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Mechanic Analysis of the Key Node in Steel Truss Bridge

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Abstract

Steel truss bridges are frequently used in bridge engineering because of their good ability of spanning capacity, construction and light self-weight. Main trusses are the critical component of steel truss bridge and the main truss are made of truss members linked by integral joints. This paper presents the mechanic performance of key joints, and the codified design of joints in steel truss girders according to the latest European norms. The results showed that the fatigue resistance of welded joints evaluation is necessary to predict, detect, and repair the crack in time for the safety service life of the bridge. The stresses of integral joint are greater than that of truss members; the stresses in the center area of the integral joint are greater than the stress at the edge.

Keywords

Welded Joint, Fatigue Resistance, Finite Element Method, Crack, Stress Intensity Factor, Steel Truss

1. Introduction

Truss is a structure of connected elements forming triangular units, and a bridge whose supporting superstructure is consisted of a truss is a truss bridge. Truss bridges are one of the oldest types of modern bridges. The girder is made of different truss components, such as the chords, vertical posts, diagonal elements, etc. The components are linked through welding and/or using high-strength bolts. In order to ease the calculation, trusses are usually assumed as pinned connection between adjacent truss members. Therefore, the truss elements like chords, verticals, and diagonals only act in either tension or compression. For modern truss bridges, gusset plate connections are generally used, then bending moments and shear forces of members should be considered for the evaluation of the real performance of the truss bridges, which is obtained by the aid of finite element software. The FEM, appeared in about 1965, was a revolution in structural analysis, because it allowed the analysis of structures of any shape, any support conditions and any loading. However, in the case of nonlinear structural analysis, physical (of stress-strain law) or geometrical one, the FEM exhibits some problems. Even in the linear case, the local stiffness matrices of FEM are complicated [1]. Previous researches on the nodes in steel truss have shown that according to the connection types, the nodes are classified to externally attached nodes, integral nodes and interpolated nodes. For the design point of view, however, the pinned connection assumption is considered for security concerns and also for simplifying the structural design and analyses. The researches are limited to the steel truss structural analysis and the performance of node but do not evaluate every type of connections.

To better understand the mechanic related to key nodes, understand connections in steel truss and choose the better one for the design, we will firstly have an overview about key node its mechanical performance, then we will study truss girder calculation based on Eurocode.

2. Key Node Overview

In steel structures, connections are very important to have the structural members connected together and have the structure function as a unit [2]. Connection types that have been explored over the years are bolted connection, riveted connection, welded connection, and welded and bolted connection [3].

2.1. Major Connection Types

1) Riveted Connection

Riveting is the particular method of connecting together pieces of metal. This process is conducted by inserting the ductile metal pins called as rivet into the holes of pieces to be joined and formed a head at the end of the rivet to prevent each metal piece from coming out. A rivet is made up of round ductile steel bar which is called as "shank" and with a head at the one end. It is made up of mild steel or high tensile steel. The riveted connections are nowadays obsolete. The understanding of this type of connections for the strength evaluation and rehabilitation for an older structure is essential. While the connection procedure for riveted connections is same as that of the bolted connections. The shank of the rivet is made up of the length to the extent through the different parts which is to be connected and with sufficient extra length for a second head to be made at the other end.

- Recurrent behavior of Riveted Connection across

Literature The recurrent behavior of riveted connection reiterating through several literatures are as follows:

a) The ultimate shear strength in riveted joints is mainly determined by the strength of the riveted materials

b) In riveted connections, stiffness of the joints, geometry of the joint, treatment and preparation of the surface, type of steel, clamping force of the fastener and the grip length have a way of affecting slip resistance.

c) Deformation of the riveted joints occurs due to the slipping of surfaces in contact after yield point is exceeded.

d) Deformation of the rivet body occurs when the end of the hole or shank and the rivet bears against each other

e) When plastic flow occurs in the connection, excessive deformation of end rivets takes place.

f) Deformation in riveted connection develops to shear failure when there is sufficient ductility.

g) Failure mode in riveted connections could be summarized into rivet shear, tension on net section and bearing at rivet holes of thinner plates [4].

