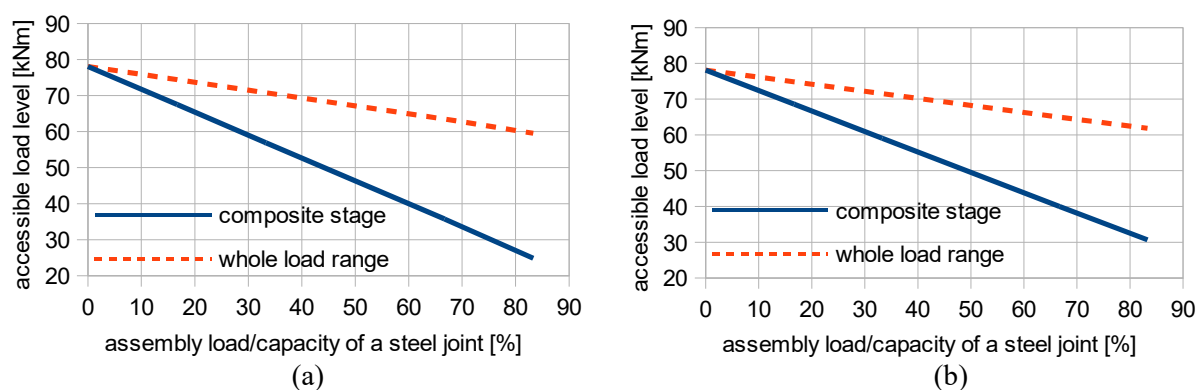


Figure 10. The load upon achieving the elastic load-bearing capacity of the composite joint in relation to the level of assembly load: a) variant A, b) variant B

The conclusions listed concern the joint, load-bearing capacity of which before execution of a concrete slab is determined by the load-bearing capacity of its top zone, and after execution of the slab by the load-bearing capacity of web. In relation to the fact that after execution of the slab, additional load shall give rise to bolts effort decrease, their influence on the load-bearing capacity of the composite joint may be neglected. Therefore, bolts shall be selected only in view of the assembly stage.

7. Alternative calculation methods

In literature [8], there may be encountered a view that consists of modelling in a calculation software of the joint component beam-column (with slightly different character than the one analysed in this work), with a characteristic of the steel joint adopted based on laboratory test and then supplementing such model with additional elements that simulate reinforcement, connected to the column as well as in a certain number of points with the steel beam. In this work, it was tried to present a different attempt, while adopting a component method to predict the behaviour of joint on both stages of work - as steel joint and composite joint.

8. Conclusions

Execution of non-propped construction influences the level of stress and value of rotation of the joint in various stages of load increase. Therefore, the path load-rotation is changed. Consequence of execution of the composite construction without vertical supports may be the increase of stress of the steel part of both frame girder and steel components of the joint, and on the other side quickening of the moment of plastification of the composite joint and certain decrease of its rotation capacity. Limiting the elastic range of composite joint behaviour may be unfavourable in view of the decrease of its load-bearing capacity and appearance on one side, and on the other side the decrease of stiffness may be advisable in

some cases. It appears that the assembly load of steel joint up to 30% of its load-bearing capacity is acceptable without a detailed analysis.

The analysis presented in this article has a distinctive nature, and the proposed method of calculations shall be treated as non-final. The aim of planned works shall be selection of a simple analytical model of joint, adjusted to the philosophy of the component method. In order to develop calculation procedures, further analysis, and maybe experimental tests, are required. In further publications, there shall be analysed various cases of composite steel-concrete frames with various assembly technology, as well as other composite joint solutions.

References

- [1] EN 1993-1-8, "Eurocode 3: Design of steel structures. Part 1-8: Design of joints".
- [2] EN 1994-1-1, "Eurocode 4: Design of composite steel and concrete structures. Part 1-1: General rules and rules for buildings".
- [3] A. Steurer, "Load-bearing behaviour and rotation capacity of bolted end plate connections", ETH Zürich, 1999, <https://doi.org/10.3929/ethz-a-003878456> (in German).
- [4] F. Kühnemund, "Verification of rotation capacity of semi-rigid steel joints", PhD thesis, Universität Stuttgart, 2003, <http://dx.doi.org/10.18419/opus-167> (in German).
- [5] A. M. Coelho, "Characterization of the Ductility of Bolted End Plate Beam to Column Steel Connections," PhD thesis, Universidade de Coimbra, July 2004.
- [6] D. Beg, E. Zupancic, I. Vayas, "On the rotation capacity of moment connections," *Journal of Constructional Steel Research*, vol. 60, pp. 601- 620, 2004.
- [7] L. Ślęczka, "Shaping and analysis of selected steel frame joints subjected to variable actions", Oficyna Wydawnicza Politechniki Rzeszowskiej, Rzeszów 2013 (in Polish).
- [8] U.I. Dissanayake, I.W. Burgess, J.B. Davison, "Modelling of plane composite frames in unpropped construction," *Engineering Structures* vol. 22, pp. 287–303, 2000.