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Simplified design equations for Plate-to-CHS T and X joints for use in codes

This paper deals with revised, simplified, consistent equations for plate-to-Circular Hollow Section (CHS) joints for inclusion in codes. After a short review of the background to these resistance equations in the current consolidated version of EN 1993-1-8 and those in ISO 14346, the background to these simplified new equations is discussed. The equations for Plate-to-Circular Hollow Section T and X joints (called TP and XP joints respectively) in the current EN 1993-1-8 are based on experimental data available up to 1991. They are further related to the equations for CHS T and X joints. Most of the data used are based on the ultimate joint resistance. A similar approach is used for the TP and XP equations in ISO 14346, but these are related to the updated equations for CHS T and X joints.

Since the drafting of ISO 14346, new consistent numerical data from Voth has become available where the resistance is not only based on the ultimate resistance but also takes the 3%d_0 joint deformation limit into account.

The new equations in prEN 1993-1-8 are based on the Voth data, the de Winkel data and the Voth-Packer equations, but use a simplified uniform presentation which permits to relate joints with an I, H and RHS brace-to-CHS chord to these basic equations. Furthermore, the presented equations are based on the case of axial compression load in the plate, which is the lower bound of the compression and tension load cases.

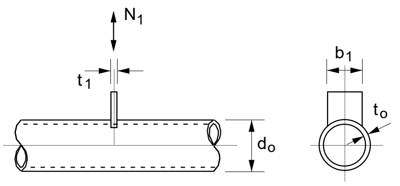
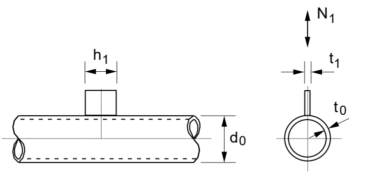
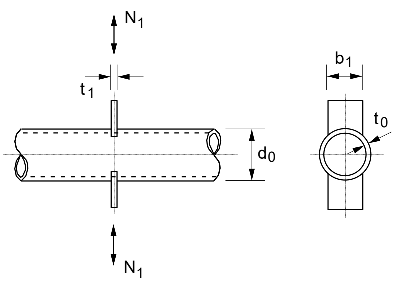
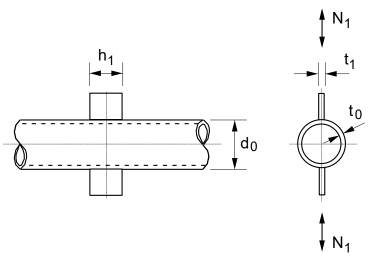
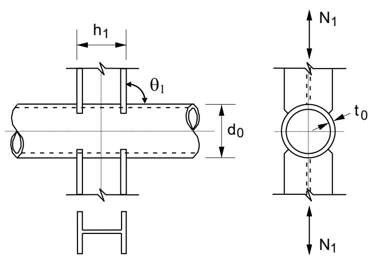
1 Introduction

This paper deals with the revised equations for transverse and longitudinal Plate-to-Circular Hollow Section (CHS) joints, which are included in the update of the current EN 1993-1-8 [3] represented by prEN 1993-1-8 [11], and the forthcoming update planned for ISO 14346 [6]. However, the main focus in this paper is in relation to the updating of prEN 1993-1-8. Just as for the ISO 14346 circular hollow section (CHS) joints, the equations in prEN 1993-1-8 have been revised with the chord stress function now based only on the maximum stress in the chord connecting face. This overcomes the confusion in EN 1993-1-8 caused by the different approaches used for CHS and Rectangular Hollow Section (RHS) joints with chord stress functions k_p and k_n, respectively. As a consequence of these changes, the geometrical part of the resistance functions also had to be revised. This had already been done for the CHS T, X and K joint resistance equations in the IIW [4] recommendations, which are the basis for ISO 14346. The recommendations of IIW [5], which are the same as those in ISO 14346, have also been included in the updated CIDECT Design Guide 1, 2nd Ed., (Wardenier et al. [24]).

Since the drafting of the IIW [5] and the ISO 14346 [6] recommendations, new numerical data became available for Plate-to-CHS T and X joints (Voth, [16]), indicated as TP and XP joints. Here, the joint design resistance is determined by considering not only the ultimate resistance but also Lu's 3%d_0 joint deformation limit (Lu et al., [8]). This deformation limit was agreed by IIW Sub-commission XV-E and introduced to avoid a separate deformation check under serviceability conditions. If the ultimate joint resistance occurs after a 3%d_0 chord indentation then, instead of the ultimate joint resistance, the resistance at the 3%d_0 chord indentation is taken as the joint resistance.

The new basic equations in prEN 1993-1-8 for transverse plate-to-CHS joints (designated as TP-1 and XP-1) and longitudinal plate-to-CHS joints (designated as TP-2 and XP-2), shown in Table 1, are developed based on the numerical Voth [16] data with the related Voth-Packer [17], [18] equations, the numerical de Winkel [25] data and the previous data bases of Makino et al. [9], [10].

Table 1. Joint classification

Transverse plate to CHS joints	Longitudinal plate to CHS joints	
TP-1 and XP-1 joints	TP-2 and XP-2 joints	TP and XP-3, 4 and 5 joints
		TP3 and XP-3 joints: combination of Type 1 and 2 TP4 and XP-4 joints: with I or H section brace TP5 and XP-5 joints: with RHS section brace
		

Voth [16] and Voth-Packer also tried to relate their equations to those included in ISO 14346 for CHS joints but showed that for the data based on the 3%d_0 deformation criterion this could not simply be done with a coefficient. In the reanalysis for prEN 1993-1-8 some of the Voth-Packer equations have been modified and presented in such a manner that the design equations for joints with I, H or RHS braces can be easily related to the equations for joints with transverse and longitudinal plates. Furthermore, EN 1993-1-8, ISO 14346 and also prEN 1993-1-8 not only give design equations for plate-to-CHS T and X joints loaded by axial force, but also for in-plane bending and out-of-plane bending.

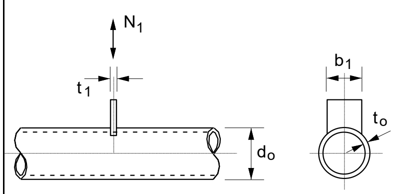
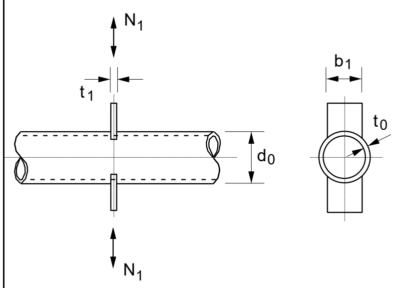
The joint resistance in prEN 1993-1-8, IIW 2012, ISO 14346 and the CIDECT design guide (Wardenier et al., [24]) is now uniformly presented as shown in Eq. 1:

$$N_{1,Rd} = f_{y0}\, t_0^2\, Q_u\, Q_f \qquad (1)$$

Here, Q_u is a function of the joint geometrical parameters $f(\beta,\gamma,\eta)$ and Q_f is a function $f(n)$ of the maximum absolute chord stress parameter n. Where β is the plate width-to-chord diameter ratio, 2γ the chord diameter-to-chord thickness ratio and η is the plate length-to-chord diameter ratio. The functions Q_u in EN 1993-1-8, ISO 14346 and those more recently developed by Voth-Packer ([17], [18]) are given in Table 2 for transverse plate-to-CHS joints and in Table 3 for longitudinal plate-to-CHS joints.

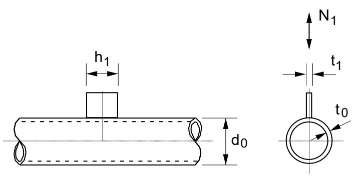
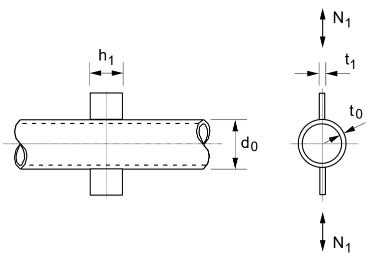
Voth and Packer produced resistance expressions that included the thickness of the transverse/longitudinal plate, as well as expressions that include a dependence on the plate loading (axial compression versus axial tension). However, in the interest of simplicity the basic versions for plate compression loading are shown in Tables 2 and 3.

Table 2. Resistance functions $Q_u = N_{1,Rd}/(f_{y0}\, t_0^2 Q_f)$ for transverse plate-to-CHS joints

Type of joint	**EN 1993-1-8** (2010)	**ISO 14346** (2013)	**Voth-Packer**, (2012, 2012a)*
TP-1 	$(4+20\beta^2)$	$\left(2.2+15\beta^2\right)\gamma^{0.2}$	$2.9(4+3\beta^2)\gamma^{0.35}$ ζ= 0.85
XP-1 	$\dfrac{5}{1-0.81\beta}$	$2.2\left(\dfrac{1+\beta}{1-0.7\beta}\right)\gamma^{0.15}$	$2\,\zeta\left(\dfrac{1+\beta}{1-0.6\beta}\right)(1+\eta)\gamma^{0.25}$ ζ= 0.85

NB: * In the Voth-Packer equations (in this Table) the effect of the weld size is neglected

Table 3. Resistance functions $Q_u = N_{1,Rd}/(f_{y0}\, t_0^2 Q_f)$ for longitudinal plate-to-CHS joints

Type of joint	EN 1993-1-8	ISO 14346	Voth-Packer, (2012, 2012a)*
TP-2 	$5(1+0.25\eta)$	$5(1+0.4\eta)$	$7.2\zeta(1+0.7\eta)$ ζ= 0.85
XP-2 	$5(1+0.25\eta)$	$5(1+0.4\eta)$	$3.5\,\zeta\left(\frac{1+\beta}{1-0.6\beta}\right)(1+0.5\eta)\gamma^{0.1}$ ζ= 0.85

NB: * In the Voth-Packer equations (in this Table) the effect of the weld size is neglected

2 Background for the equations in EN 1993-1-8 and ISO 14346

2.1 Background to EN 1993-1-8

The joint resistance equations (Sedlacek et al. [12], [13] in the current consolidated version of EN 1993-1-8 [3] for CHS T, X and K joints are based on the IIW [4] recommendations. The resistance equations for plate-to-CHS joints are based on those in the CIDECT Design Guide No 1 [22], (Wardenier et al., [22]) which were based on Kurobane [7] and Wardenier [21]. At the time of drafting the CIDECT Design Guide No. 1, no internationally agreed deformation limit existed. Therefore, the analyses were based on the ultimate resistance in the case of transverse plate-to-CHS joints. For very flexible longitudinal plate-to-CHS joints, the resistance was based on a certain not-uniformly-defined yield resistance used by the various Japanese investigators. The files for these analyses cannot be traced anymore, so the current equations in Tables 7.3 and 7.4 of EN 1993-1-8 are compared here with the database of Makino et al. [9] available at that time. Considering the database, it can be concluded that the variation in parameters is rather limited, e.g. in most cases $\beta \approx 0.5$-0.9; $2\gamma \approx 32$; $\eta \approx 0.5$ to 2. Due to the small variation in parameters, all XP joints shown in Table 1 and loaded in compression are all analysed together and shown in Fig. 1.

The data for axially loaded TP joints (Makino et al., [9]) could not be treated as one set but as two; one for the resistances based on ultimate strength as shown in Fig. 2 and the other for resistances based on yield in Fig. 3.

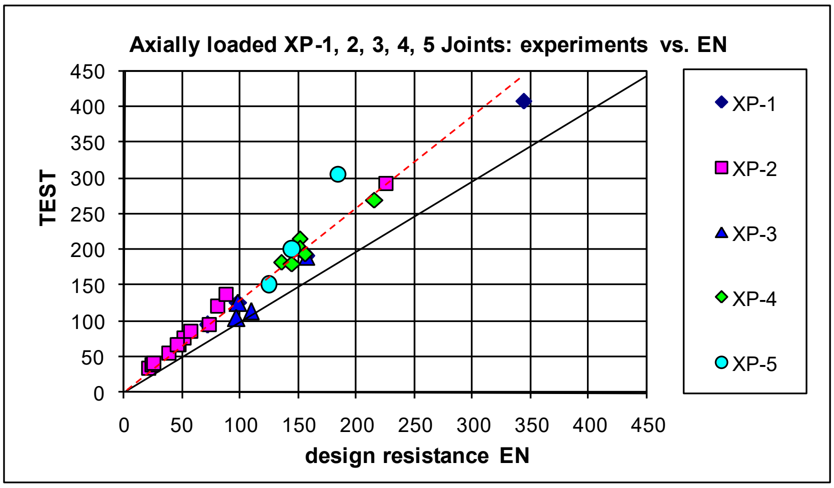


Fig. 1. Experimental data for XP joints vs. EN 1993-1-8 (m =1.37 and CoV = 11.8%). The XP-2 data are based on "yield", the others are based on ultimate resistance.

