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IMPROVEMENT OF AN ANALYTICAL MODEL FOR DETERMINING THE LOAD-CARRYING CAPACITY OF TIMBER-STEEL CONNECTIONS WITH DOWEL-TYPE FASTENERS

Summary: The second generation of Eurocode 5 is in preparation and has not yet been adopted, so the calculation of the load-carrying capacity of timber-steel connections is based solely on the rules of the first generation. Currently, load-carrying capacity is determined using expressions for thin or thick steel plates. For cases in between, linear interpolation is used. The thickness and mechanical properties of the steel plate significantly influence the behaviour of the timber-steel connection, which is why research in this field focuses on improving the expressions for the connection capacity. The aim is to define the failure modes more precisely and determine the connection capacity more accurately. This paper shows how Johansen's expressions for timber-timber connections can be adjusted for steel-timber connections by accounting for the previously mentioned parameters.

Keywords: steel-timber connections, load-carrying capacity, Eurocode 5, steel plates, dowel-type fasteners

UNAPREĐENJE ANALITIČKOG MODELA ZA ODREĐIVANJE NOSIVOSTI VEZA TIPA DRVO-ČELIK OSTVARENIH ŠTAPASTIM SPOJNIM SREDSTVIMA

Rezime: Kako je druga generacija Evrokoda 5 u fazi pripreme i još uvek nije usvojena, proračun nosivosti veza tipa drvo-čelik zasniva se isključivo na pravilima prve generacije. Zasada, nosivost veze se određuje primenom razvijenih izraza za tanke i debele čelične ploče, dok se za međuslučajeve koristi linearna interpolacija. S obzirom na to da debljina čelične ploče i njena mehanička svojstva značajno utiču na ponašanje veze drvo-čelik, istraživanja u ovoj oblasti usmerena su na unapređenje izraza za nosivost veze. Cilj ovog rada je preciznije definisanje mehanizama loma i pouzdanije određivanje nosivosti veze. U radu je prikazano kako se Johansenovi izrazi, prvobitno razvijeni za veze drvo-drvo, mogu prilagoditi vezama drvo-čelik uvažavajući prethodno navedene parametre.

Ključne reči: veze drvo-čelik, nosivost veze, Evrokod 5, čelične ploče, štapasta spojna sredstva

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1. INTRODUCTION

Over the past few decades, modern civil engineering has increasingly turned towards sustainable solutions, with timber structures gaining significance due to their environmental acceptability and low CO_2 emissions. This is accompanied by innovative materials such as glued laminated timber (GLT) and cross-laminated timber (CLT), increasingly combined with steel or concrete to achieve efficient engineering solutions. These materials are connected using conventional or innovative fasteners, sometimes with reinforcements (Milić [1]). Modern fasteners are still under development, and their application in contemporary timber connections remains limited, partly due to insufficiently developed models for calculating connection load-carrying capacity. This limitation concerns the models for determining the input parameters used in design of connection load-carrying capacity, which have not yet been developed to cover these fasteners. Therefore, manufacturers issue their own European Technical Assessments to bridge the gap relative to the currently valid standards for the design of timber structures. In addition, modern timber screws with partial or full thread are increasingly used with hardwood, representing a highly efficient strategy for achieving high-performance connections. The growing availability of hardwood-based products on the European market enables their application in contemporary timber structures of larger spans and loads. At the same time, it requires more detailed research into connection behavior, on which the reliability of these systems directly depends (Kuck and Sandhaas [2]). Reliable design of timber-steel connections is therefore a key prerequisite for the safe and efficient application of contemporary timber structures.

In Europe, the design of timber-steel connections is currently based on EN 1995-1-1:2004 [3] (first generation of Eurocode 5 – EC5-1), whose methodology relies on the European Yield Model (EYM). The load-carrying capacity calculation of timber-steel connections is based on the Johansen expressions developed for timber-timber connections. These were subsequently adapted for timber-steel connections by considering the rotation of the fastener (diameter d) within the steel plate (thickness t), characteristic of external steel plates. This implies a classification of steel plates into "thin" (allowing free rotation of the fastener, where $t/d \leq 0.5$) and "thick" (preventing rotation of the fastener, where $t/d > 1$), based on which the corresponding Johansen expressions for determining connection load-carrying capacity are selected. For intermediate steel plate thicknesses, the load-carrying capacity is determined by linear interpolation.

Previous research has shown that certain deviations exist between predicted and experimental load-carrying capacities and failure modes. This has led researchers to improve the existing expressions given in EC5-1 or to develop new models based on the European Yield Model methodology (Ai-Phien et al. [4], Zhao et al. [5], Riepe et al. [6], Yang et al. [7]). In the proposed models, the influence of plate thickness on connection load-carrying capacity is treated through simplified limiting cases, classifying the plate as thin or thick depending on the t/d ratio, that is, allowing or preventing rotation of the fastener within the plate (without or with a plastic hinge in the fastener), while the load-carrying capacity of the plate itself is neglected. Only two studies explicitly consider the influence of plate thickness on connection load-carrying capacity through the degree of restraint of the fastener within the plate (Vella et al. [8], Feng et al. [9]). However, both studies are limited to connections with cold-formed steel profiles of very small plate thicknesses (1.2 and 2.4 mm) and bolts with a diameter of 5.5 mm. By introducing a fastener restraint coefficient φ, defined as the ratio of the steel plate bending resistance M_{cfs} to the fastener yield moment M_y, the modified EYM models show better agreement with experimental results for connection load-carrying capacity and failure mode than the EC5-1 expressions.