2) Bolted Connection

A bolt is a metal pin with a head formed at one end and the shank threaded at the other end so that nut can be received. Generally, the bolts are used to connect the pieces of metals by inserting them through the holes in the metals; at the threaded end, nuts should be tightened. This connection has the advantage of flexibility in assembling parts of the structure as well as dissembling it and which is necessary if there is inspection or some routine maintenance. This type of connections is applicable for members subjected to tension or shear or both tension and shear.

Among the types of bolt connections that exist, the bearing bolted connection is the common and most popularly used connection [5]. The classification of bolted connections can be based on their behavior or reaction to geometry or loading conditions.

- Behavior of Bearing Bolted Connection

Tension in the connected members is equilibrated through the bearing stress between the bolt and the hole drilled in the plate a bearing bolted connection. In a bearing bolted connection, there is no mobilization of the bearing stress until the plates slip relative to each other and bearing is kick started on the bolt [6]. The point of critical stress in bearing bolted connection is labelled section x-x. In a bearing bolted connection, for failure to occur, the failure either takes place in the plates connected or in the bolt itself. When bearing connections are used, the structural behavior stays linear till the following happens:

- Failure of plate occurs
- Occurrence of block shear failure
- Occurrence of failure in the bolt
- Yielding of the net section of the plate due to subjection to both flexure and combined tension
- Occurrence of shear at the bolt shear plane

3) Welded Connection

Welded connection among other types of connections is very efficient in the transfer of forces from a member to another member. A typical representation of welded connection is as represented in the Figure 1. In a welded connection, the connection is formed when a melted base metal joined with the weld metal cools. The connection is classified into either fillet or butt welds. The fillet in Figure 3 is welded at two surfaces while the butt is achieved by welding two surfaces together.

- Behavior welded Connection

The behaviors of welded connection are usually expressed in lap joints splices, shear is the main design consideration, side fillets and end fillets, end fillet loaded in tension—high strength and low ductility, side fillet loaded—Limited to weld shear strength (50% tensile strength) Improved ductility and average stress in weld throat.

2.2. Analyze Research on Nodes

The node state can directly affect the serviceability performance of steel truss girder bridge. Therefore, it is of great significance for bridge working to have analysis research on nodes. As shown in Figure 2, Zhang Qiang *et al.* utilized finite element software ABAQUS to have analysis on nodes to get the conclusion that the increase in the thickness and size of the middle node caused the stress concentration to appear in the off-center variable section node [7]. Li Ying *et al.* used finite element software ABAQUS to have elastic analysis and elastic-plastic ultimate static analysis on top nodes and then worked out that improving vertical stiffening plate on nodes have no effect on node ultimate carrying capacity [8].

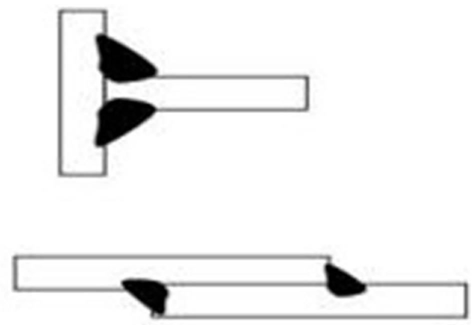

Figure 1. Fillet welds.

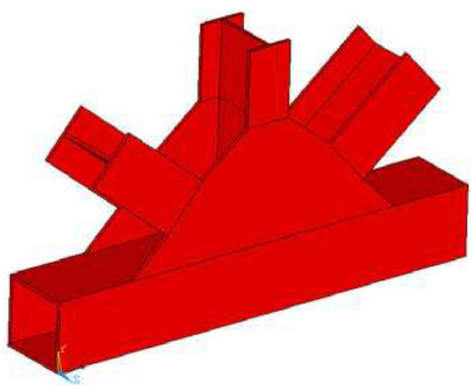

Figure 2. FE model of the major joints.

