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CONNECTIONS OF STEEL GIRDERS WITH END PLATES AND PARTIALLY PRESTRESSED BOLTS

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Rezumat

Îmbinările cu plăci de capăt și șuruburi pot fi utilizate în cazul structurilor de poduri metalice, putând conduce la soluții constructive foarte simple și economice, cu condiția modelării corespunzătoare a modului de comportare mecanică a acestor îmbinări. În lucrare se prezintă unele aspecte teoretice privind comportarea și calculul îmbinărilor cu placă de capăt și șuruburi de înaltă rezistență parțial pretensionate, la care se adaugă o platbandă sudată la talpa întinsă a grinzii. În cazul îmbinărilor solicitate la întindere, nu se recomandă pretensionarea maximă cum este cazul îmbinărilor solicitate la lunecare, deoarece efortul de întindere din acțiunile utile supraîncarcă șuruburile cu un efort ce se poate determina pe baza rigidităților relative șuruburi-placă de capăt, situație care ar putea conduce la atingerea unor solicitări critice. Pretensionarea parțială realizează un contact corespunzător între plăcile de capăt și micșorează efectul „prying force", care de asemenea solicită suplimentar șuruburile în cazul plăcilor de capăt cu o rigiditate mai redusă la încovoiere transversală. Lucrarea include un exemplu numeric prin care se evidențiază influența rigidității elementelor îmbinării și a platbandei sudate de eclisare asupra eforturilor din îmbinare, cu luarea în considerare a efectului de pretensionare a acesteia rezultat din contracția la răcire a cordoanelor longitudinale de sudură.

Cuvinte cheie: îmbinare grinzi, placă de capăt, șuruburi parțial pretensionate, rigiditate placă-șuruburi

Abstract

The connections with end plates and bolts can be used in steel bridge structures, or joining the decking beams, which can lead to very simple and economical constructive solutions. The paper presents some theoretical aspects concerning the behavior and calculation of the end plate connections which is partially pre-tensioned by high-strength bolts, and supplementary a welded cover plate is added to the tensioned flange of the plate girder. In the case of connections subjected to tension forces, it is not recommended to prestress the bolts as

much as is the case when the joints are subjected only to shear because live loads can lead to the overloading of the bolts; the overload is a function of the relative stiffness of the end plate and of the bolts. The additional welded splice at the tension flange contributes to the transfer of the large tension stresses from the extreme bolts to the welded cover plate, resulting in a significantly reduced joint assembly loading and an increased load-bearing capacity. The paper includes a numerical example highlighting the influence of the stiffness of the joint elements and of the welded splice-plate on the joint loading, considering its pre-tensioning effect from the cooling contraction of the longitudinal welding fillets.

Keywords: joint beams, end plate, partially pre-stressed bolts, plate-bolts stiffness

1. INTRODUCTION

The fundamental objective of the paper is to show that a connection with prestressed bolts under tension from exterior loading, is overloaded even when the load efforts are smaller than the value of the bolt prestressing force. The second objective is to evaluate the additional stress in the bolts, that currently in the civil engineering practice is considered to be zero, i.e., the bolts are not overloaded if the tensile stress from the loads is smaller than the prestressing force.

The originality and the novelty consist in the fact that the technical materials and the design regulations in the civil engineering field do not take into account the overloading of the prestressed bolts resulting from loads produced by the external actions.

In the design practice, the aim is to achieve the simplest joints that can be easily executed at the construction site, to ensure a clear behavior in the structure and to provide safety when subjected to static and dynamical loads. The paper presents a methodology for calculating the joint that offers the possibility of a rapid analysis of the stresses in the joint and the size of the overload of the tensioned bolts from the action of the external loads. We appreciate that the design recommendations presented in the final part of this work can be useful in the design stages of steel structures and steel bridges.

2. CONNECTIONS WITH END PLATES AND PARTIALLY PRESTRESSED BOLTS

In the case of joints with end plates and partially prestressed bolts, the transmission of the bending moment to the beam joining area shall be carried out with the end plate attached to the partially pre-tensioned high-strength bolts and a welded splice-plate of the tensioned flange of the beam. The joint bolts are

mainly subjected to tension and will instantly enter the work when applying a stretching force, so that welded parts can be used simultaneously in the joint. The mechanical model of the joint behavior, with perfectly elastic string-type devices, is shown in Figure 1.a; b [1]; the diagram of the forces that develop in the joint, prior to the separation of the end plates, is transposed in Figure 1.c [2].

