PACE-2021
International Congress on the Phenomenological Aspects of Civil Engineering
Research Article 20-23 June 2021

Usefulness of Internal Welds of Nodes in Hollow Core Profile Truss Bridges

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Keywords

Steel truss bridges,
Welding of nodes,
Weld stresses,
Alternative load path,
Truss node fatigue resistance.

Abstract

Steel truss structures are a powerful and reliable type of bridge girders. The complex part of trusses are the nodes, especially if more than 4 members are converging. Such nodes require providing internal welds; inaccessible after completion, which may be forgotten during assembling. This study assesses whether all of the internal welds are indispensable. The latter is tested for a particular bridge, consisting of hollow core profiles, intersecting at the nodes. After determining the member forces, including bending moments, an alternative load path is indicated, which must occur if the internal welds are missing. According to the relevant code, weld stresses are determined by evenly distribution of forces. Formulas are given to determine the weld stresses. In ULS some of the welds require a small amount of strengthening if the internal welds are omitted. In addition fatigue resistance was determined. Although the fatigue damage increases in this particular case it is still acceptable. Certainly for this example internal node welds may be left out, although this does not necessarily apply to more complex node cases. Further steps for research are given.

1.Introduction

In both smaller bridges and in larger roofs of buildings, steel trusses are again being used. This is related to the fact that designers have rediscovered the many advantages of trusses and also due to the strong character of this type of load-carrying structure. In particular, the use of hollow core profiles for the members is considered as an important factor. In addition, the latter have a neat view, in contrast to the rather complex members, produced in the 19th and early 20th century. Obviously, the simplicity of connecting the truss nodes is in favour of hollow core profiles, whether they be cold formed or assembled from plates.

If the truss bars can be simple, the character of truss becomes more complex for the nodes. The Warren-truss has the simplest nodes, since the number of members connecting at a node is maximum 4. This type of truss may also be used in arch bridges, especially of the tubular type (2015) [1]. In some trusses the number of bars, intersecting at the same node may reach 5. In addition, hollow core profiles may collapse due to the lateral pressure or tension perpendicular to their thin walls. This may require additional internal stiffening of the profiles at the nodes. The latter may require optimization of this type of stiffening (2014) [2] of as a result, truss nodes often need to be welded in a particular sequence and some of the welds cannot be reached, nor inspected after completion of the steel structure. The latter certainly complicates the work, and may be at the origin of misunderstandings and omissions.

The aim of the present study, still at initial phase, is to assess whether these inside welds are indispensable or what are the consequences should they be omitted. This is tested for the particular case of a rather unusual truss bridge, which serves as an example.

2. Welded nodes

2.1 General approach

In the Eurocode on steel connections EN 1993-1-8 (2015) [3], evenly distribution of normal as well as shear stresses is assumed in the throat section of fillet welds. Hence, should the problem be simulated by elaborate FE-models, uneven distributions, including high stress concentrations will be found. This would lead to a completely different stress pattern in the welds compared to the aforementioned rule. Therefore the approach is to use stress resultants as normal force and bending moments, rather than constitute continuous models. This also allows to derive a set of formulas to calculate the weld stresses.

2.2 Internal welds

To clarify the problem, the upper node of an 80 m railway truss bridge is shown in Fig.1. In this case the members are also fabricated from plates and welded.

As the blue arrows in this picture show, the force of each plate is transferred as a horizontal component to the lower flange of the horizontal member and as a vertical one to a stiffening diaphragm inside this member. The consistent force transfer from each plate requires some of the connections with single-sided welds and even infilled welds as shown in lower part of Fig. 1. Obviously these are heavy nodes and members built together from plates, which still allows these arrangements

