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EFFICIENT CONNECTION OF CROSS GIRDERS OF ARCH STEEL ROAD BRIDGES

BY

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Abstract. In steel bridges and constructions, the currently used assembling is made with common bolts and with connections with prestressed high strength bolts, to which prestressing forces are inserted in the rods after being mounted. This paper presents the mechanical behaviour of the connections made with flange-type end plates made with common bolts subjected to tension, respectively with high strength prestressed bolts, considering that the end plates have a very high stiffness or behave as flexible (semi-stiff) to rotation plates, when rotated in the vertical plane of the connection. The present paper presents briefly the arch steel bridge crossing the Someş river in the city of Cluj-Napoca, for which a feasibility study was made and where the cross girders were connected to the girder ties with end plates and cover plates welded to the tension upper flange. The calculation is performed for four constructive solutions and calculation hypotheses. It was found out that the constructive solution which is a hybrid connection of end plates joined together with non-preloaded bolts and a welded additional cover plate over the joint area in the elongated space of the flange can lead to a more uniform distribution of the stresses in the connection

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parts, the cover plate takes over a much larger stress in this case and the stresses in the bolts decrease significantly. The final part of the paper presents several conclusions and recommendations which are regarded by the authors are extremely useful in the design activity.

Keywords: steel bridges; floor girder connection; end plates; common and prestressed bolts; Eurocodes; design methodologies; calculus analysis; design recommendations.

1. Introduction

A quite accurate calculus for a connection subjected to flexure, axial force and shearing force, that was verified through experimental data, accepts that the neutral axis, respectively the gripping rotation line, in general coincides with the compressed flange load centre, Fig. 1, on condition that a stiff grip occurs with the strong end plates.

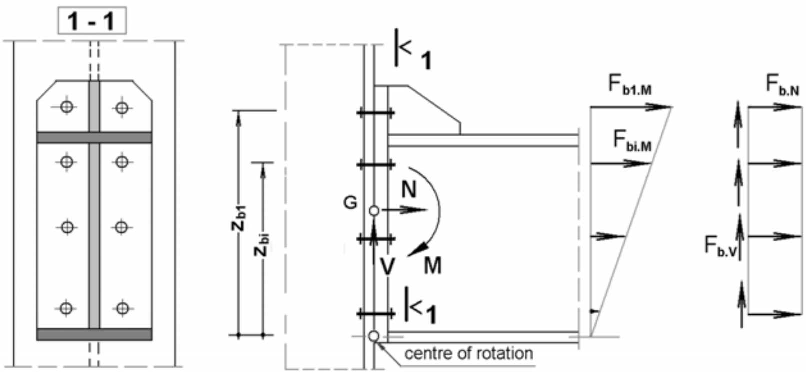


Fig. 1 – Connection with end plate and bolts.

The relationship below (1) is reached when there is an equilibrium condition between the external moment and the connection stresses:

$$M = m\sum F_{bi.M} z_{bi} \,, \quad (m \text{ – number of vertical bolt rows}) \qquad (1)$$

The tensile (stresses) forces in the most tensed and farthest bolts from the connection rotation axis can be calculated with relationship (2):

$$F_{b1.M} = M\frac{z_{b1}}{m\sum z_{bi}^2}. \tag{2}$$

It is obvious that there is a uniform distribution in the connection bolts of the axial force and the shearing force:

$$F_{b.N} = \frac{N}{n}; \qquad F_{b.V} = \frac{V}{n}, \quad (n - \text{total number of bolts}) \tag{3}$$

In order to calculate the total tension force (4), an addition of the maximum tension forces coming from the bending moment and the axial force (with their respective sign) is necessary:

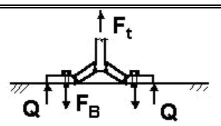
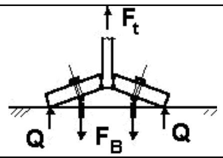
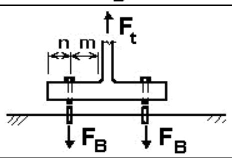
$$F_t = F_{b1.M} + F_N = M\frac{z_{b1}}{m\sum z_{bi}^2} + \frac{N}{n}. \tag{4}$$

2. The Stress State in Prestressed (Preloaded) Bolts

The transmission of the bending moment occurs through end plates that should have a size that corresponds to the stresses received.

Function of the ratio between the flange bending capacity and the connection bolt resistance to tension the situations summarised in Table 1, (European Steel Design Education Programme. ESDEP) can be met.

Table 1
Failure Modes for Bolted T-stub Connection

Case	Failure mode	Forces relation	Observations
1		$F_t = \frac{4M_{p\ell}}{m}$	Thin end plate – strong bolts Complete flange yielding
2		$F_t = \frac{2M_{p\ell} + n\sum F_B}{m+n}$	Bolt failure with flange yielding
3		$F_t = \sum F_B$	Thick end plate – weak bolts
F_B – bolt design tension resistance; $M_{p\ell} = \frac{1}{4}\ell t_f^2 f_y$; $\ell = \ell_{\text{eff}}$ (EC3-1-8)			

In steel bridges and constructions, prestressed high strength bolts are currently used and after mounting, tensile stresses are inserted in their rods.

By strongly tightening the nut, a significant tensile stress is inserted in the bolt rod of about 70,...,75% from the bolt material yield point and pressure is exerted in an area around the bolt.

Because of this, the parts joined together are tightened very strongly, so that under the action of a normal stress to the bolt rod, the relative displacement of the parts is prevented by the friction forces produced on the contact surfaces.

Consequently, different from usual bolt connections, the connections with high strength bolts subjected to external stresses will withstand by means of the friction among the parts. The magnitude of the friction forces depends upon the pressures exerted upon the parts, and implicitly upon the preload force ($F_{p.C}$) in the bolt rod and the friction coefficient.

