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Design and FEM Modelling of Steel Truss Girder Joints

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This paper critically presents the codified design of joints in steel truss girders, according to the latest european norms [1-2] , as contemporary norms in which this issue is included and processed in detail for the first time. Rules in [2] are based on simplified analytical models in combination with experimental research, so the regulations consist of semi-empirical calculation formulae which are valid in limited conditions. Thus, in engineering practice it is required to take care of parameters which affect the global or local behaviour of truss girder such as secondary bending moments, bending moments resulting from eccentric member joints, and joint deformability, so this paper gives distinct attention to these effects. Typical truss girder behaviour modelling modes using finite element method (FEM) are illustratred so to predict their values, and take them in the final estimate if needed. The steel truss girder made of hollow rectangular sections with welded joints is reviewed in detail and numerically exemplified. The accuracies are compared and recommendations are given for the application of each model.

Proračun i modeliranje spojeva rešetkastih čeličnih nosača MKE metodom

Izvornoznanstveni članak

U radu se kritički sagledava proračun spojeva rešetkastih čeličnih nosača prema novim europskim normama [1-2], kao suvremenim normama u kojima se ova problematika prvi put detaljnije kodificira. Pravila u [2] se baziraju na pojednostavljenim analitičkim modelima u kombinaciji s eksperimentalnim ispitivanjima, pa se propisi zapravo sastoje od semi-empirijskih proračunskih izraza koji vrijede u dosta ograničenim uvjetima. Stoga je u inženjerskoj praksi potrebno voditi računa o parametrima koji utječu na globalno i lokalno ponašanje rešetkastih nosača, kao što su npr. sekundarni momenti savijanja i momenti savijanja nastali od ekscentričnih spojeva štapova te deformacijska sposobnost spojeva, stoga se u ovom radu posebna pozornost posvećuje upravo tim utjecajima. U tom se smislu ilustriraju karakteristični načini modeliranja ponašanja rešetkastih nosača uz pomoć metode konačnih elemenata (MKE) kako bi se mogle procijeniti njihove vrijednosti i po potrebi uključiti u proračune. Na numeričkom se primjeru detaljnije razmatra rešetkasti nosač od šupljih pravokutnih profila i sa zavarenim spojevima, te su uspoređene točnosti i dane preporuke za primjenu pojedinih modela.

1. Introduction

Truss girders represent a very rational construction form and facilitate construction making with minimum material expenditure and very high degree of efficiency regarding construction elements. They are especially suitable for large-span structures and for greater load transfer in building construction, as well as bridge construction.

Typical truss girder calculation model implies hinge links on joint location, i.e. grid members are influenced only by axial forces. This approach facilitates calculations considerably in addition to being based on the tradition of girder design and construction.

In reality, links between individual truss elements are usually made stiff (especially welded joints). Deviations from centric connecting are common, partly due to physical inability to create fully centric links, and partly to facilitate structure construction. In these cases additional stress in truss members occurs (secondary stress), and global truss behaviour is especially influenced by joints between elements. Because of this, it becomes paramount to quantify these phenomena in order to assess their effect on actual construction behaviour. It is known that these phenomena can be tolerated to some extent with regard

Symbols/Oznake

A_i - cross-sectional area of member i (i=0, 1, 2)
- površina poprečnog presjeka štapa i (i=0, 1, 2)

b_i - overall out-of-plane width of RHS member i (i=0, 1, 2)
- ukupna širina pravokutnog kutijastog profila štapa i (i=0, 1, 2) izvan ravnine rešetkastog nosača

$b_{e,p}$ - effective width for punching shear
- djelotvorna širina pri posmičnom proboju stijenke profila

C_e - efficiency parameter, varies depending on the type of joints (T, X, K)
- koeficijent učinkovitosti, razlikuje se ovisno o tipu spoja (T, X, K)

e - eccentricity of a joint
- ekscentricitet u spojevima

f_{yi} - yield strength of member i (i=0, 1, 2)
- granica popuštanja za štap i (i=0, 1, 2)

$f(n)$ - function prestressed chord
- funkcija prednapetosti pojasa

g - gap between the brace members in a K or N joint, measured along face of chord between the toes of brace members
- razmak između štapova ispune u K ili N spojevima, mjereno između rubova zavara na štapovima ispune duž lica pojasa

h_i - overall in-plane depth of RHS member i (i=0, 1, 2)
- ukupna visina pravokutnog kutijastog profila štapa i (i=0, 1, 2) u ravnini rešetkastog nosača

t_i - wall thickness of RHS member i (i=0, 1, 2)
- debljina stijenke pravokutnog kutijastog profila štapa i (i=0, 1, 2)

$N_{i,Rd}$ - design value of the resistance of the joint, expressed with internal axial force in member i (i=0, 1, 2)
- vrijednost otpornosti spoja izražena preko uzdužne sile u štapu i (i=0, 1, 2)

β - ratio of diameter or width of brace members to that of the chord
- omjer promjera ili širina štapova ispune i pojasa

γ - ratio of the chord width or dameter to twice its wall tickness
- omjer širine ili promjera pojasa i dvije njegove debljine stijenke

η - ratio of the brace member depth to the chord dameter or width
- omjer visine štapa ispune i širine pojasa

θ_i - angle between brace member i and the chord (i=0, 1, 2)
- kut između štapova ispune i i pojasa (i=0, 1, 2)

to ductile behaviour and plastic reserves of steel, which justifies the application of aforementioned calculation model.

However, the articulated joint hypothesis is possible only if critical parts of truss girder (elements or joints) have sufficient rotation capacity. Then the secondary stress originating from bending moment can be neglected in calculations, viz. local joint effects can reduce rotation capacity of the elements and thus endanger the supposed global behaviour mechanism. Scientific research of plane-loaded truss joint behaviour began in the early sixties and is based on experimental testing with no further theoretical elaboration. Not until lately have the existing and new experimental data been associated with theory in greater detail or complexity. [3].

