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PLASTIC CAPACITY OF BOLTED RHS FLANGE-PLATE JOINTS UNDER AXIAL TENSION

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Abstract. This article considers the calculation of load-bearing capacity of flange-plate joints with bolts along two sides of rectangular hollow sections (RHS) under axial tension. It provides a review and comparison of various calculation methodologies for establishing the load-bearing capacity of RHS flange-plate joints, such as suggested in EN 1993-1-8:2005 and STR 2.05.08:2005 as well as those proposed in different countries and by other authors. Common design principles and derived results for load-bearing capacity of flange-plate joints have been analysed and compared. Following the numerical modelling, which has been done using ANSYS Workbench finite element program, the derived results for load-bearing capacity have been compared with analytical load-bearing capacity results for flange-plate joints of the same structure. The analysis has focused on one type of flange-plate joints with bolts – both preloaded and non-preloaded – along two opposite sides of the tube, with the flange thickness of 15 mm and 25 mm.

Keywords: hollow section, axial tension, prying, bolted flange-plate joint, design model, load-bearing analysis.

Introduction

Due to technological and economic indicators, trusses from cold-formed rectangular hollow sections (RHS) have become widely used in the construction of buildings established for the purposes of warehousing and sales. Usually, floor area is especially crucial in such buildings; thus, solutions with fewer columns are considered. With spans of 18, 24 and 30 metres and having in mind size restrictions for transportation, a truss must be divided into separate segments; therefore, the design of assembly connection for tensile and compressive chords should be considered. In the general case, these assembly joints can be slip-resistant, shear or tension joints. Even though all of the above-named types of joints can be used to connect truss elements, this article focuses only on flange-plate joints in axial tension as their initial displacement is the least if compared to shear joints.

Based on the effectual Lithuanian design code STR 2.05.08:2005 (STR), calculations of flange-plate joints in axial tension are made in the elastic stage, applying rigorous additional design and production restrictions (Daniunas *et al.* 2006). These measures result in a complex joint with additional stiffeners, a large number of preloaded bolts and a thick flange, faces of which might need to be milled. Therefore, the aim to reduce production costs necessitates changes in design assumptions. As an alternative to the existing code, EN 1993-1-8 (2005) allows plastic deformations that result in a more rational use of materials. It is unfortunate that EC-3 (EN 1993-1-8:2005) does not provide the design of RHS flange-plate joints, dif-

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ferently than the American AISC (Packer *et al.* 2010) or the Canadian CISC (Packer, Henderson 1997). The main aim of this article is to compare the calculation methods for RHS flange-plate joints that are provided in design codes with general assumptions of EC-3 and demonstrate the extent to which the analytical result would differ should the formulas of the Lithuanian design code STR be used.

In all of the above-named codes, the design of flange-plate joints is based on the superposition principle, i.e. – the bearing capacity of a joint equals the sum of bearing capacities of joint components. In turn, each joint component consists of usual members, such as bolts, welds, a flange, and a tube wall. As lattice truss elements are selected from the conditions of strength and buckling, the tube wall strength will be ensured during the design of assembled joints; therefore, the key task is to estimate the force that could be withstood by bolts and welds.

Deformations of flanges give rise to additional prying forces that emerge in bolts. The forces are evaluated using equivalent T-stub models. One of the most prevalent models, which is used in design codes of various countries (AISC, CISC, EC-3), has been introduced by Struik and de Back (1969). Assumptions of this model have been improved and analysed by Zoetemeijer (1974), Jaspart and Maquoi (1991), and Kulak *et al.* (2001). As a simple pair of concentrated forces can substitute the internal bending moment, a T-stub model is applicable not only for a simple tension but also for column-beam and column-base connections. A detailed analysis of the existing equivalent T-stub models has been made by Swanson (Swanson 2002).

In general, the behaviour of RHS flange-plate joints has been analysed underlining single cases of bolts either symmetrically arranged on all sides (Kato, Mukai 1982; Willibald *et al.* 2002, 2003) or on the two opposite sides of the flange (Packer *et al.* 2009). In addition, some analysis has been made on flange-plate joints with stiffeners (Semenov *et al.* 2014; Wang *et al.* 2013; Perelmuter *et al.* 2010). The mechanism of plastic hinges and the stiffness of a joint based on EC-3 have been addressed by Karlsen and Aalberg (2012), Latour and Rizzano (2013) and Heinisuo *et al.* (2012).