The prestressed high strength connections exhibit an elastic behaviour, with small deformations kept up to high values of stresses.

In the case of perfectly stiff end plates, when prestressed with a force F_v, this force will initially be in equilibrium with the compression force exerted at the contact surface, F_c. The two flanges will act as a monolith part as long as the external force F_t is smaller than the prestressing force F_v.

When an external force is applied, the stresses in the bolts will modify very little (it is supposed that flanges present a high stiffness because of the large area, by comparison with the bolt area), Fig. 2, (Kullak *et al.*, 2001).

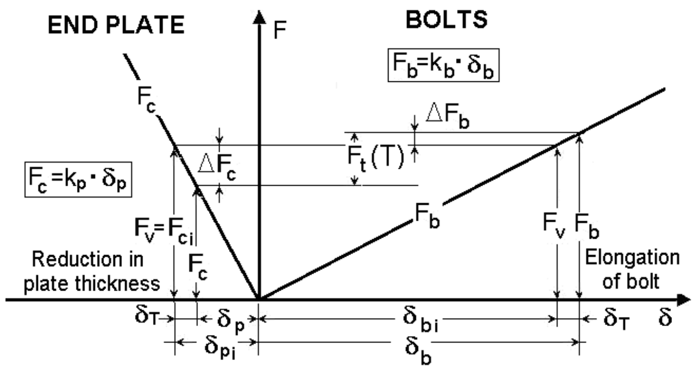


Fig. 2 – Forces in prestressed fastener before end plates separate.

One can see that an increase in the tensile force in the connection is compensated by a decrease of the compressive force between the contact plates and, to a very small extent, by the change of the stresses in the bolts. The situation is even more favourable when bolts are more elastic, relative to the connection parts, so that using longer bolts or inserting washers (possibly spring type), produces an increase in bolt elasticity and for this purpose plates need a larger thickness to get a high bending stiffness.

As seen in Fig. 2, the application of an external force F_t leads to a significant change of the compressive stress in the end plate, of value ΔF_c, compared to the increase of the tensile stress in the bolts – ΔF_b.

In the condition of an elastic behaviour of the connection, the plate is separated when $F_c = 0$, for an external force of magnitude:

$$F_t^{\text{sep.}} = F_v\left(1+\frac{k_b}{k_p}\right). \tag{5}$$

In practical design cases, ratio k_b/k_p ranges in value between 0.05,..., 0.12 and it results:

$$F_t^{\text{sep.}} = (1.05\ldots1.12)F_v. \tag{6}$$

If the plates are stiff, after the separation of the end plates, the stress in the bolts will be equal to the external tension force F_t.

For the connection to behave properly especially when the connection is subjected to fatigue, it is recommended to have a bolt pretension stress larger than the tensile stress from external forces.

The stress in the bolts will, however, change very rapidly after the pretension stress is overcome by the external tensile stress when the contact surfaces will separate and the further behaviour will be identical to that of non pretensioned (non-preloaded) bolts.

At the same time, the position of the parts contact area should be taken into consideration; this aspect is shown in Fig. 3, (European Steel Design Education Programme. ESDEP), where the end plate thickness is much diminished as compared to previous cases.

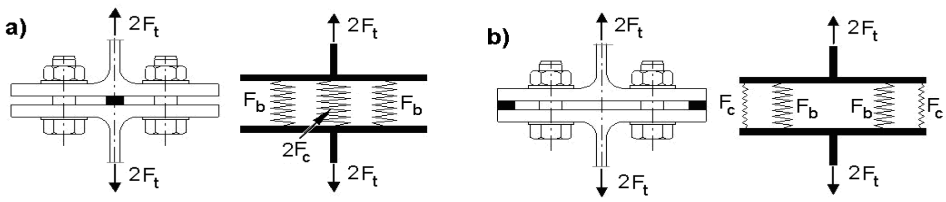


Fig. 3 – Parts of contact area.

When the contact is made in the middle, Fig. 3 *a*, the situation is that of a stiff plate in transverse direction. When the parts contact is performed externally, Fig. 3 *b*, we have the case of flexible end plates in transverse direction. In this latter case, an increase in the external tensile force will modify the stress in the bolts with about the same magnitude.

At the same time, the elongation (deformation) of the bolt increases and the contraction of the connection parts decreases; as a result, the force in the entire connection is reduced.

Actually, the stiffness of the connected package is approximately 4,...,12 times larger than the stiffness of the pretensioned bolt.

The relationship of the forces is illustrated through the forces diagram, Fig. 4, (European Steel Design Education Programme. ESDEP; Moga *et al.*, 2018).

In Figure 4.a, the contact is restrained to the central area of the connection. In this case, the force in the bolt does not increase up to the relative displacement of the parts.

In Fig. 4 *b*, the contact takes place in the marginal areas of the connection. In this situation, any tensile force will augment the plate bending and will also increase both, F_c, and the stress in the bolt by amount $\Delta F_b = K_Q F_t$.

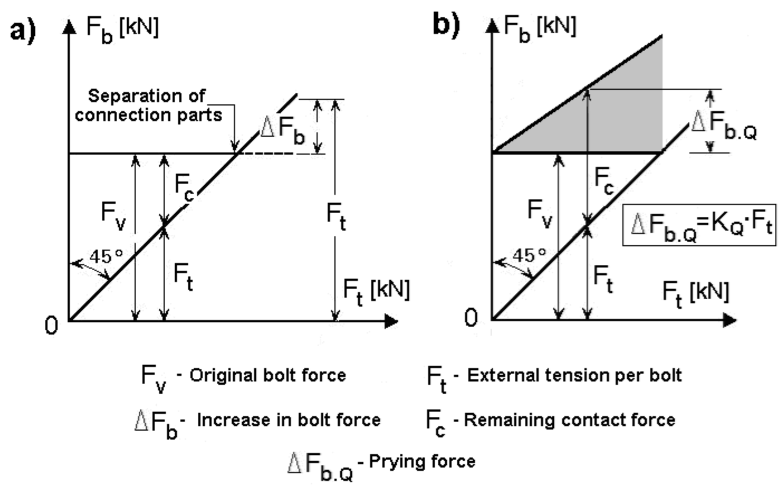


Fig. 4 – Effect of the location of the contact pressure on the bolt force.