2. Truss girder calculation according to Eurocode regulations

In the new European steel construction design regulations [1] and especially its part [2] this issue is discussed in greater detail, giving basic outlines for design truss girders. Significant innovation is represented by detailed outline of joint calculation in truss girders. It discusses procedures of calculating static joint resistance in plane or space trusses made of circular, square, or rectangular hollow sections or a combination of open and hollow sections. Characteristic types of truss joints are discussed - *K, KT, N, T, X, Y, KK, TT, XX*, and *DY* joints. Calculation rules [2] are based on simplified analytical models combined with experimental testing, so the regulations are essentially consisted of semi-empirical calculations. In order to describe joint behaviour, it is necessary to note force path, material behaviour, and joint stiffness distribution. Mathematically, this is reduced to basic models of X and K joints and in general terms, brace members force components perpendicular to chord members are discussed, because these force components cause brace plastification. Resistance of particular joints is observable through maximum design resistance of truss brace members exposed to longitudinal force and/or bending moment. Thus, it is necessary to check potential failure locations during calculation process and ascertain possible failure modes of truss girders while considering local stiffness and joint behaviour. This way, truss chord optimisation is set as dimensioning goal while controlling joint stiffness and resistance, since chord members contain up to 3/4 of truss material (with usual truss systems). Special attention is directed towards compression chord, bearing in mind that joint resistance is increased with the decrease of chord member local slenderness.

In [2] several possible ways of truss girder joint failure are discussed:

- chord face failure (by plastification) or plastic failure of the chord cross-section,
- chord side wall failure or chord web failure by yielding, crushing, or instability under the compression brace member,
- chord shear failure,
- punching shear of hollow section chord wall (crack initation leading to rapture of the brace member from the chord member),
- brace failure with reduced effective width under crack in the welds or in the brace member,
- local buckling failure of the brace member or of a hollow section chord member at the joint location.

It should also be noted that in the present version of the regulations local slenderness of the cross section is limited more strictly in order to avoid local buckling. Sometimes, multiple criteria are met (for example, chord's shear resistance is included in chord plastification formulae, etc.), so the basic joint failure modes can be reduced to chord plastification and shear puncture. In order to avoid weld failure it is recommended that the welds be stronger than joined elements and that the material is not sensitive to lamellar tearing. For regulation application purposes, maximum thickness of hollow section wall is limited to 25 mm, while minimum thickness is 2,5 mm.

Ultimate joint resistance is set by defining either maximum load on the force diagram – deformation, or equivalent load level for preset deformation limit, which is defined according to contemporary works [3] as 3 % of chord member diameter, i.e. 3 % of chord member width in rectangular and square sections measured at the joint of brace member and chord member. For serviceability limit state of use the aforementioned deformation limit is 1 %.

Formulae for joint resistance are given in terms of important geometrical parameters which depend on the dimensional relationship of brace and chord members:

- parameter β, as a proportion of average value of brace member diameter/width relative to the corresponding measuring of chord member
- parameter η, as a proportion of brace member height relative to the chord member diameter/width
- parameter γ, as a proportion of chord diameter/width relative to double thickness of its wall.

This paper thoroughly discusses welded truss constructions made of hollow rectangular cross sections as common constructional solutions in practice, although some parts of the paper are applicable to other types of truss member cross sections. Welded joints are practical and thus commonly applied. However, force transfer is often complex due to non-linear stiffness distribution along the perimeter of the joined brace. When discussing rectangular hollow sections, difference in stiffness between section corners and the centre is even greater than with circular sections, which is why the calculations are more complex due to different section orientation possibilities. Consequently, multiple ways of chord face failure are possible.

Basic analytic resistance calculation model for this group of joints is based on a yield line model, Figure 1, which assesses plastification of chord members. It should be noted that these formulae are approximate and generally give higher stress values regarding chord plastification (good estimates are given for mean values of parameter β) The model is based on equalisation of external energy provoked by an external force on the deformation δ and internal plastification energy together with lengths of plastified lines and angles of rotation θ (which can be of different configuration, marked in the figure as “a” and “b”).

$$N_{\mathrm{i}}\, sin\theta_{\mathrm{i}}\delta = \sum l_{\mathrm{i}}\phi_{\mathrm{i}} m_{\mathrm{p}}, \tag{1}$$

where:

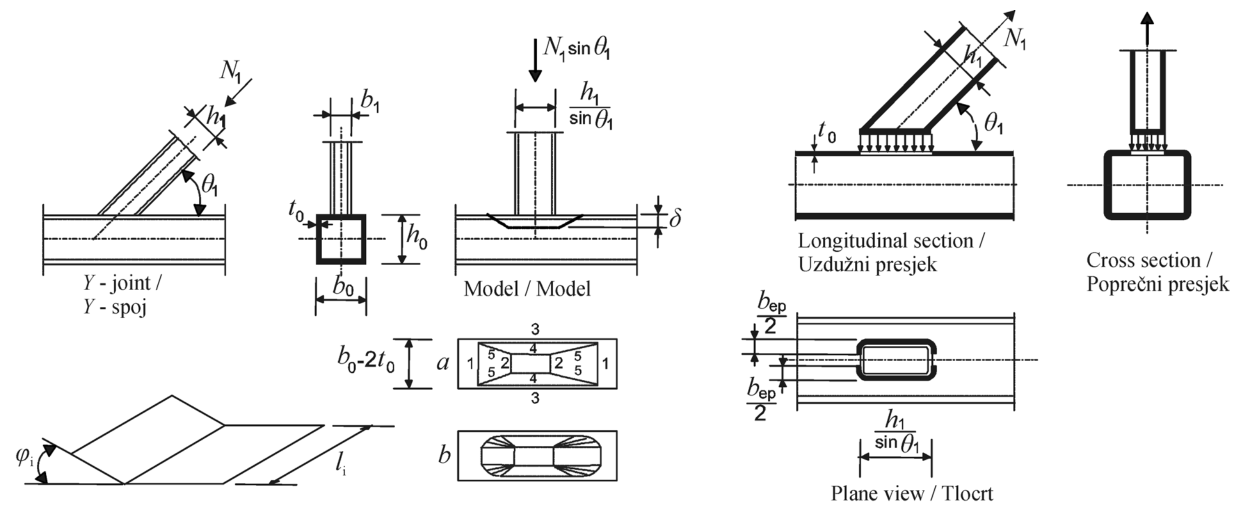
$m_p = \frac{1}{4}t_0^2 f_{y0}$,is calculated by unit length, and the meaning of other designations is visible in Figure 1.