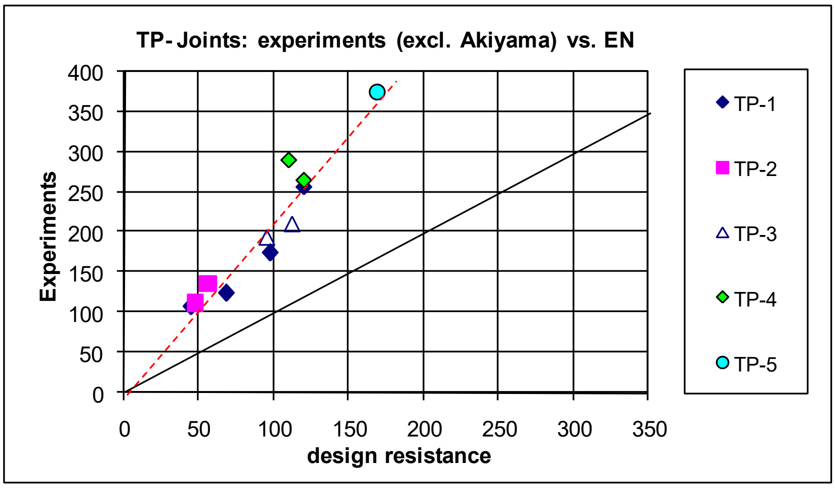


Fig. 2. Experimental data for TP joints vs. EN 1993-1-8 based on ultimate resistance

As shown in Fig. 3, the design resistance in EN 1993-1-8 is a lower bound for the Akiyama et al. [1] yield data and has an average margin of about 1.3 to the mean value. Although most of the Akiyama et al. yield data were outside the normal range of validity with high chord diameter-to-thickness ratios, these were considered to be extremely low but there was no clear explanation for it. Taking these into account also meant that the margin between the design resistance in EN 1993-1-8 and the mean of the ultimate data in Fig. 2, is about 2.

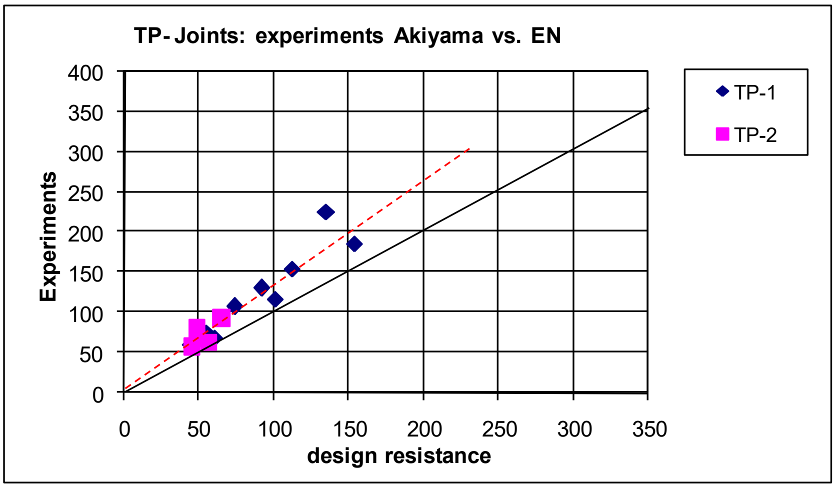


Fig. 3. Experimental data of Akiyama et al. [1] for TP-1 and TP-2 joints vs. EN 1993-1-8 based on yield

2.2 Background to ISO 14346

At the time of drafting the IIW recommendations [5], which are the basis of the ISO 14346 [6], Lu's 3%d_0 deformation criterion (Lu et al., [8]) was in use, but no detailed information was available in the Japanese experimental data for applying this deformation criterion.

In the analyses for ISO 14346 (Wardenier et al., [23]), the resistances were still based on the ultimate capacity in the case of joint types 1, 3, 4 and 5 of Table 1 and on a Japanese “not uniformly defined yield” capacity for joint type 2. In addition, numerical data for but welded XP-1 and XP-4 joints (Winkel [25]) based on limiting the maximum deformation to 3%d_0 were available. These were, as with Akiyama et al. [1], rather low compared to the ultimate capacities given in Makino et al. [9]. The additional Japanese data in Makino et al. [10] and Ariyoshi and Makino [2] were not used since the failure criteria used by the Japanese investigators were not always clear.

In addition to the data of de Winkel [25] based on the 3%d_0 deformation criterion, only one data point was available from the then-new investigation by Voth [16] on this subject. After discussion in IIW Sub-commission XV-E it was decided, until further justification of the low Akiyama and the de Winkel data, to reduce the design resistances based on the Makino et al. [9] data in such a way that the design resistance was not lower than the mean resistance found by Akiyama et al. [1] and de Winkel.

Just as discussed in section 2.1 for EN 1993-1-8, the equations for the design resistances were directly related to the (updated) ISO 14346 resistance equations of equivalent CHS T or X joints. However, for the longitudinal plate-to-CHS XP-2 and TP-2 joints, they were simplified by excluding the plate thickness effect, by assuming $\beta = 0$ and taking a lower bound for the γ effect.

Also, as for EN 1993-1-8, the data was not only analysed per type of joint but also for all XP joints and for all TP joints together. Fig. 4 shows all experimental data for XP joints used for ISO 14346 (with m = 1.27 and CoV = 8.4%). The XP-2 data (Makino et al., [9]) are based on "yield", the others are based on ultimate capacity. As shown, the scatter is smaller than for the EN 1993-1-8 equations in Fig. 1.

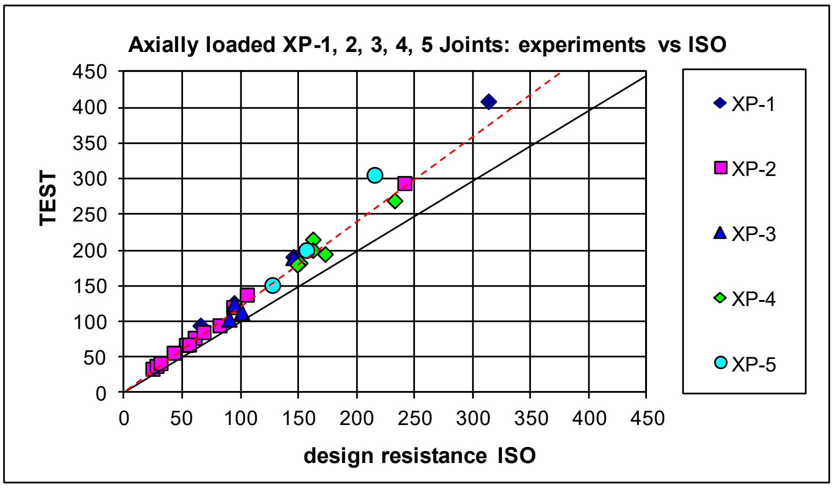


Fig. 4. Experimental data for XP joints vs. ISO 14346; XP-2 based on "yield", others based on ultimate resistance (m =1.27 and CoV = 8.4%)

As later confirmed by Voth [16], the data of de Winkel [25] agree well with the Voth data if the plate and weld thicknesses are taken into account. Considering that the de Winkel data, shown in Fig. 5, are based on the 3% deformation resistance, it can now be concluded that the design resistance for XP-1 joints in ISO 14346 is sometimes too optimistic, especially for high β.

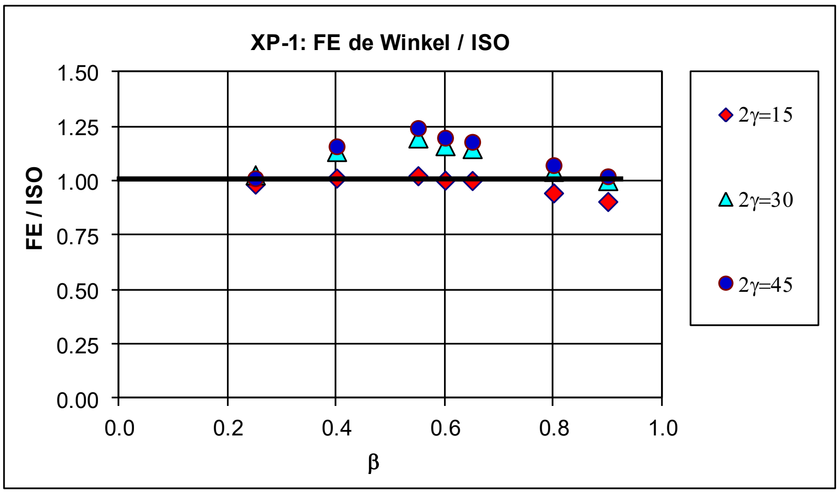


Fig. 5. FE data for XP-1 joints (de Winkel, [25]) based on 3%d_0 deformation vs. ISO 14346

Fig. 6 shows the experimental data vs. ISO 14346 for TP joints, excluding the Akiyama data for high $2\gamma > 50$ ratios. The TP-2 data are based on "yield", the others are based on the ultimate resistance.

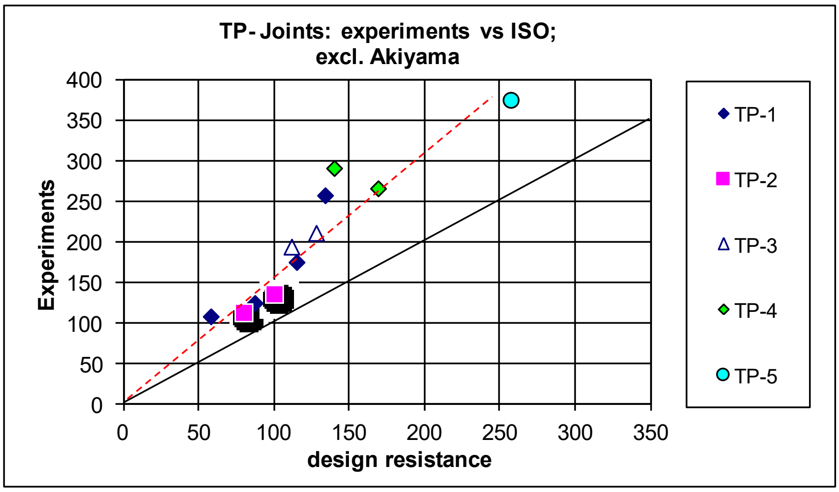


Fig. 6. Experimental data for TP joints vs. ISO 14346; excluding Akiyama data

As can be determined from Fig. 6, the ultimate resistance for TP joints in ISO 14346 has an average ratio of about 1.5 relative to the design resistance. Although the Akiyama TP-2 joint data are low and outside the validity range with ratios $2\gamma > 70$, they were indirectly taken into account by ensuring that the ISO design resistance in this range is not below the mean of these data.

3 Reanalysis for prEN 1993-1-8 and update of ISO 14346

In the reanalysis for prEN 1993-1-8, the following philosophy is used in determining the design equations for TP and XP joints:

- Where possible, relate the resistance equations to those for CHS T and X joints by the use of a constant.
- The set of equations should be logical for designers, and consistent and simple as possible, but a sufficient fit to the data.
- It should be easily possible to relate the resistance equations for joints with I or H section braces in a logical way to those for axially loaded TP and XP joints.
- It should also be possible to logically relate the resistance equations for brace in-plane bending and brace out-of-plane bending, to those for axial loading.
- The resistance equations should give an adequate margin of safety for TP and XP joints with steel grade S460, for which no experimental data exists.

The design recommendations for CHS T, X and K joints in prEN 1993-1-8 are similar to those in ISO 14346 for which the background is described in van der Vegte et al. [15].

The design recommendations for transverse and longitudinal plate-to-CHS joints are now based on the numerical data of Voth [16] and those of de Winkel [25], which both consider the ultimate resistance and the $3\%d_0$ maximum deformation criterion.

3.1 General review of the Voth and Voth-Packer equations

Voth [16] determined various possible equations, where in the best-fit equations the effect of the plate thickness and weld leg sizes is included in the β' and η' parameters, see Tables 4 and 5.

Table 4. Resistance functions Q_u for transverse plate-to-CHS joints loaded in compression

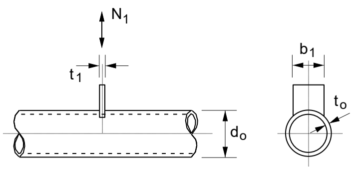
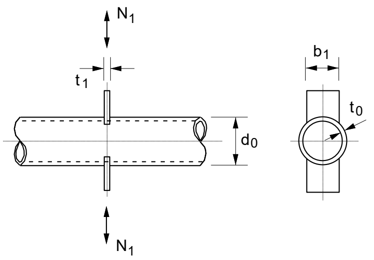
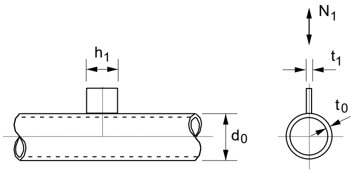
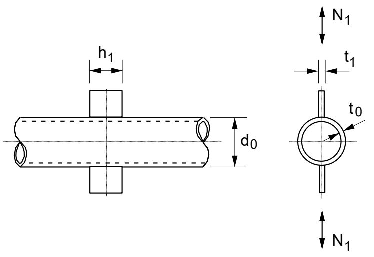
Type of joint	Voth (mean best-fit)	Voth-Packer (design)
TP-1/TP3 	$2.6\left(1+3\beta'^{2}\right)\left(1+0.6\eta'\right)\gamma^{0.35}$	$2.9\varsigma\left(1+3\beta^{2}\right)\gamma^{0.35}$ ζ= 0.85
XP-1/XP3 	$2\left(\dfrac{1+\beta'}{1-0.6\beta'}\right)(1+\eta')\gamma^{0.25}$	$2\,\zeta\left(\dfrac{1+\beta}{1-0.6\beta}\right)(1+\eta)\gamma^{0.25}$ ζ= 0.85

Table 5. Resistance functions Q_u for longitudinal plate-to-CHS joints loaded in compression

Type of joint	**Voth** (mean best-fit)	**Voth-Packer** (design)
TP-2 	$6.1(1+\beta'^{2})\,(1+0.7\eta')\,\gamma^{0.05}$	$7.2\,\zeta\left(1+0.7\eta\right)$ ζ= 0.85
XP-2 	$3.5\left(\dfrac{1+\beta'}{1-0.6\beta'}\right)(1+0.5\eta')\gamma^{0.1}$	$3.5\,\zeta\left(\dfrac{1+\beta}{1-0.6\beta}\right)(1+0.5\eta)\gamma^{0.1}$ ζ= 0.85

Voth and Packer ([17], 18], [20]) proposed simplified design equations for axially loaded TP joints with transverse and longitudinal plates, neglecting the plate thickness and weld effect. For the XP joints they included the plate thickness effect but excluded the weld leg effect. However, the figures in Voth and Packer [17], [18] showing a comparison between the FE data and the recommended equations include both the effect of plate thickness and weld leg sizes.

