AXIALLY LOADED JOINTS OF CHSINHSS

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To limit the joint deformation and the strains and to take account for the reduced deformation capacity of high strength steels (HSS), the design resistances of hollow section joints are to be reduced if high strength steels with yield strengths higher than 355 MPa are used. Furthermore, the unequal stiffness distribution along the connection perimeter of K/N gapped joints and the rotational stiffness of the joints result in secondary bending moments. Especially, for joints of high strength steels secondary bending moments may not be redistributed sufficiently due to the reduced deformation and rotation capacity of the joints and the remaining bending moments must be considered in design. Thus, ISO 14346 and CIDECT Design Guide 1 limit the yield strength to 80% of the tensile strengthfor the design of joints of high strength steels. In EN 1993-1-8 this additional limitation is not included, but it will be included it in the next revision of Eurocode 3.

Keywords: High strength steel,FEM, experimental investigations, Eurocode 3, CIDECT, ISO

1 Introduction

Due to their favorable structural behavior and due their advantages regarding design, framework constructions of steel hollow sections are gaining in popularity, even of high strength steels. Examples are given e.g. in Lan et al. (2018a). The use of high strength steels is particularly justified in cases where the higher allowable stresses ensure an economically and architecturally attractive lightweight construction, for which the costs can significantly be reduced. The use of highstrength steels can give savings in terms of weight, thus assembly and transport costs as well as cost reductions in secondary constructions such as foundations. Lower weld volumes also significantly reduce welding costs. However, these advantages are only of minor importance if reductions must be applied to design resistances for joints of high strength steel hollow sections.This publication summarizes existing investigations on CHS joints of high strength steelsand evaluates existing test results regarding the planned revision of Eurocode 3 (prEN1993-1-8).

2 Current design recommendations

2.1 *Materialfactor and reduced yield strength*

To limit the joint deformation as well as the strains and to take account for the reduced deformation capacity of high strength steels, the design resistances of hollow section joints are to be reduced if high strength steels with minimum yield strengths higher than f_y> 355MPa are used.Furthermore, the unequal stiffness distribution along the connection perimeter of K and N joints and the rotational stiffness of the joints results in secondary bending moments. Especially, for joints of high strength steels secondary bending moments may not be redistributed

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sufficiently due to the reduced deformation and rotation capacity of the joints and the remaining bending moments must be considered in design.

According to EN1993-1-8 (Eurocode 3), ISO14346 and CIDECT Design Guide 1 (Wardenier et al. 2010) the design resistances of joints of steels with a yield strength 355 MPa $<f_y \leq$ 460MPa are to be reduced by 10%. Additionally, ISO14346 as well as CIDECT Design Guides 1limit the yield strength $f_{y,red}$of the chord f_{y0} and the braces $f_{yi}$to 80% of the tensile strength f_{u0} resp. f_{ui}. However, the design resistance of joints of steels with an even higher yield strength between 460MPa and 700MPa can only be determined according to EN1993-1-12 by reducing the design resistances by 20%.In prEN 1993-1-8 an additional material factor of C_f = 0,86 obtained by linear regression is included for joints of steels with yield strengths 460 MPa $<f_y \leq$ 550 MPa (Table1). For joints of steels with a yield strength 550 MPa $<f_y \leq$ 700 MPa the resistances have to be reduced by 20 %, what is in accordance with EN 1993-1-12.

Table 1 Material factors of prEN 1993-1-8with material properties according to prEN 10210-2

Steel grade	Min. yieldstrength f_y MPa	Material factor C_f	Min. ultimate strength f_u MPa	Reduced yieldstrength $f_{y,red}$ MPa	Overall reduction $C_f f_{y,red}$ MPa	Ratio $f_y/(C_f f_{y,red})$
S355J2H	355$^{1)}$	1,0	470$^{1)}$	355	355	1,00
S460NH	460$^{2)}$	0,9	540$^{2)}$	432	389	0,85
S500MH	500$^{2)}$	0,8$^{3)}$/0,86$^{4)}$	580$^{2)}$	464	371 / 399	0,74 / 0,80
S550MH	550$^{2)}$	0,8$^{3)}$/0,86$^{4)}$	600$^{2)}$	480	384 / 413	0,70 / 0,75
S700MH	700$^{2)}$	0,8$^{3)}$	750$^{2)}$	600	480	0,69

Annotation:
1) wall thickness t$\leq$ 16 mm for the yield and $3 \leq t \leq 100$ mm for the tensile strengths.
2) wall thickness $t \leq 16$ mm for the yield and $t \leq 65$ for the tensile strengths.
3) according to EN1993-1-12.
4) according to prEN1993-1-8.