This model is suitable for *T, Y*, and *X* joints, while it can exceptionally be used for *K* joints. When discussing *K* joints, membrane stress, shear stress, and hardening have great influence on their behaviour, so semi-empirical approach must be used for calculating this joint type.

Other important model discusses possibilities of brace member punching shear. Special attention should be given to the fact that the stiffness on some parts of brace member diameter is not even, so it is possible that some scope parts do not have sufficient deformation capacity to include the total perimeter of brace member profile. Thus, only effective surface is taken into calculations (in Y, T, and X joints, brace walls along chord member are the stiffest parts), and effective width value must be set experimentally.

Along these two characteristic failure models, problems with the yielding or buckling of chord member walls may occur, as well as chord shear failure.

It has been shown that experimental indicators give sufficient information on possible failure modes relative to parameter β, but the general impression is that there are many limitations in application of some formulae and at the same time many modes of failure. Because of this, in order to simplify calculations, a general approach is adopted where a narrower formulae validity scope is targeted in order to reduce the verification of resistance to one reliable proof. Also, the approximate graphs are



a) Plastic failure of the chord face / Plastifikacija pojasnog štapa

b) Punching shear / Posmični proboj

Figure 1. Characteristic failure modes of truss connections
Slika 1. Karakteristični načini otkazivanja nosivosti spojeva

given to preliminary assess joint effectiveness in the early designing phase so that the behaviour of truss joints and members is synchronised and that the calculation is facilitated. In these diagrams, joint resistance is defined as a fraction of plastic resistance of brace member:

$$eff = \frac{N_\mathrm{i}}{A_\mathrm{i} f_\mathrm{yi}} = C_\mathrm{e} \frac{f_\mathrm{yo} t_\mathrm{o}}{f_\mathrm{yi} t_\mathrm{i}} \frac{f(n)}{sin\theta_\mathrm{i}}, \tag{2}$$

where:

C_e - is efficiency coefficient that has different esignations for different joint types (C_T,C_X,C_K),

θ_i - is the angle between the brace member and the chord member,

f_{yo}, t_o - is the yield strength and wall thickness of chord member's cross section,

f_yi, t_i - is the yield strength and wall thickness of cross section of *i*-chord member,

$f(n)$ - is the member prestress function (functions as the maximum compression force in the chord for rectangular hollow sections).

In the preliminary truss dimensioning phase the aim is to ascertain which relationship (f_{yo} t_o/ f_yi t_i) should be foreseen while achieving 100 % joint efficiency in the process, i.e. that the joint's bearing capacity does not limit the bearing capacity of the members. Application example of this type of graph for gapped *K*-joints is given in Figure 5.

3. Bending moment influence on truss girders

According to [2], it is generally allowed to calculate internal forces in girder constructions presuming the existence of hinge links in truss joints. However, it is advised that additional influences be considered apart from bending moments, that are possible to neglect only in specific cases:

- secondary bending moments,
- bending moments due to transverse load between truss nodes, and
- bending moments due to eccentric member connection in joints.

Secondary bending moments are caused by the rotational stiffness of the joints. They generally depend on absolute and relative member stiffness, static system (i.e. conditions of angle alteration between members), and the magnitude of basic stress. Consequently, secondary stress intensity assessment must be associated with the applied calculation model of actual construction. For example, Figure 2 shows truss girder segment calculation model, supposing that the forces in members are of the same value and that the stiffness and area of the chord are significantly greater than the brace member stiffness. In the process, rigid link is presumed in the member intersection, while hinge links are presumed on the members' opposite ends.

By using a relatively simple stress and deformation analysis for this model, an expression can be derived for assessing secondary stress σ_s in brace members:

$$\sigma_\mathrm{s} = \frac{3}{2}\mathrm{ctg} \quad \alpha \cdot \sigma_\mathrm{o} \cdot \frac{h}{l}. \tag{3}$$

The structure of the abovementioned expression clearly shows that the term (ctg α) represents the contribution of a selected static system, σ_o is basic member stress, while h/l is the member stiffness parameter in the girder plane (h is the section height in the truss plane while l is the system member length). It should be noted that a different expression for secondary stress would have been derived if a different model was used (for example, a model implying rigid links on the intersection and on the free ends of members).