The forces in the bolts will change after the prestressing force in the bolt is exceeded by the external tension loading; when the contact surfaces will separate, the subsequent behavior will be identical to that of the non-pre-tensioned bolts. The increase in the stretching force of the connection is compensated by a decrease in the compression stress between the contact plates and, relatively little, by the change of the stresses in the bolts. As noted in Figure 1.c, the application of an external force F_t leads to a significant change in the compression stress in the end plate, with the value ΔF_c, compared to the increase of the tension stresses in bolts, ΔF_b □ ΔF_c .

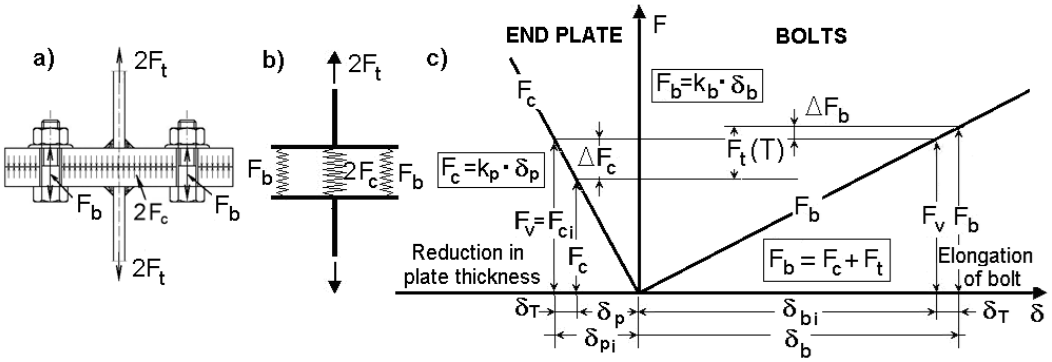


Figure 1. Forces in prestressed fastener

Before the separation of the end plates, we will have the following changes in the stresses in the bolts and in the end plate:

$$\Delta F_b = k_b \cdot \delta_T \text{ and } \Delta F_c = k_p \cdot \delta_T \quad (1)$$

It yields:

$$F_t = \Delta F_b + \Delta F_c = (k_b + k_p) \cdot \delta_T \text{ and } \delta_T = \frac{F_t}{k_b + k_p} \quad (2)$$

The constant stiffness of the connection, K, is defined, which shows the proportions between the load-bearing capacities of the joint elements, i.e. between the tensioned bolts and the compressed end plate:

$$K = \frac{k_b}{k_b + k_p} \quad (3)$$

The resulting forces in the bolts and in the end plates after the external force F_t is applied:

$$F_b = F_v + \Delta F_b = F_v + K \cdot F_t \ , \quad F_c = F_v - \Delta F_c = F_v - (1-K) \cdot F_t \qquad (4.a, b)$$

The separation of the end plate, under conditions of elastic behavior of the joint, occurs when $F_c = 0$, for an external force having the value of:

$$F_t^{sep.} = \frac{F_v}{1-K} \qquad (5)$$

In the diagram in Figure 2 it is assumed that the end plate is sufficiently thick, i.e. a high stiffness at transverse bending, and the bolts are relatively close to the web, so that it does not deform in the transverse direction. For cases of high-thickness end plates, the ratio k_b / k_p has values between 0.05...0.12 and it results $F_t^{sep.} = (1.05...1.12) \cdot F_v$; in the field of steel structures, the values can be much higher, $k_b / k_p = 0.20...0.28$. In the case of plates with low bending stiffness in the transverse direction, the contact between the end plates develops in the marginal areas of the joint due to the transverse deformation of the plate, in this case the mechanical model is shown in Figure 3.a, and the diagram of the relationship between the forces in the joint is showed in Figure 3.b, [1], [2], [3].

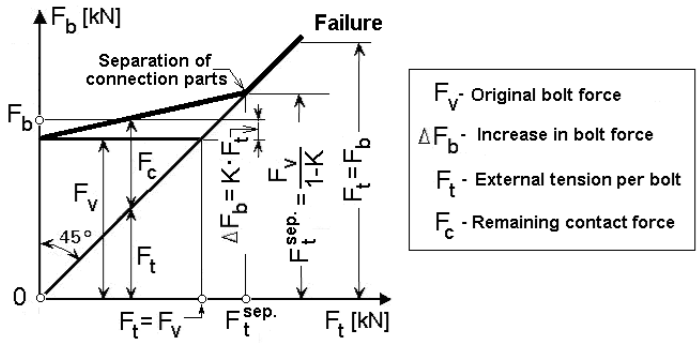


Figure 2. Bolt forces as a function of the applied load

In this case any tension force will increase the bending of the plate and the forces of the bolts by the value $\Delta F_{b.Q} = Q$, Figure 4, [2]. The prying force is directly proportional to the outer tensioning force and can be put in the general form:

$$\Delta F_{b.Q} = Q = K_Q \cdot F_t \qquad (6)$$

Some semi-empirical relationships (7.a), [1] or (7.b), [2], can lead to a rapid assessment of the marginal leverage forces, but sometimes overestimated values are obtained:

$$K_Q = \frac{1}{4}\frac{m}{n}; \quad (\frac{n}{m} \le 1.25); \qquad K_Q = \left(\frac{3m}{8n} - \frac{t^3}{20}\right)_{U.S.units} = \left(\frac{3m}{8n} - \frac{t^3}{328}\right)_{S.I.units} \qquad (7.a, b)$$

