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NODES OF HOLLOW CORE PROFILE TRUSS BRIDGES WITH INCOMPLETE WELDS

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Among the various types of steel bridge girders, trusses seem one of the most powerful and reliable. The individual members may be simple, but the nodes are the complex parts, especially if more than 4 members are connected. In view of the complexity, internal welds become inevitable and are inaccessible after finishing, which is overlooked easily during construction. The proposal is to eliminate those of the inside welds that may be dispensable. If some of them would be eliminated, the consequences may be acceptable, or require minor modifications only. The assumption is tried out on the example of a structure replacing a historic bridge. This structure is composed of rectangular hollow profiles, connected at nodes. Member forces and bending moments, have been determined. If the internal welds are about to be eliminated, an alternative load path is found. The calculation of stresses in the various fillet welds is based on the assumption of evenly distribution of forces. The weld stresses can be calculated by the derived formulas. The results show some welds require strengthening, due to the elimination of the more complicated internal welds. This concerns ultimate limit state. Thus, the possibility is confirmed that some of the internal welds may be eliminated, although there still is no proof the method may be applied to more complex nodes. Research must be continued to clarify this. For the type of hollow core members being considered, omitting of some internal welds may well be an acceptable alternative.

Keywords: Welded truss nodes, Eliminating internal welds, Alternative load path, Equal stress distribution in welds.

1 INTRODUCTION

Steel trusses are again being used often for smaller bridges and for larger roofs of buildings. Indeed, designers have found again the many advantages of trusses, one of these being this type of structure's strong character. This tendency is being fostered by the qualities of hollow core profiles for members, since the latter have the advantage of easy use and connection, with low amount of welding material. In addition, hollow core profiles have a cool view, which is in contrast to the assembly of plates and angle profile structures, produced in the 19th and early 20th century. Whether the RHS profiles be assembled by welding from plates, or are hollow core profiles, the simplicity of the fabrication of these nodes fosters this type of truss construction.

Thus, the design and construction of the truss bars is simple, but the connecting nodes are not. The latter are the complex part of truss beams, depending on the type of truss. The simplest of nodes are those found in the Warren-truss. This is due to the fact that no more than 4 bars are intersecting in this type. Warren trusses can successfully be applied for tubular arch bridges, as in Van Bogaert *et al.* (2015). There are types of trusses where 5 or more bars are intersecting at the nodes. The use of hollow core profiles may introduce another difficulty, since their resistance to lateral facial pressure is rather low and introduces transverse bending in the profile wall. This type of bending stress combines with axial stress and bending and can cause critical conditions in the thin-walled members. As a result, additional stiffening of the hollow core members at the nodes is necessary. The stiffeners can even be optimized to provide increased strength or fatigue resistance as shown by Stael *et al.* (2014). This implies that the fabrication truss nodes often requires welding in a well determined sequence. In addition some of locations welds cannot be reached for welding, and inspection of the welds, after completion becomes virtually impossible. This may very well lead to omission of certain welds and confusion with the fabricator.

In the present study, which has just started, the necessity of the inside welds is evaluated. Obviously, the main question is what would be the consequence should they be eliminated. This is demonstrated in the case of a unique truss bridge that serves as an example.

2 WELDED TRUSS NODES

The throat dimension of fillet welds is determined according to the recommendations for steel connections of EN 1993-1-8 (European Committee for Standardization 2005). This standard assumes that both normal and shear stress are evenly distributed across the weld length. This is in opposition to stress calculation by FE-element models, widely used to deal with this type of design. This implies there is no even distribution of the various stresses, which does not comply with the code. In addition, FE-models detect high stress concentrations, which are difficult to evaluate. The stress distribution in the fillet welds, as determined by the models of FE, is completely uneven and thus contradicts the above rule. Therefore, the approach is to use normal and shear force as well as bending moments instead of building continuous models. In this way, a consistent set of formulae can be derived that allow the welding stresses to be calculated. To illustrate the problem, Figure 1 shows the upper chord node of an 88 m long railway truss bridge. In this case, the welded parts are made of plates of different thicknesses.