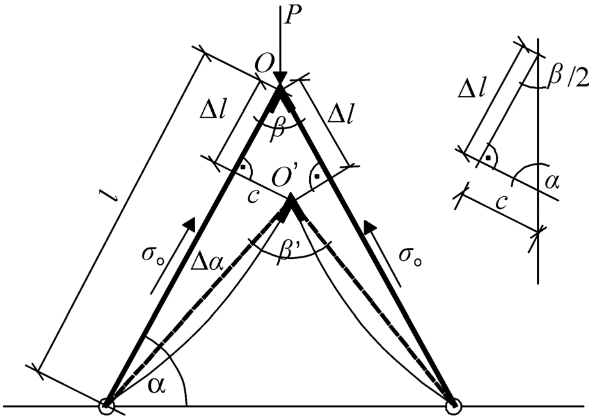


Figure 2. Example of design model for estimation of secondary stress
Slika 2. Primjer proračunskog modela za procjenu sekundarnih naprezanja

Expression 3 shows that secondary stress will be lower with members whose height in the truss plane h is smaller than the member length l, so this ratio is very important when assessing the magnitude of secondary stress due to rigid links. Thus, in [2] is also stated that the observed secondary stress can be neglected in the calculations if l/h ratio has corresponding values (for example, for truss constructions in building construction the minimum ratio value for the stress to be neglected is 6). It should be noted that in case of fatigue danger this stress can have significant influence and must be taken into account.

Moments originating from eccentric joint links can be neglected when calculating the resistance of chord and brace tension resistance, while in joint resistance calculations they can be neglected only if the eccentricity element e is in the following interval:

$$-0,55h_o \le e \le 0,25h_o, \tag{4}$$

where h_o is the height of chord member in the truss plane.

However, with compression chords the link eccentricity must always be taken into account. For eccentricities that fall under abovementioned parameters, they are taken only for compression chord resistance calculations in such a way that the total moment is distributed between chord members on each side of the joint proportional to their relative stiffness I/l (where l is system length of the member). If the eccentricity value e exceedes values given in the interval above, then it has to be taken into consideration with joint resistance calculations, and the total moment is distributed among all the elements connected in the node. Truss resistance of joints that are aditionally loaded with bending moments is generally resolved the same way as joints with axial stress, bearing in mind that the chord plastification and punching shear mechanisms are somewhat modified.

4. Numeric modelling of truss girders

Past analyses make it apparent that the truss girder behaviour modelling should consider global, as well as local behaviour of construction elements and joints. The Finite Element Method (FEM), depending on the requested accuracy and correctness of the model, differentiates betweeen:

- member model of girder joints with or without considering possible secondary influences, or influences due to joint eccentricity,
- space girder models which can be completely made of *shell* type space elements, or a combination of shell and *beam* elements,
- isolated truss girder joint models.

Beam model application is the simplest and the most acceptable in engineering terms, and it is generally possible to conduct linear or non-linear analysis (material and/or geometrical). The most widely applied model in practice is the one implying hinge links in nodes, when the result is only axial forces in members. Then the potential bending stress resulting from more precise analyses is considered secondary, since it does not influence the balance and is generally possible to be neglected (if it occurs within reasonable limits). However, if the truss is made as a frame system (node link is “rigid” instead of hinge), then the bending stress calculated in this way is not seen as secondary stress, since it can influence the longitudinal force values in such models. In the case when the joint eccentricities are modelled, one of the models in Figure 3 is used. By selecting different links between brace members or between brace and chord members, link eccentricity bending moment can be influenced (in accordance with previously described moment reallocation [2]). With this type of modelling, it is not possible to consider local effects that appear in joints as consequence of their deformation characteristics, not even by applying non-linear (elastoplastic) analysis. In other words, if material elastoplasticity was taken into consideration, it would consider only stress redistribution (i.e. gradual bending moment decrease in chord elements due to rotational stiffness with simultaneous increase of longitudinal force), caused by ductile behaviour of steel.

Truss space models in which the truss elements are modelled by *shell*-type plane finite elements are very precise and they include global and local system behaviour. These models are used only for research purposes because of creation complexity of this model, which largely depends on truss cross section types, on the static system complexity (i.e. space truss), and the calculation duration (especially non-linear analysis).

An acceptable modelling approach could be the combination of beam and plane elements in such a way that the joint area is modelled by plane elements, and other areas by beam elements. The connection is made through rigid links (i.e. master-slave). This way the model creation complexity is significantly reduced, calculation results include both global and local joint element behaviour, and most importantly, modelled joints are exposed to actual boundary conditions existing in the construction itself. Aforementioned advantage is especially related to so-called isolated models that include only specific (usually critical) joints, but only when it is impossible to fully include actual influences that appear in that joint at its actual location in the construction [4].

5. Numeric example

Truss girder modelling types using FEM shall be explained and exemplified by means of a simple truss construction. Calculation results shall be critically analysed and compared in relation to the codified approach [2].

Truss sketch together with measures, used cross sections, and load distribution is shown in Figure 4. Initial force is 200 kN, and the presented calculation results show the factor which is multiplied by initial force in order to assert actual load. Steel quality is S235, and the models presume idealised elastoplastic steel behaviour diagram. Welding material exhibits the same properties as the basic material and sufficient weld size is presumed, so it is not necessary to consider this mode of failure. In the process of overlap joints and gap joints construction, gap and eccentricity tollerance guidelines given in [2] were followed. Software packages used for modelling are Autodesk Robot [5] for beam model construction, and ADINA [6] for space model construction (shell-type finite elements with 4 nodes were used).

Characteristic truss details are given in Table 1.

Figure 5 shows the procedure of approximate definition of critical joint efficiency (joint 4 in Figure 2) by using preliminary diagrams. Table 1 data show that the critical joint in the truss girder is joint 2 in Figure 4, since the calculated load (according to linear theory) achieves its 100-percent efficiency, while other joints under same load are less used. The relevant failure mode in the process was chord member face plastification. Force in compression brace member 1 for achieving 100-percent efficiency in joint 2 was $N_{1,\mathrm{Rd}}$=153.7 kN. Figure 5 shows that by applying approximate procedure, with the aid of finished diagrams, a more conservative force for this compression diagonal can be derived ($N_{1,\mathrm{Rd}}$=137.5 kN).