1. General model of prying action

The eccentric position and flange deformations amount to additional prying forces in a bolt. The impact of the forces on the load-bearing capacity of the joint is determined by the analysis of equivalent T-stub members. One of the most popular methods for the determination of the load-bearing capacity has been offered by Struik and de Back. This method describes three failure modes of a T-stub in tension: full plasticity of a flange-plate (Fig. 1a); bolt failure with partial flange-plate plasticity (Fig. 1b); and bolt failure (Fig. 1c). The key advantage of the method lies in its relatively simple equations, in which the equilibrium of shear forces and bending moments can be easily disturbed. They have no empirical coefficients; therefore, this model has no restrictions related to bolt diameter or strength as well as the least thickness of the flange. Rather conservative load-bearing capacity results could be named as a shortcoming in cases where thin flanges are used as the model does not consider material strengthening and expressed membrane behaviour. Considering the symmetry of a member, Table 1 providesthe side-by-side comparison of equations for one bolt from EC-3 and the modified Struik and de Back model for an equivalent T-stub model. The modified Struik and de Back method was chosen instead of the original one because it considered the weld just as EC-3; meanwhile, other assumptions of the method remain unchanged and correspond to the original method.

The exceptional characteristic of the model by Struik lies in the fact that the three failure modes are described using equilibrium equations with the quantity α. This relative quantity shows the number of times the bending moment beside the T-stub wall is greater than the bending moment beside the bolt. In

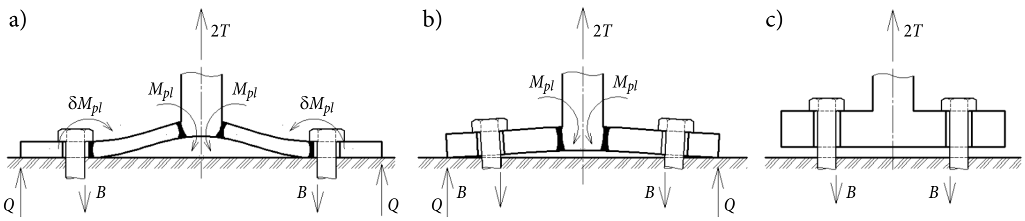
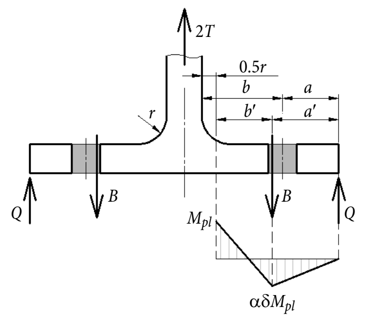
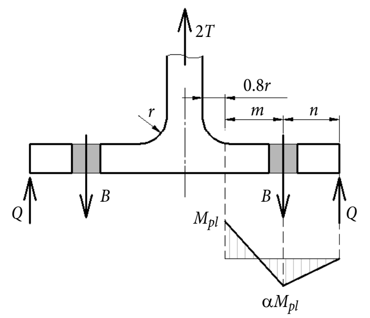


Fig. 1. Failure modes of a T-stub model: full plasticity of a flange-plate (a); bolt failure with partial flange-plate plasticity (b); bolt failure (c)

