Interaction between Theory and Practice in Civil Engineering and Construction
Edited by Komurlu, R., Gurgun, A. P., Singh, A., and Yazdani, S.

ISBN: 978-0-9960437-2-4

# CONSIDERATION ON TRUSS AND SPLICE JOINT DESIGN MODELS IN EXTREMAL LOADING

LILITA OZOLA

*Dept of Structural Engineering, Latvia University of Agriculture, Jelgava, Latvia*

In this study the behavior of elements and joints in hard loaded Warren trusses is analyzed theoretically with the purpose of disclosing the sections and elements which possibly can become extremely stressed under the design load proposed being real before the ultimate limit state. The significance of truss topology selected has been brought forth, since unfavorable loading of a tensioned chord connection is discussed in this paper. Three types of tensioned splice joints have been examined considering expected elongations of bolts in order to determine additional stresses induced due to the redistribution of tensile forces. An assumption of a continuous beam model on elastically deformed discrete supports has been adopted as a design model for the examination of stresses possibly caused in the sections of bolts. It has been proved that a joint with extended end-plates over both flanges of chord I profile may be accepted as the one which is safe and robust enough for use in tensile chords of bearing structures. The joint type with end-plates extended over a more tensioned flange only and the one with connecting bolts all hidden between flanges has been subjected to sharp criticism due to the extremely nonuniform behavior and the overloading of bolts. The results of this numerical case study promote a deeper understanding and help assessing the end-plate joint behavior since they lack the uniqueness of the solution recommended by the building codes, which is particularly significant when hard-loaded structures have been designed for covering the spans of public building areas.

*Keywords*: Steel structures, Truss topology, Tensile joints, End-plate connections, Bolt force, Prying action.

## 1 INTRODUCTION

During the recent few decades the end-plate connections in hard loaded tension chords made of I-profiles have not been chosen as a priority of research themes, since hollow section profiles prevail in construction. Nevertheless Kim and Madugula (2010) studied flanged connections and proved the T-stub model design equations for leg members of guyed lattice towers. Mohamadi-shooreh and Mofid (2008) performed FEM-based parametric analysis of bolted end-plate splices of beams. Some research works have been devoted to the analysis of high strength steel end-plate connections (Coelhoa and Bijlaard 2007, Yamaguchia *et al.* (2004) focusing on the ductility effects.

Failure of the roof trusses in Riga (2013) raised some disputable topics in the truss design area dealt with the conformity of the design model. The methodology proposed for the modeling of end-plate connection behavior has been detailed in previous work by Ozola (2015). In this study three types of tensioned splice joints are compared, considering possible additional force (the so called "prying effect") due to the deformation of a bolt and the action of bending moment.

## 2 CONSIDERATION ON TRUSS DESIGN MODEL

Unjustified assumptions involved in the calculation model usually are caused by the simplifications representing the joint behavior. A detailed analysis of different static models proposed for the simulation of the behavior of the bearing structure becomes more significant for hard loaded trusses, and particularly when the junction has been put into a heavily loaded tensioned chord element. It is a routine in design to define the design model of the truss representing the web members as a pin connected to the continuous chords. In most cases no more static trials have been carried out and generally not needed. Nevertheless, such simplification leads to overlooking some important portion of stresses in some special cases.

The opinion suggested in this current paper is illustrated by a practical example of a hard loaded Warren truss with an effective span of 18 m, height 2 m, and vertical design load applied directly to the upper chord through concrete panels q= 65,7 kN/m.

Welded connections between chord members and diagonals provide rigidity of a joint capable of preventing any rotation of the ends of members when the structure endures the load. The model shown in Figure 1 is applicable for the determination of a full scale internal force assembly: bending moments, shear and axial forces which must be all taken into consideration. Using the conventional model some portion of internal forces is ignored. Particularly significant are the differences of axial force values for tensioned diagonals at the supports depending on the calculation model used, see joint loading schemes A' (from simplified model) and A (from correct model) in Figure 1. Besides, it is important to note that bending moment produces an additional portion of normal stress perpendicular to the throat of the filled weld at the joint.