Furthermore, by using FEM, the following calculations have been derived:

- non-linear truss calculations in beam models together with:
  - presumed rigid centric links between members,
  - presumed rigid overlap joints,
  - presumed rigid joints with gap.
- non-linear calculations on a 3D truss model made by using shell-type finite elements,
- non-linear calculations on the combined model consisting of beam elements and shell elements,
- non-linear calculations of isolated truss girder joints.

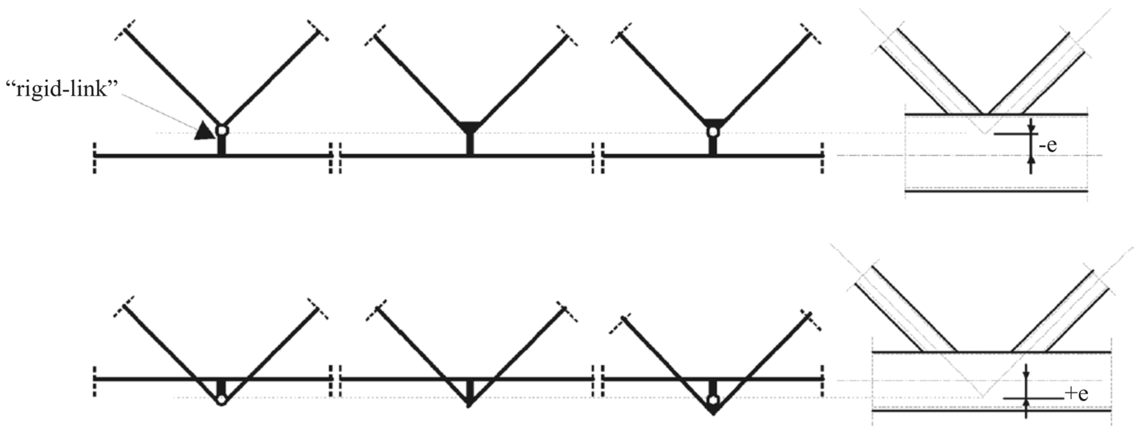


Figure 3. Various modelling methods of truss bar models
Slika 3. Varijante modeliranja rešetki štapnim modelima

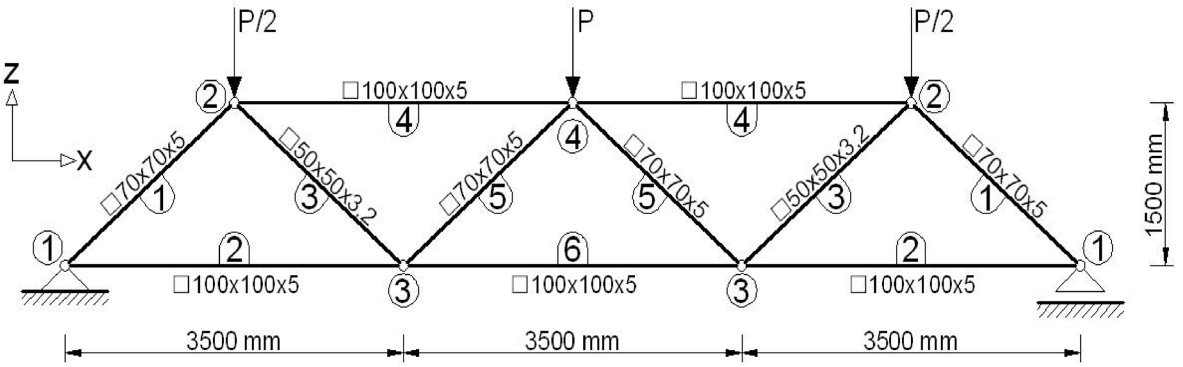


Figure 4. Schematic representation of truss girder used in analysis
Slika 4. Shematski prikaz rešetkaste konstrukcije korištene u proračunima

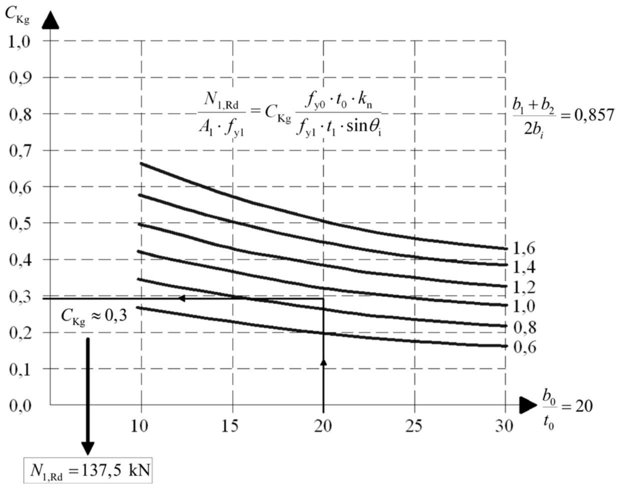


Figure 5. Diagram of efficiency of braces for K joints with gap
Slika 5. Dijagrami za određivanje učinkovitosti K spojeva s razmakom

Figure 6 shows force factor-displacement diagrams, where there is a global vertical displacement of joint 3 (z axis). It is visible that the results for various calculation models are very similar, which is expected considering that the eccentricity and secondary stress limitation guidelines were followed.

However, it is apparent that there is still a significant difference in the limit load of space and beam models due to possible inclusion of local joint behaviour in the space models (Figure 7). Consequently, according to 3D space models, the force in critical diagonal 1 is $N_{1,Rd}$=229,8 kN, while for beam models it is $N_{1,Rd}$=289,6 kN (but that force is actually related to global failure of diagonal 1 and not its joints). By comparing obtained results and the codified procedure it is visible that the formulae in [2] give somewhat conservative estimates of joints resistance (the difference is around 33 % compared to 3D space truss models), which is expected considering predescribed suppositions for their defining. Similar observations can be found in other works, for example [7].