Since the final version of the second generation of Eurocode 5 is expected by the end of 2027, the available draft prEN 1995-1-1:2023 [10] (EC5-2) introduces significant novelties in the calculation of timber-steel connections. It extends the existing expressions and failure mechanisms developed for timber-timber connections to timber-steel connections. The new methodology directly accounts for the influence of the steel plate on connection load-carrying capacity, modifying the existing expressions for timber-timber connections through a corrected existing coefficient β ($=f_{h2}/f_{h1}$). The influence of plate thickness t_{h2} is reflected through the embedment strength f_{h2} in the steel plate, whose value varies depending on the t_{h2}/d ratio. Thanks to this modification, it is no longer necessary to separately consider thin and thick plates, eliminating the strict boundary between them. Instead, a continuous transition between different failure modes is achieved, representing a significant advantage over the existing expressions from EC5-1. A major advantage of the revised load-carrying capacity expressions is their independence from material type. By applying the appropriate embedment strength for the connection elements, the calculation covers combinations of all materials, with the possibility of direct verification of embedment strength through failure mechanisms a and b (see expressions in Chapter 2).

However, studies confirming the novelties introduced in the new generation of Eurocode 5 (EC5-2) are lacking, and certain inconsistencies and inaccuracies introduced by the proposed expressions have been identified ([11], [9]). Their uncritical application can lead to results that significantly deviate from the actual connection behavior. Therefore, a more detailed analysis and modification of the proposed expressions for timber-timber connection load-carrying capacity is needed, in order to enable their adequate application to timber-steel connections, ensure a more reliable estimation of connection load-carrying capacity, and more precisely define failure mechanisms for a wider range of steel plate thicknesses. Precise prediction of load-carrying capacity and failure mechanisms is essential to ensure ductile connection behavior and prevent brittle failures, such as timber splitting or local failures of steel plates. In this paper, an improved analytical model is proposed, based on the EC5-2 methodology, which explicitly considers the influence of plate thickness for a wide range of t/d ratio values, taking into account the mechanical properties of both the plate and the fastener, whereby only the Johansen component of connection load-carrying capacity is considered, without the contribution of the rope effect. In the absence of experimental tests, validation of the proposed model was carried out through numerical simulations of timber-steel connections, covering a wide range of plate thicknesses and different mechanical characteristics of materials (fastener, plate, and timber).

2. REVIEW OF EXISTING EXPRESSIONS FOR CONNECTION LOAD-CARRYING CAPACITY

In the following, the methodological approach of existing models (within the first and second generations of Eurocode 5 (EC5-1 [3] and EC5-2 [10]), will be presented in detail, with a focus on how the influence of plate thickness is treated in the calculation of connection load-carrying capacity. Attention is given to the expressions for calculating the load-carrying capacity of timber-steel connections with external steel plates in single-shear connections, since the expressions for double-shear and multiple-shear connections are based on them. The pre-factors (1.05 and 1.15) preceding the ductile failure modes (with plastic hinges in the fastener) are omitted from the presented expressions. They compensate for differences in the partial safety factors γ_M for timber and steel and for the influence of moisture and load duration (modification factor k_{mod}) and are applied when determining the design value of connection load-carrying capacity. The influence of the "rope" effect is also deliberately omitted, so that only the Johansen component of connection load-carrying capacity is considered. According to Eurocode 5, the "rope" effect is added to the Johansen component using simplified rules that in many cases do not agree with experimental research, and it significantly increases the load-carrying capacity and changes the connection failure mode. This portion of the load-carrying capacity has been omitted here to avoid the influence of these secondary effects.

2.1. FIRST GENERATION OF EUROCODE 5 (EN 1995-1-1:2004)

According to the currently valid Eurocode 5 (EC5-1), the influence of the steel plate on connection load-carrying capacity is treated by classifying it according to the t/d ratio and accordingly applying the corresponding Johansen expressions (expressions (1) and (2)), which are defined depending on the connection failure mode (Figure 1). The connection load-carrying capacity is determined by taking the calculated minimum value obtained from the presented expressions, based on which the failure mode of the connection that will occur is estimated (that is, which expression gave the minimum value). If the t/d ratio falls between the prescribed limits (0.5 and 1), linear interpolation of the calculated load-carrying capacities for these two limiting cases is performed. This implicitly considers that for a t/d≤0.5 ratio, free rotation of the fastener within the plate is achieved, while for t/d>1 ratios, rotation is prevented, implying complete restraint of the fastener within the plate (that is, the occurrence of a plastic hinge in the fastener). In these models, only the influence of timber (t_1 and f_{h1}) and the fastener (d and $M_{y,Rk}$) is considered, while the mechanical characteristics of the plate and its load-carrying capacity are neglected. Such an approach is a rough approximation of the real behavior of timber-steel connections: the boundary conditions are strictly defined without clear theoretical grounding regarding the degree of restraint or the influence of the mechanical characteristics of the plate and the fastener, while intermediate cases are described by a simple linear function between the limiting cases. This calculation approach does not allow precise prediction of mixed failure modes (between the limiting cases), neither does it account for the influence of local plate behavior (bearing and tilting) on connection load-carrying capacity, which results in significant deviations from experimentally obtained results.

Thin steel plates in single-shear connections (t/d≤0.5):

$$F_{v,Rk} = min\begin{cases} 0{,}4\, f_{h1,k} \cdot t_1 \cdot d & (a) \\ \sqrt{2 \cdot M_{y,Rk} \cdot f_{h1,k} \cdot d} & (b) \end{cases} \tag{1}$$

Thick steel plates in single-shear connections (t/d>1):

$$F_{v,Rk} = min\begin{cases} f_{h1,k} \cdot t_1 \cdot d & (c) \\ f_{h1,k} \cdot t_1 \cdot d \left[\sqrt{2 + \frac{4M_{y,Rk}}{f_{h1,k} \cdot d \cdot t_1{}^2}} - 1\right] & (d) \\ 2\sqrt{M_{y,Rk} \cdot f_{h1,k} \cdot d} & (e) \end{cases} \tag{2}$$

where:
$F_{v,Rk}$ is the characteristic load-carrying capacity per shear plane per fastener;
f_{h1} is the characteristic embedment strength in the timber member;
t_1 is the smaller of the thickness of the timber side member or the penetration depth;
d is the fastener diameter;
$M_{y,Rk}$ is the characteristic fastener yield moment.