Another phenomenon, usually not considered in practice, is the option to insert or remove the middle vertical bar called zero one in static sense, see joint C in Figure 1. A middle bar promotes an effect of obstruction of the deformation of truss contour, and thereby internal forces result in a close relationship with the section stiffness's of the elements involved in both the middle bar and bottom chord elements. The loading situations around the middle point of the bottom chord are of particular interest (see joint C Figure 1) when one solves the connection problem.

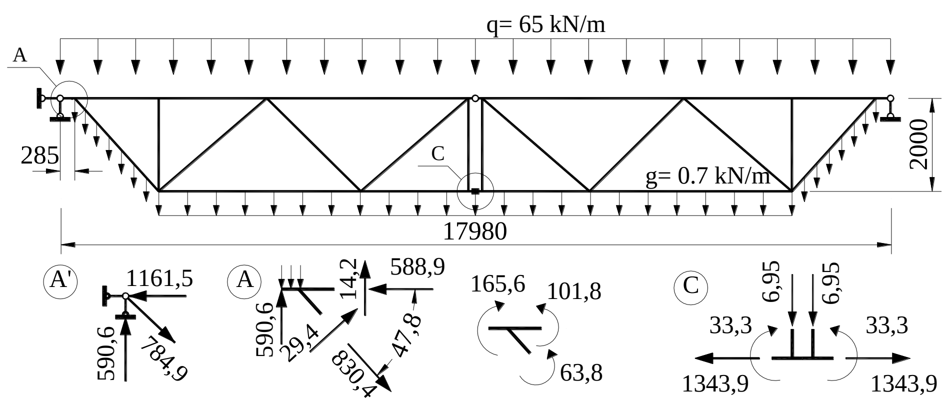



Figure 1. Rigid model of Warren truss and force diagrams for more attractive joints (axial forces in kN, bending moments in kNm).

## 3 TYPES AND MODELS OF END PLATE CONNECTIONS

Different variants may be chosen for joining together the tensioned bottom chord elements made of single I sections, see Figure 2(a). The variant with extended end plates over both chord flanges (ExTC) was used many decades ago but nowadays it is subjected to criticism due to the unpleasant impression from an architectural point of view. The splice joint variant with end plates extended over a bottom flange only (ExT) in order to compensate an additional tensile force due to bending moment may be another option, but a variant with joining bolts hidden between flanges (CL) is becoming more acceptable by architects.

In this study the end-plate is modeled as a continuous beam on discrete elastically deformed supports which represent joining bolts, see Figure 2(b), and loaded by concentrated forces transferred by the flanges and the web of the I-section.