For the evaluation to design resistances, they proposed a factor of $\zeta = 0.85$ to the mean resistance, which is, depending on the scatter, somewhat higher than that used in the procedure for ISO 14346 (Wardenier et al., [23]) which is (including a $\gamma_M = 1.1$) close to 0.80. The method used in ISO 14346 to derive the design equations is based on that published by Strating [14] which was the basis for the procedure used in the EN codes.

Considering the Voth and Voth-Packer equations in Tables 4 and 5 more in detail shows that:

- The $f(\beta)$ or $f(\beta')$, and the $f(\eta)$ or $f(\eta')$, functions are different from those used for CHS T and X joints because the governing numerical data did not fit well with the functions used for the CHS joints.
- The $f(\beta)$ or $f(\beta')$ and $f(\eta)$ or $f(\eta')$ in the Voth best-fit functions differ for the various types of joints.
- For TP joints the plate thickness is not considered in the recommended Voth-Packer equations but for the XP joints it is, which is not consistent.
- Furthermore, the η effect is in all cases considerably stronger than expected relative to previous analyses based on the ultimate resistance only.
- The γ effect in the TP-1 joint equation is stronger than that for the XP-1 joint, which is consistent with that in the CHS T and X joint equations.
- However, for TP-2 joints the γ effect is smaller than that for XP-2 joints, which contradicts the trend for TP-1 and XP-1 joints as well as for CHS X joints.

Considering the above-mentioned philosophy and observations, it was decided:

- To base both the TP and XP joint equations on butt-welded plates with a thickness calibrated to 10 mm, thus in principle neglecting the effect of larger plate thicknesses and weld sizes; this not only gives a lower bound but also a consistent approach.
- To evaluate the $f(\beta)$ and $f(\eta)$ functions to be more consistent.
- To check whether the γ effect in the simplified Voth-Packer equation for XP-2 joints can also be neglected.

Further, the Voth and the de Winkel data are derived from well-calibrated numerical analyses of joints with mild steel S355, thus steel with a better ductility than that of S460. As a consequence the additional margin of safety for joints loaded in tension, compared to those loaded in compression, may be reduced or may even disappear. Since no experimental evidence for these joints is available, care has to be taken in formulating resistance equations for TP and XP joints which can be assumed to be valid up to and including S460.

3.2 Transverse plate-to-CHS joints

Transverse plate-to-CHS (TP-1) joints

Voth [16] used an FE model with compensating chord end bending moments, resulting in moments in the chord at the plate connection which are nearly zero, thus with a chord stress function $Q_f = 1.0$.

The analysis of the Voth data including the plate thickness and weld leg sizes with the best-fit functions for β' and η' (see Table 4) gives a mean of m = 1.24 and a CoV = 9.7%, comparable to that in Voth [16]. The results are presented in Fig. 7. Voth and Packer [18] simplified the best-fit Voth equation by excluding the plate and weld size effect, see Table 4. However, this is based on the Voth data which had a relatively large plate thickness and large fillet welds. To also cover butt-welded joints with smaller practical plate thicknesses, the analysis here is standardized on a practical minimum plate thickness of 10 mm.

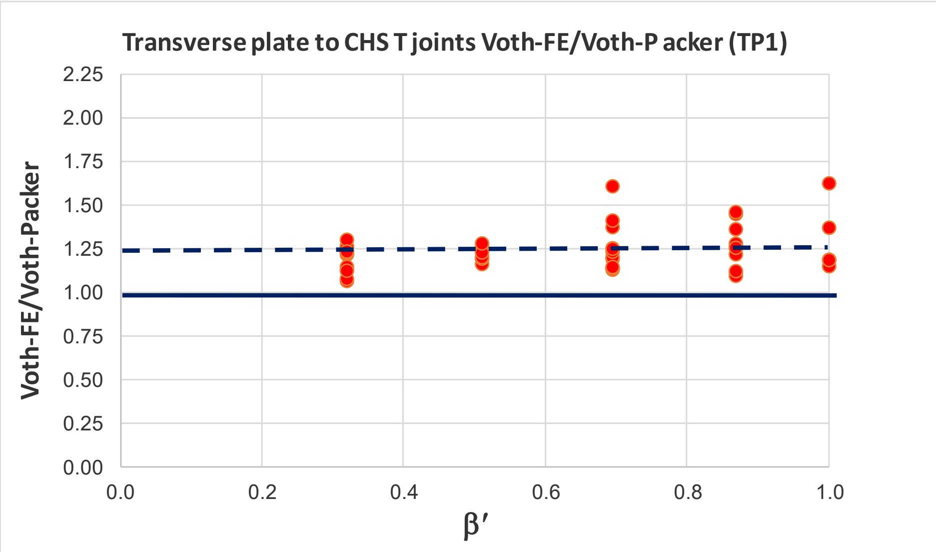


Fig. 7. Comparison of the Voth FE data vs. the best-fit Voth equation for transverse plate-to- CHS TP-1 joints loaded in compression, incl. effect of plate and weld leg size in β' and η' (m = 1.24, CoV = 9.7%); excluding data deleted by Voth

The Voth data are all based on an effective ratio η' = 0.21 which includes the plate thickness with the weld leg sizes, related to the chord diameter. For a butt-welded plate with an assumed practical minimum thickness of t_1 = 10 mm it gives, for the Voth data, a f(η) = (1 + 0.6×10/219) = 1.03. Thus, the coefficient for the modified Voth-Packer design equation based on butt-welded TP-1 joints with a plate thickness effect of 1.03 for t_1 = 10 mm and ζ = 0.85 will now be 2.6x1.03x0.85 = 2.3. Therefore, the resulting prEN equation is given by Eq. (2) and Fig. 8 shows the comparison between the Voth data and the modified Voth-Packer equation for prEN 1993-1-8.

$$Q_u = 2.3\left(1+3\beta^2\right)\gamma^{0.35} \tag{2}$$

Since the prEN design equation is based on the lower resistances of joints with a smaller plate thickness of 10 mm, compared to the Voth-Packer equation, it results in a larger mean value of m = 1.54 and a CoV = 9.8% for the Voth data.

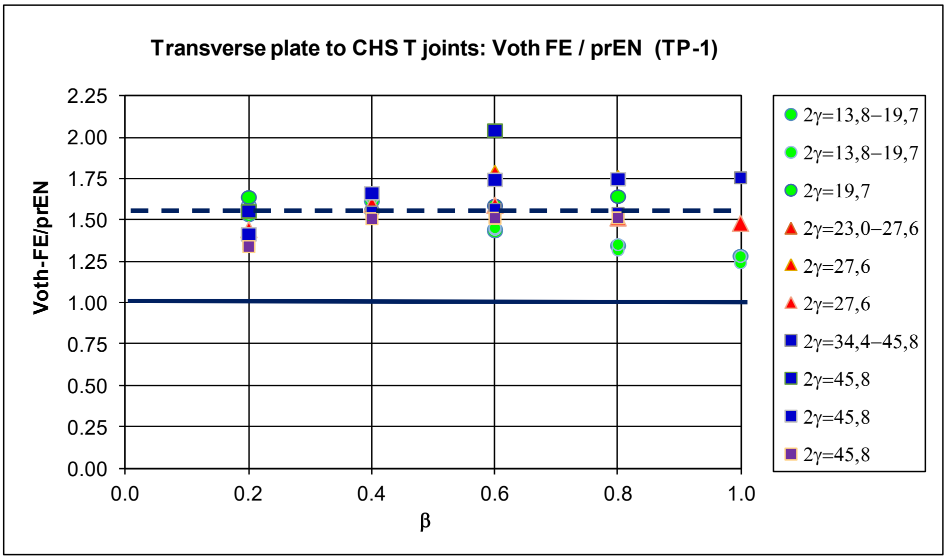


Fig. 8. Voth FE data vs. the modified Voth-Packer equation for transverse plate-to-CHS TP-1 joints loaded in compression, adopted for prEN 1993-1-8 (m = 1.54, CoV = 9.8%); excluding data deleted by Voth

Transverse plate-to-CHS (XP-1) joints

Because of the relatively larger influence of the plate thickness and the weld leg for transverse plate-to-CHS (XP-1) joints than for TP-1 joints, Voth included both effects in the best-fit equation by means of the functions for β' and η' in Eq. 3, see also Table 4:

$$Q_u = 2\,\zeta\left(\frac{1+\beta'}{1-0.6\beta'}\right)(1+\eta')\gamma^{0.25} \quad \text{with } \zeta = 0.85 \text{ from Voth-Packer (2012a)} \qquad (3)$$

Fig. 9 shows the Voth FE data related to Eq. (3) based on β′ and η′. Voth and Packer [18] proposed to include only the effect of the plate thickness and exclude the effect of the weld leg in β and η, as shown in Fig. 10, which results in an increased mean value m from 1.21 to 1.47 or it gives an additional margin between the Voth data and the equation of 1.47/1.21 = 1.21.

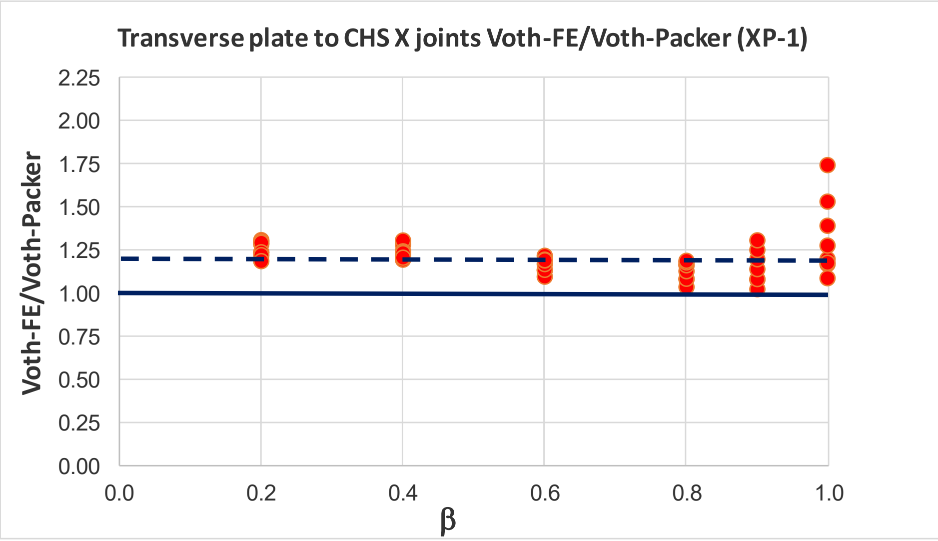


Fig. 9. Voth FE data (1.2 ≤ t_1/t_0 ≤ 4.0) vs. Voth Eq.(3) for transverse plate-to-CHS XP-1 joints loaded in compression, incl. effect of plate and weld leg size in β ' and η' (m = 1.21, CoV = 9.8%)

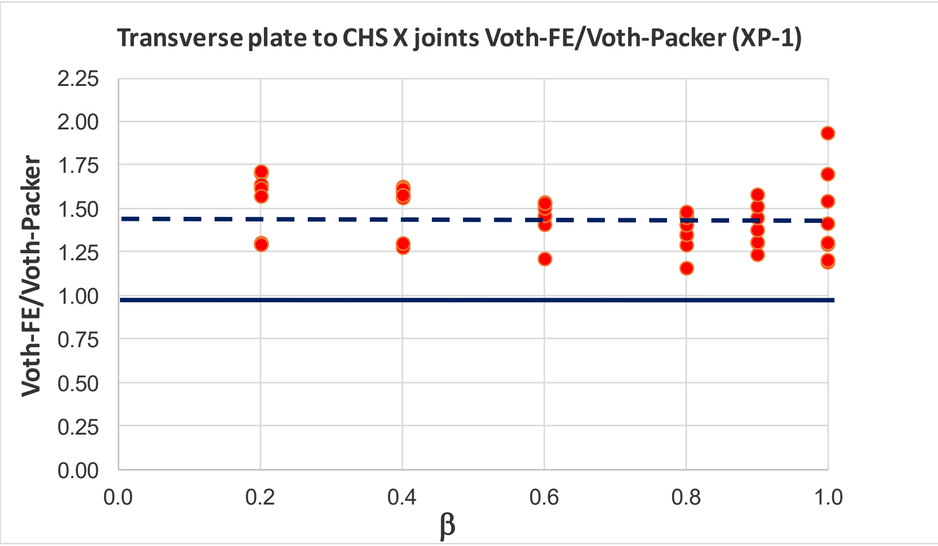


Fig. 10. Voth FE data (1.2 ≤ t_1/t_0 ≤ 4.0) vs. Voth-Packer for transverse plate-to-CHS XP-1 joints loaded in compression, excl. weld leg effect in β and η (m = 1.47, CoV = 11.5%)

Excluding also the effect of the plate thickness in η, would result in an additional increase of the mean value m from 1.47 to 1.61, or it would give a further margin between the Voth data and the design equation of 1.61/1.47 = 1.10. As shown in Fig. 11, when the plate thickness and the weld leg are excluded, this gives relative to Fig. 9 a margin of 1.61/1.21 = 1.33.