Table 1. Types of T-stub models

Modified Struik and de Back model	EC-3, Zoetemeijer
	
Equilibrium equation: $$\begin{cases} T\cdot b' = \left[1+(\alpha\cdot\delta)\right]M_{pl} \\ Q\cdot a' = \alpha\cdot\delta\cdot M_{pl} \\ B = T+Q \end{cases};\qquad (1a)$$	Equilibrium equation: $$\begin{cases} T\cdot m = \left[1+\alpha\right]M_{pl} \\ Q\cdot n = \alpha\cdot M_{pl} \\ B = T+Q \end{cases};\qquad (1b)$$
Relationship of bending moments: $$\alpha = \frac{1}{\delta}\left(\frac{T\cdot b'}{M_{pl}} - 1\right);\qquad (2a)$$	Relationship of bending moments: $$\alpha = \frac{T\cdot m}{M_{pl}} - 1;\qquad (2b)$$
Prying force: $$Q = T\left(\frac{\alpha\cdot\delta}{1+\alpha\cdot\delta}\right)\left(\frac{b'}{a'}\right);\qquad (3a)$$	Prying force: $$Q = T\left(\frac{\alpha}{1+\alpha}\right)\left(\frac{m}{n}\right);\qquad (3b)$$
Strength when α = 1 (Fig. 1a)	
$$T_a = \frac{1+\delta}{b'}M_{pl};\qquad (4a)$$	$$T_a = \frac{2}{m}M_{pl};\qquad (4b)$$
Strength when 1 ≥ α ≥ 0 (Fig.1b)	
$$T_b = \frac{B_{yd}\cdot a' + M_{pl}}{a'+b'};\qquad (5a)$$	$$T_b = \frac{B_{yd}\cdot n + M_{pl}}{m+n};\qquad (5b)$$
Strength when α = 0 (Fig. 1c)	
$$T_c = B_{yd}\qquad (6)$$	

the physical sense, the opening of the plastic hinge beside the bolt is the result of the hinge formation beside the T-stub wall; therefore, according to the module, α cannot be greater than one. Another dimensionless quantity is δ, which is found only in expressions based on the original Struik and de Back model. It is used to evaluate the impact of a bolt hole on the formation of plastic hinge:

$$\delta = 1 - \frac{d_h}{l_{eff}}.\qquad (7)$$

Here, d_h is the diameter of the bolt hole; l_{eff} is the effective length of a T-stub tributary to one bolt.

The location for the formation of plastic hinges depends on the stiffness as the increase in the cross-section beside the T-stub wall, the lever arm of the external force and bolt resultant force. The location of the resultant force itself depends on the distribution of pressure under the bolt head. In the Struik and de Back model, the bolt force is shifted sidewise by a half of the nominal diameter of the bolt; on the contrary, the EC-3 makes an assumption that the pressure under the bolt head is evenly distributed, and the resultant force is concentrated beside the bolt axis. According to Jaspart and Maquoi (1991), none of these assumptions correspond to the true behaviour as the assumption by Struik and de Back overestimates the local effect of the bolt and EC-3 does not consider the bending of the bolt. Yet another general quantity of models is the plastic bending moment, which is calculated by the formula:

$$M_{pl} = l_{eff}\frac{t^2 f_{yd}}{4}.\qquad (8)$$

Here, t is the thickness of the flange of a T-stub model; f_{yd} is the design steel yield strength.

The equivalent T-stub model can be graphically represented as a line OABC, as a dependence of the resultant force of the T-stub model on the flange thickness (Fig. 2). Its sections represent the following: OA – the zone of the flange-plate thicknesses, in which the mechanism of two hinges can develop while attaining the maximum of prying forces; in the thickness zone AB, the bolt fails earlier than the maximum value of prying forces institution reached ($Q_i < Q_{max}$); BC – support force of the T-stub model does not depend on the thickness of the plate. Yet another exceptional part of this figure is the section OBCD. Theoretically, the external force, which is a part of this section of the figure, neither causes the rise of prying forces nor causes a partial yield of the plate, which is especially important for structures affected by fatigue loading. Expressions of marginal thicknesses of the plate are given in Table 2.

As Figure 2 demonstrates, the AB part of the curve of the modified Struik and de Back model (Swanson 2002) is shorter than that of the EC-3. In the case of point A, the difference between the two thicknesses of marginal failure thicknesses of these two models amounts to 2.20 mm; and in the case of point B – 5.28 mm.

2. Calculation of flange thickness for RHS joints

The available design regulations for tubular profile flange-plate joints (AISC; CISC; CIDECT), which are based on the Struik and de Back model, underscore two design cases with bolts arranges on two sides of the tube (Fig. 3a) and around the entire contour (Fig. 3b and 3c). Cases with bolts arranged beyond the tube boundary are considered an exception to this grouping; therefore, the codes do not provide such calculations.