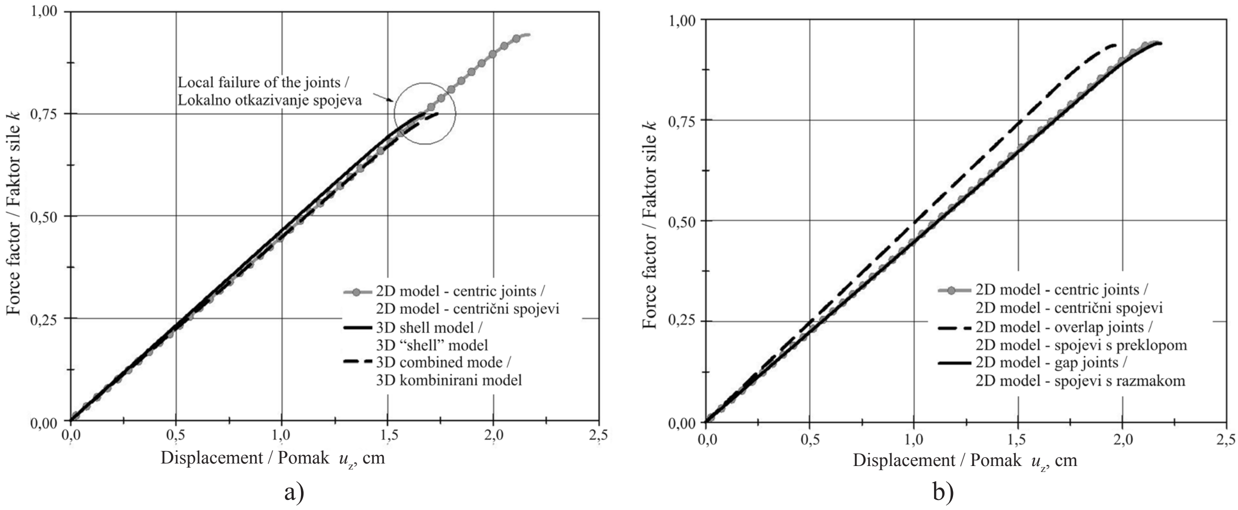


Figure 6. Force factor – displacement diagram for analyzed models
Slika 6. Dijagram faktor sile - pomak za korištene modele

It is also visible that the results for the combined 3D model and the full 3D shell are almost identical (Figures 6a and 8), which justifies this type of modelling example since it is much simpler. It should be noted that for exaple purposes all 3D combined model joints are modelled by shell elements, but that does not have to be the case. It is sufficient to model critical details that are easy to determine preliminarily (as described earlier).

Table 1. Codified design of characteristic truss connections according [2]
Tablica 1. Kodificirani proračun otpornosti karakterističnih detalja prema [2]

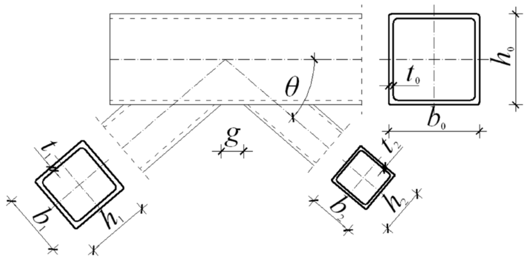
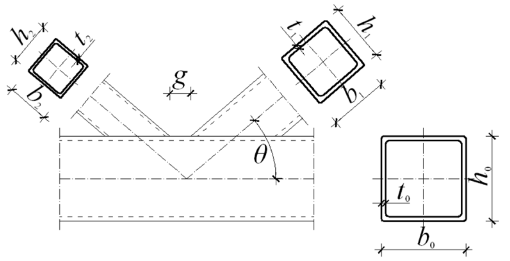
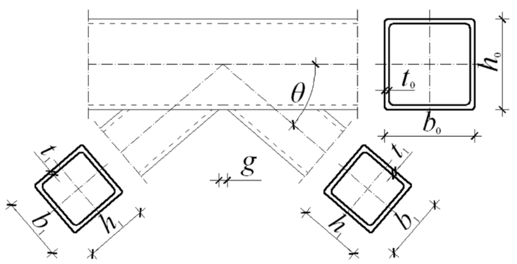
EFFICIENCY OF *K* - JOINTS / UČINKOVITOST *K* SPOJEVA		
PLASTIC FAILURE OF THE CHORD FACE / PLASTIFIKACIJA LICA POJASA $N_{\mathrm{i,Rd}} = \dfrac{\dfrac{8.9 \cdot k_{\mathrm{n}} \cdot f_{\mathrm{y0}} \cdot t_0^2 \sqrt{\gamma}}{\sin\theta_{\mathrm{i}}}\beta}{\gamma_{\mathrm{M5}}}$ $k_{\mathrm{n}} = 1,3 - \dfrac{0,4 \cdot n}{\beta},\ k_{\mathrm{n}} \le 1,0$ $\gamma = \dfrac{b_0}{2\,t_0}$ $f_{\mathrm{y0}} = 235\ \mathrm{N/mm^2}$ $\beta = \dfrac{b_1 + b_2 + h_1 + h_2}{4 b_0}$		
	JOINT 2 / SPOJ 2 $f_{y0} = f_{y1} = 235\ \mathrm{N/mm^2}$ $b_0 = h_0 = 100$ m $t_0 = 5{,}0$ mm $b_1 = h_1 = 70$ mm $t_1 = 5{,}0$ mm $b_2 = h_2 = 70$ mm $t_2 = 3{,}2$ mm $\theta = 40{,}60°$	$k_n = 1{,}035 \rightarrow k_n = 1{,}00$ $\gamma = 10{,}00$ $\beta = 0{,}60$ $N_{1,\mathrm{Rd}} = 152{,}45$ kN **100%**
	JOINT 3 / SPOJ 3 $f_{y0} = f_{y1} = 235\ \mathrm{N/mm^2}$ $b_0 = h_0 = 100$ m $t_0 = 5{,}0$ mm $b_1 = h_1 = 70$ mm $t_1 = 5{,}0$ mm $b_2 = h_2 = 70$ mm $t_2 = 3{,}2$ mm $\theta = 40{,}60°$	$k_n = 0{,}964$ $\gamma = 10{,}00$ $\beta = 0{,}60$ $N_{1,\mathrm{Rd}} = 144{,}22$ kN **53,3%**
	JOINT 4 / SPOJ 4 $f_{y0} = f_{y1} = 235\ \mathrm{N/mm^2}$ $b_0 = h_0 = 100$ m $t_0 = 5{,}0$ mm $b_1 = h_1 = 70$ mm $t_1 = 5{,}0$ mm $\theta = 40{,}60°$	$k_n = 1{,}072 \rightarrow k_n = 1{,}00$ $\gamma = 10{,}00$ $\beta = 0{,}70$ $N_{1,\mathrm{Rd}} = 177{,}86$ kN **43,2%**