If the bolts have a high rigidity (large diameter and small length), the two effects, i.e. overloading the bolts due to the redistribution of the stretching force between the bolts and the plate and the prying forces accumulate.

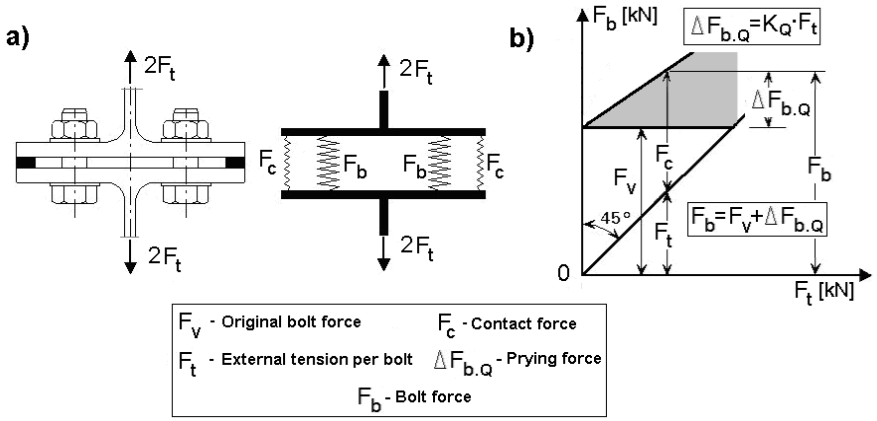


Figure 3. Connection with end plates having small transverse stiffness

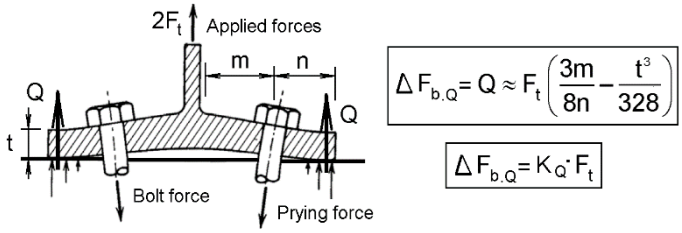


Figure 4. Joint deformation and the prying forces [2]

The relative stiffness of the connection elements

Design rules and technical literature do not provide a relatively simple and practical methodology for assessing the rigidity of the joint parts, so the Shigley's method [4] used in the field of mechanical engineering is proposed for assessing the relative rigidity of the bolts and of the end plate. The stiffness of a bolt is given by the stiffness of those of the two rod zones, with and without thread, Figure 5.a, considered to be two springs linked in series, [4]:

$$\frac{1}{k_b}=\frac{1}{k_1}+\frac{1}{k_2} \quad \Rightarrow k_b=\frac{k_1 k_2}{k_1+k_2}; \quad k_1=\frac{A_s E}{\ell_1};\ k_2=\frac{A_b E}{\ell_2} \tag{8}$$

where: A_b – the gross area of the bolt; A_s – the effective area of the bolt.

The constant of the bolt stiffness k_b is found:

$$k_b=\frac{A_s A_b E}{A_b \ell_1 + A_s \ell_2} \approx \frac{A_b E}{\ell} \tag{9}$$