Depending on a situation, a check or design problem can be solved. During a check, the thickness of a flange is known; therefore, the entire calculation focuses on finding the minimum T-stub tensile capacity: $T_{Rd} = \min(T_a, T_b, T_c)$. Solving the design problem, the calculation focuses on finding the minimum number of bolts from the condition that the external force per bolt would not exceed the design bolt tensile capacity ($T_{Ed} \leq B_{Rd}$); maximum thickness of the plate is calculated according to the expression (10). In the case when plastic deformations are permitted, the thickness of the plate can be reduced to t_{req}, which is determined using Equation (12).

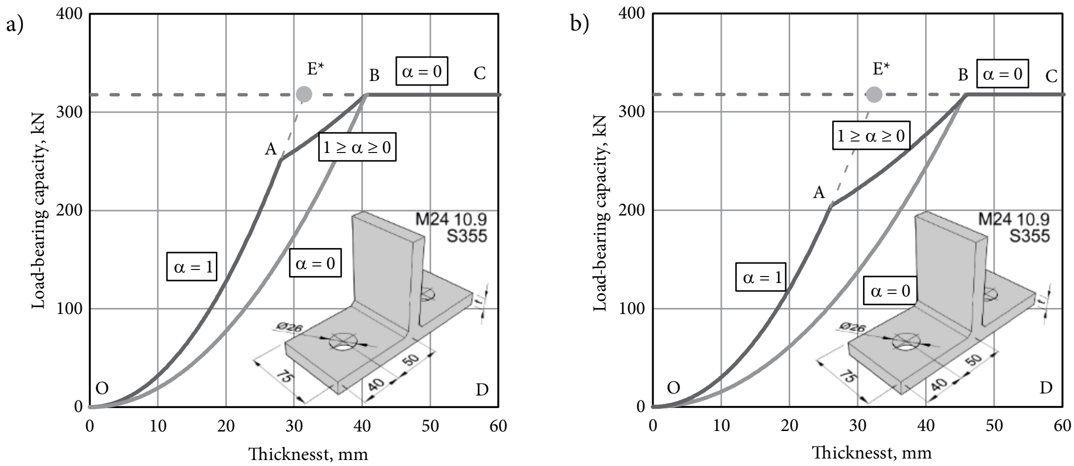


Fig 2. Relationships between the T-stub load bearing capacity and flange thickness: modified Struik and de Back model (a); EC-3 (b)

Table 2. Expressions of calculations for flange marginal thicknesses under different failures

Modified Struik and de Back model		EC-3, Zoetemeijer	
$t_A = \sqrt{\dfrac{4b'B_{yd}a'}{l_{eff}f_{yd}\cdot((1+\delta)(a'+b')-b')}}$;	(9a)	$t_A = \sqrt{\dfrac{4B_{yd}\cdot n\cdot m}{2l_{eff}f_{yd}\cdot(m+n)-l_{eff}f_{yd}m}}$;	(9b)
$t_B = \sqrt{\dfrac{4B_{yd}b'}{l_{eff}f_{yd}}}$;	(10a)	$t_B = \sqrt{\dfrac{4B_{yd}m}{l_{eff}f_{yd}}}$;	(10b)
$t_{E*} = \sqrt{\dfrac{4B_{yd}b'}{l_{eff}f_{yd}\cdot(1+\delta)}}$.	(11a)	$t_{E*} = \sqrt{\dfrac{2B_{yd}m}{l_{eff}f_{yd}}}$.	(11b)