It is known that fatigue cracks may occur in the welded joints under the long-term dynamic loading as shown in **Figure 3**. When the chord members of a steel truss are connecting with the gusset plate by welding, the key joints of the steel truss bridge are in a complex stresses state. Under the long-term dynamic loading, it may cause the brittle fracture and even lead to collapse of the bridge [9]. Hence, it is important to study the fatigue crack for the safety of the steel truss bridges.

The welded joints between the flange and gusset plates are subjected to tensile stresses and other adverse effects on fatigue resistance, which lead to fatigue damages. In general, the adverse effects comprise stress concentration, weld defects and residual stresses at welded joints [9] [10] [11] [12]. Under cyclic loading, cracks tend to initiate at the welded joints and compromise the long-term durability of the truss bridge. The effects of stress concentration, weld defect and residual stresses on fatigue resistance of welded joints have been examined [9]-[17]. The stress concentration of steel truss joints has been studied through finite element analysis [9] and experimentation [10]-[17]. Different methods for evaluating fatigue resistance of welded joints have been suggested [18] [19] [20], with consideration of stress concentration and weld residual stress. Nominal stress method is recommended by several design specifications for steel structures, such as Eurocode 3 [18]. To solve engineering problems of crack by elastic fracture mechanics, it should first consider the stress intensity factor [21]. As for it, there are lots of researches on stress intensity factor. China aeronautical establishment compile a manual of it [22]. However, there are many stress intensity factors for irregular structure which cannot be found in the manual. With the development of computer technology and finite element method, the crack can be by simulated with finite element software such as ANSYS, **Figures 3-8** give an

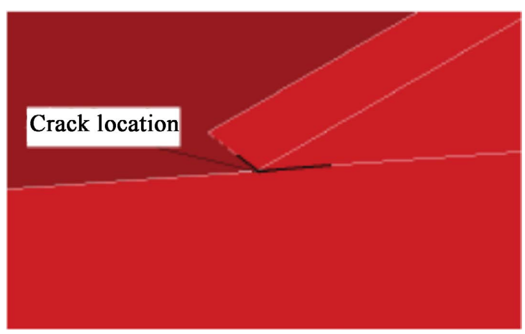

Figure 3. Crack location.

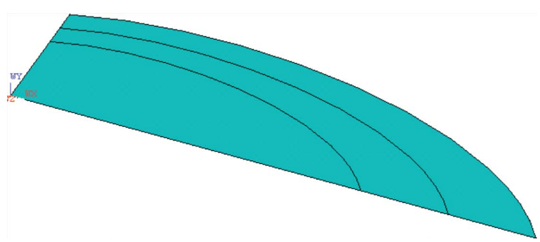

Figure 4. Three elliptic areas.

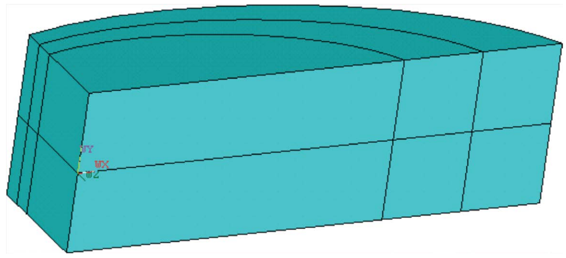

Figure 5. Cracked part.

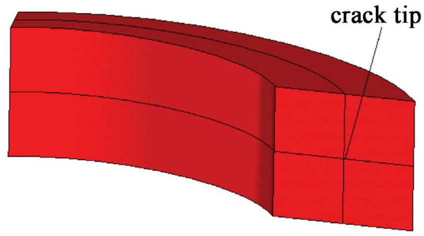

Figure 6. Crack tip.

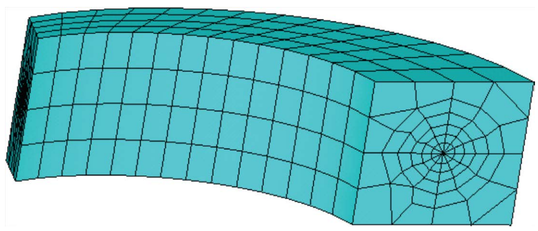

Figure 7. Mesh the cracked party.