Neither EN 1993-1-8 nor EN 1993-1-12 include an additional limitation of the yield strength to $f_y \leq 0{,}8 \cdot f_u$but it is included in the current versionof prEN 1993-1-8 for the check of punching shear and brace failure. For joints of high strength steels with a yield strength higher than 550 MPa and less than or equal to 700 MPa the additional limitation results in a further reduction of 11% compared to EN 1993-1-12 if punching shear or brace failure is governing.

2.1 *Determination* of the joint design resistance

The design resistance for chord (face) failure (CFF) for T, Y and X joints and for K and N joints with gapaccording to EN 1993-1-8, ISO 14346 and CIDECT Design Guide 1 as well asthe current version of prEN 1993-1-8 are given in Table 2.

There, there-evaluations of the results of the experimental and numerical investigations are based on joint resistances determined withthe $3\% \cdot d_0$ deformation limit of Lu et al. (1994). Therefore, modifications to the design resistance of chord (face) failure are applied to ISO 14346, CIDECT Design Guide1 and prEN 1993-1-8.These modifications are not only related to modified influences of the width ratio β and the chord slenderness γ but to a completely revised influence of the chord stress reduction which now takes account of reductions due totensile loaded chordsand the joint configuration as well (Table2).Also, the influence of the gap is modified. Furthermore, in contrast to the determination of the chord stress reduction k_p in EN 1993-1-8 the chord stress reduction $Q_f$in ISO 14346, CIDECT Design Guide 1 and prEN1993-1-8 are based on a negative compressive chord stress and on the maximum chord stress $\sigma_{0,Ed}$in saddle of the joint.

Table 2 Design resistance for chord(face) failure of axially loaded CHS

	EN 1993-1-8	ISO 14347[1)]& CIDECT DG 1[1)] and prEN 1993-1-8
T, Y	$C_f \dfrac{\gamma^{0,2} k_p f_{y0} t_0^2}{\sin\theta_1}(2{,}8 + 14{,}2\beta^2)/\gamma_{M5}$	$C_f \dfrac{f_{y0} t_0^2}{\sin\theta_1}(2{,}6 + 17{,}7\beta^2)\gamma^{0,2} Q_f/\gamma_{M5}$
X	$C_f \dfrac{k_p f_{y0} t_0^2}{\sin\theta_1}\left(\dfrac{5{,}2}{1 - 0{,}81\beta}\right)/\gamma_{M5}$	$C_f \dfrac{f_{y0} t_0^2}{\sin\theta_1}\left(\dfrac{2{,}6 + 2{,}6\beta}{1 - 0{,}7\beta}\right)\gamma^{0,15} Q_f/\gamma_{M5}$
K, N (gap)	$C_f \dfrac{k_g k_p f_{y0} t_0^2}{\sin\theta_1}\left(1{,}8 + 10{,}2\dfrac{d_1}{d_0}\right)/\gamma_{M5}$	$C_f Q_g \dfrac{f_{y0} t_0^2}{\sin\theta_1}(1{,}65 + 13{,}2\beta^{1,6})\gamma^{0,3} Q_f/\gamma_{M5}$
with	$k_g = \gamma^{0,2}\left(1 + \dfrac{0{,}024\gamma^{1,2}}{1 + e^{(0,5g/t_0 - 1,33)}}\right)$	$Q_g = 1 + \dfrac{1}{1{,}2 + \left(\frac{g}{t_0}\right)^{0,8}}$
	$k_p = \begin{cases} 1 - 0{,}3n_p(1 + n_p) \leq 1{,}0 & (n > 0)^{2)} \\ 1{,}0 & (n \leq 0) \end{cases}$	$Q_f = (1 - \lvert n \rvert)^{C_1} \geq 0{,}4$
		T, Y, X: $C_1 = \begin{cases} 0{,}45 - 0{,}25\beta & (n < 0)^{2)} \\ 0{,}20 & (n \geq 0) \end{cases}$
		K, N withgap: $C_1 = \begin{cases} 0{,}25 & (n < 0)^{2)} \\ 0{,}20 & (n \geq 0) \end{cases}$

Annotation: 1) The yield strength f_{yi} and f_{y0} should not exceed 80% of the tensile strength f_{ui} and f_{ui}.
2) Compression.