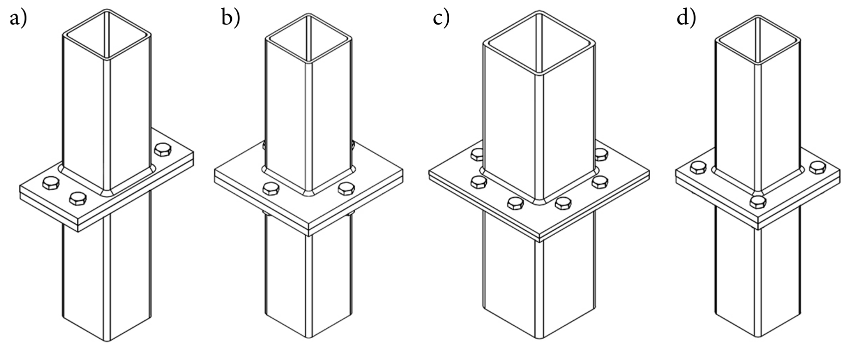


Fig. 3. Examples of RHS flange-plate joints: bolts on two opposite sides of the tube (a); bolts arranged symmetrically along the entire contour(a, b); bolts arranged beyond the tube boundary (d)

Table 3. Thickness of the flange according to the ultimate limit state

Modified Struik and de Back model	EC-3, Zoetemeijer
$t_{req} = \sqrt{\dfrac{4T_{Ed}b'}{l_{eff} f_{yd}(1+\delta\alpha_{Ed})}}$ (12a)	$t_{req} = \sqrt{\dfrac{4T_{Ed}m}{l_{eff} f_{yd}(1+\alpha_{Ed})}}$ (12b)

Ultimately, when the external force per bolt amounts to the bolt strength ($T_{Ed} = B_{Rd}$), and the coefficient of the relationship between acting moments amounts to zero ($\alpha_{Ed} = 1$), Equation (12) transforms into Equation (11). In this case, the least thickness of the plate allowed corresponds to the apparent point E* (Fig. 2). Once the thickness of the plate is selected, checking is made, i.e. the condition $T_{Ed} \leq B_{Rd}$ is checked using Equation (1), or $T_{Ed} \leq T_{Rd}$ – using Equation (5).

It is important to state that the direct application of the T-stub model for RHS end plate joints is debatable."Depending on the position of the bolts along RHS wall, and the relationship between the thicknesses of the end-plate and the RHS wall, the end plate inside the section may be subjected to significant bending. This bending causes both bending and axial deformation of RHS walls, most prominent in the parts away from the corners" (Karlsen, Aalberg 2012).This phenomenon complicates the direct application of the T-stub model, because of changes in places where plastic hinges will form. For elimination of this inaccuracy, Packer *et al.* (2009) suggested using a greater external force and a bolt force lever arm: $b' = b - 0.5d_b + t$.

Willibald *et al.* (2002, 2003) investigated flange-plates with bolts on four sides of tubular cross section (Figs 3b, 3c). Based on his own experimental test results, Willibald concluded that theoretical assumptions for a T-stub had better compliance without increased lever arm of forces suggested by Packer. Then, a question arises about a possibility to apply this assumption to RHS flange-plate joints with other geometrical configuration (Fig. 3), because bending at a wall of a hollow section is inherent for all types of RHS lange-plate joints. For example, Karlsen and Aalberg (2012) in their calculations of joint with bolts on two sides of RHS did not take into account possible changes in a shape of plastic lines and concluded that for flanges in thicknesses of 8 and 10.1 mm, the prediction of failure type was exact, but the capacity of a joint was diminished. This conclusion was based only on four tests; therefore, it is impossible to answer about the correlation between EC-3 T-stub model and tests results.

3. Numerical simulation of the behaviour of a flange-plate

One of the main difficulties in determining the bearing capacity of a T-stub in tension is the calculation of the effective length. In a general case, yield lines form in three-dimensional patterns and each arrangement of bolts should have numerous failure schemes with a joint instantaneously turning into a mechanism. Applying the virtual work principle to these schemes, the least effective yield line length must be determined from among all mechanisms. The length corresponds to the minimum total potential energy. The scope of this problem is too wide. Some separate solutions can be found in EC-3 tables. For the numerical analysis of this article, the case of a flange-plate joint was selected with bolts on both sides (Fig. 3a). In such a case, the effective length of a T-stub model per bolt equals to half of the joint width (75 mm).