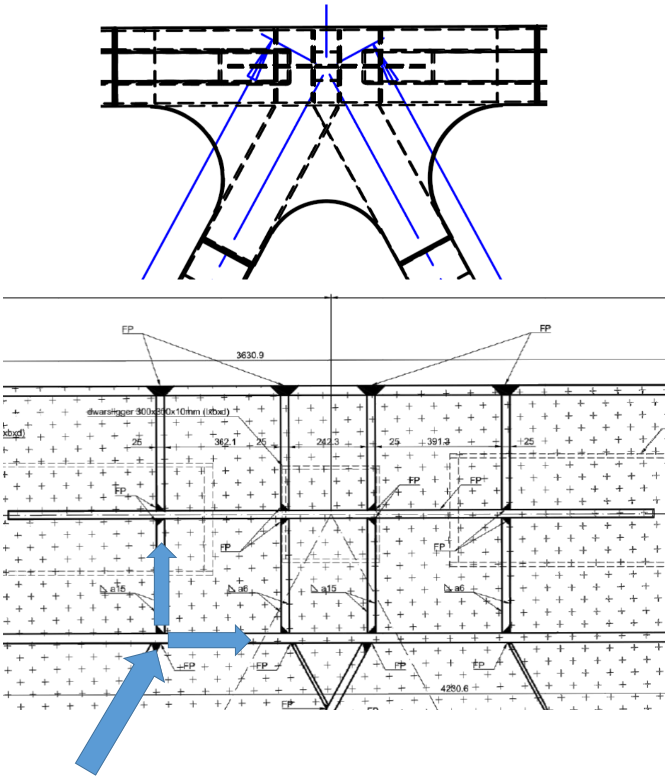

Figure 1. Force transfer in node

If light, cold formed members are used, the figures from EN 1993-1-8 allow for partial intersection of the profiles. This implies interrupting parts of vertical or sloping bars, provided the remaining parts be connected by welding and lateral crushing of the hollow core profiles is resisted. From this, one may question whether all connections and especially the internals are always needed, provided other connections are capable of transferring the member forces. This is exactly what is verified in the following example.

2.3 Truss bridge

The truss bridge under construction will replace a similar bridge across the canal from Pommeroeul to Antoing in the South-West of Belgium. This small canal was established in 1826, the present bridge being constructed 100 years later. Hence, the ‘Royal Bridge’ as it is sometimes called is almost 100 years old and suffers from heavy corrosion. In spite of the historic significance and the heritage value of this bridge, the owner decided corrosion was too extended to refurbish the bridge and it should be replaced. Both in the old and the new bridge the road surface does not correspond nor to the upper members, nor to the lower ones. This is a particular feature of this bridge and complicates the analysis. The vertical bars are subjected to compression rather than tension. Fig. 2 shows the fabrication model of the new bridge, including the high crossbeams that allow the rise of the road pavement above the lower members. Fig. 3 displays both truss members fabricated in the workshop.

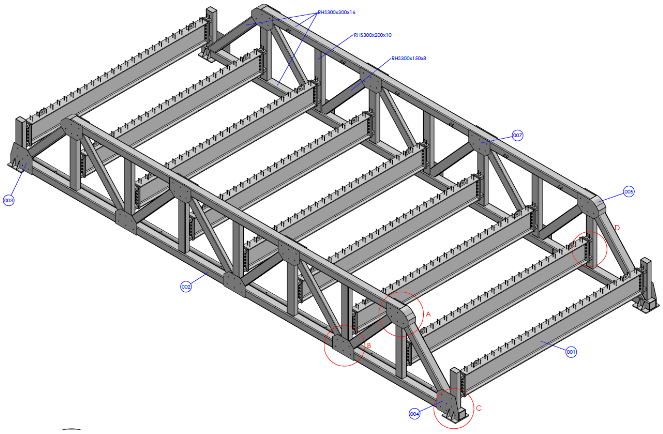

Figure 2. Fabrication model of new truss bridge

Figure 3. Trusses in workshop

As will be argued later, it is not believed that the introduction of compression, instead of tensile force in the lower part of the vertical members, does not modify the problem if the internal welds.

3. Node analysis

3.1 Member forces

Fig. 4 shows the FE-model of the bridge. As mentioned, beam models have been used, to be certain of deriving normal force and bending moments. The nodes have been considered as stiff, since there is no upper bracing and the lateral wind loads may cause out-of-plane bending.