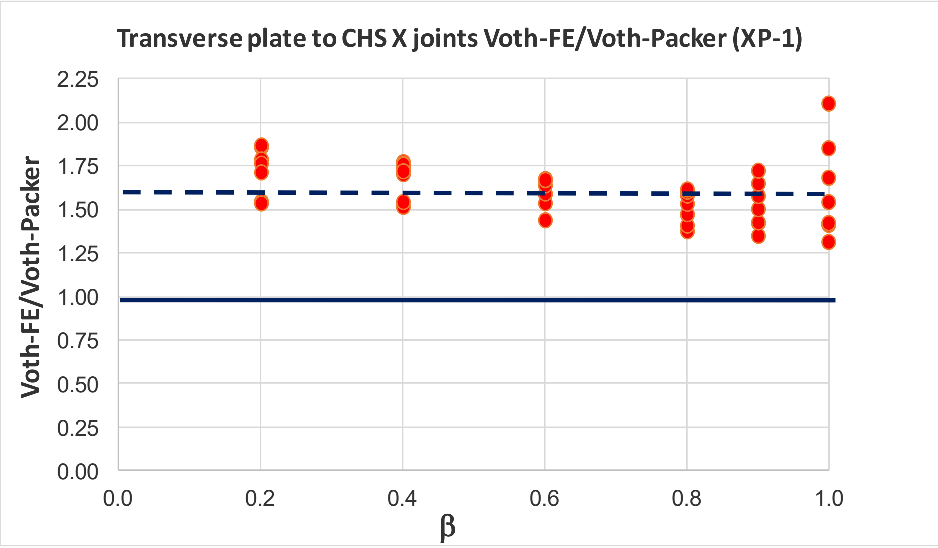


Fig. 11. Voth FE data ($1.2 \leq t_1/t_0 \leq 4.0$) vs. modified Voth-Packer Eq. *excl. effect of plate and weld leg sizes* *(m = 1.61, CoV = 10.1%)*

For transverse plate-to-CHS XP-1 joints, the Voth-Packer equation has no direct relationship with the CHS X joint equation, because the β effect differs. Therefore, it has been checked whether a further consistent simplification would be possible.

Voth [16] reported that his data and those of de Winkel [25] for the same geometry agreed well with each other. Checking the de Winkel (XP-1) data more in detail shows that these can be simplified and represented by a Q_u term with a β function similar to that in the Voth-Packer equation for TP-1 joints (Table 4), but with a constant of 2.4, see Eq. 4 and Fig. 12:

$$Q_u = 2.4\left(1+3\beta^2\right)\gamma^{0.25} \quad (4)$$

With a correction for a plate thickness of 10 mm for the Voth data of f(η) = (1+10/219) = 1.05 and a factor ζ = 0.85 adopted by Voth-Packer (2012a) results in the prEN Eq. (4a):

$$Q_u = 2.1\left(1+3\beta^2\right)\gamma^{0.25} \quad (4a)$$

This equation is adopted in prEN 1993-1-8. Comparison of the Voth data vs. the prEN 1993-1-8 (Eq. 4a) in Fig. 13 gives a mean of 1.59 and CoV = 10.8%, which is comparable to that for the modified Voth-Packer equation in Fig. 11 with m = 1.61 and CoV = 10.1.

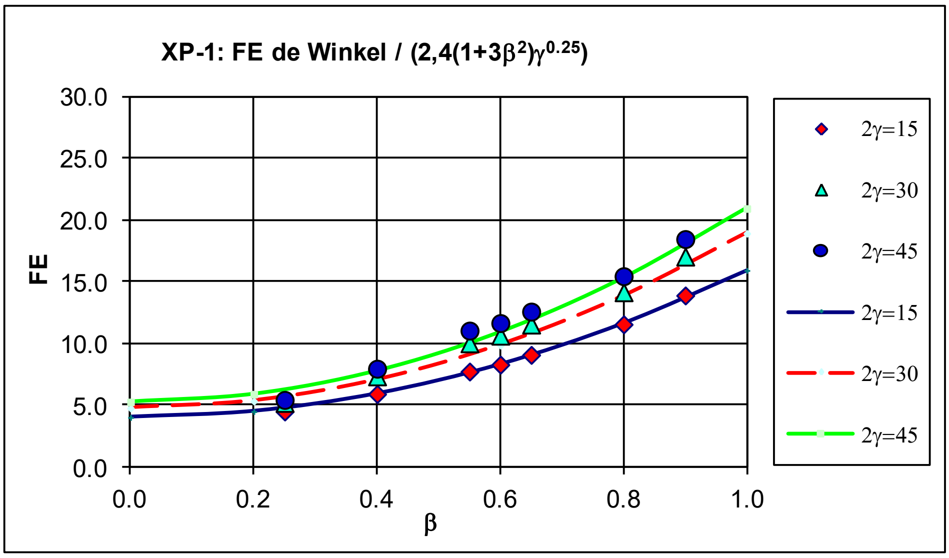


Fig. 12. De Winkel FE data for transverse plate-to-CHS X joints vs. a function $Q_u = 2.4(1+3\beta^2)\gamma^{0.25}$

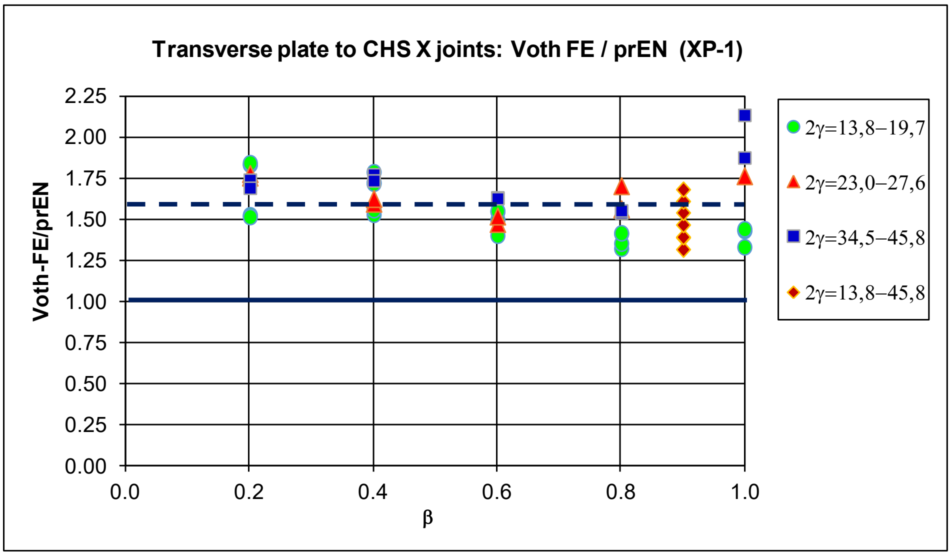


Fig. 13. Voth FE data with $1.2 \leq t_1/t_0 \leq 4.0$ *vs. the prEN-1993-1-8 equation for transverse plate-to-CHS XP-1 joints, loaded in compression (m = 1.59, CoV = 10.8%)*

3.3 Longitudinal plate-to-CHS joints

Longitudinal plate-to-CHS (TP-2) joints

The best-fit equation of Voth [16] from Table 5, with the effect of the plate and weld thickness included in β' and η' and an adopted $\zeta = 0.85$, versus the Voth FE data is shown in Fig. 14. As indicated before, Voth used an FE model with compensating chord end bending moments, thus no chord stress reduction has to be taken into account, i.e. $Q_f = 1.0$.

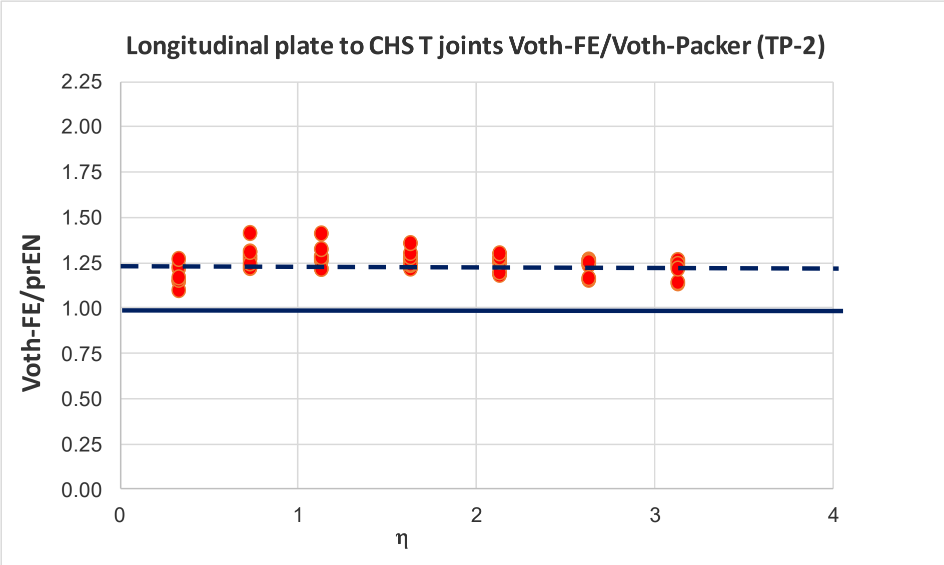


Fig. 14. Comparison of the Voth FE data vs. the best-fit Voth equation for longitudinal plate-to-CHS TP-2 joints loaded in compression, <u>incl. effect of plate and weld leg size</u> in β′ and η′ (m = 1.24, CoV = 5.0%).

As shown in Table 5, in the Voth-Packer [16] design equation for longitudinal plate-to-CHS TP-2 joints, the effect of the plate thickness and the weld leg, and also the small gamma effect, are excluded. In the best-fit equation of Voth in Table 5 the effect of the plate and weld thickness is included in the $f(\beta') = (1+\beta'^2)$ function. Considering the plate thickness and the weld leg sizes used to derive this β' parameter gives, for all Voth data, a $\beta' = 0.21$ with a $f(\beta') = 1.04$. Butt-welded TP-2 joints with a plate thickness of 10 mm give, for the Voth numerical specimen, a $\beta = 10/219 = 0.045$ which has a negligible effect. Thus, based on TP-2 joints with a plate thickness of 10 mm the coefficient in the Voth-Packer resistance equation could be reduced by 1.04.

The η effect in the Voth and Voth-Packer equations with $(1+0.7\eta)$ for TP-2 joints deviates considerably from that in EN 1993-1-8 which is $(1+0.25\eta)$, and that in ISO 14346 which is $(1+0.4\eta)$, thus it is not in line with what may be expected based on previous analyses. Considering the Voth FE data in Fig. 14 in more detail shows that the function $(1+0.7\eta)$ is largely influenced by the data for $\eta < 1.0$. For $\eta \geq 1.0$ the function $(1+0.4\eta)$ gives a better fit and also agrees with that used in ISO 14346.

Re-analysis all Voth data with the function $(1+0.4\eta)$, in relation to the Voth-Packer [17] equation with $\eta = 1.0$ as a reference point, gives a calibration correction of (1+0.7)/(1+0.4) = 1.21. This results in the prEN term for Q_u for TP-2 joints with a longitudinal plate with a correction of the constant 7.2 by a reduction factor of 1.04 for thickness, an increase of 1.21 for the η effect , and $\zeta = 0.85$, resulting in: (7.2/1.04)x1.21x0.85 = 7.1. Thus the prEN Q_u function for TP-2 joints with a longitudinal plate with $\eta = 1.0$ is now given by:

$$Q_u = 7.1(1+0.4\eta) \tag{5}$$

This modified Voth-Packer equation versus all Voth FE data for inclusion in the prEN is shown in Fig. 15.