Calculations were made using *ANSYS Workbench* software, in the Static Structural analysis system. Once the joint symmetry is inputted, with regard to two perpendicular planes, the joint and the bolt were meshed by SOLID187 finite elements. The bolt is simplified to a pin (Fig. 4), the smaller cross-section area of which is equal to the area of tensile stress. The initial bolt pretension equals 110 kN, with the package consisting of a nut and a washer attached to each other by Boolean operation, which allows diminishing the number of contact friction surfaces to three, i.e. between flange plates, between the bolt head and the flange, and between the washer and the flange. The formulation of contact surfaces is augmented Lagrangian. The tube is connected to the plate by a 5 mm thick full penetration butt weld. Non-linearity of materials is modelled by bilinear isotropic hardening with E_{tan} = 900 MPa for steel and E_{tan} = 1100 MPa for bolt material; mechanical properties are given in Table 4.

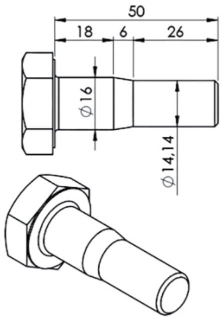


Fig. 4. Calculation model of a bolt

Designing the bolt strength under tension, codes EC 3 and STR 2.05.08:2005 use the safety factor multiplied by the characteristic strength of the bolt material according to the ultimate strength. The numerical value of this coefficient in the EC-3 is equal to the relationship: k_2 / γ_{M2}, where γ_{M2} is the partial safety factor of the load-bearing capacity of bolts, and k_2 considers the type of bolt head. In a numerical sense, to accept the recommended γ_{M2} values means that EC-3 relationship changes from 0.5 to 0.72. In the code STR2.05.08:2005, the reliability coefficient equals 0.5.

For the purpose of analysis, two groups of models were selected with flange thicknesses equal to 15 and 25 mm. Analytical solutions of these models, in which no partial reliability factors were used and bolt strength accepted according to the yield stress, are depicted in Figure 5. The relationships between loads and deformations are depicted in Figure 6.

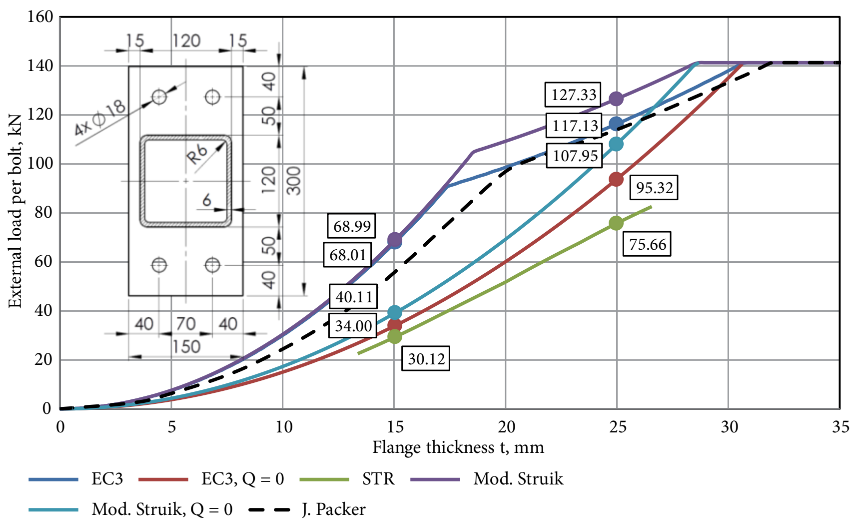


Fig. 5. Analytical solutions of the model

Table 4. Strength and mechanical properties of joint materials

Property	SteelS355 (EC-3)	Bolt 10.9 (ISO 898-1)
Modulus of elasticity E	210 000 MPa	201 000 MPa
Poisson's ratio υ	0.3	0.3
Yield stress f_y, f_{yb}	355 MPa	900 MPa
Tensile strength f_u, f_{ub}	490 MPa	1000 MPa
Extension at fracture A_t	0.15	0.09