Fig. 4 clearly shows the relevant nodes for the issue being discussed, located on the lower horizontal members and closest to the bearings. Hence the 3rd node counted from the support is considered. The loads include the steel structure’s dead weight, the concrete road slab, the pavement as well as LM 1 from Rurocode EN 1991-2 (2004) [4], as well as wind load according to EN 1991-1-4 (2010) [5]. Load combinations have been made with the proper factors, although the effect of lateral wind was not factored with the relevant ψ-factor. Hence, the effect of wind is somewhat overestimated.

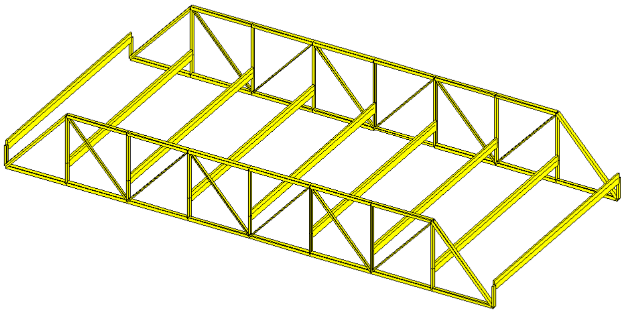

Figure 4. Beam element model of truss bridge

Table 1 shows the normal and shear forces as well as the bending moments in the 5 members that are intersecting in node 3, expressed in kN or kNm. The x-axis corresponds to the member axis, whereas the y axis is perpendicular to the truss plane. Obviously, because of the stiffness of the nodes, bending moments in the lower edge members cannot be neglected.

Table 1. Member forces node 3

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

To substantiate the importance of the bending moments, the total normal stress in both the horizontal members, 44% of stress in the left member and 37% in the right hand member are due to bending. This also applies to the vertical member, since 44.7 % of the total stress is due to bending.

3.2 Node welds

The purpose of these welds is evident. A and B are fillet welds to connect the vertical member to the There are several welds to be distinguished in the 5 member node 3. Names from A to F were given and are shown in Fig. 5. upper flange of the lower edge bar. C and D have an identical function for the diagonal bars, whereas E and F connect the latter to the vertical bar. Clearly, A are the only welds that become inaccessible after assembling of the node and may be those that are omitted or become questionable regarding the usefulness, since the member force of the flange of width d might transit via welds F towards welds D to connect with the lower edge member.

As indicated in Fig. 4, the slope of the diagonal members is given by the angle β whereas the cross-sectional dimensions of the RHS profiles are shown. The thickness of each flange is called t_b, t_d etc.. the index referring to the flange width. The letter a is the throat dimension of the fillet weld, possibly supplemented by the index corresponding to the length. Obviously, an approximation is being made in considering RHS-profiles to correspond to actual rectangular cross sections. In order to rectify this, the thickness t_n might be adapted accordingly.

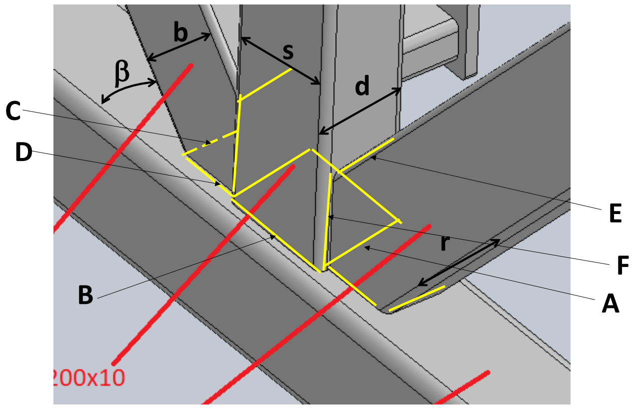

Figure 5. Various welds in 5-member node

It is assumed that the normal force of each member is equally distributed among the 4 flanges of the profile, according to their cross section area. In addition, bending moments are resisted by compressed and tension flanges, the contribution of webs being neglected. With these assumptions, the following equations can be derived for calculating the stresses in the weld throats.

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

Both normal and shear weld stresses are calculated according to EN 1993-1-8 and are of equal magnitude. In weld B these stresses are found from Eq. (2).