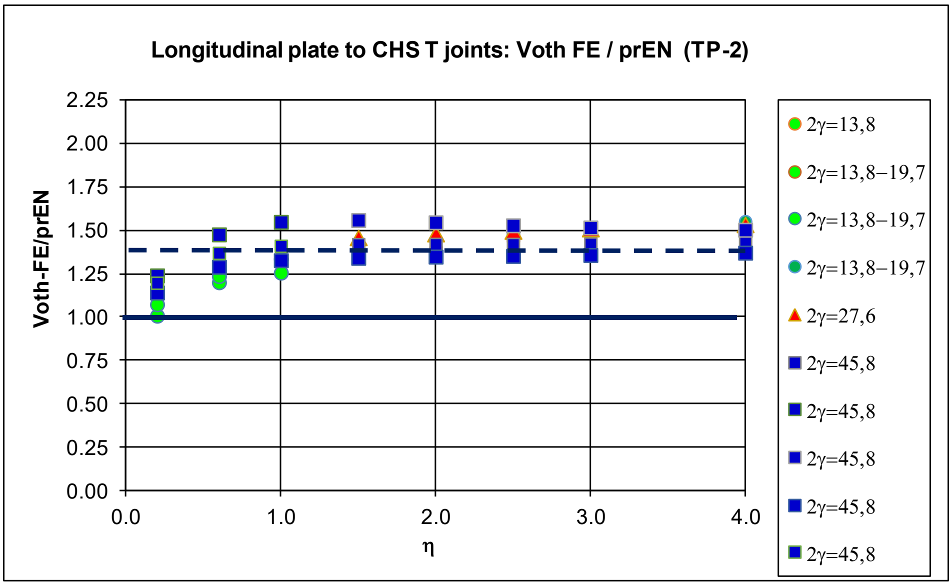


Fig. 15. Voth FE data vs. the modified Voth-Packer equation (5) for longitudinal plate-to-CHS TP-2 joints, adopted for prEN 1993-1-8; joints loaded in compression (m = 1.38, CoV = 10.3%)

This comparison with all Voth FE data gives a mean of 1.38 and a CoV of 10.3%. However, for the data with a validity range of $\eta \geq 0.6$ it gives a mean of 1.42 and a CoV of 6.7%. An appropriate validity range for Eq. (5) is $0.6 \leq \eta \leq 4$.

Longitudinal plate-to-CHS (XP-2) joints

The equation for longitudinal plate-to-CHS XP-2 joints by Voth-Packer [18] is a simplification of the Voth data in that the parameters β' and η' are simplified to β and η, thus the effect of the weld size is not taken into account. For TP-2 joints Voth and Packer have included the plate thickness effect in the Q_u equation by the same function as used for XP-1 joints, see Tables 4 and 5.

For XP-2 joints with a longitudinal plate, Fig. 16 shows the Voth FE data versus the Voth [16] equation, thus taking the plate thickness and the weld leg sizes into account in the β' and η' functions. The data are based on FE analyses of XP-2 joints with thick longitudinal plates with $1.2 \leq t_1/t_0 \leq 4.0$.

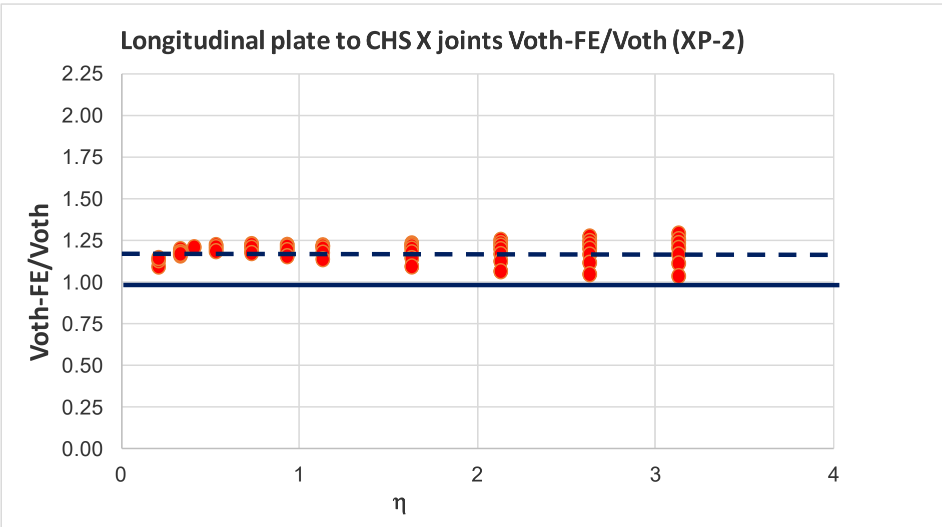


Fig. 16. Voth FE data vs. the Voth best-fit equation for longitudinal plate-to-CHS XP-2 joints loaded in compression, incl. the effect of plate thickness and weld legs in β′ and η′ (m = 1.19, CoV = 4.7%)

Fig. 17 shows the Voth data vs. the Voth-Packer [18] design equation if only the plate thickness effect is included in the design equation but the effect of the weld leg is excluded. If the weld size is neglected in the design equation then, compared to Fig. 16, an additional margin of 1.47/1.19 = 1.23 between the Voth FE data and the design equation based on the plate thickness only, results.

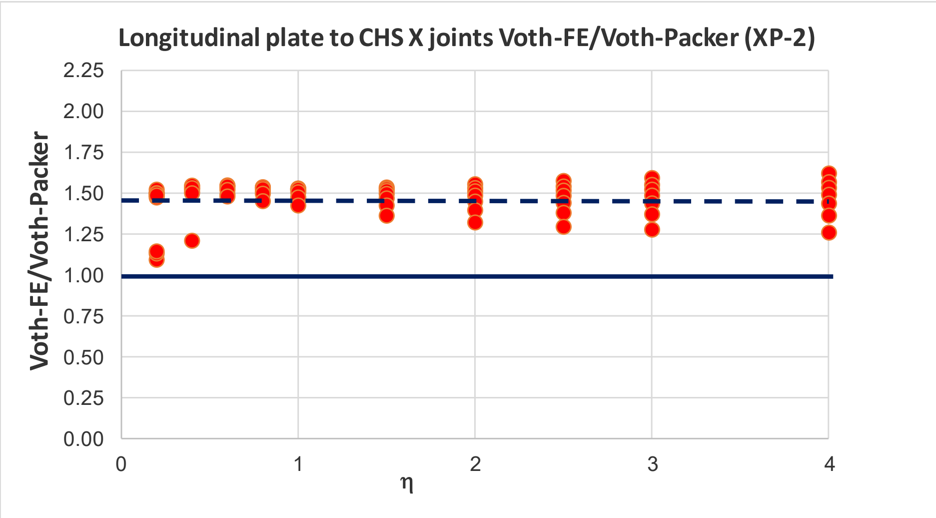


Fig. 17. Voth FE data vs. the Voth-Packer design equation for longitudinal plate-to-CHS XP-2 joints loaded in compression, including only the effect of plate thickness in β (m = 1.47, CoV = 7.1%).

For consistency with TP-2 joints, it was decided also to exclude the effect of plate thickness and weld leg size for XP-2 joints and also to base the design equation conservatively on equivalent Voth data with a butt-welded plate of 10 mm (or a β =10/219.1 = 0.046). For a joint with a butt-welded thin plate of 10 mm the $f(\beta)$ effect would be 1.075. Since all Voth FE data are based on relatively thick plates and welds with $\beta' = 0.21$, the calculated combined plate thickness and weld effect in $f(\beta')$ is 1.38. Thus, if the design equation is based on data of XP-2 joints with thin longitudinal plates of 10 mm, the Voth data with thick plates and welds would be a factor 1.38/1.075 = 1.28 higher than in Fig. 16, resulting in a mean of about 1.19x1.28 = 1.53.

As shown later on in Fig. 18, the comparison of the Voth data versus the prEN 1993-1-8 equation gives a mean of 1.59, which is close to 1.53.

Furthermore, it is notable that the Voth-Packer equation for XP-2 joints contains a ($\gamma^{0.10}$) effect while the equation for TP-2 joints has no γ effect, which is different to that for CHS T and X joints, and to that for TP-1 and XP-1 joints where the γ effect is larger ($\gamma^{0.35}$) for TP-1 joints than for XP-1 joints ($\gamma^{0.25}$). Further investigation has shown that the γ effect is only applicable to the data for $\eta < 1.0$, as shown in Fig. 18, where the γ effect in the equation can be excluded by using an average value for $f(\gamma) = 1.3$.

As for TP-2 joints, for XP-2 joints it is decided to incorporate the η effect by a function $(1+0.4\eta)$ with a calibration correction of (1.5/1.4) referenced to $\eta = 1.0$.

For XP-2 joints the Q_u function for the design resistance in prEN 1993-1-8, with $\eta = 1.0$ as a calibration point, can now be given by Eq. 6. This uses the constant of 3.5 from the Voth-Packer equation multiplied by ζ =0.85, the thickness effect correction of 1.075, the gamma effect of 1.3 and the calibration 1.5/1.4 at $\eta = 1.0$, which results in: 3.5x0.85x1.075x1.3x(1.5/1.4) = 4.4.

$$Q_u = 4.4\left(1 + 0.4\eta\right) \quad (6)$$

Fig. 18 shows all Voth FE data versus the modified Voth-Packer equation adopted for the prEN. As shown, the validity range can be extended to $0.6 \le \eta \le 4.0$.

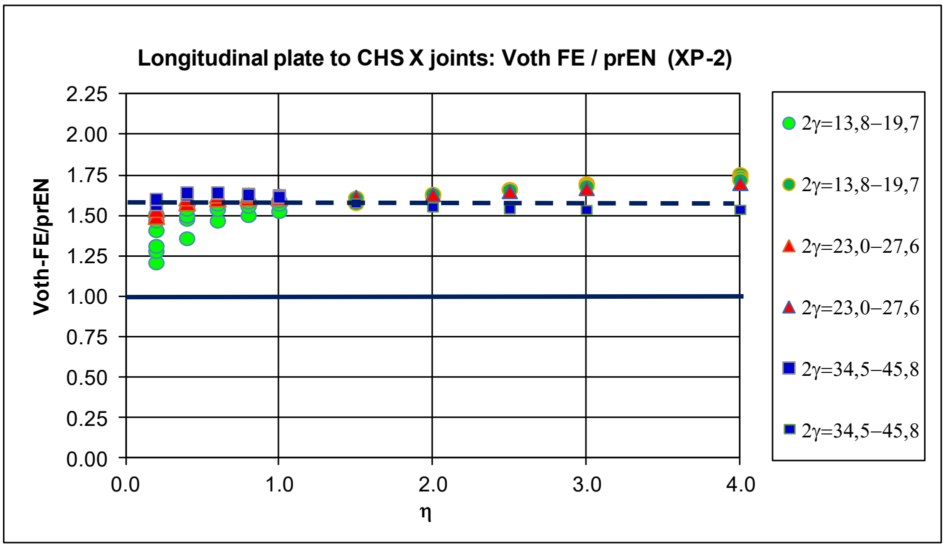


Fig. 18. Voth FE data vs. the prEN Eq. (6) for longitudinal plate-to-CHS XP-2 joints loaded in compression (m = 1.59, CoV = 6.1%)

3.4 Reanalysis based on the ultimate resistances of experiments for comparison with EN 1993-1-8 and ISO 14346

For comparison with Figs. 1 and 4 for EN 1993-1-8 and ISO 14346, respectively, Fig. 17 shows a comparison between the ultimate resistance of the XP tests of Washio et al. (see Makino et al., [10]) and the design resistance equations in prEN 1993-1-8 which incorporate a 3%d_0 maximum deformation criterion. For the data for XP Type 1 to Type 5 joints, the mean is 1.45 with a CoV of 6.2%. As shown, compared to Figs. 1 and 4, the CoV is reduced from 11.8% and 8.4% to 6.2%. For these Japanese tests the plate thicknesses were generally lower than in the Voth FE analyses.

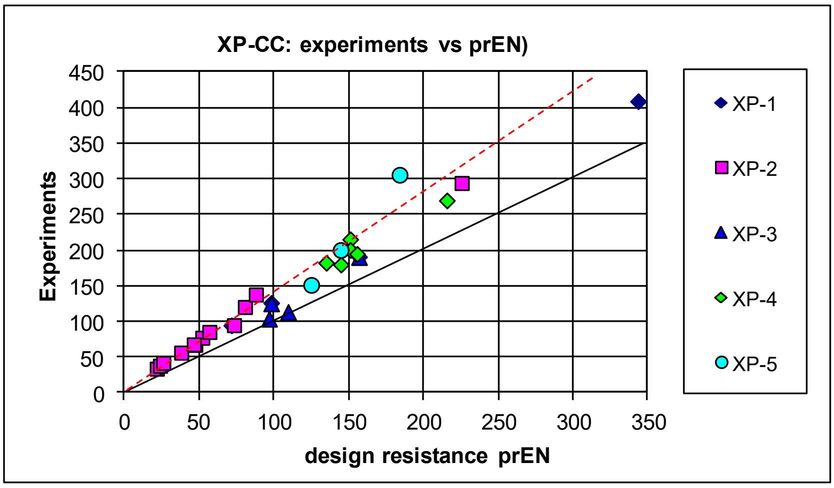


Fig. 17. Comparison of Washio XP experiments N_{1u} vs. prEN 1993-1-8 design resistances (m = 1.45 and CoV = 6.2%)

Fig. 18 shows the load deformation diagrams for three XP-1 joints with 2γ = 31.8 and different β ratios (0.5, 0.7 and 0.9). For these joints with a chord diameter of 165.2 mm, the 3%d_0 criterion is 5 mm, for which the resistances are almost the same as the ultimate resistances.