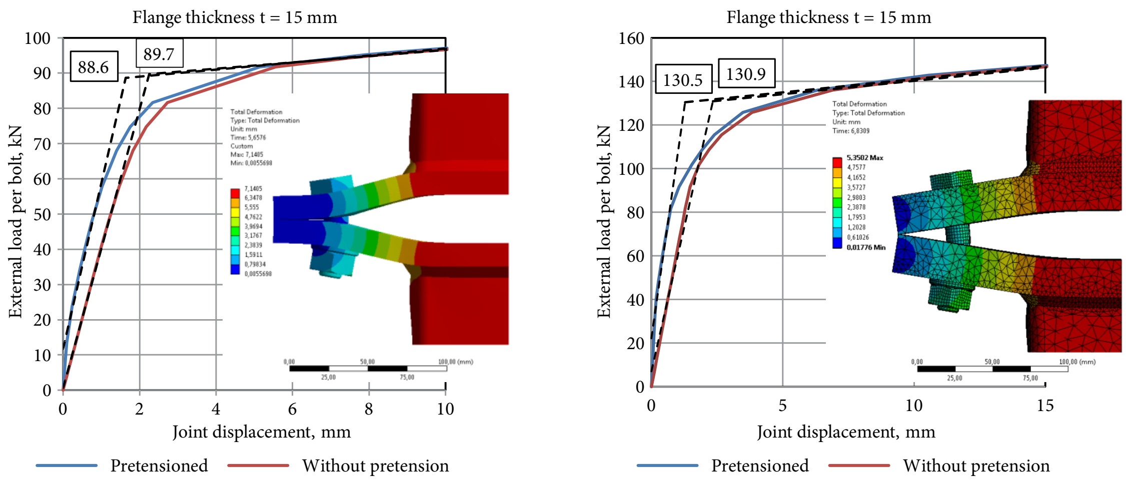


Fig. 6. Relationship between the jointload anddeformation

As depicted in Figure 5, for the Struik and de Back model with Packer's modified lever arm, the load for the formation of a one-hinge mechanism is 10% lower than for the modified Struik and de Back model and 4.4% lower than for EC-3. Because Struik and de Back and EC-3 models are conservative, in the case of a mechanism of two hinges, results with the Packer assumption would be even more conservative.

The main difference between joints with bolts that are preloaded and non-preloaded lies in the stiffness of the joint, i.e. in the case of preloaded bolts, it is close to infinity. In joints with non-preloaded bolts, tensile forces emerge instantaneously as soon as the external load is applied (Fig. 7). On the contrary, in joints with preloaded bolts, the rise of tensile forces is delayed because, at the initial stage, tensile forces must eliminate the advance compression of flanges. As soon as flange compression force approaches zero, the joint starts gapping and the bolt takes over all of the forces, i.e. the relationship between the force in the bolt and the joint load stops being linear (Fig. 7). In the analysis of structures unaffected by dynamic loads, the initial state of bolt stress has no impact on the load-bearing capacity (Fig. 6), yet must be considered during the analysis of the redistribution of internal forces.

To avoid brittle failure, it should be considered that the effective weld length is shorter than the perimeter of the tube (Fig. 8). It depends on the stiffness of the flange as well as dimensions of the tube, i.e. rounding of corners. Packer *et al.* (2010) states, that the approximate effective length of the fillet weld should be taken equal to the sum of lengths of tube side edges, alongside which bolts are arranged.