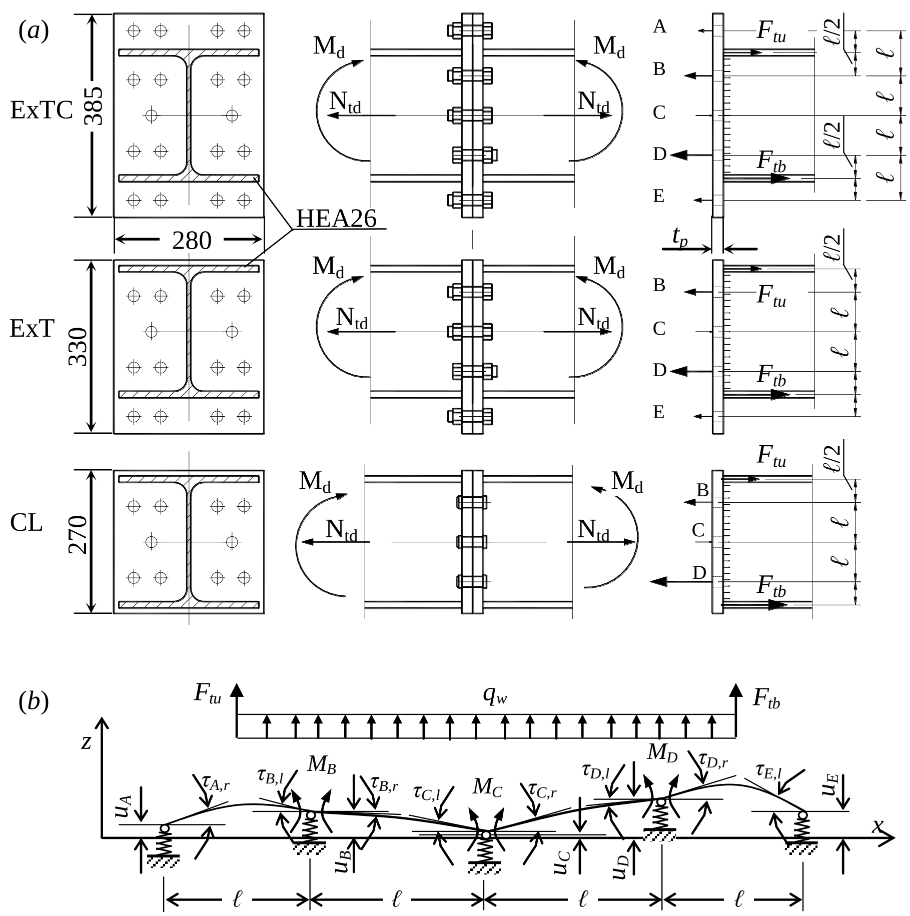



Figure 2. Tensioned splice joints: (a) structural variants ExTC, ExT, CL of splice joints (Ftu= 401 kN, Ftb=682 kN, qw=11.6 kN/m); (b) design model of behavior of joint ExTC.

## 4 NUMERICAL TESTS

First-order structural analysis and dimensioning of truss, have been performed according to conditions stated by the Eurocodes, see design model in Figure1.

Numerical tests of end-plate connections under extremal loading have been performed basing on a continuous beam theory (Marti 2013). Continuous beam on the rigid supports, represented by bolts, has been defined as an initial design model of the end-plate loaded by forces transferred by flanges mainly as well as by web of I-profile. Forces in bolts determined in a rigid model using the three-moment equation method have been taken as input data for the further calculation cycles assuming bolt elongations as elastic deformation of supports. Support settlement values of the end-plate have been determined as proposed elongations of an individual bolt in the row due to axial force action according to Hooke's law. Then a new equation system for a corresponding type of connections (Eq.(1) for ExTC, Eq.(2) for ExT, and Eq.(3) for type CL) has been formulated and solved taking into consideration the additional rotations of the beam axis due to different settlements of the supports, see Figure 2(b). Calculation cycles have been repeated until the bending moment values of the two last cycles differ insignificantly.

$$\begin{cases} \dfrac{2\ell}{3EI}M_B+\dfrac{\ell}{6EI}M_C+\dfrac{u_C-u_B}{\ell}-\dfrac{u_B-u_A}{\ell}+\dfrac{1}{EI}\left(\dfrac{F_{tu}\ell^2}{16}+\dfrac{q_w\ell^3}{8}\left(\dfrac{3}{16}+\dfrac{1}{3}\right)\right)=0 \\ \dfrac{\ell}{6EI}M_B+\dfrac{2\ell}{3EI}M_C+\dfrac{\ell}{6EI}M_D+\dfrac{u_D-u_C}{\ell}-\dfrac{u_C-u_B}{\ell}+\dfrac{q_w\ell^3}{12\,EI}=0 \\ \dfrac{\ell}{6EI}M_C+\dfrac{2\ell}{3EI}M_D+\dfrac{u_E-u_D}{\ell}-\dfrac{u_D-u_C}{\ell}+\dfrac{1}{EI}\left(\dfrac{F_{tb}\ell^2}{16}+\dfrac{q_w\ell^3}{8}\left(\dfrac{3}{16}+\dfrac{1}{3}\right)\right)=0 \end{cases} \tag{1}$$