The increase of the stress in the bolts is explained by the phenomenon ”prying action”, with lateral lever forces development (”prying forces”), due to the transverse deformation of the flexible end plate Fig. 5, (Kullak *et al.*, 2001).

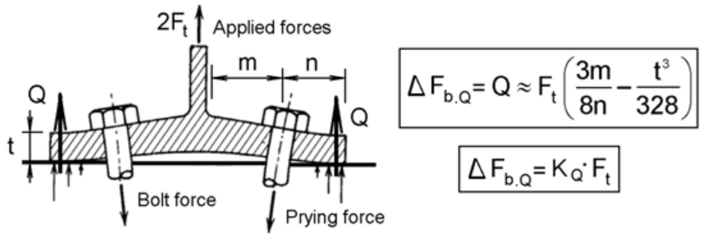


Fig. 5 – Prying action.

The percentage of force transmission depends upon the connection elastic behaviour, and, in general, a) mode is important, through contact in the bolt area.

3. Mixed (Hybrid) End Plate and Cover Plate Connections

In the design practice, the connections aimed at should be as simple as possible, ready to be built easily on the building site and to provide a clear behaviour in the structure as well as safety in service at static and dynamic loads. When the bolts placed on the girder depth do not provide for the strength of the connection and the development of the end plate beyond the girder cross section could affect the constructive solution in the connection area (such as the placement of the reinforcements in the slab), one can opt for a solution in which additional cover plates welded to both flanges or one only cover plate welded to the tension girder flange is chosen, Fig. 6.

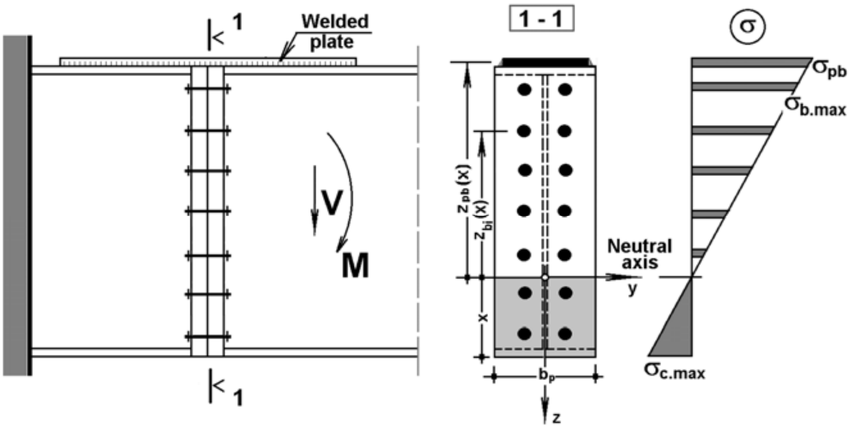


Fig. 6 – Connection made with an end plate and a cover plate welded to the tension flange.

To calculate the connection for the bending moment, the hypothesis of the rotation versus the compressed flange axis does not correspond to the actual behaviour of the connection because the plate from tension flange, whose section area is larger than the area of a pair of bolts, prevents the end plates to separate and a more precise calculus is to be performed by finding first the position of the neutral axis situated on the depth of the end plate and yielding a compressed surface whose size depends on the strength of the cover plate.

The position of the neuter axis is found by making the static moment of the compressed surface equal to the static moment of the tensile members:

$$\frac{1}{2}b_p x^2 = A_{pb} z_{pb}(x) + 2\sum_{1}^{n_t} A_b z_{bi}(x) \quad \Rightarrow x\,. \tag{7}$$

Knowing the position of the neutral axis, one can calculate the unit stresses with the Navier equation:

$$\sigma_{bi} = \frac{M_{\mathrm{Ed}}}{I_{b.p}} z_{b.i}\,. \tag{8}$$

The inertia moment of the connection section with respect to the neutral axis is:

$$I_{b.p} = \frac{b_p x^3}{3} + n_{bt} I_b + 2A_b \sum_{1}^{n_t} z_{bi}^2 + \frac{b_{pb} t_{pb}^3}{12} + b_{pb} t_{pb} z_{pb}^2\,. \tag{9}$$

Verification of the connection elements:

In accordance with EC3-1-8, (DSC to Eurocode 3, 2003), the bolt connections subjected to tension are calculated in one of the following ways:

- Not prestressed D grade bolts

In this category, one uses bolts belonging to quality grades 4.6 up to 10.9 inclusively. Bolts are not necessarily prestressed.

This category should not be applied in the case of connections that are frequently subjected to variations of the tensile force.

In the area of the joint between the end plates, bolts are verified through the relationship of interaction between the moment and shearing force, in accordance with EC3-1-8, Table 3.4:

$$\frac{F_{v.Ed}}{F_{v.\mathrm{Rd}}}+\frac{F_{t.\mathrm{Ed}}}{1.4F_{t.\mathrm{Rd}}}\leq 1\,, \tag{10}$$

where:

$$F_{V.\mathrm{Ed}}=\frac{V_{\mathrm{Ed}}}{n_b}\,;F_{t.\mathrm{Ed}}=\sigma_{b.\max}\cdot A_b\,;F_{V.\mathrm{Rd}}=\frac{\alpha_v\,f_{ub}\,A_b}{\gamma_{M2}}\,;\;F_{t.\mathrm{Rd}}=\frac{0.90\,f_{ub}\,A_s}{\gamma_{M2}}\,.$$

- Prestressed E grade bolts

In this category, one includes bolts from quality grades 8.8 and 10.9, with controlled tightening, according to the Reference Standards: Group 7.