Figure 9 shows details of joint 2 isolated model, and Figure 10 shows parallel results of 3D *shell* force-displacement results, the combined model results, and the isolated model results. The designation $u_{z,r}$ marks the greatest relative shift of the chord member wall with welded brace members relative to the opposite wall.

A relatively good diagram overlap can be noticed, and once more the emphasis should be put on the importance of the proper boundary condition modelling in isolated node models.

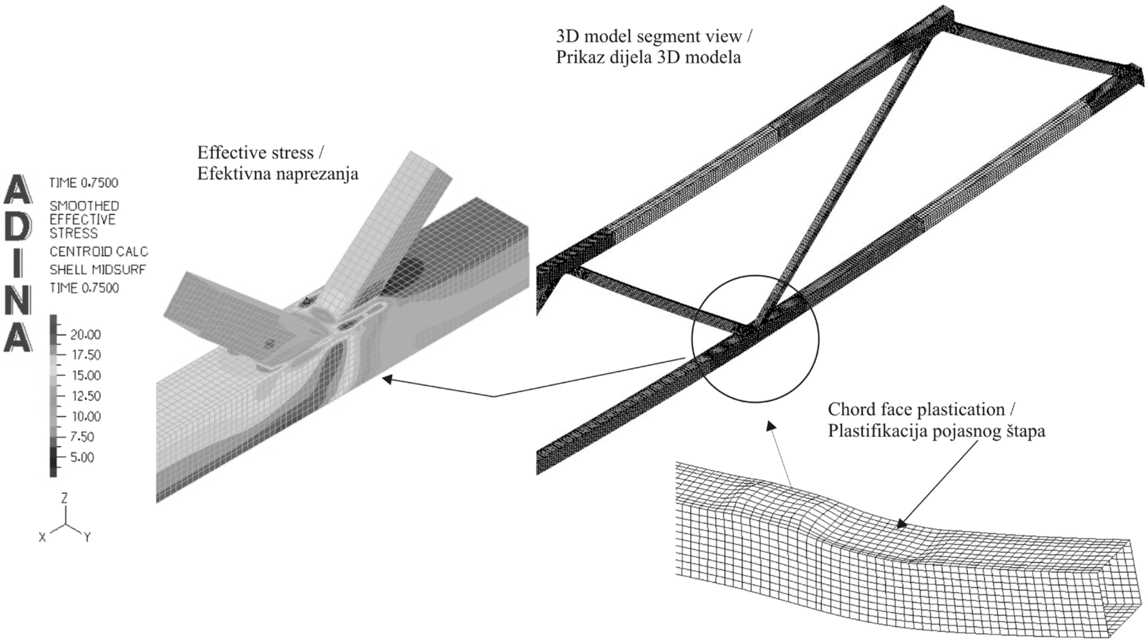


Figure 7. Part of 3D model (connection 3)
Slika 7. Prikaz dijela 3D modela (spoj 3)