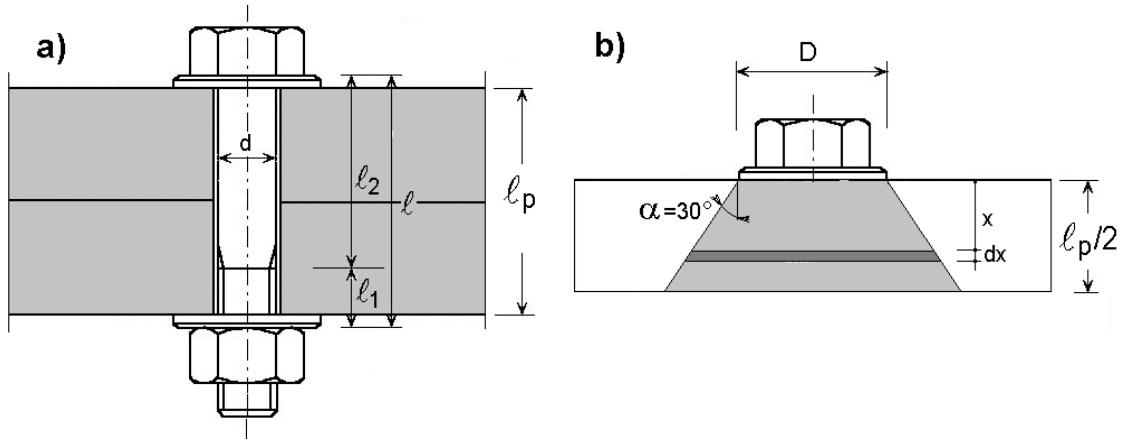


Figure 5. Characteristics for calculating stiffness values [4]

For the determination of the stiffness constant of the plate, one can start from the deformation of the cone frustum on which the pressure resulting from the tightening of the bolt is exerted, Figure 5.b, considering a pressure distribution at an angle $\alpha = 30^o$.

$$d\delta = \frac{Fdx}{EA(x)} \quad \Rightarrow \delta = \frac{F}{\pi E}\int_0^{\ell_p/2}\frac{dx}{f(x)} \tag{10}$$

The stiffness of two springs connected in series is obtained:

$$k_p = \frac{1}{2}\frac{F}{\delta} = \frac{\pi dE \cdot tg\alpha}{2 \cdot \ln[f(tg\,\alpha; t; d; D)]} \tag{11}$$

The end plate stiffness constant can be written as an exponential equation [4]:

$$k_p = 0.787 \cdot d \cdot E \cdot e^{0.628\frac{d}{\ell_p}} \tag{12}$$

3. HYBRID CONNECTIONS WITH END PLATES JOINED WITH PARTIAL PRESTRESSED BOLTS AND A WELDED COVER PLATE

If the bolts placed on the depth of the web do not provide resistance for the connection, and the development of the end plate outside the beam section would affect the constructive solution in the area of the joint (such as the laying of the reinforcement), the solution with additional welded plates to both flanges or a single welded plate on the tension flange of the beam may be chosen, as it is shown in the Figure 6.

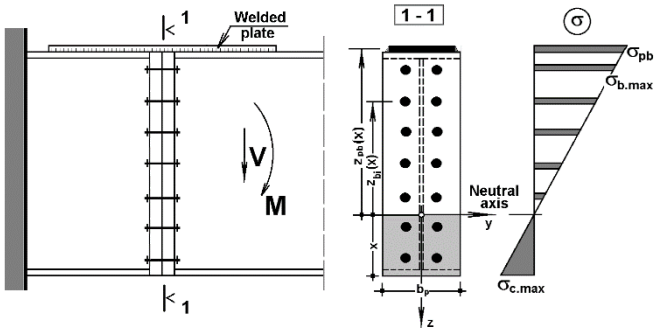


Figure 6. Connection with end plates and a welded cover plate

The position of the neutral axis is obtained by equalizing the static moment of the compressed surface to the static moment of the elongated bolts:

$$\frac{1}{2}b_p x^2 = A_{pb} z_{pb}(x) + 2\sum_{1}^{n_t} A_b z_{bi}(x) \quad \Rightarrow x \tag{13}$$

The inertia moment of the joint section after the neutral axis will be:

$$I_{b.p} = \frac{b_p x^3}{3} + n_{bt} I_b + 2A_b \sum_{1}^{n_t} z_{bi}^2 + \frac{b_{pb} t_{pb}^3}{12} + b_{pb} t_{pb} z_{pb}^2 \tag{14}$$

Knowing the position of the neutral axis, the tension stresses in the cover-plate and the tension forces in the marginal bolts can be calculated:

$$\sigma_{pb} = \frac{M_{Ed}}{I_{b.p}} z_{pb} \text{ (a)}; \quad \sigma_{b1} = \frac{M_{Ed}}{I_{b.p}} z_{b1} \text{ (b)}; \quad N_{b1} = \sigma_{b1} \cdot A_b \text{ (c)}. \tag{15.a;b;c}$$

3.1. Pretension stresses in the cover-splice plate due to contraction of the welding fillets

In the vicinity of the welding fillets, in the so-called thermally influenced zone (ZIT), where the temperature was higher than the value of 550…600^{o}C, at cooling it tends to contract, but free deformation is prevented by areas where the deformations have not exceeded the elasticity limit. For this reason, stretching stresses occur in the vicinity of the welding fillets and compression stresses in the rest of the area, Figure 7. The estimation of the size of the plasticized area in the vicinity of the welding fillets, in which tensile stresses developed at cooling, are equally to the steel yielding stress f_y, can be made with equation (16), [5]:

$$c = \frac{C \times A_w}{f_y \times t} = \eta \times t \tag{16}$$

where: $C = 6000\ N/mm^2$ - coefficient with stress units established on tests basis; A_w – area of the weld fillets cross-section.