$$\begin{cases} \dfrac{2\ell}{3EI}M_C+\dfrac{\ell}{6EI}M_D+\dfrac{u_D-u_C}{\ell}-\dfrac{u_C-u_B}{\ell}+\dfrac{q_w\ell^3}{12\,EI}-\dfrac{\ell}{6EI}\left(\dfrac{F_{tu}\ell}{2}+\dfrac{q_w\ell^2}{8}\right)=0 \\ \dfrac{\ell}{6EI}M_C+\dfrac{2\ell}{3EI}M_D+\dfrac{u_E-u_D}{\ell}-\dfrac{u_D-u_C}{\ell}+\dfrac{1}{EI}\left(\dfrac{F_{tb}\ell^2}{16}+\dfrac{q_w\ell^3}{8}\left(\dfrac{3}{16}+\dfrac{1}{3}\right)\right)=0 \end{cases} \tag{2}$$

$$\frac{2\ell}{3EI}M_C+\frac{u_D-u_C}{\ell}-\frac{u_C-u_B}{\ell}+\frac{q_w\ell^3}{12\,EI}-\frac{\ell}{6EI}\left(\left(F_{tu}+F_{tb}\right)\frac{\ell}{2}+\frac{q_w\ell^2}{4}\right)=0 \tag{3}$$

where $\ell$ is distance between bolt rows, see Figure 2, $E$ is the modulus of elasticity of steel, $I$ is the moment of inertia of the end-plate section, $u_A$, $u_B$, $u_C$, $u_D$, $u_E$ are the displacements of end-plate supports (bolt elongations), $M_B$, $M_C$, $M_D$ are the bending moments supposed to be active at the corresponding plate sections, $F_{tu}$ and $F_{tb}$ are the tensile forces transferred to the end plate by upper and bottom flanges correspondingly, as well as $q_w$ is the load transferred by the web assumed uniformly distributed.

## 5 RESULTS AND DISCUSSION

It has been proved by the results of the current study that for any of the joint types bending moments generated in the tensile bottom chord sections around the middle bar even of reasonably moderate values itself affects significantly the force distribution between the bolts depending on the location.

Geometric sizes, maximal expected stress values and steel consumption for each joint type are presented in Table 1. It has been found that in the case of the variant ExT solution the increase of tensile force in bolts near the flanges may be expected 1.7 times more due to the non-uniform elongations of bolts between rows B, C, D, E, see Figure 2, in comparison with the force distribution assuming the initial rigid stage, see Table 2.

The joint type with extended end-plates (ExTC) is highly recommended as in the case of failure of extremely stressed bolts at level D the tension force will be transferred by bolts at E and C levels, and due to the assumption for robustness it would be likely to use the same number of bolts at the middle level C as it is near the flanges.

Architectural assumptions often result in the use of end-plate connections of type CL when all the bolts must be placed between flanges. It is clear that this type is detrimental because a large increase of tension forces can occur due to bending moment, moreover in the limit state it is a highly unsafe connection without any contingency elements for short term transferring of forces released by failed bolts.

It has been revealed by numerical tests that in the range of stresses up to 100 MPa the variation of tensile force in the joint type ExT has been affected significantly by the ratio of moments of inertia of plate and bolt's sections (I_plate/I_bolts), see Fig.3.

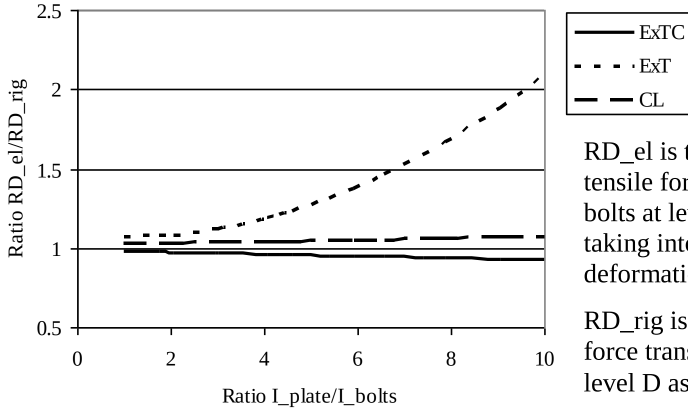
A phenomenological approach may be preferred for use by the designer when the definition of an adequate structural model has become the subject for decision making but experimental tests are quite expensive. Indeed, it is not possible to achieve a complete similarity between the assumed design model and a real structure. However, a characteristic tendency will be disclosed.