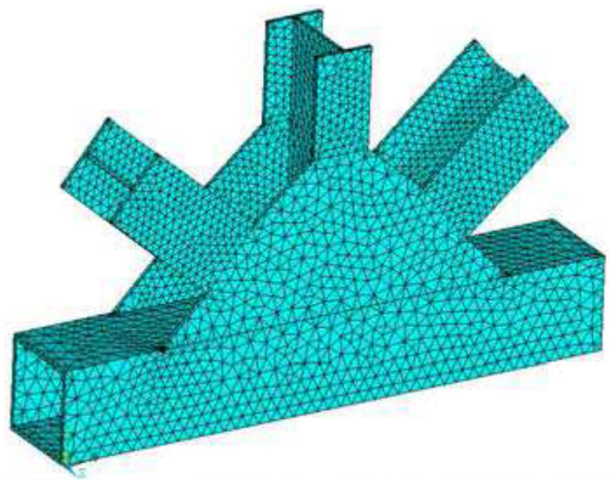

Figure 8. Meshing the other parts.

illustration to the process of finite element methode. Therefore, the first step is model the structure with cracks using ANSYS, then calculate the stress intensity factor.

3. Calculation of the Stress Intensity Factor

There are two models for the calculation of stress intensity factor. One is multiple degrees of freedom propagation model (**Figure 9(a)**), which has the advantages of high accuracy but in slow operation speed. The other is two degrees of freedom propagation model (**Figure 9(b)**), which is more accordant with reality and convenient for ANSYS calculation. Therefore, the stress intensity factors are calculated by it.

It is known that the stress intensity factor keeps increasing with the increased crack length. It is very convenient for us to establish node with different crack length. In solid modeling method, it can only change the initial parameter in APDL commands of ANSYS. According to the measuring result in the bridge. The cyclic stress amplitude is shown in reference [22]. It assumes the initial length of the crack is 2 cm and 1 cm in two directions. the result of stress intensity factor can be seen in **Table 1**.

4. Truss Girder Calculation Based on Eurocode

Typical truss girder analyzation or calculation model implies hinge links on joint location, *i.e.*, grid members are influenced only by axial forces. This way eases calculations considerably in addition to being based on the tradition of girder design and construction. Deviations from centric connecting are frequent, partly due to physical inability to create fully centric links, and partly to ease structure construction.

The joints method determines forces at the truss joints or nodes using FBD's (free body degree). The general assumptions for this method are:

1) All truss elements are estimated rigid, they never bend.

2) A force applied to the truss structure will only produce compression or tension on the elements.

3) Tension—compression forces' directions are parallel to the elements.

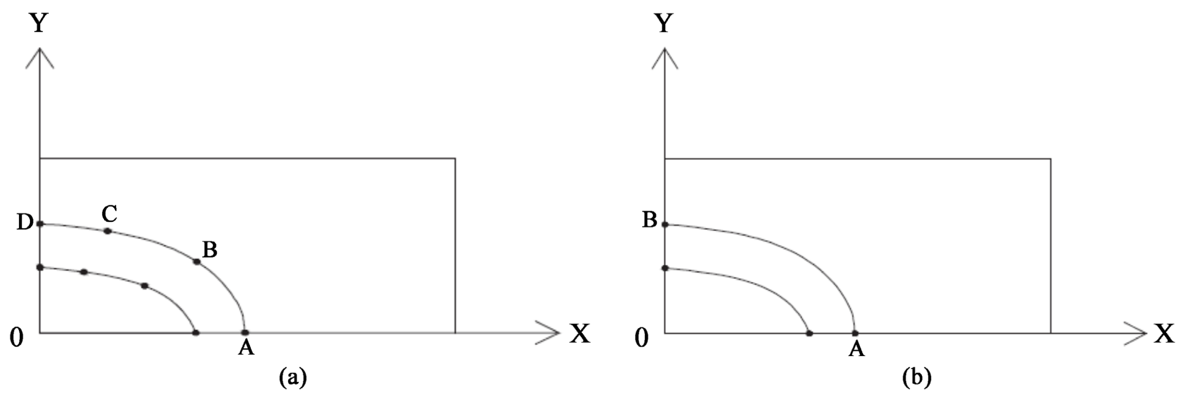

Figure 9. (a) Multi-degree of freedom crack (cm); (b) Two degree of freedom crack (cm).