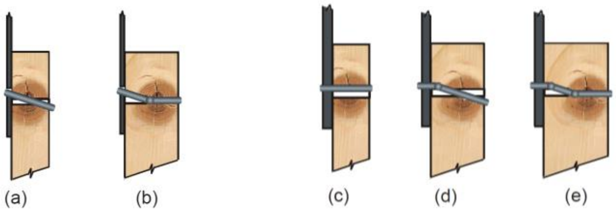


Figure 1: Failure modes for timber-steel connections according to EC5-1

2.2. SECOND GENERATION OF EUROCODE 5 (prEN 1995-1-1:2023)

With this significant contribution of the second generation of Eurocode 5 (EC5-2), the basic expressions developed for timber-timber connections have been generalized (expression (3), Figure 2) and extended to other materials, including steel in timber-steel connections, emphasizing the importance of the ratio of embedment strengths in both connection elements. Through the adopted modification of the existing coefficient β (expression (4)), the influence of the steel plate (through t_{h2} and f_{h2}) is taken into account in the connection load-carrying capacity and depends on the t_{h2}/d ratio. As stated, owing to this modification, the strict boundary between thin and thick plates is eliminated, and they no longer need to be considered separately; instead, a continuous transition between different failure modes is achieved. Although this introduced modification appears to take into account, to some extent, the degree of restraint of the fastener within the plate through the t_{h2}/d ratio and the correction of f_{h2} (i.e., β), it nevertheless remains strictly tied to the newly adopted limits (dependent on the t_{h2}/d ratio) prescribed for determining the coefficient k_{pl}, which reduces the embedment strength of the steel plate f_{h2}. These limits are the same ones that were previously used to classify plates as thin and thick, i.e., $k_{pl}=0.5$ for the case $t_{h2}/d\leq 0.5$, $k_{pl}=1$ for the case $t_{h2}/d>1$, while for intermediate values of the t_{h2}/d ratio, the coefficient k_{pl} is determined by linear interpolation. There is an error in the notation of the coefficient k_{pl} in the draft EC5-2 (prEN 1995-1-1:2023), where instead of t_{h2}/d the notation d/t_{h2} currently appears (see [11]). An explanation for the adoption of this coefficient was given by Bruhl in [11], where it is stated that comparing the equations according to EC5-1 and EC5-2 indicates that for plates with a t_{h2}/d ratio smaller than 1, significant increases in load-carrying capacity occur (see Figure 7a in [11]). A significant influence on these increases in connection load-carrying capacity results from the formation of a plastic hinge in the plate for t_{h2}/d ratios considerably smaller than 1 (see Figure 6 in [11]), i.e., that for smaller t_{h2}/d ratios, a higher tensile strength value of the steel plate f_{uk} is required compared to larger t_{h2}/d ratios in order for a plastic hinge to form in the fastener within the steel plate. It is also stated in [11] that no experimental test results exist for a detailed assessment of this effect, and in order to account for the effect of partial restraint, the coefficient k_{pl} has been introduced, by which the embedment strength of steel f_{h2} is reduced by 50% for t_{h2}/d ratios smaller than 0.5. Through this procedure, for $t_{h2}/d\leq 0.5$ ratios, the connection load-carrying capacity is fictitiously reduced, thereby compensating for the previously mentioned significant increases in load-carrying capacity, as well as the effects of the gap between the fastener and the hole, and local plastic deformation of the plate in the vicinity of the fastener, which are not covered by the primary Johansen component of load-carrying capacity. In the only study addressing this issue, Feng et al. [9] compared the load-carrying capacities calculated according to EC5-2 with the experimental results and found that the calculated values significantly underestimate the test data. They therefore proposed a correction of the expression for k_{pl} for ratios $t_{h2}/d\leq 1$ and further suggested that the Johansen expressions could be corrected to account for the degree of restraint through the coefficient φ, following Vella et al. [8]. Although, as a rule, the embedment strength of steel f_{h2} should be determined according to prEN 1993-1-8:2023 [12] (EC3-1.8), the draft EC5-2 recommends a value of 600 MPa for all types of steel. According to this, EC5-2 distances itself from the high f_{h2} values that can be determined according to EC3-1.8 ($f_{h2}\approx 3\cdot f_{uk}$, where f_{uk} is the tensile strength of the plate) and prescribes considerably lower values, on the order of $1.5\cdot f_{uk}$ to $2\cdot f_{uk}$, corresponding to the case where deformations due to bearing of the steel must be limited (according to EC3-1.8).

$$F_{D,k} = min\begin{cases} f_{h,1,k}\cdot t_{h1}\cdot d & (a) \\ f_{h,2,k}\cdot t_{h2}\cdot d & (b) \\ \frac{f_{h,1,k}\cdot t_{h1}\cdot d}{1+\beta}\left[\sqrt{\beta+2\beta^2\left[1+\frac{t_{h2}}{t_{h1}}+\left(\frac{t_{h2}}{t_{h1}}\right)^2\right]+\beta^3\left(\frac{t_{h2}}{t_{h1}}\right)^2}-\beta\left(1+\frac{t_{h2}}{t_{h1}}\right)\right] & (c) \\ \frac{f_{h,1,k}\cdot t_{h1}\cdot d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_{y,k}}{f_{h,1,k}\cdot d\cdot t_{h1}^2}}-\beta\right] & (d) \\ \frac{f_{h,1,k}\cdot t_{h2}\cdot d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\frac{4\beta(1+2\beta)M_{y,k}}{f_{h,1,k}\cdot d\cdot t_{h2}^2}}-\beta\right] & (e) \\ \sqrt{\frac{2\beta}{1+\beta}}\sqrt{2\cdot M_{y,k}\cdot f_{h,1,k}\cdot d} & (f) \end{cases} \quad (3)$$

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} \tag{4}$$

$$f_{h,2,k} = k_{pl} \cdot 600 \tag{5}$$

where:

$F_{D,k}$ is the characteristic load-carrying capacity per shear plane per fastener;

$f_{h1,k}$ is the characteristic embedment strength in member 1 (timber);

$f_{h2,k}$ is the characteristic embedment strength in member 2 (steel plate);

k_{pl} is the factor accounting for fastener clamping in the steel plate (for outer steel plates: k_{pl}=0,5 for $t_{h2}/d \leq 0{,}5$, k_{pl}=1 for $t_{h2}/d>1$, with interpolation for intermediate ratios t_{h2}/d);

t_{h1} is the thickness of member 1 (timber);

t_{h2} is the thickness of member 2 (steel plate);

β is the ratio of embedment strengths $f_{h2,k}/f_{h1,k}$.