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

In a similar manner the weld stresses in C and D are found from Eqs. (3a and b) and (4a and b) respectively. However the square root in (1) and (2) must now be replaced by functions of the correct angle between the weld throat and the axis being considered.

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

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

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

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

For the problem under consideration, the calculation of welds E and F are more important, since they will play an important part if the internal weld A would be omitted. Equations 5a and b apply to weld E and 6a and b correspond to weld type F.

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

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

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

$$\tau_l = \left(\frac{N\,b\,t_b}{2r\,t_r+2\,b\,t_b} + \frac{M_z}{r}\right)\frac{\sin\beta\cos\beta}{b\,a_b} \qquad (6b)$$

It is noticed that in weld F part of the member force is transformed to longitudinal shear in the weld. This is because the connection is not perpendicular to the member axis.

Table 2 shows the various stresses in the weld throats, as well as the vonmises equivalent stresses. The latter demonstrate that A and B show insufficient strength if the material corresponds to S 355. This is due to the fact that LM 1 does not entirely apply to this structures, since the road is of minor importance and the NAD for the code would allow to design for a moderate load as 0.8-times the traffic load. However, this does not influence the purpose of the present study.

Table 2. Weld stresses node 3

weld	σ_t	τ_t	τ_l	vonmis
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

4. Eliminating internal welds

If the internal weld A is omitted, the force from the flanges with length d will be transmitted to the lower member as shear force through weld F and introduce additional stress in D. This alternative load path is more likely than transfer though weld E since the stiffness is considerably higher.

Obviously, part of the vertical member flanges becomes inactive for load transfer. This increases the vertical stress in the remaining cross section. In addition, stress concentrations will occur, which will depend on the radii of the rounding of the profiles and the detailing. Hence, the alternative should be properly prepared. It is believed careful detailing will allow the solution, especially since the heaviest member is concerned and general buckling cannot occur at this location.

In the case of the bridge being discussed the stress in the vertical member increases from 103.3 MPa to 192.4 MPa, or by 86%, although still acceptable. This result may well be of general nature, since in most trusses the members themselves are not critical, the nodes being the determining locations. Should this also be true for other cases, the welds might be adapted to the alternative load path.

As a consequence stresses will increase in welds F and D. Eq (7) renders the additional vertical shear stress in weld F, whereas (8) gives the additional stress in D.

$$\Delta\tau_l = \left(\frac{N\,b\,t_b}{2s\,t_s+2\,b\,t_b} + \frac{M_y}{s}\right)\frac{1}{b\,a_b\,\cos\beta} \qquad (7)$$

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

Again for the particular case under discussion, the new values of weld stresses are listed in Table 3.

Clearly, welds D en F have to be increased. However, in practice this would mean to increase the throat dimension from 5 to 7 mm, which also complies with the parent metal strength.

Table 3. Modified weld stresses node 3

weld	σ_t	τ_t	τ_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

Consequently in this particular case and as far as ultimate limit state is concerned, the internal welds may be omitted, provided larger weld thickness is used, compatible with the parent metal thickness. Obviously this conclusion does not have a general character, although it may be applicable to other cases.

5. Verification of fatigue strength

As the structure considered for this study is a road bridge, fatigue resistance may be important. As mentioned before, this is a local road and the number of lorries crossing daily the bridge is limited. Nevertheless, the verification of the alternative without internal welds should also be verified for fatigue.

According to recommendations (2018) [6], Fatigue load model FLM 3 has been used. Obviously, the effect on vertical loads is moderate, although for the forces in the sloping bars the change of sign, due to the movable loads, introduces large stress variations. Hence for these members, fatigue damage becomes more important. This is a general characteristic for trusses, easily verified when looking at the influence lines for the various bars of a truss structure.

Fatigue verifications is required both for welds and for parent material. In general the parent material condition is more detrimental than is the resistance of the welds. This appears to apply also to this particular truss bridge.