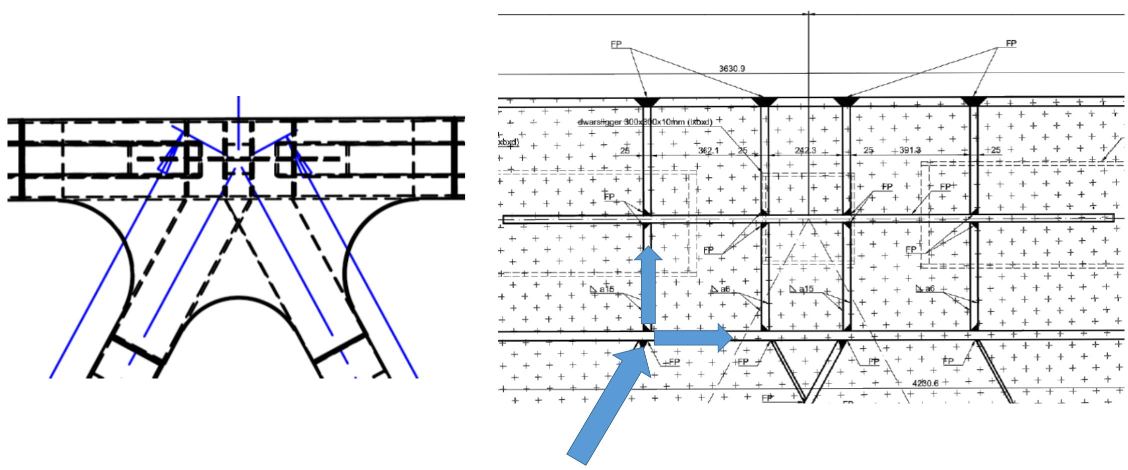

Figure 1. Force transfer in node.

The force carried by each plate is divided in a horizontal and vertical component at each meeting point with a horizontal flange and a vertical diaphragm stiffener. This is clearly indicated by the blue arrows in Figure 1. The diaphragm stiffeners are internal elements of the node. Thus a consistent transfer of forces is derived, each plate corresponding to a certain force. At each node the equilibrium of forces is established. In certain cases, there is no other way to connect the various plates than by welding from a single side or even infilled as exhibited un the right part of Figure 1.

According to the figures shown in EN 1993-1-8 connecting hollow core profiles by partial intersecting of the members is acceptable (European Committee for Standardization 2005). This implies that both the vertical and sloping bars are partially interrupted, provided the remaining parts be connected by welding and there is sufficient resistance to have the hollow core profiles not to be crushed in the direction perpendicular to their axes. This inevitably raises the question whether all these welded connections are always needed, especially the hidden or internal ones. There may very well be other connections, which allow different load paths for member forces. In the following example an alternative load path is found and the necessity of internal welds is examined.

The truss bridge, serving as an example to explore the effect of eliminating internal node welds, has been built to replace an almost identical structure that allows crossing the canal running from Pommeroeul to Antoing (South-West part of Belgium). This small canal was built in 1826. Hence the new bridge comes after 100 years of service of the former one. For some reason unknown the former structure was sometimes called the “Royal Bridge”. As heavy corrosion was found on the replaced bridge and in spite of the historic importance and the heritage value of the old bridge, the owner did decide the corrosion was too extended to have the bridge refurbished and choose to replace it. As can be seen in Figure 2 the level of the road surface does not coincide with the upper chord members, and neither to the lower ones. This unusual arrangement applies both to the old and the new bridge, thus complicating the analysis. Since the vertical traffic load is located above them, the vertical bars are compressed rather than to they are subject to tension. Figure 2 also exhibits the model for fabrication of the replacing bridge. In Figure 3 the fabrication in the workshop can be seen.

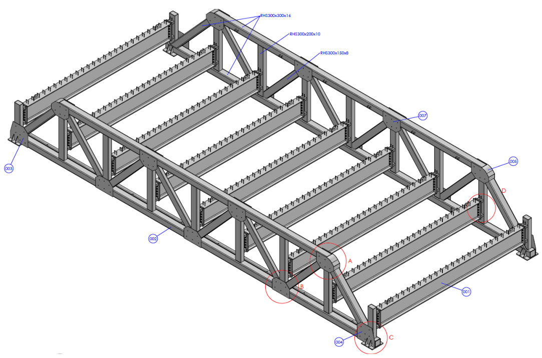

Figure 2. Fabrication model of new truss bridge.