Table 1. Stress intensity factor ($N \cdot mm^{3/2}$).

	K1	K2	K3
A	156.6	103.5	21.3
B	135.7	85.4	15.6

4) Any force on a truss element is transmitted to its ends.

5) A truss structure in equilibrium means that every joint or node is at equilibrium.

6) Once determined the value of a tension or compression force at one of the ends of an element, the complementary force at the other end of the element will be equal but in opposite direction. (Equilibrium condition).

In the new European steel construction design regulations [23] and especially its part [24] Significant innovation is represented by detailed outline of joint calculation in truss girders. procedures of calculating static joint resistance are discussed. Calculation rules [24] are done from simplified analytical models associated to experimental testing, so the regulations are essentially consisted of semi-empirical calculations. Resistance of particular joints is apparent through maximum design resistance of truss brace members under longitudinal force and/or bending moment. Thus, it is necessary to check probable failure locations during the process of calculations and ascertain possible failure modes of truss girders while considering local stiffness and joint behavior. This way, truss chord optimization is set as dimensioning goal while controlling joint stiffness and resistance, as chord members contain up to 3/4 of truss material (with usual truss systems).

Particular attention is orientated towards compression chord, having in mind that joint resistance is increased with the decrease of chord member local slenderness. In [24] various possible ways of truss girder joint failure are discussed:

- chord face failure (by plasticization) 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 initiation 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 this version of the regulations local slenderness of the cross section is strictly limited in order to avoid local buckling. Sometimes, many criteria are met (for example, chord's shear resistance is included in chord plasticization formula, etc.), so the basic joint failure modes can be reduced to chord plasticization and shear puncture. In order to avoid weld failure, it is advisable that the welds be stronger than joined elements and that the material is not sensitive to lamellar tearing. For regulation application sakes, maximum thickness of hollow section wall is limited to 25 mm, while minimum thickness is 2.5 mm.

Ultimate joint resistance is made by defining either maximum load on the force diagram—deformation, or equivalent load level for preset deformation limit, which is defined based on contemporary works [25] 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 above-mentioned deformation limit is 1 %. Formula for joint resistance is 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 function to the corresponding measuring of chord member
- parameter η, as a proportion of brace member height function to the chord member diameter/width
- parameter γ, as a proportion of chord diameter/width function to double thickness of its wall.

This paper exclusively discusses welded truss constructions made of hollow rectangular cross sections as usual constructional solutions in practice, although some parts of the paper are useful to other types of truss member cross sections. Welded joints are practical and thus frequently applied. However, force transfer is usually complex because of the non-linear stiffness distribution along the perimeter of the joined brace. Basic analytic resistance calculation model for this group of joints is based on a yield line model, Figure 10, which assesses plasticization of chord members. It should be noted that these formulas are approximate and generally give higher stress values regarding chord plasticization (good estimates are given for mean values of parameter β) The model is based on equalization of external energy provoked by an external force on the deformation δ and internal plasticization energy together with lengths of plasticized lines and angles of rotation θ

$$N_i\left(\sin\theta\right)\delta = \sum l_i \phi_i m_p \tag{1}$$

where: $m_p = \frac{1}{4} t_0^2 f_{y_0}$ is calculated by unit length, and the meaning of other