The calculus for verification in these connections is presented in EC3-1-8, (DSC to Eurocode3, 2003).

The theoretical resistance to sliding of a prestressed bolt from grade 8.8 or 10.9 is calculated with:

$$F_{s.\mathrm{Rd}}=\frac{k_s\,n\,\mu}{\gamma_{M3}}F_{p.C}\,, \tag{11}$$

where: k_s is the function of the hole type; n – number of friction surfaces; μ – friction coefficient.

The calculated prestressing force is assessed with relationship:

$$F_{p.C}=0.7\,f_{ub}\,A_s\,. \tag{12}$$

The theoretical shearing force in the limit state of normal service should not exceed the calculus resistance to sliding.

The ultimate theoretical shearing force should not also exceed the bearing pressure resistance to the hole.

For C grade connections, the theoretical resistance to sliding of a bolt is found with relationship:

$$F_{s.\mathrm{Rd}} = \frac{k_s\, n\, \mu}{\gamma_{M3}} (F_{p.C} - 0.8 F_{t.\mathrm{Ed}}) . \tag{13}$$

4. Applicative Examples

The present application makes an analysis of the state of stresses from the connection of a cross girders to the deck cross-tie of a road bridge with the structure on steel arches, steel concrete deck made up from two cross-ties, cover plates and cantilevers for the pavement, a concrete slab acting together with the deck flooring.

Fig. 7, (XC PROJECT – Cluj-Napoca) presents the arch steel bridge crossing the Someş River in the city of Cluj-Napoca, for which a Feasibility Study was made, where the cross girders were connected to the girder ties with end plates and cover plates over the tension upper flange.

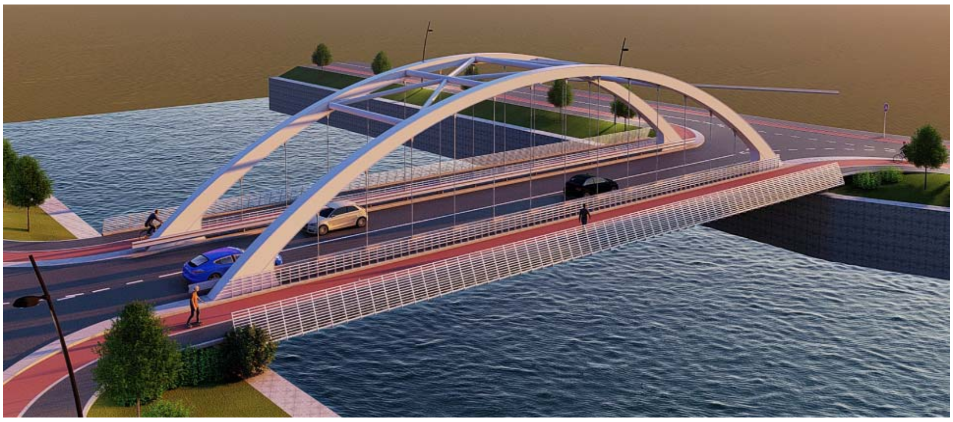


Fig. 7 – Rendered after the Feasibility study.
Bridge crossing Someş river in Cluj-Napoca (XC PROJECT– Cluj-Napoca).

The bridge superstructure is made up from a mixed steel and concrete deck on two steel arches in caisson section, with a variable height section in the vertical plane.

The projected span is 47.00 m, the abutment clearance is 46.00 m, the distance between the axes of the longitudinal girders and the arch axes is 9.00 m, and the overall width of the deck is 18.00 m.

The deck has a carriage way of 7.80 m width, with two 3.50 m lanes and safety spaces of 2 × 0.40 m; laterally, on 4.50 m supports there are 1.50 m pedestrian sidewalks and 2.00 m wide bicycle lanes.

The strength bridge members – arches and girders – are briefly presented below.

Arches are designed with a variable caisson section comprised between 800 × 1,000 mm at the supports and 800 × 700 mm at the vault. The caisson is stiffened inside with longitudinal stiffeners and transverse diaphragms among the walls.

The arch sag measured between the longitudinal girder axis and the vault arch axis is 10.00 m.

Arches are formed of three portions, the mounting connections being performed with prestressed high strength bolts and with welding, the result being a fully closed and tight section, where the internal corrosion is practically negligible, due to lack of air and humidity.

To provide the horizontal plane stability (in transverse direction) connecting elements are supplied, made from circular pipes placed out of the traffic size.

The deck plates are formed of lane steel girders and the reinforced concrete slab is covered by asphalt pavement.

The lane girders are made up from:

– two longitudinal girders with a half open caisson section (the lower flange having voids), of 700 × 800 mm section, which also work as arch cross-bar;

– cross girders, in double T section, with a variable height to provide also the 2% transverse slope of the track contained between 800 and 900 mm.

– the end cross girders – calculated so as to allow the raising of the bridge flooring to replace support devices and maintain the end zone of the flooring;

– the side pavement supports – made with double T variable section, supporting the way for the sidewalks and bicycle lanes.

The metal girders are made from steel quality S355M: f_y = 355 N/mm^2; f_u = 470 N/mm^2.

The bolts to be used are M 27 – grade 10.9, with the calculated characteristics: f_{yb} = 900 N/mm^2; f_{ub} = 1,000 N/mm^2; A_s = 4.59 cm^2; A_b = 6.16 cm^2.

The calculation stresses in the connection, bending moment and shearing stress compatible with those for this road bridge are compatible with the computer-based calculation giving: M_{Ed} = 1,400 kN.m; M_{Ed} = 1,050 kN.