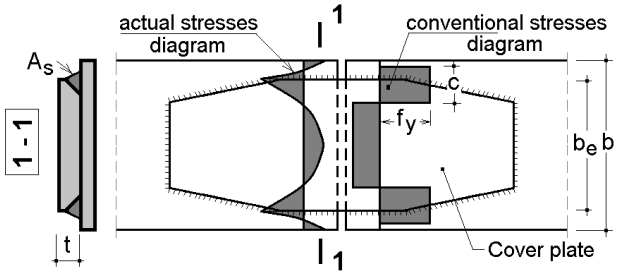


Figure 7. Residual stresses due to contraction of the welded fillets

When the welding fillets and the adjacent plasticized areas are cooling, a compression stresses will be developed in the plate which balances the result of the tensioning stresses with the size, [6]:

$$N_{c.w}=2\,c\,t\times f_y \tag{17}$$

This compression force is reduced in the center of gravity of the joint to a compression force $N_{c.w}$ and to a bending moment $M_w=N_{c.w}\times z_{pb}$, [6]. The reduction of the tension stresses in the bolts can be evaluated with equation (18):

$$\sigma_{c.w}=\frac{N_{c.w}}{A_{eff.}^{conn.}}+\frac{M_w}{I_{b.p}}z_{bi} \tag{18}$$

The effective area of the joint area, $A_{eff.}^{conn.}$ is the sum of the bolt areas in the tension area and the compressed area of the end plate: $A_{eff.}^{conn.}=A_{b.t}+A_{p.c}$.

3.2. Final stresses and forces in the connection elements

Combining the effect of overloading the bolts due to the external tension forces, the effect of the prying force and the effect of contraction when cooling the welding fillets, the state of stresses in the elements of the joint – the cover-splice plate and bolts will be as follows:

- The stresses in the cover-splice plate:

$$\sigma_{pb}=\sigma_{pb}^{M_{Ed}}-(\sigma_{pb}^{N_{c.w}}+\sigma_{pb}^{M_w})=\frac{M_{Ed}}{I_{b.p}}z_{pb}-\left(\frac{N_{c.w}}{A_{eff.}^{conn.}}+\frac{M_w}{I_{b.p}}z_{pb}\right) \tag{19}$$

- The tension forces in the most loaded bolts:

$$F_b=F_v+\Delta F_b+\Delta F_{b.Q}-F_{b.w} \tag{20.a}$$

$$F_b=F_v+(K+K_Q)\times A_b\times\left(\frac{M_{Ed}}{I_{b.p}}z_{b1}\right)-A_b\left(\frac{N_{c.w}}{A_{eff.}^{conn.}}+\frac{M_w}{I_{b.p}}z_{b1}\right) \tag{20.b}$$

4. DESIGN ANALYSIS

This application analyses the state of stresses in the connection between the cross-girder and the stiffening-beam of the deck of a road bridge having the resistance structure on metal tied-arches, with a composite steel-concrete slab. The steel girders are made from steel S355M. The bolts to be used are M 30 - grade 10.9, partially prestressed. The forces for the connection design, bending moment and shear force, compatible with those resulting from the structure of a road bridge with a span of 48.00 m, four lanes of traffic and sidewalk consoles and cycle paths of 4.00 m width, are M_{ed}=1800 kNm and V_{Ed}=1200 kN.

The analysis is performed under the following calculation assumptions: the end plates are flexible (semi-stiff) to rotation; the bolt state of stress is determined for a prestressed force $F_v=0.5F_{p.C}$; the thickness of the end plates has the value of 40 mm and 30 mm. The constructive composition of the deck is shown in Figure 8.

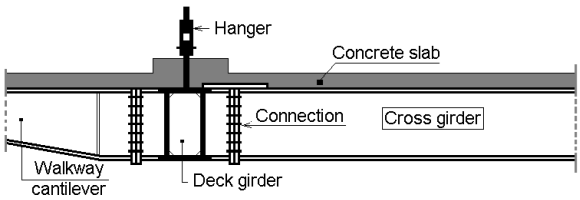


Figure 8. Constructive solution of the deck connection

The constructive composition and the scheme for calculating the connection forces are shown in Figure 9. The position of the neutral axis is evaluated using eq. (13), leading to x=23.5 cm.

The moment of inertia of the connected section about the neutral axis, eq. (14), leads to $I_{b.p}=59.7x10^4$ cm^4.