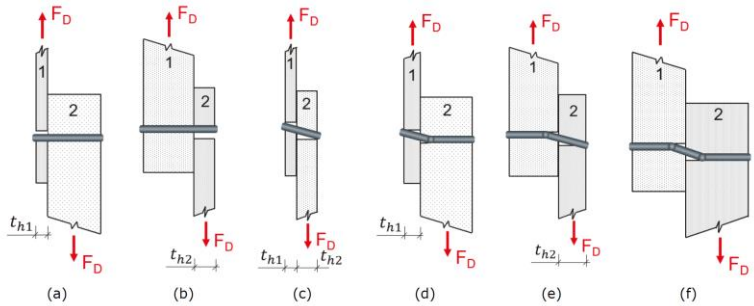


Figure 2: Failure modes for timber-steel connections according to EC5-2

3. IMPROVED ANALYTICAL MODEL FOR TIMBER-STEEL CONNECTIONS

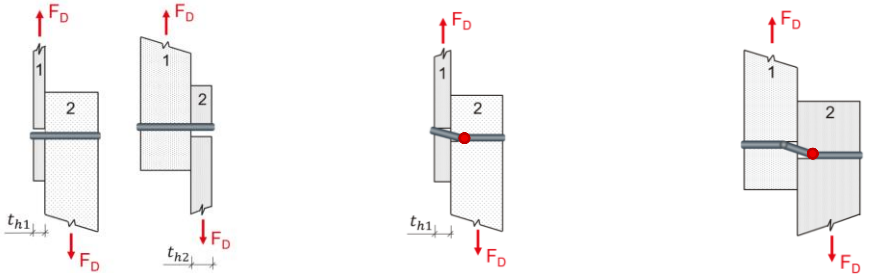
A question arises as to whether the approach introduced by EC5-2 as a modification is on the safe side, and whether, through this fictitious modification of the coefficient β (≠const.), which is otherwise responsible for accounting for the different properties of timber (different timber species or the influence of the angle of force relative to the grain direction), its fundamental role within the Johansen expressions for connection load-carrying capacity has been compromised. Starting from the model proposed by Vella et al. in [8], which has proven to be a reliable solution, improved expressions for calculating the load-carrying capacity of timber-steel connections have been derived (expression (6)). Since the model is based on the moment capacity of thin plates M_{csf}, a correction of the plate moment capacity was carried out in order to cover a wider range of steel plate thicknesses applied in timber-steel connections.

If, in the existing expressions for timber-timber connections according to EC5-2 (expression (3)), the degree of restraint of the fastener within the plate is taken into account through the coefficient $\varphi=M_{spl}/M_{y,k}$ (≤1.0), and considering the modification of only two types of ductile connection failure (failure modes (f) and (d)) into which this effect can be incorporated, a set of four expressions (expression (6)) is obtained, covering all connection failure modes. In the absence of a more precise definition of the moment capacity of the steel plate, M_{spl} can be determined using expression (7). This approach is based on the approximation of a rectangular stress distribution f_{h2} across the plate thickness t_{h2}, where the embedment strength of the steel plate is determined according to EC3-1.8 [10] as $f_{b,RD}=F_{b,Rd}/(d\cdot t_{h2})=k_m\cdot\alpha_b\cdot f_{y,k}$. In deriving expression (7), the yield strength $f_{y,k}$ was adopted instead of the tensile strength $f_{u,k}$ of the plate, in order to account for the gradual plastification of the plate (resulting from bearing and tilting) and the non-uniform stress distribution in thicker plates, which influences the degree of restraint of the fastener within the plate through the coefficient φ. In the limiting case when $M_{spl}\rightarrow 0$ (when $\varphi\rightarrow 0$), the failure mechanisms are modified directly through the coefficient φ as follows: failure mechanism (f) transitions into failure mechanism (e) (f→e, where this failure mode in expression (6) is now referred to as failure mode (f-e)), while failure mechanism (d) transitions into failure mechanism (c) (d→c, where this failure mode in expression (6) is now referred to as failure mode (d-c)), while failure mechanisms (a) and (b) remain identical to those defined in the EC5-2 draft (Figure 3). Furthermore, since the embedment strength of the steel plate is defined as $f_{h2}=k_m \cdot \alpha_b \cdot f_{u,k}$ (according to EC3-1.8), and depends solely on the mechanical properties of the plate ($f_{u,k}$) and the fastener ($f_{ub,k}$), the ratio $\beta=f_{h2,k}/f_{h1,k}$ remains constant. By implementing the moment capacity of the plate M_{spl} and a constant value of the coefficient β, the need for establishing strict boundaries between thin and thick plates is eliminated. The degree of restraint of the fastener within the plate (quantified by the coefficient φ) becomes directly dependent on the mechanical and geometric characteristics of the connection, such that a continuous transition between different failure mechanisms is established through the modified expressions. By implementing the coefficient φ, a considerably more precise description of connection behavior is achieved, since it directly defines the conditions for the formation of a plastic hinge in the fastener within the plate, thereby eliminating the approximations present in previous models, and the plastic deformation of the fastener within the steel plate becomes exactly defined, which contributes to the reliability of the load-carrying capacity calculation. It should also be emphasized that this approach makes it possible to describe transitional failure mechanisms between those strictly defined by Johansen theory. In this way, a reliable estimation of the degree of plastification of the fastener within the steel plate is achieved, leading to a more precise prediction of the failure mechanism itself.