Fig. 19 shows a comparison between the ultimate resistances of the TP tests of Washio et al. (from database of Makino et al., [9], [10]) and the prEN 1993-1-8 design resistance. For these Japanese data for TP Type 1 to Type 5 joints (excluding Akiyama) the mean is 1.76 with a relatively high CoV of 15.4%, mainly caused by the relatively low TP-2 data.

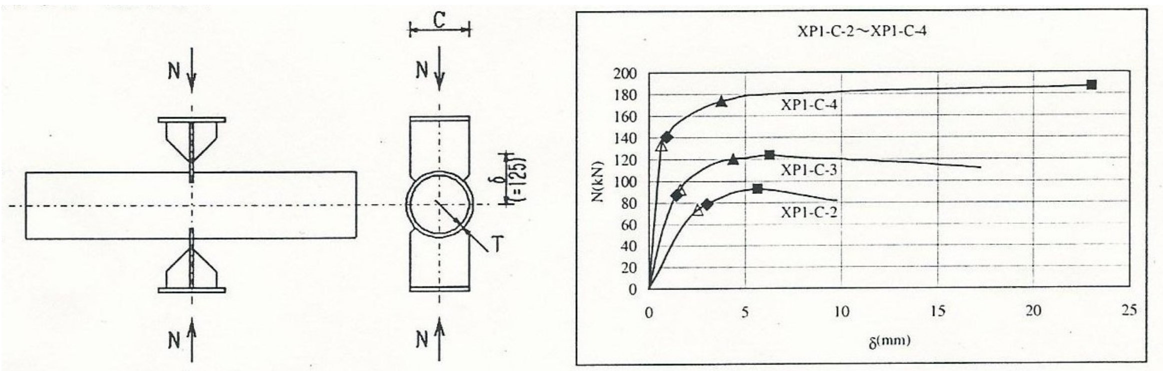


Fig. 18: Some load vs. chord deformation diagrams with different yield and ultimate resistance criteria, for XP-1 joints with 2γ = 31.8, β = 0.5, 0.7 and 0.9, (Makino et al., [10])

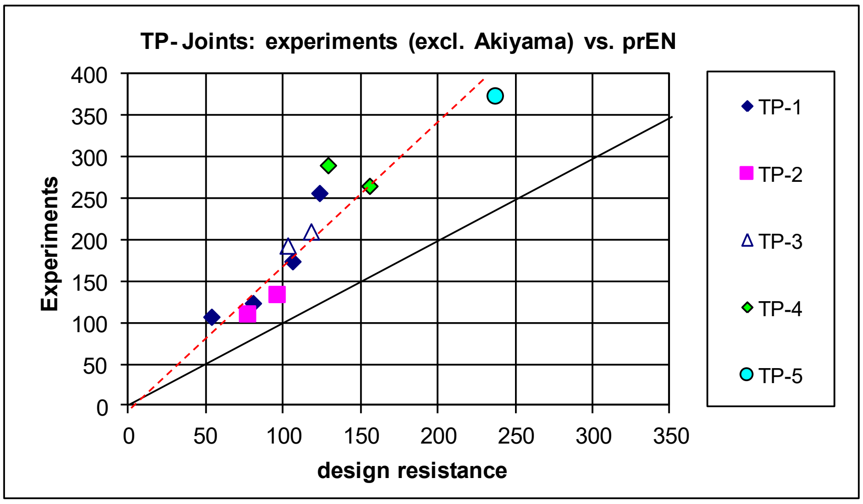


Fig. 19. Comparison of the Washio TP experiments N_{1u} vs. the prEN 1993-1-8 design resistances (m = 1.76, CoV = 15.4%)

4 Plate-to-CHS joints loaded in tension versus loading in compression

Another aspect to consider is whether separate equations should be given for compression and tension brace loading. Until now, it has been usual practice in design guides and codes to base the design resistance conservatively only on the resistance for compression loading, to avoid potential failures due to insufficient deformation capacity when loaded in tension.

Transverse plate-to-CHS joints

For similar FE models, Figs. 20 and 21 show the ratios between the Voth FE resistances for tension-loaded and compression-loaded TP-1 and XP-1 joints with transverse plates. The data in Fig. 20 for TP-1 joints have a small β effect with a large γ effect, with the lowest ratios for joints with the lowest 2γ values (ratios 1.10 – 1.13 for 2γ = 13.8 with β = 0.8 – 1.0).

The data in Fig. 21 for XP-1 joints with transverse plates also give the lowest ratios for the low 2γ values (1.12 for β = 0.2 and 1.15 for β = 1.0).

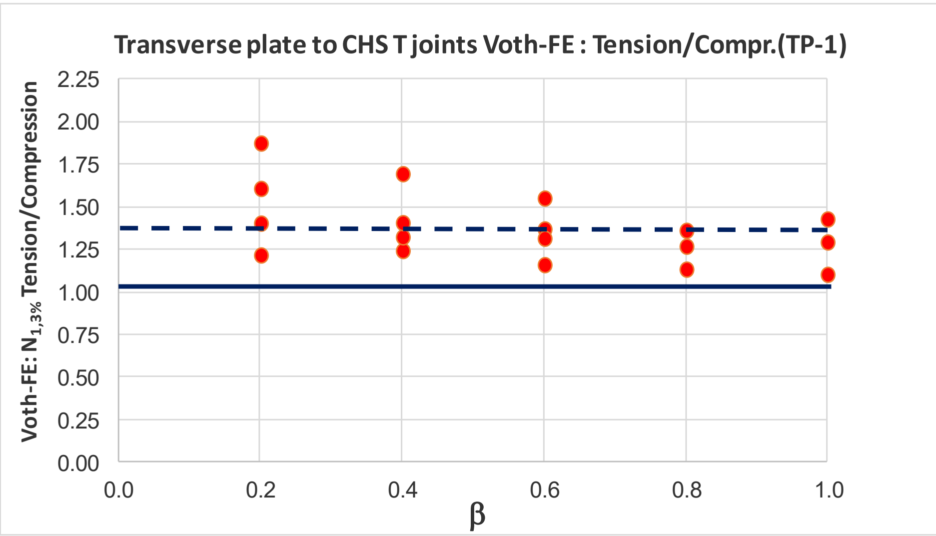


Fig. 20. TP-1 joints with a transverse plate: Tension/Compression (Voth data from similar models); (m = 1.37, CoV = 14.6%)

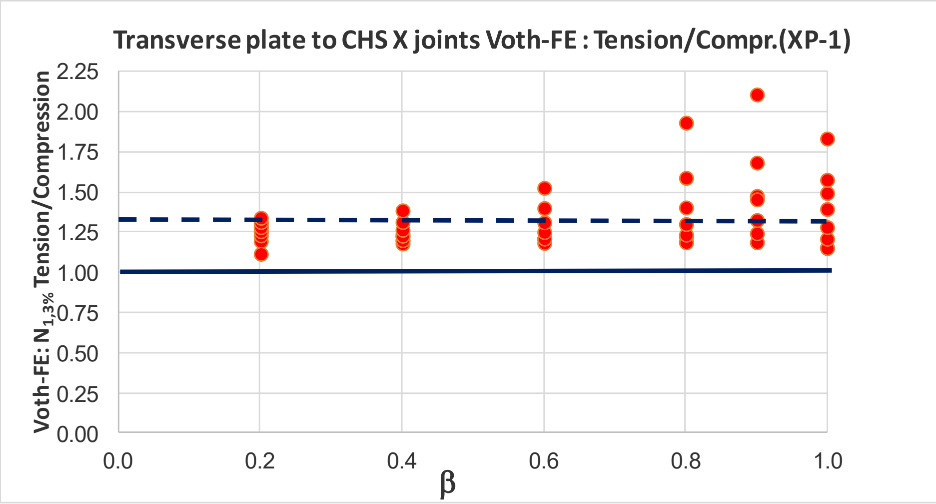


Fig. 21. XP-1 joints with a transverse plate: Tension/Compression (Voth data from similar models); (m = 1.34, CoV = 15.1%)

Longitudinal plate-to-CHS joints

For similar FE models, Figs. 22 and 23 show the ratios between the Voth FE resistances for tension-loaded and compression-loaded TP-2 and XP-2 joints with longitudinal plates. For these joints with a longitudinal plate the ratio between the resistance for tension-loaded and compression-loaded joints is reasonably constant, with a small scatter band.

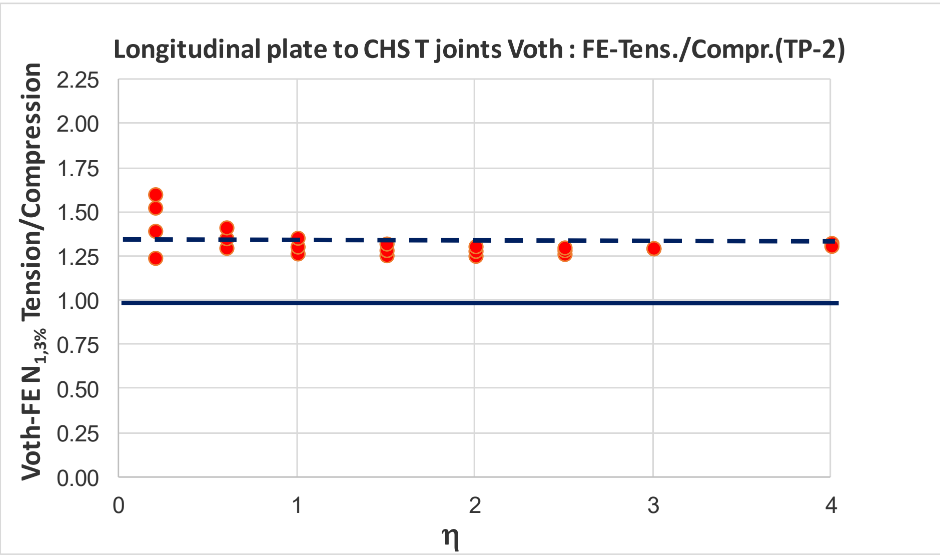


Fig. 22. TP-2 joints with a longitudinal plate: Tension/Compression (Voth data from similar models); (m = 1.34, CoV = 6.5%)

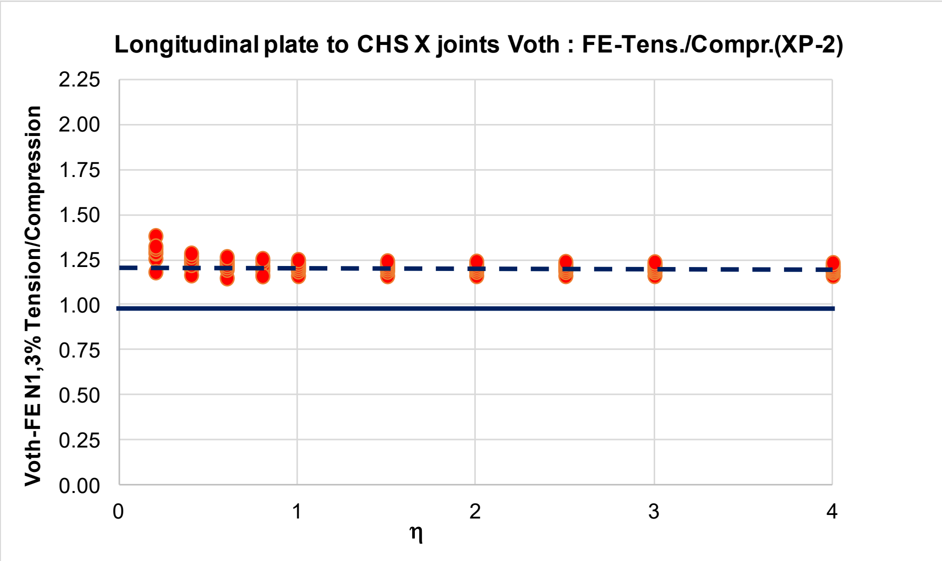


Fig. 23. XP-2 joints with a longitudinal plate: Tension/Compression (Voth data from similar models); (m = 1.22, CoV = 3.5%)

Evaluation

As shown in Voth and Packer [17] and copied here as Fig. 24, the joints loaded in tension ("left" in the figure) have, compared to compression loading, a low deformation capacity and failure may occur close to the 3%d_0 limit. This requires, in the opinion of the authors, a larger margin between failure and the design resistance.

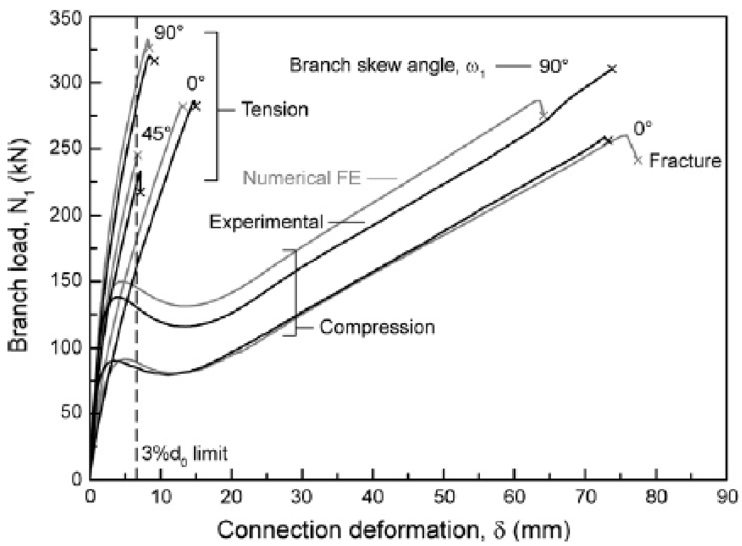


Fig. 24. Load deformation behaviour for tension- and compression-loaded TP joints (Voth and Packer, 2012)

Remark: In Fig. 24 the increased right part (after about 15 mm) in the load-deformation diagram for compression loading may not be representative for some applications in practice and only occurs for short chord length specimens after large displacements.