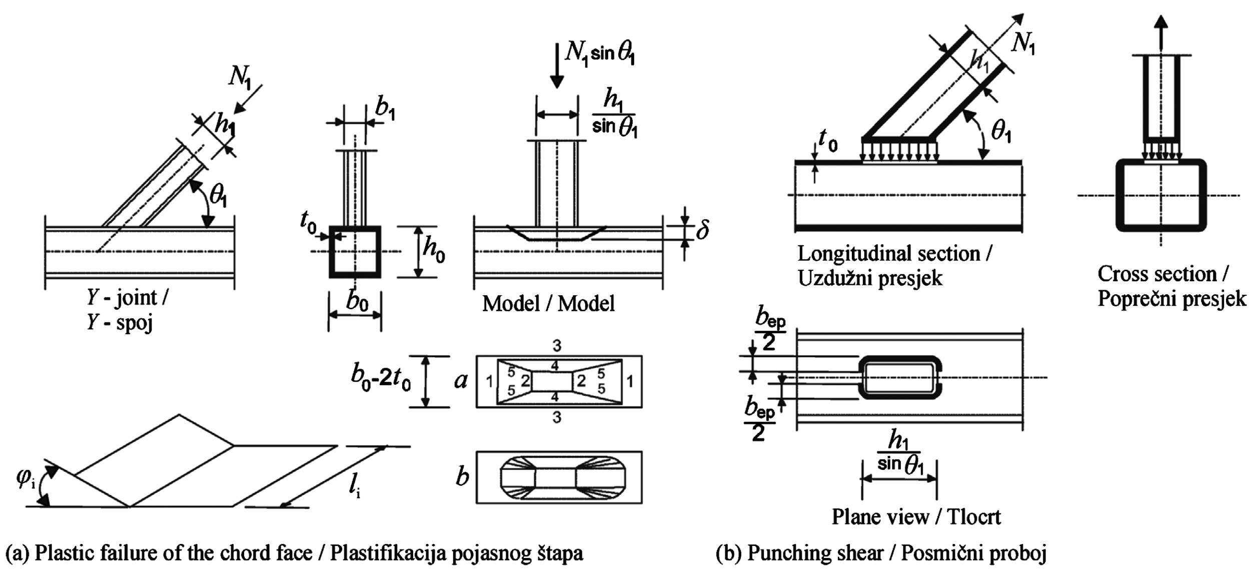

Figure 10. Characteristic failure modes of truss connections.

designations.

It has been proved 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 formula and at the same time many modes of failure. Thus, in order to ease calculations, a general approach is adopted where a narrower formula validity scope is targeted in order to reduce the verification of resistance to one reliable proof. Also, the approximate graphs are given to preliminary assess joint effectiveness in the early designing phase so that the behavior of truss joints and members is synchronized and that the calculation is facilitated. In these diagrams, joint resistance is defined as a fraction of plastic resistance of brace member:

$$eff = \frac{N_i}{A_i f_{y_i}} = C_e \frac{f_{y_0} t_0}{f_{y_i} t_i} \frac{f(n)}{\sin \theta_i} \tag{2}$$

where:

C_e—is efficiency coefficient that has different designations for different joint types (*CT*, *CX*, *CK*),

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

f_{y_0}, t_0 —is the yield strength and wall thickness of chord member's cross section,

f_{y_i}, 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 step, the goal is to ascertain which relationship ($f_{y_0} t_0 / f_{y_i} t_i$) 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.

5. Bending Moment Influence

According to [24], it is usually permitted to calculate internal forces in girder constructions supposing the existence of hinge links in truss joints. However, it is recommended to take additional influences into consideration apart from bending moments, that are able to be neglected 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 made by the rotational stiffness of the joints. They usually depend on absolute and relative member stiffness, static system (*i.e.*, conditions of angle alteration between members), and the magnitude of basic stress. That is why, secondary stress intensity assessment must be couple with the applied calculation model of actual construction. For example, **Figure 11** gives a look of truss girder segment calculation model, presuming that the forces in members are of the same value and that the stiffness and area of the chord are considerably greater than the brace member stiffness. During the process, rigid