$$F_{IM,k} = min \begin{cases} f_{h,1,k} \cdot t_{h1} \cdot d & (a) \\ f_{h,2,k} \cdot t_{h2} \cdot d & (b) \\ \frac{f_{h,1,k} \cdot t_{h1} \cdot d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)(\varphi \cdot M_{y,k})}{f_{h,1,k} \cdot d \cdot t_{h1}{}^2}}-\beta\right] & (d-c) \\ \sqrt{\frac{2\beta}{1+\beta}} \cdot \sqrt{2 \cdot M_{y,k} \cdot (1+\varphi) \cdot f_{h,1,k} \cdot d} & (f-e) \end{cases} \tag{6}$$

$$M_{spl} = \frac{\alpha_1 \cdot d \cdot t_{h2}^2}{4} \cdot f_{y,k} \tag{7}$$

where:
$F_{IM,k}$ is the load-carrying capacity per shear plane per fastener (improved expression);
φ is the ratio of plate capacity moment to yield moment, $M_{spl}/M_{y,k}$ ($\leq 1{,}0$);
M_{spl} is the plastic moment of the steel plate;
$f_{y,k}$ is yield strength of steel plate;
$f_{h,2,k}$ is the embedment strength of member 2 (steel plate, $f_{h2,k}=k_m \cdot \alpha_b \cdot f_{u,k}=\alpha_1 \cdot f_{u,k}$ acc. to EC3-1.8 [12], where k_m is 1 (for steel grades up to S460) or 0,9 (steel grades equal to or higher than S460), and $\alpha_b=\min(e_1/d_0;3 \cdot f_{ub,k}/f_{u,k};3)$, where $f_{ub,k}$ ultimate tensile strength of fastener and $f_{u,k}$ ultimate tensile strength of steel plate).



(a) (b) (d-c) (f-e)

Figure 3: Failure modes of improved analytical model for timber-steel connections

4. NUMERICAL SIMULATION OF EXPERIMENTAL TESTING OF TIMBER-STEEL CONNECTIONS

For the purpose of validating the improved analytical expressions, numerical models were developed using the FEMAP with NX NASTRAN software package. These models represent modified numerical models originally presented in the doctoral dissertation by Milić ([1], [13]). For the purposes of this study, two groups of numerical models were analyzed.

The first group of models was developed to validate the adopted material models, which was performed using the results of bolt bending tests. A complete overview of these numerical models is provided in paper [1]. For the improved analytical solution, the numerical M_y-α relationship was determined (Fig. 4a), from which it is possible to determine the characteristic yield moment value of the fastener M_y [14]. According to the numerical M_y-α relationship, the maximum yield moments are $M_{y(4.6),max}$=73510 Nmm and $M_{y(12.9),max}$=226155 Nmm, respectively. The yield moment, required for the design calculations according to EC5-2, is determined for the angle $\alpha=\alpha_1\cdot(2.78\cdot\rho_k/f_t)^{0.44}+\alpha_2$, as defined by the EN 409 [15]. Beyond the diameter-dependent rotation angle, the determination of the yield moment in accordance with EN 409 explicitly incorporates both characteristic timber density ρ and fastener tensile strength f_t, ensuring a more accurate representation of the connection's yield behavior [2]. Following this procedure, the yield moments M_y for bolt grade 4.6 are obtained as $M_{y(4.6,C18)}$=69463 and $M_{y(4.6,D30)}$=72208 Nmm, while for grade 12.9, it is $M_{y(12.9,C18)}$=222768 and $M_{y(12.9,D30)}$=224668 Nmm.

The second group of models represents a continuation of the research, analyzing a timber-to-steel-plate connection made using an M10 bolt. A multi-parametric numerical analysis was conducted to encompass all relevant failure modes. By varying the geometric and material characteristics, the following components were included: Timber classes: C18 and D30; Bolt grades: 4.6 and 12.9; Steel plate grades: S235 and S355; Steel plate thicknesses: 4 mm, 8 mm, and 12 mm. To enable a comparison between the numerical results and analytical solutions, the mechanical properties of all materials were adopted as nominal values. The improvement of the analytical expression is performed only for the part of the equation that accounts for the Johansen embedding strength, meaning that the rope effect was not analyzed in the numerical model.

To simulate the embedment behavior of timber under the pressure of the bolt shank parallel to the grain, equivalent line elements of the 2-node ROD type (two nodes with three degrees of freedom each) were modeled. The material model for timber was adopted according to the proposal by Gikonyo et al. [14] (Figure 4b), where the non-linear behavior is defined as a function of the curve shape parameter a and the density ρ. The curve shape parameter was adopted according to the recommendation (a=3), while the timber density was adopted according to the normative values: ρ_m=380 kg/m^3 (timber class C18), and ρ_m=640 kg/m^3 (timber class D30).