The fatigue damage in the parent material of the vertical and sloping bars, all welded connections being present, reaches 0.40 and 0.59 respectively. If the internal welds are omitted, these values become 0.60 and 0.59, the conditions of the diagonal members being unchanged. These data are obtained, notwithstanding the low fatigue class of the vertical member. The fatigue category for parent material, connected by fillet welding to a perpendicular member is 36 MPa, which may be increased to 45 MPa. The conditions to apply the latter category are indeed fulfilled.

Fatigue damage of the welds has been calculated by determining variations of normal and shear weld stresses and applying the formula (8.3) from EN 1991-1-9. However, due to the two types of shear stress variations the formula had to be extended to Eq. (9).

$$\left(\frac{\Delta\sigma_{Et}}{\Delta\sigma_c}\right)^3 + \left(\frac{\Delta\tau_{Et}}{\Delta\tau_{ct}}\right)^5 + \left(\frac{\Delta\sigma_{El}}{\Delta\tau_{cl}}\right)^5 \leq 1 \qquad (9)$$

The values of $\Delta\sigma_c$, $\Delta\tau_{ct}$ and $\Delta\tau_{cl}$ were taken from table 8.7 of EN 1991-1-9 [7]. The result can be seen in Table 4, both for the case including all welds and for the case where internal welds are omitted.

Table 4. Fatigue strength of weld stresses node 3

all welds	$\Delta\sigma_t$	$\Delta\tau_t$	$\Delta\tau_l$	Damage
A	47.41	47.41		0.22
B	36.77	36.77		0.09
C	13.18	34.92		0.02
D	14.40	14.40	18.38	0.00
E	13.43	34.92		0.02
F	11.73	11.73	39.37	0.00
internal not	$\Delta\sigma_t$	$\Delta\tau_t$	$\Delta\tau_l$	Damage
A				
B	36.77	36.77		0.09
C	13.18	34.92		0.02
D	56.98	56.98	18.38	0.26
E	13.43	34.92		0.02
F	11.73	11.73	57.75	0.20

All values of the damage are smaller than 1. As expected, the damage increases in D and F, although weld A initially showed the largest damage. In table 4, no increase of the weld throat thickness has been considered.

Clearly, in this particular case fatigue is a less detrimental condition than ultimate limit strength of the welds. However, the increase of the fatigue damage certainly is similar. For other cases, as major bridge located on heavy traffic highways or railway bridges, fatigue resistance may be a more critical requirement.

6. Discussion

In the example under discussion, the nodes are not supplemented with gusset plates as it is customary in larger truss bridges, having members and nodes composed of welded plates, as shown in Fig. 6. Gusset plates locally increase the cross section areas and provide multiplication of the node strength. However, the transition between member forces may be more complicated than shown in Fig. 1.

Figure 6. Truss node with gusset plates

Obviously, omission of internal welds should show a symmetrical pattern. If just one of both welds A would not exist, heavy additional bending would originate in the vertical member and the effect on the remaining welds would certainly be inacceptable. In addition, the deviation of the member vertical force may just shift the location of intersection of the node members and thus introduce additional bending in the lower edge beam.

This leads to another requirement concerning the geometry of the structure. The members converging at a node should certainly be concurrent. If the bars are not concurrent, and similar to the previous discussion, additional bending is introduced in the lower edge member.

Taking out those welds, which are perpendicular to the flange of the lower edge member and originate from the heavily loaded vertical member, avoid bending of the upper flange of this horizontal bar. Consequently, the failure mode e of table 7.4 of EN 1993-1-8 is eliminated. This certainly is an asset of the proposal.

Due to these various limitations, it is rather unsure that the simplification being presented might be generalized. Nevertheless, the equations (1) to (8) allow to calculate the various welds in this type of connection and to test whether internal welds might be avoided.

In future, more complicated and critical cases will be explored, including those with gusset plates. The outcome should be compared to consistent FE-models, albeit these will necessarily reveal high stress concentrations, in opposition to the recommendations for design of welded connections. Should the opportunity arise, experimental verification is recommendable, although it can be imagined small scaled plastic models can allow further exploring of the system.

Declaration of Conflict of Interests

The author declares that there is no conflict of interest. He has no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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