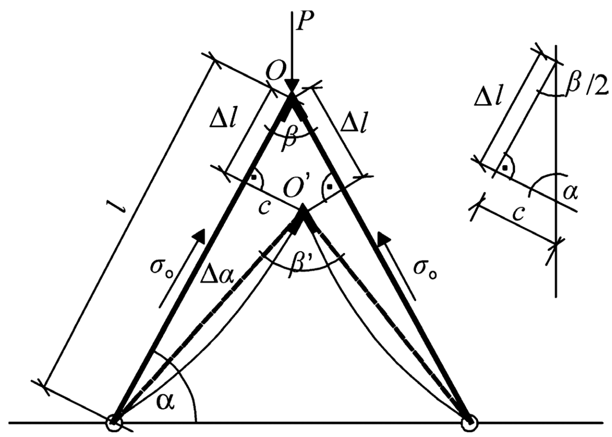

Figure 11. Example of design model for estimation of secondary stress.

link is supposed in the member intersection, while hinge links are supposed on the member's opposite ends. An expression can be derived by using a relatively simple stress and deformation analysis for this model to assess secondary stress σ_s in brace members:

$$\sigma_s = \frac{3}{2}\operatorname{ctg}\alpha \cdot \sigma_0 \frac{h}{l} \tag{3}$$

The structure of the aforementioned expression shows that the term (ctgα) represents the contribution of a selected static system, σ_0 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).

Equation (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 in the evaluation of the magnitude of secondary stress due to rigid links. That is why, in [24] is also indicated 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). We have to note that in case of fatigue danger this stress can have important influence and must be taken into account. Moments from eccentric joint links can be neglected when calculating the resistance of chord and brace tension resistance, whist in joint resistance calculations they can be neglected only if the eccentricity element e is in the following interval:

$$-0.55h_0 \le e \le 0.25h_0 \tag{4}$$

where h_0 is the height of chord member in the truss plane.

Nevertheless, with compression chords the link eccentricity have to be taken into account. For eccentricities that are under aforementioned parameters, they are taken only for compression chord resistance calculations so 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 exceeds values given in the interval above, then it should be considerate with joint resistance calculations, and the total moment is distributed among all the elements connected in the node. Truss resistance of joints that are additionally loaded with bending moments is generally resolved the same way as joints with axial stress, bearing in mind that the chord plasticization and punching shear mechanisms are somewhat modified.

6. Summary

In this paper, we can easily understand that key nodes are the most important components when it comes to steel truss bridge. The stresses of integral joint are actually greater than those of truss member. Therefore, key joints are subjected to varying tensile stresses and other adverse effects on fatigue resistance, and thus susceptible to fatigue damages due to the crack of the welded joint. The solid modeling method adopts the idea of breaking up the whole into parts. It was suitable to establish complicated configured model and could be carried on easily by change parameters in ANSYS which is simple and efficient to solve crack problem. Associated with the truss girder design calculation of different code (Eurocode, Chinese code etc.) we can easily predict, detect and repair crack and therefore guaranty the safety service ability of the bridge.

Conflicts of Interest

The author declares no conflicts of interest regarding the publication of this paper.

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Symbols

A_i—cross-sectional area of member i (i = 0, 1, 2)

b_i—overall out-of-plane width of RHS member i (i = 0, 1, 2)

$b_{e,p}$—effective width for punching shear

C_e—efficiency parameter, varies depending on the type of joints (T, X, K)

e—eccentricity of a joint

f_{y_i}—yield strength of member i (i = 0, 1, 2)

$f(n)$—function prestressed chord

g—gap between the brace members in a K or N joint, measured along face of chord between the toes of brace members

h_i—overall in-plane depth of RHS member i (i = 0, 1, 2)

t_i—wall thickness of RHS member 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)

β—ratio of diameter or width of brace members to that of the chord

γ—ratio of the chord width or dameter to twice its wall tickness

η—ratio of the brace member depth to the chord dameter or width

θ_i—angle between brace member i and the chord (i = 0, 1, 2)