The prEN design equations presented earlier, all based on compression-loaded T and X joints, have been shown to give adequate safety margins and are standardized on a relatively low plate thickness (10 mm), which is a conservative measure. Under tension loading, some transverse T and X joints may have a resistance that is as low as ~5% above the compression-loaded counterpart (see Figs. 20 and 21). Such tension-loaded joints also have a lower deformation capacity (especially with β~1.0) and thus may only marginally meet safety margins for joints of low ductility. Hence, the oft-observed "favourable" resistance of tension-loaded joints relative to compression-loaded joints has not been included in these recommendations. Moreover, all of the foregoing experimental and numerical studies have been performed on mild steel CHS with a yield strength up to and including S355. For higher strength steels it is recommended to check if the brace tension loading case becomes more critical than the brace compression loading case.

5 Axially loaded through plate-to-CHS joints

Voth and Packer [20] also investigated through plate joints and showed that the resistance can conservatively be given by the sum of the resistances for compression loading and that for tension loading of TP joints. This applies both for transverse and longitudinal plate-to-CHS (TP-1 and TP-2) joints.

For prEN 1993-1-8, as discussed in section 4, since it was decided not to give separate design rules for compression and tension loading cases, the design resistance of through plate joints can simply be taken conservatively as twice that for compression plate loading.

Voth and Packer [19] also investigated skew plate joints and showed that the resistance can be given by a combination of the transverse plate-to-CHS and the longitudinal plate-to-CHS joint resistance. However, it was decided not to include this case in the updated EN-1993-1-8.

6 Combined transverse and longitudinal cruciform plate-to-CHS (TP-3 and XP-3) joints

For axial loading, the connection of a transverse plate with the CHS chord is much stiffer than that of a longitudinal plate, thus the axial resistance is governed by the resistance of the transverse-plate-to-CHS (TP-3 or XP-3) joint.

For in-plane bending, the resistance is governed by that of the longitudinal plate-to-CHS (TP-2 or XP-2) joints and for out-of-plane bending the resistance is governed by that of the transverse plate-to-CHS (TP-1 or XP-1) joints. These joints are also not explicitly included in prEN 1993-1-8, because it is clear that for axial loading the longitudinal plate connection is weaker compared to the transverse plate connection and hardly contributes. For bending, the plate at the neutral axis does not contribute to load transfer.

7 TP and XP joints loaded by bending moments

Transverse plate-to-CHS joints

Joints with transverse plates are not suitable for the transfer of in-plane-bending moments, for which the resistance is set to zero.

For TP-1 and XP-1 joints with a transverse plate and loaded by an out-of-plane bending moment, no test data or numerical data are available. However, based on the relationship between axially loaded CHS X joints and CHS X joints loaded by out-of-plane bending, the out-of-plane bending moment resistance is taken as $0.5b_1N_{1,Rd}$ (see EN 1993-1-8 and ISO 14346). This has also been used in prEN 1993-1-8 for TP-1 and XP-1 joints (see Tables 6 and 7).

Longitudinal plate-to-CHS joints

Joints with longitudinal plates are not suitable for the transfer of out-of-plane-bending moments, for which the resistance is set to zero.

For TP-2 joints with longitudinal plates loaded by in-plane-bending only limited experimental test data are available (Makino et al., 1998) and generally only for the higher 2γ ratios and η ratios up to 2.5. Fig. 25 shows that the relationship $0.7h_1N_{1,(TP\text{-}2)}$ between the test data for in-plane-bending and the axial resistance for TP-2 joints gives a lower bound with a CoV of 5.5%. For XP-2 joints with a longitudinal plate, Fig. 26 shows that the relationship $h_1.N_{1,(XP\text{-}1)}$ between the test data (Makino et al., [10]) for in-plane-bending and the axial resistance for XP-2 joints presents an approximate lower bound with a CoV of 7.3%. Because of the small number of data points and a limited η range, it was decided to use the same relationship as for TP-2 joints here; i.e. using $0.7h_1$ (see Tables 6 and 7).

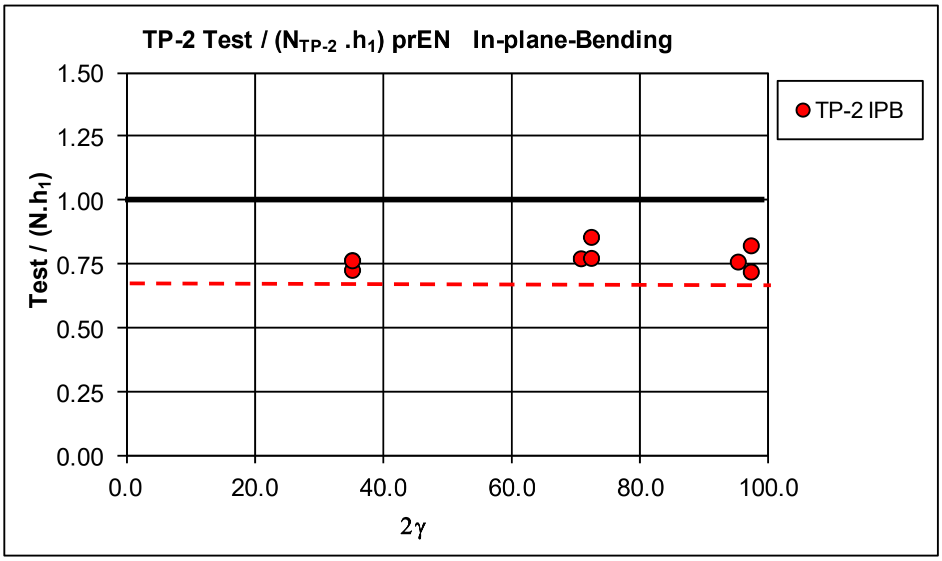


Fig. 25. Comparison of the TP-2 data (Makino et al. [10]) vs. the prEN 1993-1-8 design resistance equation for axial loading

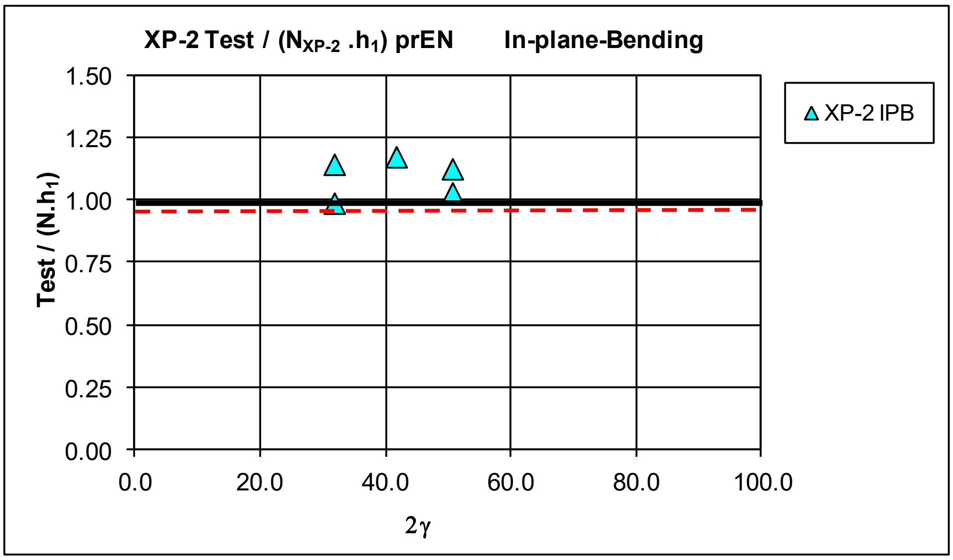


Fig. 26. Comparison of the XP-2 data (Makino et al. [10]) for bending-in-plane with the prEN 1993-1-8 design resistance for axial loading

8 I or H section brace-to-CHS (TP-4 and XP-4) joints

For the determination of the η effect in TP-4 and XP-4 joints with I or H section braces (with the brace oriented as shown in Tables 6 and 7), only test data based on ultimate load were used for the evaluation of the design equations for EN 1993-1-8 and ISO 13346. For EN 1993-1-8, these gave an η function (1+0.25η) with a CoV = 5.6% and for the ISO an η function (1+0.4η) with a CoV = 5.7%. The scatter is therefore similar, but in the available tests η only varied between 1 and 2 and there are only six XP-4 tests. Since (1+0.4η) is adopted for the axially loaded TP-2 and XP-2 equations, this has also been chosen for the TP-4 and XP-4 joint equations.

The Q_u for the design resistance of TP-4 joints is now given by (see Table 6):

$$Q_{u,(TP\text{-}4)} = Q_{u,(TP\text{-}1)} \left(1 + 0.4\eta\right) \tag{7}$$

and for XP-4 (see Table 7):

$$Q_{u,(XP\text{-}4)} = Q_{u,(XP\text{-}1)} \left(1 + 0.4\eta\right) \tag{8}$$

For I or H section brace-to-CHS (TP-4 and XP-4) joints the axial load resistance has to be limited to twice that for an axially loaded transverse plate-to CHS (TP-1 or XP-1) joint.

Since the TP-2 and XP-2 joints already have a large ratio in the order of 1.5 between the ultimate resistance and the resistance based on a deformation of $3\%d_0$, no further statistical treatment has been applied.

De Winkel [25] investigated XP-4 joints under in-plane moment loading, with web and without web, which showed that the effect of the web is negligible. In his analysis the moment resistances were taken at a rotation of 0.06 rad. which agrees with a rotation of $0.06/\eta$ for $\eta = 1.0$ or a local deformation of $0.03d_0$. Further, as shown in Fig. 27, relating the bending moment resistance to the axial resistance of a comparable XP-1 joint shows a lower bound for $h_m.N_{1,(XP-1)}$.

For TP-4 joints under in-plane moment loading, no data are available based on the $3\%d_0$ deformation limit, but these can be treated conservatively in a similar manner to the XP-4 joints above (see Table 6).

For out-of-plane bending no test data or numerical data are available. Based on the relationship used for the resistance between out-of-plane bending versus the resistance for axial loading for CHS X joints in the current EN 1993-1-8 and in ISO 14346, the out-of-plane moment capacity is taken as $0.5b_1N_{1,Rd}$. This has also been used for TP-1 and XP-1 joints, as also used here for TP-4 and XP-4 joints (see Tables 6 and 7).

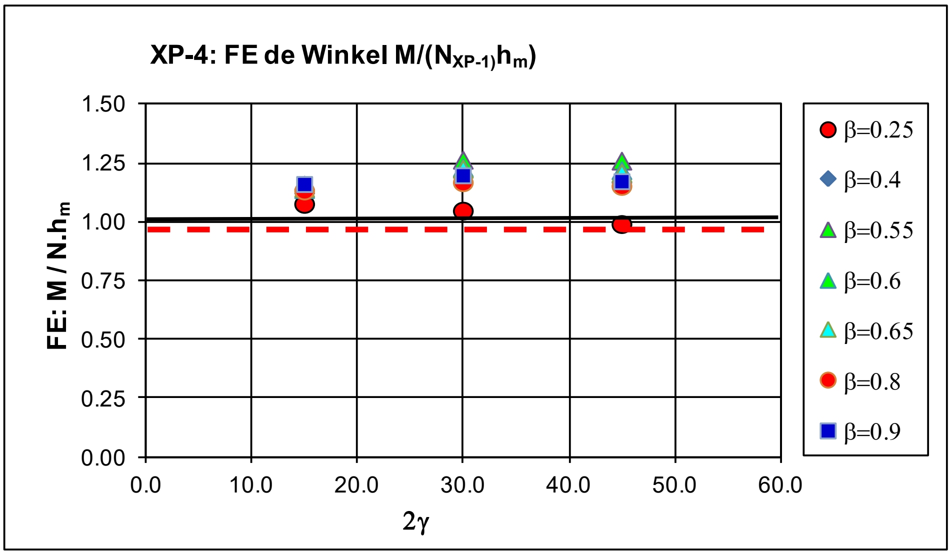


Fig. 27. XP-4 In-Plane-Bending/($N_{1,(XP-1)}.h_m$) based on the de Winkel [25] data (rotation limit $0.06/\eta$) vs. 2γ

For joints with RHS-to-CHS (TP-5 and XP-5) joints, no data are available based on the $3\%d_0$ deformation limit but, just as in the current EN 1993-1-8 and ISO 14346, both may be conservatively treated as TP-4 or XP-4 joints. The punching shear check, in this case, has to be applied to the shear area at the RHS section outer perimeter.