Average stress in the cover plate, eq. (15.a) is: $\sigma_{pb}=1930$ daN/cm^2<f_y/γ_{M0}

Stresses in the marginal bolts, eq. (15.b) are: $\sigma_{b1}=1643$ daN/cm^2

Tension force in the bolt, eq. (15.c): $N_{b1}=116$ kN<$F_{t.Rd}=404$ kN,

where, [7]: $F_{t.Rd}=\dfrac{0.90\, f_{ub}\times A_s}{\gamma_{M2}}=404$ kN.

Maximum compression stress in the cover plate is:

$$\sigma_p=\frac{M_{Ed}}{I_{b.p}}z_p=551\ \text{daN/cm}^2<<f_y/g_{M0}=3550\ \text{daN/cm}^2$$

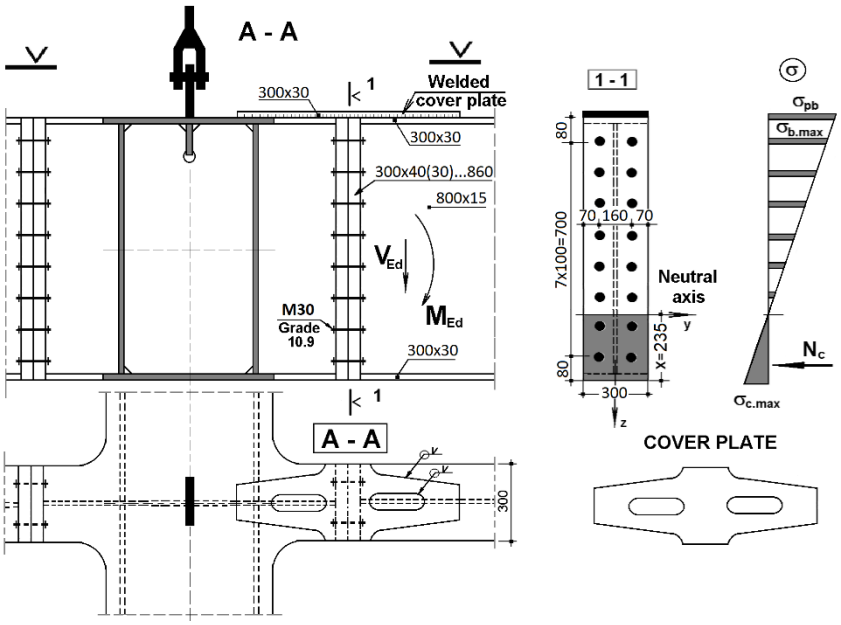


Figure 9. Joint with end plates and a cover-splice plate

Verification at the combined action of tension and shearing in the area of the connection shall be carried out with the interaction equation, [7]:

$$\frac{F_{V.Ed}}{F_{V.Rd}}+\frac{F_{t.Ed}}{1.4\times F_{t.Rd}}=0.47<1, \text{ where: } F_{V.Ed}=\frac{1200}{16}=75\text{ kN};\ F_{t.Ed}=N_{b1}=116\text{ kN}.$$

$$F_{V.Rd}=\frac{\alpha_v\, f_{ub}\, A_b}{\gamma_{M2}}=282.4\text{ kN};\ F_{b.Rd}=\frac{k_1\,\alpha_b\, f_u\, d\, t}{\gamma_{M2}}=612\text{ kN}>F_{V.Rd}>F_{V.Ed}$$

It is analyzed the situation where the bolts M30–grade 10.9 in the joint are prestressed by a force $F_v=0.5\times F_{p.C}\approx 200\text{ kN}$, where: $F_{p.C}=0.7\, f_{ub}\, A_s=392.7\text{ kN}$. The bolts stresses are evaluated with the maximum forces previously obtained: $F_v=200\text{ kN}$ and $F_t=N_{b1}=116\text{ kN}$. The forces in the elements of the joint shall be evaluated for two values of the thickness of the end plate: Case 1: t_p = 40mm; Case 2: t_p = 30mm.

4.1. Case 1: t_p = 40mm

The constant stiffness of the bolt, from eq. (8) is $k_b=0.74\,E\text{ daN/cm}$. The constant stiffness of the plate, eq. (12), $k_p=2.988\times E\text{ daN/cm}$. The constant stiffness of the joint: K=0.20. The forces in the bolts after loading with effort F_t will be, according to eq. (4.a), $F_b=223.2\text{ kN}$.

The plate separation force, eq. (5), $F_t^{sep.}=250\ \text{kN}=1.25F_v$.