The calculus is developed in the following hypotheses:

– Case 1 – connection with perfectly stiff to rotation end plates placed between the flanges (along the web depth) and with no additional cover plates;

– Case 2 – connection with flexible (semi-stiff) to rotation end plates placed between the flanges (along the web depth) and with no additional cover plates;

– Case 3 – connection with perfectly stiff to rotation end plates and with an additional cover plate over the joint area at the tension flange of the cross girder;

– Case 4 – connection with flexible (semi-stiff) to rotation end plates and with an additional cover plate over the joint area at the tension flange of the cross girder;

For the numerical example, paper (Moga *et al.*, 2018) was used.

The constructive scheme of the connection is presented in Fig. 8.

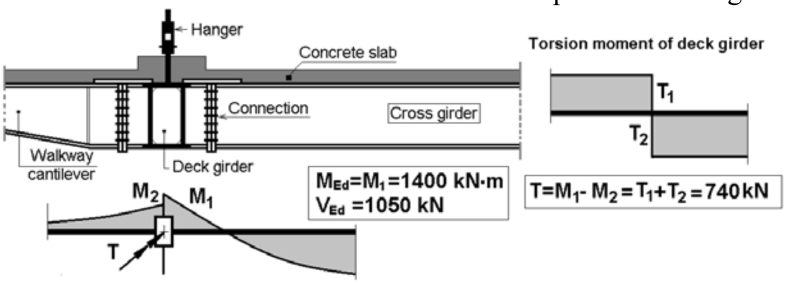


Fig. 8 – The constructive scheme of the connection and calculated forces.

Case 1 – Connection with perfectly stiff to rotation end plates and with no additional cover plates

The constructive scheme of the connection and the stress assessment diagram is given in Fig. 9 *a*.

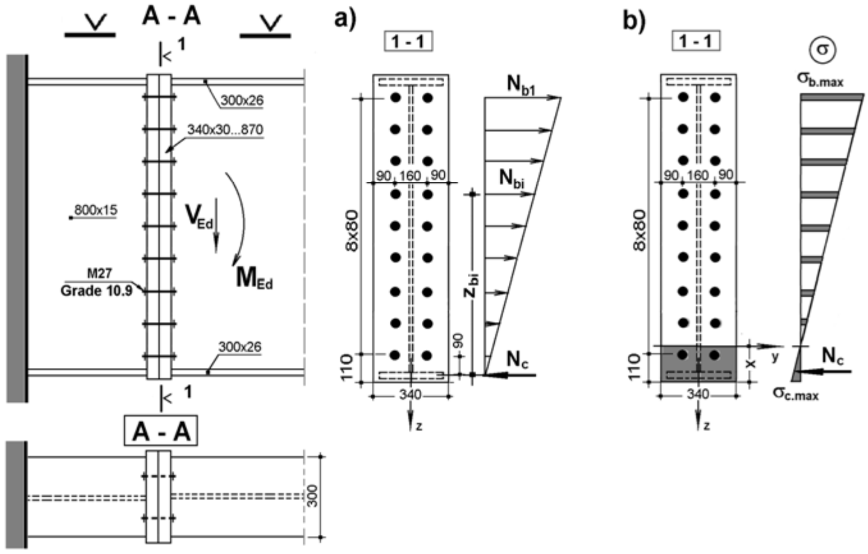


Fig. 9 – Connection with end plates and no additional cover plates.

$$\sum z_i^2 = 9^2 + 17^2 + 25^2 + 33^2 + 41^2 + 49^2 + 57^2 + 65^2 + 73^2 = 18{,}969\,\text{cm}^2. \quad (14)$$

The axial forces in the marginal bolts:

$$N_{b1} = \frac{M_{\text{Ed}}}{2\sum z_{bi}^2} z_{b1} = \frac{1{,}400\times10^4}{2\times18{,}969} 73\times10^{-2} = 269.4\,\text{kN} < F_{t.\text{Rd}} = 330.5\,\text{kN}\,, \quad (15)$$

$$F_{t.\text{Rd}} = \frac{0.90 f_{ub} A_s}{\gamma_{M2}} = \frac{0.90\times10{,}000\times4.59}{1.25}\cdot10^{-2} = 330.5\,\text{kN}. \quad (16)$$

The verification for the mixed tension and shearing stress in the joint area is made with the interaction relationship:

$$\frac{F_{v.\text{Ed}}}{F_{v.\text{Rd}}} + \frac{F_{t.\text{Ed}}}{1.4F_{t.\text{Rd}}} \le 1;\ \frac{58.3}{246.4} + \frac{269.4}{1.4\times330.5} = 0.82 < 1, \quad (17)$$

where: $F_{V.\text{Ed}} = V_{\text{Ed}}/n_b = 1{,}050/18 = 58.3\,\text{kN}$; $F_{t.\text{Ed}} = N_{b1} = 269.4\,\text{kN}$;

$$F_{V.\text{Rd}} = \frac{\alpha_v f_{ub} A_b}{\gamma_{M2}} = \frac{0.5\times10{,}000\times6.16}{1.25}\cdot10^{-2} = 246.4\,\text{kN}.$$

For the pressure on the hole, the bearing bolt force is (EC 3-1-8):

$$F_{b.\mathrm{Rd}} = \frac{k_1\,\alpha_b\,f_u\,d\,t}{\gamma_{M2}} = \frac{2.5\times 0.7\times 4{,}700\times 2.8\times 3}{1.25}\cdot 10^{-2} = 552.7\,\mathrm{kN} > \\ > F_{V.\mathrm{Rd}} > F_{V.\mathrm{Ed}}. \tag{18}$$

Case 2 – Connection with flexible (semi-stiff) to rotation end plates and with no additional cover plates

The constructive scheme of the connection and the stress assessment diagram is given in Figure 9.b.