For modeling the bolt and the steel plate, an elastoplastic constitutive model was used, with the onset of yielding according to the von Mises criterion and multilinear behavior in the plastic range, as shown in Figure 4c. The bolt and the steel plate were modeled using 8-node SOLID finite elements (FE). The bolt head and shank were modeled as a single body, where the bolt shank has a diameter of 10 mm, and the head has a diameter of 16 mm and a length of 7 mm. The FE mesh size was set such that the bolt shank diameter (d = 10 mm) is divided into 6 parts, giving a mesh size of 1.67 mm. The FE mesh size of the steel plate was adopted such that the plate is divided through its thickness into 4, 8, and 12 segments, depending on the plate thickness (4, 8, and 12 mm). Since the bolt shank was modeled using volumetric elements and the timber using line elements, the contact between them was established by introducing intermediate surface elements of the 4-node PLATE type. These elements form the boundary of the hole surface – the contact shell. The connection between the surface elements of the contact shell and the line elements is realized through shared nodes. The contact shell elements form a cylindrical surface that surrounds the bolt shank, located at a distance of 0.10 mm from it. Since the contact shell serves as an intermediary between the line and volumetric elements, its influence on the mechanical behavior of the connection was minimized by applying a linear elastic material model, with modulus of elasticity of E = 0.001 GPa and a thickness of 0.1 mm. The contact definition consists of a contact pair (target surface and contact surface) and the specific interaction behavior between them. Due to symmetry, only one half of the connection was analyzed. The model loading was applied as a prescribed nodal displacement, with an adopted value of 16 mm.

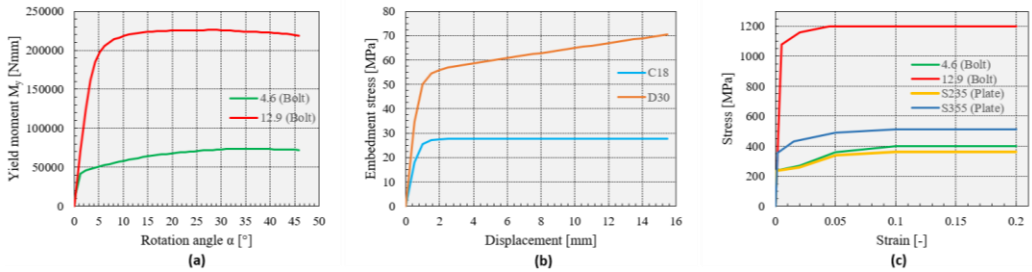


Figure 4: (a) Adopted embedment stress-displacement curves for timber classes C18 and D30 (b) Relationship between the bolt yield moment M_y and rotation angle α for bolt grade 4.6 and 12.9 and (c) Adopted stress-strain curves for used steel and bolt grades (right)

5. RESULTS

For validating the improved model for calculating the load-carrying capacity of timber-steel connections and for comparison with the numerical results and approach to EC5-2 (Table 1), mean values of the material mechanical properties given in Chapter 4 were used. While characteristic values are used in design to ensure safety, mean values enable a consistent comparison when analytical modeling is compared with the numerical simulations used to determine connection load-carrying capacity at the prescribed displacement of 15 mm [14].

Table 1: Load-carrying capacities of timber-steel connections determined through numerical simulations ($F_{v,num}$), according to EC5-2 ($F_{D,k}$) and IM ($F_{IM,k}$)

	$F_{v,num}$ [kN]			$F_{D,k}$ [kN]			$F_{IM,k}$ [kN]		
Sim. number	**4mm**	**8mm**	**12mm**	**4mm**	**8mm**	**12mm**	**4mm**	**8mm**	**12mm**
4.6/S235/C18	7.82 [e]	8.86 [f]	8.84 [f]	6.05 [e] -22.6%	7.85 [e] -11.4%	8.57 [f] -3.1%	7.24 [f-e] -7.4%	8.64 [f] -2.5%	8.64 [f] -2.3%
4.6/S235/D30	11.01 [e]	13.64 [f]	13.74 [f]	9.12 [e] -17.2%	11.60 [e] -14.9%	13.49 [f] -1.8%	11.49 [f-e] +4.3%	13.77 [f] +1.0%	13.77 [f] +0.2%
4.6/S355/C18	8.48 [f-e]	8.92 [f]	8.87 [f]	6.05 [e] -28.6%	7.85 [e] -12.0%	8.57 [f] -3.4%	7.51 [f-e] -11.4%	8.67 [f] -2.8%	8.67 [f] -2.3%
4.6/S355/D30	12.84 [f-e]	13.65 [f]	13.44 [f]	9.12 [e] -29.0%	11.60 [e] -15.0%	13.49 [f] +0.4%	12.39 [f-e] -3.5%	13.90 [f] +1.8%	13.90 [f] +3.4%
12.9/S235/C18	10.88 [e]	13.94 [d-c]	15.95 [d]	10.62 [e] -2.4%	11.65 [e] -16.4%	13.23 [c] -17.1%	11.61 [f-e] +6.7%	12.89 [d-c] -7.5%	14.33 [d] -10.1%
12.9/S235/D30	15.61 [e]	22.43 [e]	25.37 [f]	16.06 [e] +2.9%	17.44 [e] -22.2%	20.55 [e] -19.0%	18.23 [f-e] +16.8%	21.06 [f-e] -6.1%	24.30 [f] -4.2%
12.9/S355/C18	11.51 [e]	14.59 [d-c]	15.52 [d]	10.62 [e] -7.7%	11.65 [e] -20.1%	13.23 [c] -14.8%	11.80 [f-e] +2.6%	13.27 [d-c] -9.0%	14.37 [d] -7.4%
12.9/S355/D30	16.89 [e]	23.88 [f-e]	25.53 [f]	16.06 [e] -4.9%	17.44 [e] -27.0%	20.55 [e] -19.5%	18.91 [f-e] +12.0%	22.99 [f-e] -3.7%	24.52 [f] -4.0%

Based on the numerical simulations, Table 1 presents the connection load-carrying capacities $F_{v,num}$ determined for all the listed combinations of input parameters. In addition to the load-carrying capacities determined through numerical simulations, Table 1 also provides the calculated values of connection load-carrying capacities obtained using the analytical expressions according to EC5-2 ($F_{D,k}$) and the improved expressions - IM ($F_{IM,k}$). The table indicates the failure modes in square brackets, as well as the deviations of the analytical models ($F_{D,k}$ and $F_{IM,k}$) relative to the numerical values.