The following shall be noted: $F_b=223.2\ \text{kN}\begin{cases}<F_{t.Rd}=404\ \text{kN}\\<F_t^{sep.}=250\ \text{kN}\end{cases}$

It can be concluded that in the extreme bolts of the joint, the design resistance to tension is not exceeded before the separation of the end plates. The results obtained are graphically translated into the diagram in Figure 10.

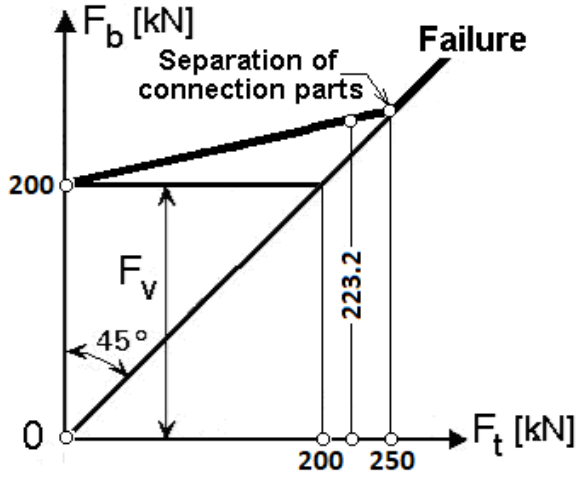


Figure 10. Bolt forces in connection

Effect of the prying force

In this case the prying force effect is reduced in the marginal (extreme) bolts, because the transverse deformation of the end plate is prevented by the cover plate.

If this aspect is neglected, it would result in the prying force, eq. (6):

$$\Delta F_{b.Q}=\left(\frac{3m}{8n}-\frac{t^3}{328}\right)\times F_t=43\ \text{kN};\quad K_Q=0.19$$

Pretension effect due to the contraction of the welding fillets

The size of the plasticized area in the vicinity of the welding fillets, eq. (16) and the resulting forces, eq. (17) are: $c=3.4\ \text{cm}=1.13\times t$; $N_{c.w}=724\ \text{kN}$.

$$F_{b.w}=A_b\left(\frac{N_{c.w}}{A_{eff.}^{conn.}}+\frac{M_w}{I_{b.p}}z_{b1}\right)=37.7\ \text{kN}$$

The total tension force in the bolts: $F_b=F_v+\Delta F_b+\Delta F_{b.Q}-F_{b.w}=228.5\,\text{kN}$

4.2. Case 2: t_p = 30mm

The constant stiffness of the bolt, from eq. (8) is $k_b = 0.93E$ daN/cm. The constant stiffness of the plate, eq. (12), $k_p = 3.23 \times E$ daN/cm. The constant stiffness of the joint: K=0.22. The forces in the bolts after loading with effort F_t will be, according to eq. (4.a), $F_b = 225.5$ kN.

The plate separation force, eq. (5): $F_t^{sep.} = 256.4 \text{ kN} = 1.28 F_v$.

The following shall be noted: $F_b = 225.5 \text{ kN} \begin{cases} < F_{t.Rd} = 404 \text{ kN} \\ < F_t^{sep.} = 256.4 \text{ kN} \end{cases}$

It can be concluded also, that in the extreme bolts in the joint, the design resistance to tension is not exceeded before the separation of the end plates.

Effect of the prying force

In this case the prying force effect is small in the marginal bolts, because the transverse deformation of the end plate is prevented by the cover plate.

If this aspect is neglected, it would result in the prying force:

$$\Delta F_{b.Q} = Q = K_Q \times F_t = \left(\frac{3m}{8n} - \frac{t^3}{328} \right) \times F_t = 69 \text{ kN}; \quad K_Q = 0.306$$

The total tension force in the bolts will be:

$$F_b = F_v + \Delta F_b + \Delta F_{b.Q} - F_{b.w} = 256.8 \text{ kN}$$

The results of the numerical analysis are presented in Table 1.

Analyzing the results obtained in the numerical analysis, the following aspects can be highlighted:

- in case of pretension of the bolts, following the application of the external tension forces, the tension force from pretension of the bolts will be increased as a result of the redistribution of the loading in proportion to the stiffness of the connection parts, with values of approximately 11 % in relation to the pretension effort, representing 20.22% of the external tension force;
- the prying force effect that occurs as a result of the transverse deformation of the end plate increases the pretension effort by 20...34%, representing 37...59% of the external tension force;
- the pretension effect due to the cooling contraction of the welding fillets for joining the cover plate at the tension flange of the girder, reduces the maximum tension force in the bolts by about 19% of the pretension force.

In the works [10], [11] and [12] other aspects regarding the behavior of the connections with end plates bolted and a supplementary welded cover-splice plate on the tensioned flange of the cross-girder are presented.