With the help of equation (7), one obtains the position of the neutral axis, respectively $x=z_p=13.86$ cm.

The inertia moment:

$$I_{b.p} = \frac{b_p x^3}{3} + n_{bt} I_b + 2A_b \sum_1^{n_t} z_{bi}^2 = \frac{34\cdot 13.86^3}{3} + 16\cdot\frac{\pi 2.7^4}{64} + \\ +2\times 6.16\left(5.14^2 + 13.14^2 + 21.14^2 + 29.8^2 + 37.14^2 + 45.14^2 + \right. \\ \left. +53.14^2 + 61.14^2\right) = 172{,}057\,\mathrm{cm}^4. \tag{19}$$

The stresses in the upper bolts:

$$\sigma_{b1} = \frac{M_{\mathrm{Ed}}}{I_{b.p}} z_{b1} = \frac{1{,}400\times 10^4}{17.21\times 10^4}\cdot 61.14 = 4{,}974\,\mathrm{daN/cm^2}. \tag{20}$$

The tension force in the bolt:

$$N_{b1} = \sigma_{b1} A_b = 4{,}974\times 6.16\times 10^{-2} = 306.4\,\mathrm{kN} < F_{t.\mathrm{Rd}} = 330.5\,\mathrm{kN}, \tag{21}$$

where:

$$F_{t.\mathrm{Rd}} = \frac{0.90\,f_{ub} A_s}{\gamma_{M2}} = \frac{0.90\times 10{,}000\times 4.59}{1.25}\cdot 10^{-2} = 330.5\,\mathrm{kN}. \tag{22}$$

The maximum compressive stress in the end plate:

$$\sigma_p = \frac{M_{\mathrm{Ed}}}{I_{b.p}} z_p = \frac{1{,}400\times 10^4}{17.21\times 10^4} 13.86 = 1{,}127\,\mathrm{daN/cm^2} < f_y/\gamma_{M0} = 3{,}550\,\mathrm{daN/cm^2}. \tag{23}$$

The verification for the mixed tension and shearing stress in the joint area is made with the interaction relationship:

$$\frac{F_{v.\mathrm{Ed}}}{F_{v.\mathrm{Rd}}}+\frac{F_{t.\mathrm{Ed}}}{1.4F_{t.\mathrm{Rd}}}\le 1; \qquad \frac{58.3}{246.4}+\frac{306.4}{1.4\cdot 330.5}=0.90<1, \tag{24}$$

where: $F_{V.\mathrm{Ed}}$ = 58.3 kN; $F_{t.\mathrm{Ed}} = N_{b1}$ = 306.4 kN; $F_{V.\mathrm{Rd}}$ = 246.4 kN; $F_{b.\mathrm{Rd}}$ = = 552.7 kN > $F_{V.\mathrm{Rd}} > F_{V.\mathrm{Ed}}$.

Case 3 – Connection with perfectly stiff to rotation end plates and with an additional cover plate over the joint area at the tension flange

The constructive scheme of the connection and the stress assessment diagram is given in Fig. 10 *a*.

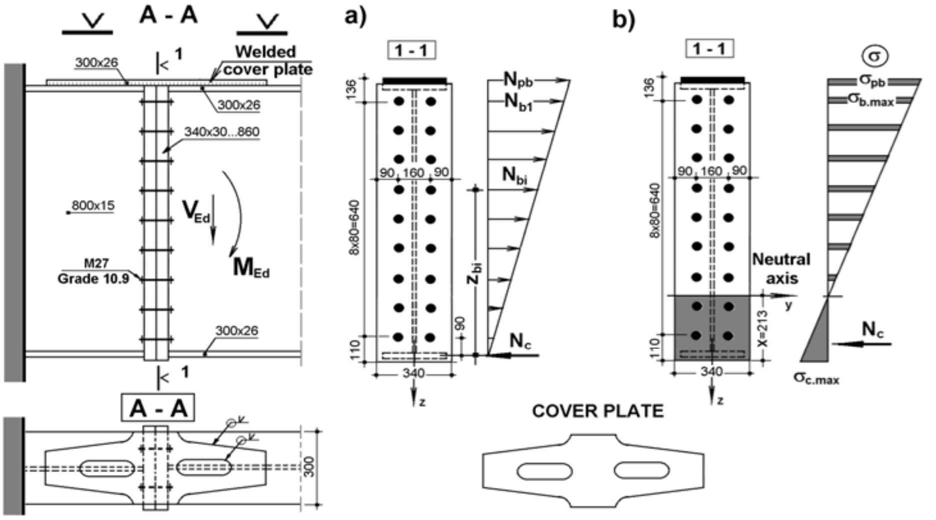


Fig.10 – Connection with end plates and a cover plate.

$$\sum z_i^2 = 9^2+17^2+25^2+33^2+41^2+49^2+57^2+65^2+73^2+85.3^2 = \\ =26{,}245\,\mathrm{cm}^2; \tag{25}$$

$$N_{pb}=\frac{M_{\mathrm{Ed}}}{\sum z_{bi}^2}z_{pb}=\frac{1{,}400\times10^4}{26{,}245}\cdot 85.3\times10^{-2}=455\,\mathrm{kN}. \tag{26}$$

The axial stress in the cover plate will be:

$$\sigma_{pb}=\frac{N_{pb}}{A_{pb}}=\frac{455\times10^2}{30\times2.6}=583\,\mathrm{daN/cm^2} << f_y/\gamma_{\mathrm{M0}}=3{,}550\,\mathrm{daN/cm^2}. \tag{27}$$