In Figure 5, to enable comparison of the numerical load-carrying capacities $F_{v,num}$ and to better understand the behavior of the analytical expressions depending on the t/d ratio, a comparison is presented of the load-carrying capacity predictions according to the analytical expressions of EC5-2 ($F_{D,k}$) and the improved expressions ($F_{IM,k}$), together with predictions according to EC5-1 ($F_{v,RK}$). For the purpose of comparing the estimated failure modes obtained through the analytical expressions according to EC5-2 and IM, Figure 6 shows the failure modes obtained through the numerical simulations.

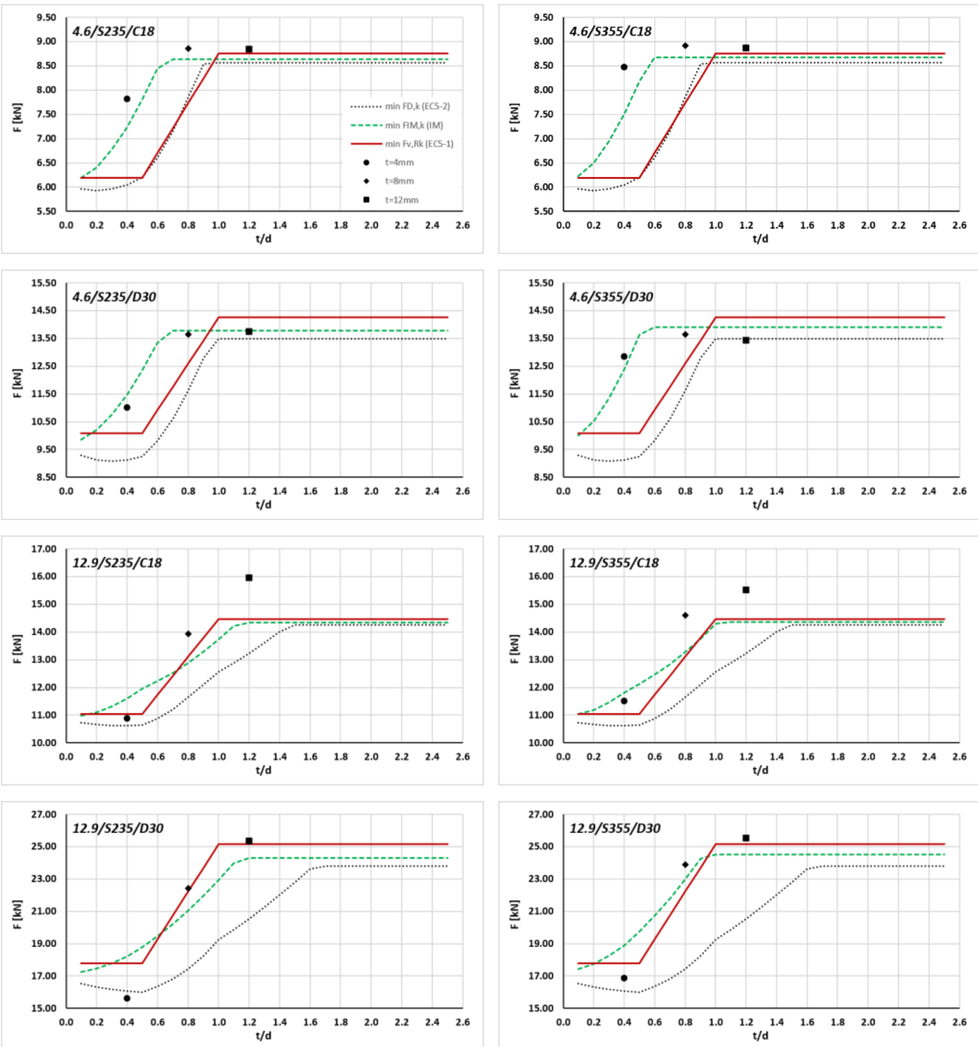


Figure 5: Comparison of the analytical expressions according to EC5-1 ($F_{v,Rk}$), EC5-2 ($F_{D,k}$), IM ($F_{IM,k}$) and the numerical values $F_{v,num}$

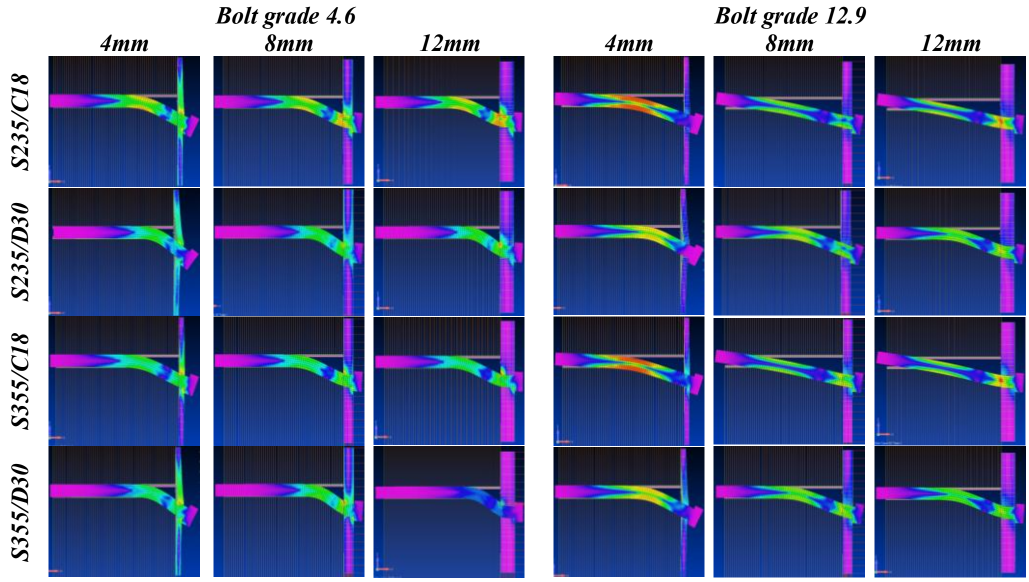


Figure 6: Failure modes of the investigated connections and stress distribution in the plate and fastener from numerical simulations