9 Evaluation for design rules

As discussed in section 3, the procedure used to determine the design resistance for hollow section joints for IIW 2009 and ISO 14346 (van der Vegte et al., [15]) with a partial safety factor of $\gamma_M = 1.1$, generally results in a reduction factor of about 0.80 to the mean data. This means a total factor of about 1.25 between the mean of the data base and the design resistance. Since the plate joint design resistances herein are conservatively based on joints with a butt-welded thin plate with a plate thickness of 10 mm, the resulting mean values for the Voth data for axial compression-loaded plate-to-CHS are considerably higher due to the relatively thick plates and welds.

Some joints with transverse plates, loaded in tension with the lowest data at $\beta = 0.8$ - 1.0 for S355 marginally meet a margin of 1.25 if the margins for compression and the additional margin for the tension/compression loading ratio are combined.

This is the minimum margin accepted for joints with a low deformation capacity, although it may be expected that for the joints considered, the secondary effects will be minimal.

To cover joints with steels above S355 up to S460, ISO 14346 uses a material factor of $C_f = 0.9$ in combination with a yield stress limit of $f_y \leq 0.8f_u$, which is also used here.

In addition to the equations discussed before, the joints should be checked for plate failure by gross yielding which can be considered as "member" failure. Chord punching shear failure is another limit state that may need to be checked, for some joint types, but it is noteworthy that this failure mode is incorporated implicitly in the Voth numerical data, on which the plate-to-CHS joint design recommendations herein are based. To limit the strains, and considering that these joints have high strain concentration factors, the punching shear limit state (when applied to non-plate joints or to plate-to-CHS joints with a yield stress f_y >355 N/mm^2) should be based on an elastic approach, as shown in Tables 6 and 7.

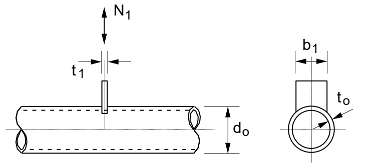
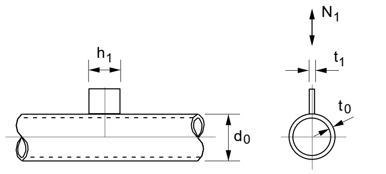
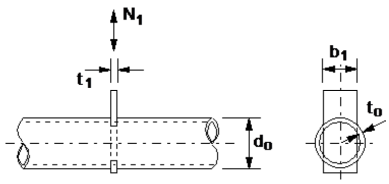
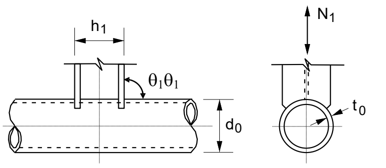
9 Conclusions

The resulting design recommendations for plate-to-CHS T and X (TP and XP) joints, with transverse and longitudinal plates as well as with an I, H or RHS brace, are conservatively based on TP and XP joints with butt-welded plates and a plate thickness of 10 mm. They are summarised in Tables 6 and 7 and presented as in prEN 1993; i.e. separately for TP and XP joints. An alternative could have been to include a plate thickness effect function of $(1+0.4\eta)$ for TP-1 and XP-1 joints and $(1+3\beta^2)$ for TP-2 and XP-2, which would also give consistent equations. However, this will change the relation with the equations for the joints with I, H or RHS braces.

10 Acknowledgement

Appreciation is extended to Dr. A.P. Voth for making his FE data files and background reports available for a re-analysis of the relevant functions, for prEN 1993-1-8.

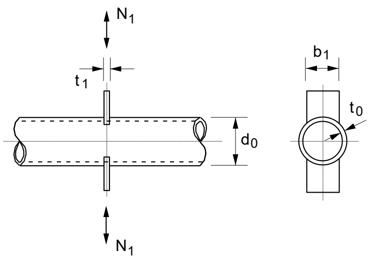
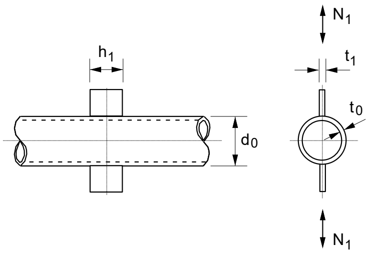
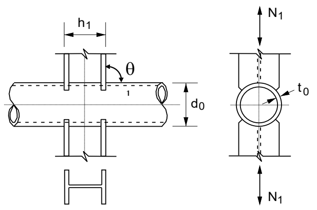
Table 6. Design resistances of welded T-type joints connecting plate, I, H or RHS to CHS chords

Type of TP joint	Design resistance	
Transverse Plate*	**Chord failure**	
	$N_{1,Rd} = 2.3\,C_f\ f_{y0}\ t_0^2\left(1+3\beta^2\right)\gamma^{0.35}Q_f/\gamma_{M5}$	$M_{ip,1,Rd} = 0$ $M_{op,1,Rd} = 0.5\,b_1\ N_{1,Rd}$
Longitudinal plate*	**Chord failure**	
	$N_{1,Rd} = 7.1 C_f f_{y0}\ t_0^2\left(1+0.4\eta\right)Q_f/\gamma_{M5}$	$M_{ip,1,Rd} = 0.7 h_1\ N_{1,Rd}$ $M_{op,1,Rd} = 0$
Through plate*	**Chord failure**	
	$N_{1,Rd} = 2N_{1,Rd}$ for a transverse and for a longitudinal plate T joint	
I, H or RHS brace	**Chord failure**	
	$N_{1,Rd} = N^*_{Rd}(1+0.4\eta)$ but $\le 2N^*_{Rd}$ with $N^*_{Rd} = N_{1,Rd}$ for a transverse plate T joint	$M_{ip,1,Rd} = h_{1,m}\ N_{1,Rd}/(1+0.4\eta)$ $M_{op,1,Rd} = 0.5 b_1\ N_{1,Rd}$
	Chord punching shear	(for $b_1 \le d_0 - 2t_0$)
I section brace (with axial loading or out-of-plane bending) and RHS brace	$\frac{N_{1,Ed}}{A_1} + \frac{M_{ip,1,Ed}}{W_{el,ip,1}} + \frac{M_{op,1,Ed}}{W_{el,op,1}} \le 0.58 C_f f_{y0}\frac{t_0}{t_1}$ (t_1 = flange thickness I section brace)	
I or H section loaded by in-plane bending	$\frac{N_{1,Ed}}{A_1} + \frac{M_{ip,1,Ed}}{W_{el,ip,1}} + \frac{M_{op,1,Ed}}{W_{el,op,1}} \le 1.16 C_f f_{y0}\frac{t_0}{t_1}$	(t_1 = flange thickness)

Chord stress factor $Q_f = \left(1-\lvert n\rvert\right)^{C_1} \ge 0.3$		$n < 0$ (compr.)	$n \ge 0$ (tension)
	T, Y and X joints	$C_1 = 0.25$	$C_1 = 0.20$

Material factor C_f = 1.0 for f_{y0} and $f_{y1} \le 355$ N/mm^2 ; **C_f = 0.9** for $355 \le f_{y0}$ or $f_{y1} \le 460$ N/mm^2		
Validity Range (in addition to the limits given for CHS T and X joints in prEN 1993-1-8		
General	$\theta_1 = 90°$	$f_{y0} \le 0.8 f_{u0}$ and $f_{y1} \le 0.8 f_{u1}$
Plates	Transverse plate: $\beta = \frac{b_1}{d_0} \ge 0.25$	Longitudinal plate: $0.6 \le \eta = \frac{h_1}{d_0} \le 4$
I, H and RHS sections		$0.6 \le \eta \le 2.5$
*Note: Plates also have to be checked based on loads from the connected members		

Table 7. Design resistances of welded X-type joints connecting plate, I, H or RHS to CHS chord

Type of XP joint	Design resistance	
Transverse Plate*	**Chord failure**	
	$N_{1,Rd} = 2.1 C_f \, f_{y0} \, t_0^2 \left(1+3\beta^2\right)\gamma^{0.25} \, Q_f / \gamma_{M5}$	$M_{ip,1,Rd} = 0$ $M_{op,1,Rd} = 0.5 \, b_1 \, N_{1,Rd}$
Longitudinal plate*	**Chord failure**	
	$N_{1,Rd} = 4.4 C_f f_{y0} t_0^2 \, (1+0.4\eta) \, Q_f / \gamma_{M5}$	$M_{ip,1,Rd} = 0.7 \, h_1 \, N_{1,Rd}$ $M_{ip,1,Rd} = 0$
I, H or RHS brace	**Chord failure**	**I, H or RHS brace**
	$N_{1,Rd} = N^*_{Rd} \, (1+0.4\eta)$ but $\leq 2N^*_{Rd}$ with $N^*_{Rd} = N_{1,Rd}$ for a transverse plate X joint	$M_{ip,1,Rd} = h_{1,m} \, N_{1,Rd} / (1+0.4\eta)$ $M_{op,1,Rd} = 0.5 b_1 \, N_{1,Rd}$

	Chord punching shear (for $b_1 \leq d_0 - 2t_0$)
I section brace (with axial loading or out-of-plane bending) and RHS brace	$\frac{N_{1,Ed}}{A_1} + \frac{M_{ip,1,Ed}}{W_{el,ip,1}} + \frac{M_{op,1,Ed}}{W_{el,op,1}} \leq 0.58 C_f f_{y0} \frac{t_0}{t_1}$ (t_1 = flange thickness I section brace)
I or H section loaded by in-plane bending	$\frac{N_{1,Ed}}{A_1} + \frac{M_{ip,1,Ed}}{W_{el,ip,1}} + \frac{M_{op,1,Ed}}{W_{el,op,1}} \leq 1.16 C_f f_{y0} \frac{t_0}{t_1}$ (t_1 = flange thickness)

Chord stress factor $Q_f = \left(1-\lvert n \rvert\right)^{C_1} \geq 0.3$		n < 0 (compr.)	n ≥ 0 (tension)
	T, Y and X joints	$C_1 = 0.25$	$C_1 = 0.20$

Material factor C_f = 1.0 for f_{y0} and $f_{y1} \leq 355$ N/mm^2 ; **C_f = 0.9** for $355 \leq f_{y0}$ or $f_{y1} \leq 460$ N/mm^2

Validity Range (in addition to the limits given for CHS T and X joints in prEN 1993-1-8		
General	$\theta_1 = 90°$	$f_{y0} \leq 0.8 f_{u0}$ and $f_{y1} \leq 0.8 f_{u1}$
Plates	Transverse plate: $\beta = \frac{b_1}{d_0} \geq 0.25$	Longitudinal plate: $0.6 \leq \eta = \frac{h_1}{d_0} \leq 4$
I, H and RHS sections		$0.6 \leq \eta \leq 2.5$
*Note: Plates also have to be checked based on loads from the connected members		

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Keywords

CHS, Circular hollow section joints, EN 1993-1-8, ISO 14346, design recommendations.

Abbreviations and symbols

Organisations

CIDECT   Comité International pour le Développement et l'Etude de la Construction Tubulaire

CHS Circular Hollow Section

IIW International Institute of Welding

ISO International Standards Organisation

RHS Rectangular Hollow Section

Symbols

A_1 cross sectional area of plate or brace

C_1 exponent in chord stress function $Q_f = f(n)$

C_f material factor

CoV coefficient of variation

$M_{ip,1,Ed}$ design applied in-plane bending moment in plate or brace

$M_{op,1,Ed}$ design applied out-of-plane bending moment in plate or brace

$M_{ip,1,Rd}$ design resistance of a joint, expressed in terms of in-plane bending moment in plate or brace

$M_{op,1,Rd}$ design resistance of a joint, expressed in terms of out-of-plane bending moment in plate or brace

N_1 applied axial force in plate or brace

$N_{1,Ed}$ design applied axial load in plate or brace

$N_{1,Rd}$ design resistance of a joint, expressed in terms of axial load in plate or brace

Q_f chord stress function (in ISO 14346)

Q_u function in the design resistance equations accounting for the effect of the geometrical parameters β, γ and η

$W_{el,ip,1}$ elastic section modulus for in-plane bending moment in plate or brace

$W_{el,op,1}$ elastic section modulus for out-of-plane bending moment in plate or brace

b_1 transverse plate width or width of brace, normal to the plane of the joint

d_0 chord diameter

f_{u0} ultimate stress of chord

f_{u1} ultimate stress of plate or brace

f_{y0} yield stress of chord

f_{y1} yield stress of plate or brace

h_m depth of I, H or RHS section minus flange thickness (h_1-t_1), in the plane of the joint

h_1 longitudinal plate length or depth of brace, in the plane of the joint

$h_{1,m}$ h_1-t_1

k_n chord stress function in EN 1993-1-8 for RHS joints, based on maximum chord stress

k_p chord stress function in EN 1993-1-8 for CHS joints, based on chord prestress

m mean of test results

n maximum normal stress (due to axial load plus bending) in the chord connecting surface

t_0 chord thickness

t_1 plate or brace flange thickness

β transverse plate width to chord diameter ratio

β' transverse plate width (or plate thickness of longitudinal plate) to chord diameter ratio, including the weld leg sizes

2γ chord diameter to thickness ratio

γ_M, γ_{M5} partial factor

η longitudinal plate length to chord diameter ratio

η' longitudinal plate length (or plate thickness of transverse plate) to chord diameter ratio including the weld leg sizes

ζ resistance factor proposed by Voth-Packer to obtain the design resistance from the mean value

θ_1 brace-to-chord angle

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