Table 1. Characteristic indicators for tested joints.

| **Specification** | **Values for variants** | | |
|---|---|---|---|
| | **ExTC** | **ExT** | **CL** |
| Bolt diameter, mm | 18 | 22 | 24 |
| Maximal tensile stress expected in a bolt section, MPa | 68 | 73 | 93 |
| Thickness of end-plate, mm | 20 | 28 | 36 |
| Maximal bending stress expected in an end-plate section, MPa | 59 | 71 | 74 |
| Steel consumption for bolts and end-plates, kg | 38 | 46 | 48 |

Table 2. Reactive forces in bolts assuming rigid and elastically deformed bolts.

| **Bolt row** | **Reactive forces, kN, per bolt row assuming rigid/ elastic supports** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | **Variant ExTC** | | | **Variant ExT** | | | **Variant CL** | | |
| | **rigid** | **elastic** | **ratio** | **rigid** | **elastic** | **Ratio** | **rigid** | **elastic** | **ratio** |
| A | 160.2 | 170.9 | 1.07 | - | - | - | - | - | - |
| B | 406.5 | 368.4 | 0.91 | 769.7 | 854.7 | 1.11 | 835.6 | 926.3 | 1.11 |
| C | -79.1 | -21.5 | 0.27 | -335.4 | -577.3 | 1.72 | -731.0 | -912.4 | 1.25 |
| D | 599.7 | 556.1 | 0.93 | 663.8 | 892.4 | 1.34 | 1257.1 | 1347.8 | 1.07 |
| E | 274.4 | 287.8 | 1.05 | 263.7 | 191.9 | 0.72 | - | - | - |




RD_el is the final value of tensile force transferred to the bolts at level D (see Figure 2) taking into account elastic deformation of bolts

RD_rig is value of tensile force transferred to the bolts at level D assuming rigid model.

Figure 3. Relationship of force variation in bolts versus ratio of moments of inertia.

## 6 CONCLUSIONS

- It has been proved by this numerical study that serious consequences may be expected due to the disability of traditional design models to reveal more unfavorable internal forces in specific elements of a structural system.
- It is necessary to include recommendations for a detail design of end-plate connections between tensioned I profile chord elements in structural codes.
- Splice joint variant with end-plates extended over flanges (ExTC) may be acknowledged as proper solution regarding both normal behavior under service loads and providing some robustness in the case of overloading.

## References


Coelhoa, A. M. G. and Bijlaard, F. S. K., Experimental behavior of high strength steel end-plate connections, *Journal of Constructional Steel Research*, Elsevier, 63, 1228–1240, 2007

Kim, Y.J. and Madugula, M. K. S., Behavior of bolted circular flange connections subject to tensile loading, *International Journal of Steel Structures*, Springer, Vol.10(1), 65-71, 2010.

Marti, P. *Theory of structures: fundamentals, framed structures, plates and shells*/ Peter Marti; transl. by Philip Thrift, Ernst&Sohn, Berln, 2013

Mohamadi-shooreh, M.R. and Mofid, M., Parametric analyses on the initial stiffness of flush end-plate splice connections using FEM, *Journal of Constructional Steel Research*, Elsevier, 64, Issue 10, 1129–1141, 2008

Ozola, L., The analysis of splice joint behavior in heavily loaded tensile chords of warren trusses, *Proceedings of the 1st Pan-American Congress on Computational Mechanics and XI Argentine Congress on Computational Mechanics*, Idelsohn, S.R., Sonzogni, V., Coutinho, A., Cruchaga, M., Lew, A., Cerrolaza M. (Eds.), 507-517, Buenos Aires, 2015.

Yamaguchi, T., Suzuki Y., Kitada T. and Sugiura K., Effect of tensile and shear force on mechanical behavior of high strength bolted tensile joints, *Journal of Constructional Steel Research*, Elsevier, 60, 1545–1560, 2004