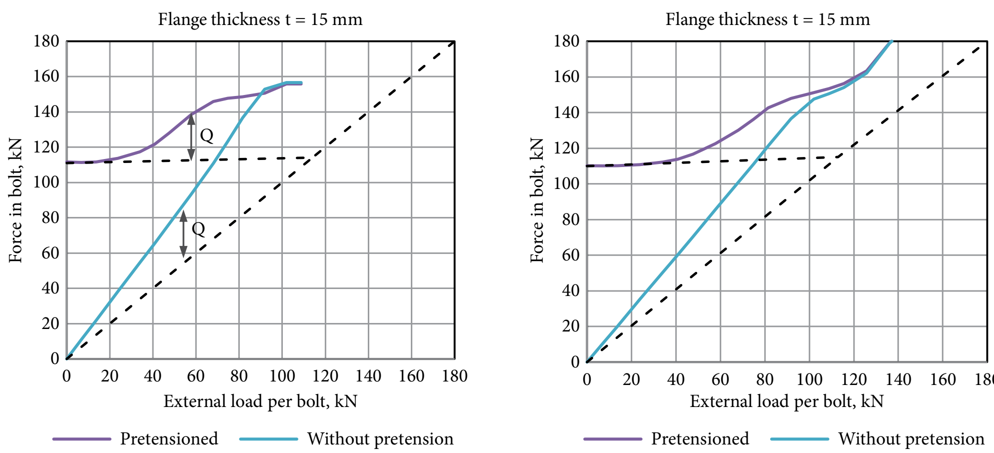


Fig. 7. Relationship between force in bolt and external load

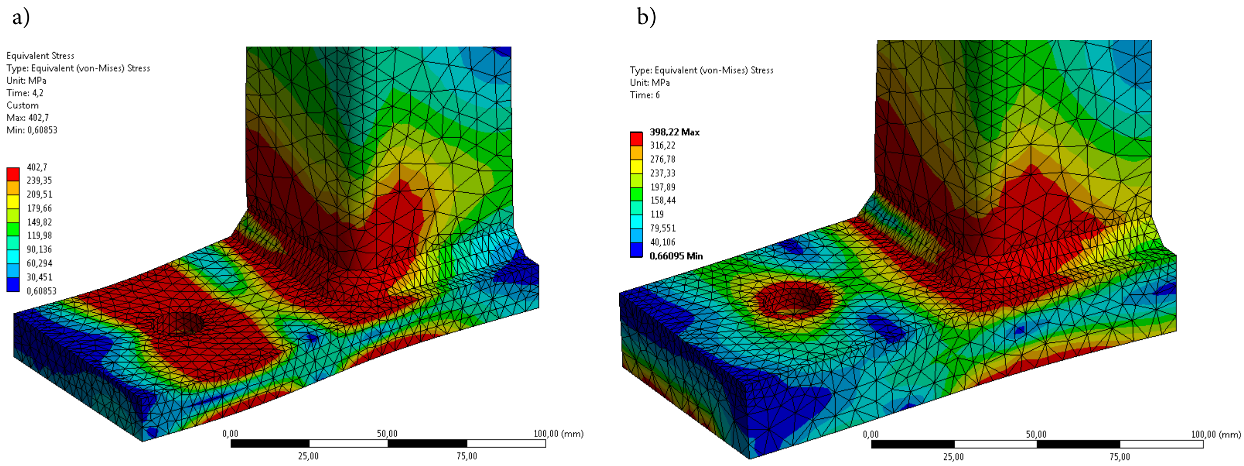


Fig. 8. Von Mises yield stresses: flange thickness of 15 mm (a); flange thickness of 25 mm (b)

Conclusions

1. Although from the first glance the modified Struik and de Back T-stub model does not seem similar to EC-3 due to different designations, it defines the same limit states. If bolt hole can be disregarded and the pressure is accepted as equally distributed underneath the bolt head, equations of both models would be the same. These models are universal and suitable for the design of joints with preloaded and non-preloaded bolts.
2. In calculations of joints with non-preloaded bolts, such as column bases, prying forces arise as soon as the load is applied; therefore, it seems irrational to design such bolts based on the condition that prying forces do not emerge (Q = 0).
3. The difference between the analytical and numerical models, in the case where the mechanism of two hinges emerges (t = 15 mm), amounts to 23%. For the case where one hinge emerges (t = 25 mm), the difference reduced to 10% if calculated according to EC-3; and to 2.6% if calculated based on the modified Struik and de Back model. If the partial reliability coefficient is used, which is applied in the STR code, then, according to the STR, the bearing capacity of the joint would equal to the gapping force (Fig. 7).
4. In the case of the analysed joint, the assumption by Packer did not serve the purpose as the estimated load for the formation of a two-hinge mechanism was conservative in comparison with EC-3, modified Struik and de Back model and numerical simulation results. One-hinge failure load was 4.5% less than that of EC-3, but failure modes for both cases corresponded to numerical simulation results (Fig. 8). To assess the possibility to apply EC-3 T-stub model for joints with bolts on two opposite sides of RHS, additional experimental and numerical investigations are required.

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