Figure 3. New truss bridge in workshop.

3 ANALYSIS OF NODES

The FE-model of the bridge is similar to Figure 2. Beam elements have been in this model. In doing so, one may be certain to derive normal force and bending moments. As no upper bracing is present, lateral wind can introduce out of plane bending. Hence, the nodes are given identical stiffness as the truss bars.

Figure 4 shows a node connecting 5 truss bars, the lower chord member being continuous. The location corresponds to the 5 bar connecting node closest to the bearings. Hence this corresponds to the node Nr 3, counted from the support. The loads consist of the dead weight of the steel structure, the concrete road slab, the pavement, the traffic load represented by LM 1 from Eurocode EN 1991-2 (European Committee for Standardization 2004), and the effect of wind according to EN 1991-1-4 (European Committee for Standardization 2010). The relevant load cases are combined by multiplying the effects with the appropriate γ and ψ factors. It should be mentioned that effect of lateral wind was not affected by the relevant ψ-factor, thus somewhat overestimating this effect.

Table 1 exhibits the axial and shear forces as well as bending moments in the 5 truss bars connected at node 3, and expressed in kN or kNm. The x-axis is coincides with the axis of the member and the y axis is perpendicular to the plane of the truss. Since the nodes have equal bending stiffness as the bars, bending moments in the lower edge members are non-negligible.

The node 3, connecting 5 members includes several distinguishable welds. In Figure 4 they are named by letters from A to F. The fillet welds A and B connect the vertical bars to the bottom edge bar's top flange. The diagonal bars serve the same function as C and D, whereas E and F connect the latter to the vertical bar. Clearly, A are the only welds that become inaccessible once the node is assembled, and they may be removed or questioned as to their usefulness, because the member force of the flange of width d might transit via welds F to link with the lower edge member via welds D. The force of the vertical member's lateral plates of width d will be transmitted to weld E and, as a result, to the webs of the sloping member.

Table 1. Node 3: Forces in members.

Member force (kN or kNm)	**N**	**V_y**	**V_z**	**M_y**	**M_z**
Horizontal member left	1384.	-33.95	4.18	22.56	75.2
Horizontal member right	1393.	-24.41	5.02	20.89	53.7
Vertical	-576.6	8.60	-35.6	-27.47	-3.7
Sloping left	774.95	-4.62	-0.24	-0.04	-4.7
Sloping right	-53.24	2.22	-0.20	-0.99	-2.1

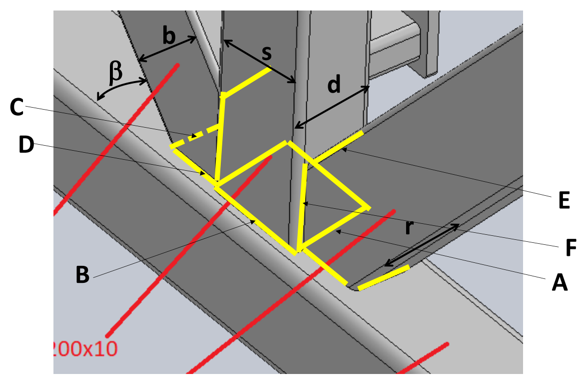

Figure 4. Various welds in 5-member node.

The angle β displayed in Figure 4 is the inclination of the sloping members. In addition, the figure shows the cross-section's dimensions of all RHS profiles. The flange thickness of each member is called t_b, t_d etc. The index of these symbols refers to the corresponding flange width. As commonly used, a is the dimension of each fillet weld's throat and is has an index according to its length. The cross-section area of the RHS-profiles is assumed to correspond to a rectangular section, which may somewhat overestimate the real value, because of the rounded angles. The thickness t_n may be modified as to identify the cross-section area to the real value.