Table 1. Centralized parameters

Design parameters	**Case 1** t_p **= 40 mm**	**Case 2** t_p **= 30 mm**
Maximum tensile force in non-prestressed bolts N_{bl} [kN]	116	
Interaction Shear and Bending $\frac{F_{V.Ed}}{F_{V.Rd}} + \frac{F_{t.Ed}}{1.4 \times F_{t.Rd}}$	0.47	
Bolt stiffness k_b	0.74·E	0.93·E
End plate stiffness k_p	2.988·E	3.23·E
Coeficient of connection stiffness K	0.20	0.22
ΔF_b [kN]	23.2	25.5
$\Delta F_{b.Q}$ [kN]	43.0	69.0
$\Delta F_{b.w}$ [kN]	-37.7	-37.7
Total maximum tensile force in overloaded bolts F_b [kN]	228.5	256.8

5. PRACTICAL APPLICATIONS OF THE THEORETICAL ISSUES IN STRUCTURES

5.1. Bridge over Someș River in Cluj Napoca

Figure 11 [8] presents a steel bridge on arches over the Someş River in the municipality of Cluj-Napoca, for which the Feasibility Study was drawn up, a bridge where the solution of connection between the cross-girders and the tie-beams by end plates and cover-splice plate at the tensioned upper flange was used. The superstructure of the bridge consists of a composite steel-concrete deck and two steel box arches, with a variable depth section. The span of the deck is of 46.00 m, the distance between the axes of the stiffened-beams and the axes of the arches is 9.00 m, and the total width of the deck is of 18.00 m.



Figure 11. Bridge over Someș River in Cluj Napoca. Rendering after the Feasibility Study [8]

5.2. Bridge over Cibin River in Sibiu

Figure 12 shows the bridge over the Cibin River in the municipality of Sibiu, in operation now, [8] and [9], the bridge where the solution of joining the cross-girders with stiffening-girders with bolted end plates and a welded cover-splice plate at the tensioned flange was also used. The bridge has a single span of 34.00 m with a total length of 38.50 m, 35.30 m is the span of the superstructure, and the clearance of the bridge is 32.90 m. The bridge has a roadway with four lanes to ensure all traffic relations at intersections and sidewalks to ensure pedestrian and bicycle crossing of the river.



Figure 12. Bridge over Cibin River in Sibiu City [8]

The new bridge superstructure is made of four steel arches, two on each side. The arches near the roadway are vertical and connected at the bottom by means of two main tie-beams. The marginal arches are inclined and have the main role in ensuring the stability of the vertical arches. The arches span is of 34.0 m, the rise is about 8.10 m, and are made of circular pipe with the diameter Φ508 mm and a wall thickness of 40 mm. The steel quality is S355M. The secondary arches that are inclined, are designed with a cross-section of circular pipe with the

diameter Φ508 mm and the wall thickness of 20 mm and are linked to the main vertical arches through 7 horizontal bars made of Φ406.4 mm pipes. The arch curvature was achieved by hot bending of the sections, using the phenomenon of electromagnetic induction, at a factory from the field of extractive industry. The horizontal ties of the arches are the main longitudinal girders of the deck bridge that have a composite cross-section. The steel part is made of hot rolled members HE 1000B, made of five sections, connected using SIRP. The vertical hangers ensure the connection between the deck and the arches; there are 14 vertical steel hangers with a circular Φ70 mm cross-section made of S460M on each side of the bridge. On the sidewalks area, the crossbeams are continued with steel cantilevers.

6. CONCLUSIONS AND RECOMMENDATIONS

Connections with end plates and bolts can be used in case of steel structures which can lead to very simple and economical solutions, provided that the mechanical behavior of these joints is properly modeled. To use this solution, it is necessary that the depth of the beam section is large enough to allow the arrangement of the necessary number of bolts to take up the joint loading.

As stated before, in civil engineering design regulations, the overloading of the prestressed bolts produced by external actions is not considered. The recommendation resulting from the current paper is that for important joints under tension (of the flange type), a verification should be made to take into account the overloading of the bolts.

Pretension of end plate fastening bolts increases the bolt forces and therefore generally it is not recommended for such joints. To ensure a perfect contact between the end plates we recommend initial prestressing of the bolts at a force of 20…50% of the pretension force, $F_{p.C}$. The design rules and technical literature do not provide a relatively simple and practical methodology for assessing the stiffness of end plates. The connection calculation methodology presented in the paper provides the possibility of a quick analysis of the safety of the joint. The effect of the prying force is important in case of relatively small thickness end plates, when it is possible to develop transverse deformations that lead to the appearance of forces that require additional bolts from the joint. The pretension effect due to the cooling contraction of the welding fillets is in this case favorable, but the developed force having a relatively low value, and which cannot be accurately assessed, is not considered when calculating the joint.

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