The forces in the marginal bolts:

$$N_{b1} = 0.5\frac{M_{\mathrm{Ed}}}{\sum z_i^2} z_{b1} = 0.5 \cdot \frac{1{,}400\times10^4}{26{,}245} \cdot 73 = 194.7\,\mathrm{kN} < F_{t.\mathrm{Rd}} = 330.5\,\mathrm{kN}\,, \quad (28)$$

where:

$$F_{t.\mathrm{Rd}} = \frac{0.90 f_{ub} A_s}{\gamma_{\mathrm{M2}}} = \frac{0.90\times10{,}000\times4.59}{1.25} \cdot 10^{-2} = 330.5\,\mathrm{kN}\,.$$

The verification for the mixed tension and shearing stress in the joint area is made with the interaction relationship:

$$\frac{F_{v.\mathrm{Ed}}}{F_{v.\mathrm{Rd}}} + \frac{F_{t.\mathrm{Ed}}}{1.4 F_{t.\mathrm{Rd}}} \le 1; \qquad \frac{58.3}{246.4} + \frac{194.7}{1.4\times330.5} = 0.66 < 1, \quad (29)$$

where: $F_{V.\mathrm{Ed}}$ = 58.3 kN; $F_{t.\mathrm{Ed}} = N_{b1}$ = 194.7 kN; $F_{V.\mathrm{Rd}}$ = 246.4 kN; $F_{b.\mathrm{Rd}}$ = 552.7 kN > $F_{V.\mathrm{Rd}} > F_{V.\mathrm{Ed}}$.

Case 4 – Connection with flexible to rotation end plates and with an additional cover plate over the joint area at the tension flange

The constructive scheme of the connection and the stress assessment diagram is given in Fig. 10 *b*.

With the help of equation (7), one obtains the position of the neutral axis, respectively $x = z_p = 21.3$ cm.

The inertia moment:

$$I_{b.p} = \frac{b_p x^3}{3} + n_{bt} I_b + 2A_b \sum_1^{n_t} z_{bi}^2 + \frac{b_{pb} t_{pb}^3}{12} + b_{pb} t_{pb} z_{pb}^2 = \frac{34\times21.3^3}{3} +$$
$$+14 \cdot \frac{\pi 2.7^4}{64} + 2\times6.16\left(5.7^2 + 13.7^2 + 21.7^2 + 29.7^2 + 37.7^2 + \right. \quad (30)$$
$$\left. +45.7^2 + 53.7^2\right) + \frac{30\times2.6^3}{12} + 30\times2.6\times66^2 = 547{,}518\,\mathrm{cm}^4.$$

The average stresses in the cover plate:

$$\sigma_{pb} = \frac{M_{\mathrm{Ed}}}{I_{b.p}} z_{pb} = \frac{1{,}400\times10^4}{54.75\times10^4}\cdot 66 = 1{,}688\,\mathrm{daN/cm^2} < f_y/\gamma_{\mathrm{M0}} = 3{,}550\,\mathrm{daN/cm^2}. \quad (31)$$

The stresses in the marginal bolts:

$$\sigma_{b1} = \frac{M_{\mathrm{Ed}}}{I_{b.p}} z_{b1} = \frac{1{,}400\times10^4}{54.75\times10^4}\cdot 53.8 = 1{,}376\,\mathrm{daN/cm^2}. \quad (32)$$

The bolt tension stress:

$$N_{b1} = \sigma_{b1} A_b = 1{,}376\times6.16\times10^{-2} = 84.76\,\mathrm{kN} < F_{t.\mathrm{Rd}} = 330.5\,\mathrm{kN}, \quad (33)$$

where:

$$F_{t.\mathrm{Rd}} = \frac{0.90 f_{ub} A_s}{\gamma_{\mathrm{M2}}} = \frac{0.90\times10{,}000\times4.59}{1.25}\cdot10^{-2} = 330.5\,\mathrm{kN}.$$

The maximum compression stress in the end plate:

$$\sigma_p = \frac{M_{\mathrm{Ed}}}{I_{b.p}} z_p = \frac{1{,}400\times10^4}{54.75\times10^4}\cdot 21.3 = 545\,\mathrm{daN/cm^2} << f_y/\gamma_{\mathrm{M0}} = 3{,}550\,\mathrm{daN/cm^2}. \quad (34)$$

The verification for the mixed tension and shearing stress in the joint area is made with the interaction relationship:

$$\frac{F_{v.\mathrm{Ed}}}{F_{v.\mathrm{Rd}}} + \frac{F_{t.\mathrm{Ed}}}{1.4F_{t.\mathrm{Rd}}} \le 1; \qquad \frac{58.3}{246.4} + \frac{84.76}{1.4\times330.5} = 0.42 < 1,$$

where: $F_{V.\mathrm{Ed}}$ = 58.3 kN; $F_{t.\mathrm{Ed}} = N_{b1}$ = 84.76 kN; $F_{V.\mathrm{Rd}}$ = 246.4 kN; $F_{b.\mathrm{Rd}}$ = 5527 kN > $F_{V.\mathrm{Rd}} > F_{V.\mathrm{Ed}}$.

Table 2 shows the main connection stresses and the stress degree resulting from the M-V interaction for the four constructive solutions (cases) and calculation methods.

Table 2
Connection Forces

Forces in connection	Case 1	Case 2	Case 3	Case 4
Stresses in cover plate σ_{pb} [daN/cm^2]	–	–	583	1,688
Maximum tensile force in bolts $N_{b.\max}$, [kN]	269.4	306.4	194.7	84.8

$\frac{F_{v.\mathrm{Ed}}}{F_{v.\mathrm{Rd}}}+\frac{F_{t.\mathrm{Ed}}}{1.4\cdot F_{t.\mathrm{Rd}}}$	0.82	0.90	0.66	0.42

One can see that applying the modelling in Case 4, the stresses in the connection parts are more uniformly distributed, the cover plate takes over a larger stress than when using the simplified calculus (being about 2.9 larger) while the stresses in the bolts diminish (they are about 2.3 smaller).