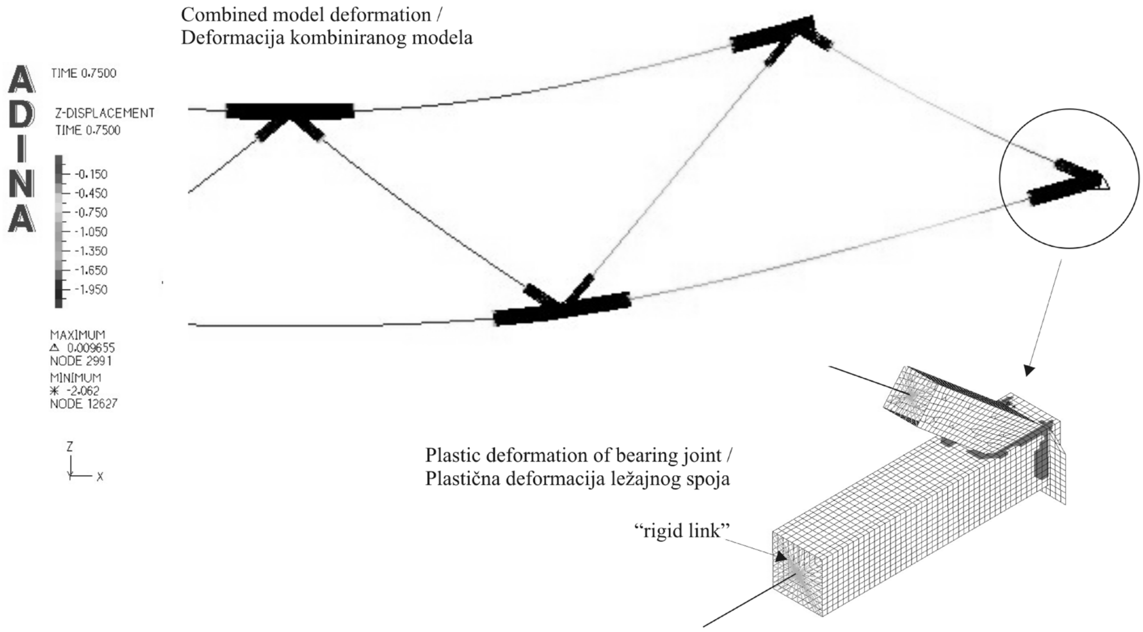


Figure 8. View of the combined model
Slika 8. Prikaz kombiniranog modela

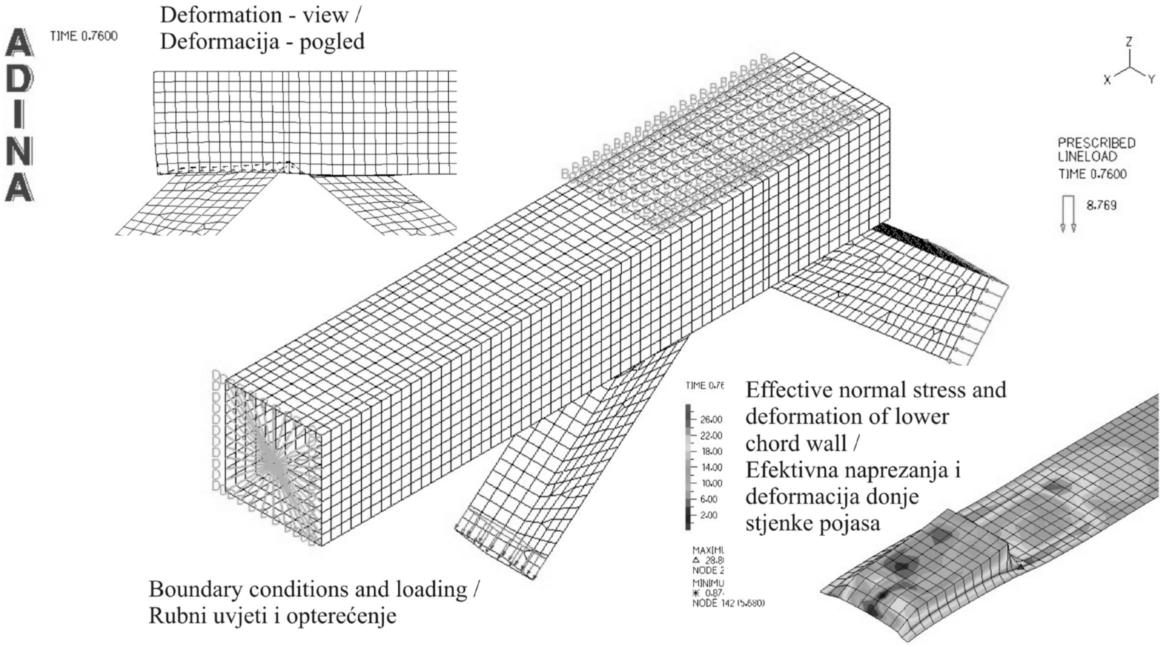


Figure 9. View of the isolated model of connection 1
Slika 9. Prikaz izdvojenog modeliranja detalja 1

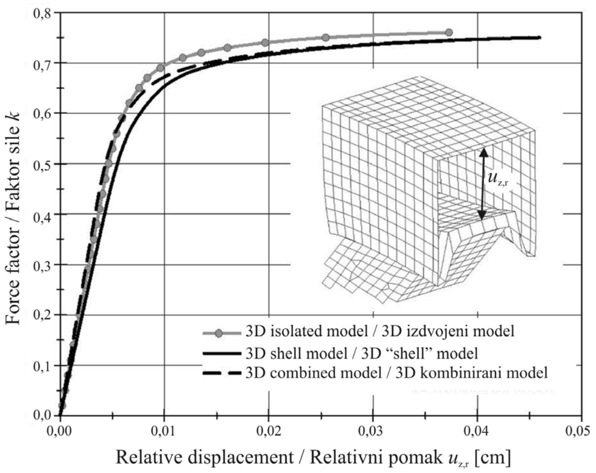


Figure 10. Diagram force – relative displacement for various models
Slika 10. Dijagram sila-relativni pomak za različite modele

6. Conclusion

The paper gives a survey of truss girder calculation model according to new Eurocode regulations [1, 2]. Joint design rules in [2] are set in semi-empirical formulae and are valid in very limited dimension and geometry conditions of truss girders. Therefore the need of more detailed calculations arises frequently in engineering practice, especially on local joint level. Thus, this paper gives and comments on modelling modes of this construction type using the Finite Element Method (FEM). All modelling possibilities have been analysed on a numeric example of a simpler truss and the calculation results have been compared. Bilinear steel behaviour has been preset and non-linear calculations have been conducted.

Calculation results point to possible areas of individual model application, as well as common need to use combined models considering the global and local construction behaviour. It especially concerns cases not falling under strict tollerancies and dimensional limitations of Eurocode regulations. This system, after using FEM, exhibits slightly greater limit stress values than is bound by regulations.

Based on the aforementioned findings, the final conclusion is that, considering a relatively narrow area of regulation validity [2] and its conservativeness, this area requires further experimental and numeric tests.

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