6. DISCUSSION AND CONCLUSION

Based on the presented results, it can be concluded that the proposed IM expressions, in terms of load-carrying capacity prediction and the achieved failure mode, show very good agreement with the numerical simulations, which are considered to represent the realistic behavior of timber-steel connections that would be obtained through experimental testing. It can be observed that the maximum deviations of the IM expressions relative to the numerical values are approximately ±10%, with the exception of connection model 12.9/S235/D30 (+16.8%), while the values according to EC5-2 underestimate the numerical values in most cases by approximately 10 to 30%. While the IM expressions follow the behavioral trend in terms of failure mode prediction and consequently connection load-carrying capacity for all t/d ratios, as can be seen in Figure 5, the expressions according to EC5-2, due to the introduced modification for t/d ratios below 1, yield significantly reduced load-carrying capacity values and different failure modes in certain cases. In the case where a bolt of grade 12.9 is applied, this trend is particularly pronounced when using EC5-2, and it can be observed that the formation of a plastic hinge in the fastener shifts to t/d ratios as high as 1.6. It should be emphasized that the expressions according to EC5-1 also provide good predictions in terms of both load-carrying capacity and failure mode, specifically for t/d ratios greater than 0.5. Furthermore, it can be observed that the trend (load-carrying capacity versus t/d) of the IM expressions is quite close to the expressions according to EC5-1 in the case of a grade 12.9 bolt, whereas for a grade 4.6 bolt this trend differs considerably for t/d ratios below 1. This indicates that the boundaries between thin and thick plates are defined very strictly in EC5-1, and the degree of restraint of the fastener in the plate is not taken into account when considering possible failure modes, which consequently means that the connection load-carrying capacity cannot be estimated more accurately. Considering the stress state and the formation of a plastic hinge in the fastener, both within the timber and at the interface with the steel plate (Figure 6), as well as the value of the coefficient φ in the IM expressions, it can be concluded that the behavior of the fastener within the connection can be very well captured through the coefficient φ. This conclusion is of particular significance, as it serves to draw designers' attention to the connection behavior, that is, to ensure a ductile failure mode during design and to avoid brittle failures. The observations noted above, based on the presented results, indicate that the introduction of coefficient φ yields a more realistic stress state within the connection, namely the formation of a plastic hinge in the fastener at the contact with the plate, and that the proposed IM expressions have performed very well in comparison to the modification introduced by EC5-2. Since the IM expressions, in the absence of a better-defined moment capacity of the steel plate, are based on the introduced approximation using a uniform stress distribution in the plate, further research directions should focus on its more accurate determination. Additionally, for further validation of the IM expressions, experimental investigations need to be conducted in order to fully assess their limitations and applicability.

REFERENCES

[1] Milić M. Predlog novog rešenja za sprezanje čelika i unakrsno lameliranog drveta primenom ojačanih moždanika, doktorska disertacija, Građevinsko-arhitektonski fakultet, Univerzitet u Nišu, 2023.

[2] Kuck E., Sandhaas C. Load-bearing behaviour of partially threaded screws in hardwood, Wood Material Science & Engineering, 17 (4), 2022, 283-298.

[3] EN 1995-1:2004. Eurocode 5: Design of timber structures - Part 1-1: General – Common rules and rules for buildings, European Committee for Standardization (CEN), Brussels, Belgium.

[4] Ho A.-P., Ziegler J. W., Seim W. Extended analytical model for shear-loaded steel-to-timber connections with screws and profiled nails, Engineering Structures, 364, 2026, 123160.

[5] Zhao E., Song Q., Zhang X., Li S. Shear performance of H-shaped steel to novel inorganic-bonded bamboo composite connections: Experimental tests and prediction models, Engineering Structures, 331, 2025, 119975.

[6] Riepe J., Díaz J., Schänzlin J. An elasto-plastic model for the deformation of dowel-type fasteners based on Johansen's theory, Engineering Structures, 344, 2025, 121315.

[7] Yang D., Li H., Hong C., Lorenzo R. Performance of single-bolted laminated flattened-bamboo connection with steel plate under compression parallel to grain – An experimental and numerical study, Construction and Building Materials, 401, 2023, 132774.

[8] Vella N., Gardner L., Buhagiar S. Analytical modelling of cold-formed steel-to-timber connections with inclined screws, Engineering Structures, 249, 2021, 113187.

[9] Feng Z. R., Wang Y. C., Cunningham L. S. Behaviour of screw shear connections between oriented strand board and cold-formed steel sections under ambient and post-heating conditions: An experimental and analytical investigation, Engineering Structures, 334, 2025, 120209.

[10] prEN 1995-1:2023. Eurocode 5: Design of timber structures - Part 1-1: General rules and rules for buildings; draft prEN 1995-1-1:2023, European Committee for Standardization (CEN), Brussels, Belgium.

[11] Bruhl F. Eurocode 5 – stiftförmige Verbindungsmittel, 26. Internationales Holzbau-Forum 2022, Band I Prolog VI, 2022, 246-256.

[12] prEN 1993-1-8:2023. Eurocode 3: Design of steel structures – Part 1-8: Design of joints. European Committee for Standardization (CEN), Brussels, Belgium.

[13] Milić M., Vacev T., Petronijević P., Nešović I., Zorić A., Paunović S., Matejević Nikolić B. Experimental Research and FE Model of a Bolted Steel-CLT Composite Connection, Periodica Polytechnica Civil Engineering, 68 (1), 2024, 8-22.

[14] Gikonyo J. W., Schweigler M., Bader T. K. Beam-on-Foundation modelling of dowel-type single fastener connections in cross laminated timber, Engineering Structures, 303, 2024, 117519.

[15] EN 409:2009. Timber structures - Test methods – Determination of the yield moment of dowel type fasteners, European Committee for Standardization (CEN), Brussels, Belgium.