Use of prestressed bolts

In this case, an analysis of the M27-group 10.9 bolts in the connection are prestressed to the force $F_{p.C}$:

$$F_{p.C}=0.7\,f_{ub}\,A_s=0.7\times 1{,}000\times 459\times 10^{-3}=321.3\,\mathrm{kN}. \tag{35}$$

The bearing shearing force of a 10.9 prestressed bolt will be:

$$F_{s.\mathrm{Rd}}=\frac{k_s\,n\,\mu}{\gamma_{\mathrm{M3}}}F_{p.C}=\frac{1.0\times 2\times 0.5}{1.25}\cdot 321.3=257\,\mathrm{kN}>F_{V.\mathrm{Rd}}=246.4\,\mathrm{kN}, \tag{36}$$

where: k_s = 1.0 – bolts in usual holes; n = 1; μ = 0.5 – friction surface class A.

As the contact force in the compressed area counterbalances the traction force applied to the tensed area, it is not necessary to reduce the connection resistance to sliding (DSC to Eurocode 3, 2003).

It is noticed that the theoretical sliding strength is very near to the theoretical shearing stress so that preloading the bolts is not justified, though the connection behaviour to fatigue is improved.

If the aim is sliding not to occur in the ultimate limit state (C grade bolts), the theoretical bolt resistance to sliding is calculated with (13).

In the case under analysis, one obtains $F_{s.\mathrm{Rd}} < F_{V.\mathrm{Rd}}$, when the connection will behave as a connection made with non prestressed bolts.

5. Conclusions and Design Recommendations

The connections made with end plates and bolts can be used for steel bridges to connect the deck girders leading to simple and economical constructive solutions on condition that the mechanical behaviour of the connections is properly modelled. The positioning of a cover plate on the tension flange of the girder substantially improves the general behaviour of the connection as stresses in the joint area are made uniform and big stresses in the bolts are transferred to the added cover plates. The cover plate should be welded to the flange only after all the bolts in the connection are well tightened.

The prestressing of the end plate gripping bolts is not necessary and consequently it is not recommended in the case of such connections; to provide a perfect contact between the end plates, we recommend to tighten the bolts at a stress value of 10,...,20% from the theoretical prestressing stress value.

When bolts that are previously subjected to traction from external loads are prestressed, an additional load of 5,...,12% is recommended, if the tension stress does not exceed the prestressing force and only for stiff end plates.

With flexible (semi-stiff) end plates with transverse direction, the tensile stress F_t produced by the external loads (of value $\Delta F_b = K_Q F_t$) overlaps the initial prestress load and connection safety can become critical or can exceed the bolt strength capacity.

The design norms and the literature do not provide a relatively simple and practical methodology to evaluate the end plates stiffness.

The solution of connecting cross girders to the longitudinal girder proposed for the bridge in the presentation, made with a bolted end plate and a cover plate welded over the joint is much more advantageous than the ”classical” solution with cover plates and bolts on both flanges and to the web from the steel material consumption (steel sheets for cover plates and bolts) viewpoint and simplicity of work and shorter time of building as well.

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SOLUŢIE DE ÎMBINARE EFICIENTĂ A ANTRETOAZELOR PODURILOR METALICE RUTIERE PE ARCE

(Rezumat)

La podurile şi construcţiile metalice se folosesc, în mod curent îmbinările de montaj realizate cu şuruburi obişnuite şi îmbinările realizate cu şuruburi de înaltă rezistenţă pretensionate, cărora după montare li se introduc eforturi de preîntindere în tijă.

În lucrare se prezintă modul de comportare mecanică a îmbinărilor cu placă de capăt tip flanşă, realizată cu şuruburi obişnuite solicitate la întindere, respectiv cu şuruburi de înaltă rezistenţă pretensionate, presupunând că plăcile de capăt au o rigiditate foarte mare sau se comportă ca plăci flexibile (semi-rigide), la rotire în planul vertical al îmbinării. Îmbinările cu şuruburi de înaltă rezistenţă pretensionate au o comportare elastică, cu deformaţii mici ce se menţin până la valori mari ale eforturilor.

În lucrare se prezintă pe scurt podul metalic pe arce pentru traversarea râului Someş în municipiul Cluj-Napoca, pentru care s-a întocmit Studiul de fezabilitate, pod la care s-a utilizat soluţia de îmbinare a antretoazelor de grinda-tirant cu plăci de capăt şi platbandă-eclisă la talpa superioară întinsă.

Calculul se dezvoltă în următoarele patru soluţii constructive şi ipoteze de calcul:

– îmbinare cu plăci de capăt perfect rigide la rotire, dispuse pe zona dintre tălpi (pe înălţimea inimii), fără eclise suplimentare;

– îmbinare cu plăci de capăt flexibile (semi-rigide) la rotire, dispuse pe zona dintre tălpi (pe înălţimea inimii), fără eclise suplimentare;

– îmbinare cu plăci de capăt perfect rigide la rotire, cu o eclisă suplimentară, dispusă peste zona rostului, la talpa întinsă a grinzii;

– îmbinare cu plăci de capăt flexibile (semi-rigide) la rotire, cu o eclisă suplimentară, dispusă peste zona rostului, la talpa întinsă a grinzii.

S-a constatat faptul că, ultima soluţie constructivă şi modelarea prezentată conduce la obţinerea unei distribuţii mult mai uniforme a eforturilor în piesele din îmbinare, platbanda-eclisă preia în acest caz un efort mult mai mare decât prin utilizarea calculului simplificat, iar eforturile în şuruburi scad foarte mult.

În partea finală a lucrării sunt prezentate câteva concluzii şi recomandări pe care autorii le consideră deosebit de utile în activitatea de proiectare.