Each member's normal force is distributed according to the cross section area, the 4 flanges of the profile are considered to be evenly distributed. Furthermore, compressed and tension flanges resist bending forces, with the contribution of webs being overlooked. The following equations for determining the stresses in the weld throats are derived using these assumptions.

$$\sigma_t = \tau_t = \left(\frac{N\,d\,t_d}{2\,d\,t_d+2\,s\,t_s} + \frac{M_y}{s}\right)\frac{\sqrt{2}}{2\,d\,a_d} \tag{1}$$

Normal and shear stress in of the weld are determined according to EN 1993-1-8 and have the same value (European Committee for Standardization 2005). Eq. (2) renders both stresses in weld B. The latter also follows directly from the method recommended by the standard.

$$\sigma_t = \tau_t = \left(\frac{N\,s\,t_s}{2\,d\,t_d+2\,s\,t_s} + \frac{M_y}{d}\right)\frac{\sqrt{2}}{2\,s\,a_s} \tag{2}$$

Similarly, the stresses in welds C and D are calculated from Eq. (3a) and (3b) or (4a) and (4b). However, the square root in Eqs. (1) and (2) must now be replaced by functions of the correct angle between the weld throat and the relevant.

$$\sigma_t = \left(\frac{N\,r\,t_r}{2\,r\,t_r+2\,b\,t_e} + \frac{M_y}{b}\right)\frac{t_r\,\sin(45°-0.5\,\beta)}{2\,b\,t_b\,a_c} \quad \text{(3a)}, \qquad \tau_t = \left(\frac{N\,r\,t_r}{2\,r\,t_r+2\,b\,t_e} + \frac{M_y}{b}\right)\frac{t_r\,\cos(45°-0.5\,\beta)}{2\,b\,t_b\,a_c} \quad \text{(3b)}$$

$$\sigma_t = \left(\frac{N\,b\,t_b}{2\,r\,t_r+2\,b\,t_e} + \frac{M_z}{r}\right)\frac{t_b\,\sin(45°-0.5\,\beta)}{r\,t_r\,a_b} \quad \text{(4a)}, \qquad \tau_t = \left(\frac{N\,b\,t_b}{2\,r\,t_r+2\,b\,t_e} + \frac{M_z}{r}\right)\frac{t_b\,\cos(45°-0.5\,\beta)}{r\,t_r\,a_b} \quad \text{(4b)}$$

Deriving throat thickness of welds E and F is more important, as the internal weld A is eliminated. Weld type E is represented by Eqs. (5a) and (5b), while weld type F is represented by Eqs. (6a) and (6b).

$$\sigma_t = \left(\frac{N\,r\,t_r}{2r\,t_r+2\,b\,t_b} + \frac{M_y}{b}\right)\frac{\sin(45°-0.5\,\beta)}{r\,a_r} \quad \text{(5a)}, \qquad \tau_t = \left(\frac{N\,r\,t_r}{2r\,t_r+2\,b\,t_b} + \frac{M_y}{b}\right)\frac{\cos(45°-0.5\,\beta)}{r\,a_r} \quad \text{(5b)}$$

$$\sigma_t = \tau_t = \left(\frac{N\,b\,t_b}{2r\,t_r+2\,b\,t_b} + \frac{M_z}{r}\right)\frac{\sqrt{2}\cos^2\beta}{2\,b\,a_b} \quad \text{(6a)}, \qquad \tau_l = \left(\frac{N\,b\,t_b}{2r\,t_r+2\,b\,t_b} + \frac{M_z}{r}\right)\frac{\sin\beta\cos\beta}{b\,a_b} \quad \text{(6b)}$$

Table 2. Weld stresses node 3.

weld	σ_t	τ_t	τ_l	vonmises
A	248.92	248.92		497.84
B	313.87	313.87		627.75
C	61.78	160.80		285.28
D	69.84	69.84	89.11	208.16
E	61.82	160.80		285.29
F	56.86	56.86	89.11	191.71

It is important to remember that in the weld F, some of the element's force is transferred into longitudinal shear. This is because the connection is not perpendicular to the axis of the bar. The various stresses in the welds, as well as the equivalent vonmises stresses, are shown in Table 2.

4 ELIMINATION OF INTERNAL WELDS

If the internal weld of type A does not exist, the force in the d-length flanges is passed on to the lower member as shear force via weld F, causing extra stress in D. Because the stiffness of this path is significantly higher than that of weld E, this alternate load path is more likely. It is obvious that a portion of the flanges of the vertical member does not contribute to transferring loads. This results in a raise of vertical stress in the remaining cross section. Furthermore, stress concentrations will appear, which will be determined by the radii of the profile rounding and detailing. As a result, prior to manufacturing, the alternative should be adequately prepared. Since this is a new solution, it is expected that careful detailing will make it practical to implement it, since buckling does not appear at this location.

For the bridge taken as example, the increase of stresses in the vertical member equals 86% as the value rises from 103.3 MPa to 192.4 MPa, which is still acceptable. This confirms the fact that the members of truss girders are seldom critical, since the nodes are more detrimental parts. If this also applies to other cases, internal forces can be adapted to alternative load paths. Consequently, the stress levels increase accordingly in welds F and D. Eq. (7) allows to determine the increase of vertical shear stress in weld F, and Eq. (8) renders the stress increase in D.

$$\Delta\tau_l = \left(\frac{N\, b\, t_b}{2s\, t_s + 2\, b\, t_b} + \frac{M_y}{s}\right)\frac{1}{b\, a_b\, \cos\beta} \quad (7) \qquad \Delta\sigma_t = \Delta\tau_t = \left(\frac{N\, b\, t_b}{2s\, t_s + 2\, b\, t_b} + \frac{M_y}{s}\right)\frac{\sqrt{2}}{2\, b\, a_d\, \sin\beta} \quad (8)$$

Table 3 exhibits the new values of weld stresses and shows welds D and F are to be increased. In practice the throat dimension should increase from 5 to 7 mm. This also closely corresponds to the strength of the parent metal. Therefore, in this particular case, internal welding can be omitted if a larger weld thickness is used, corresponding to the thickness of the parent metal. Yet this applies to ultimate limit state only.

Table 3. Modification of weld stresses node 3.

weld	s_t	t_t	t_l	vonmis	increase
B	313.87	313.87		627.75	0.00
C	61.78	160.80		285.28	0.00
D	293.35	293.35	89.11	606.66	1.91
E	61.82	160.80		285.29	0.00
F	56.86	56.86	301.90	535.12	1.79

5 CONCLUSION

It is implied that, certainly for the example truss bridge, internal welds of truss nodes may be eliminated. This requires reinforcement of other fillet welds. The absence of welds perpendicular to the lower edge member's flange and transmitted from the vertical member, in particular, eliminates any bending effect of the horizontal chord's upper flange. As a result, failure mode e of EN 1993-1-8 Table 7.4 cannot occur, which should encourage acceptance of the proposal. Due to a variety of factors, it is unclear whether the idea being offered has a broad scope. However, using Eqs. (1) to (8), one may calculate the various welds in this type of connection and see if internal welds can be avoided.

References

European Committee for Standardization, *Eurocode 1 Actions on Structures - Part 2 Traffic Loads on Bridges (+AC 2010)*, EN 1991-2, 2004.

European Committee for Standardization, *Eurocode 3 Design of Steel Structures - Part 1-8 Design of Joints (+ AC 2005 + 2009)*, EN 1993-1-8, May, 2005.

European Committee for Standardization, *Eurocode 1 Actions on Structures - Part 1-8 General Actions - Wind Actions (+ AC 2009 + 2010),* EN 1991-1-4, April, 2010.

Stael, D., De Backer, H., and Van Bogaert, P., *Determining the SCF's of Tubular Bridge Joints with An Alternative Method*, Journal of Constructional Steel Research, 101, October, 2014.

Van Bogaert, P., Stael, D., De Pauw, B., and De Backer, H., *Framed or Triangular Brace Arrangements in Tubular Arches*, Proceedings of IABSE Conference 2015, 62-63, Nara, Japan, May 13-15, 2015.