In addition to chord (face) failure, punching shear has tobe checked if $d_i \leq d_0 - 2t_0$. In principle, the design resistance for punching shear is obtained in all recommendations with Equation (1). However, in ISO 14346 and CIDECT Design Guide 1 $1/\sqrt{3}$ is replaced by 0,58, what results in minor deviations of the design resistances for punching shear (Table3).

$$N_{i,Rd} = C_f \frac{f_{y0}}{\sqrt{3}} \pi d_i t_0 \frac{1 + \sin\theta_i}{2\sin^2\theta_i} / \gamma_{M5} \quad (1)$$

Of course, member checks as e.g. checking the chord shear resistance orchecking the axial chord resistances reduced due to interaction effects,must be considered in design. For the determination of joint resistances these checks are not relevant.

3 Experimental and numerical test evidence for joints of circular hollow sections (CHS)

Makino et al (1996) maintain a database covering results of experimental and numerical investigations of T, Y, X and K joints with gap or overlap of CHS carried out until 1996. Axially loaded joints as well as joints loaded by in-plane bending with and without chord prestresses are included in the database. If available measured dimensions and material properties are provided otherwise nominal values are given. Various researchers contribute to the database and it would exceed the limits to mention all here.The deformation limit of (Yura et al. 1981) are used for the determination of the joint resistances for chord (face) failure.Unfortunately, no load deflections curves or results of local deformation measurements are recorded in the database, thus only little information of the deformation and rotation capacity of the joints are available.

In 1998 Noordhoek et al. (1998) experimentally investigate seven axially loaded X joints with brace angles of $\theta_1 = 90°$ of cold formed CHS in grades FeE355, FeE450 and Fe E 700, four under compression and three in tension. The joint resistances are determined based on the 3%·b_0 deformation limit. All joints have chords with a nominal diameter of $d_0 = 450$ mm and braces of

d_i = 370 mm, both have a thickness of $t_0 = t_i$ = 10 mm. Noordhoek et al. (1998) also report measured dimensions and material properties, weld dimensions and imperfections of the investigated joints. Additionally, load-deflections and results of strain measurements are given for all tests.

Puthli et al. (2010, 2014) experimentally investigate symmetric X joints with brace angles of θ_1 = 90° of CHS in steel grades S 355, S 460, S 690 and a small number of joints in grade S 770. 31 tests are carried out under compression, 18 in tension and 23 are loaded by in-plane bending (IPB). Additional chord loads are not applied in the experimental investigations. The specimens have slenderness ratios 2γ between 11 and 31, diameter ratios β varying from 0,38 to 1,00 and wall thickness ratios τ between 0,22 and 1,14.

The investigation of the geometrical influence on the material factor $C_f$is also carried out by Puthli et al. (2010, 2014) on basis of numerical investigations for symmetric X joints with brace angles of θ_1 = 90°. For axially loaded joints fillet welds with a throat thickness according to CIDECT Design Guide 1 are considered if the wall thickness of the braces doesn'texceed 8 mm. For thicker brace sections butt welds are used in the numerical model.

All joints have a chord diameter of d_0 = 323,9mm. By varying the thickness t_0 and t_1 of the sections as well as the brace diameter d_1, joints with diameter ratios of $0,6 \leq \beta \leq 0,8$, chord slenderness ratios of $15 \leq 2\gamma \leq 40$ and wall thickness ratios of $0,4 \leq \tau \leq 1,0$ are investigated for axially loaded joints. Since no significant influence of the thickness ratio τ on the reduction factors$C_f$is observed for axially loaded joints, the investigated parameter ranges of joints under IPB are reduced to joints with a thickness ratio of τ = 1,0. Furthermore, joints with a slenderness of the braces d_i/t_i< 10 or d_i/t_i> 50 are not considered by the parameter studies.

For the numerical investigations a nonlinear behavior (E = 210 GPa), taking isotropic strain hardening into account, is used. The yield strengths f_{y0} and f_{y1} and ultimate strengths $f_{u0}$and $f_{u1}$are taken from standards. Since no information of the uniform strains A_{gt} for high strength steels is available, they are assumed toA_{gt} = 10% for axially loaded joints. For joints loaded by IPB, realistic uniform strains provided from steel producers Dillinger Hüttenwerke, voestalpine and Vallourec are used (Fleischer et al. 2008).

Lan et al. (2018b) numerically determine joint resistances of 30 CHS X joints under compression of steel gradeS 700 and 30 of steel grade S 900(Ma et al. 2015) using the $3\% \cdot d_0$ deformation limit.For grade S 700 (E = 214 GPa) a mean yield strength corresponding to the stress at plastic strain of 0,2% of $f_{y,m} = \sigma_{0,2}$ = 772 MPa and a mean tensile strength of $f_{u,m}$ = 816 MPa at a uniform strain of ε_u = 4,64% are used. For grade S 900 (E = 210 GPa), a mean yield strength of $f_{y,m} = \sigma_{0,2}$ = 1054 MPa and a tensile strength $f_{u,m}$ = 1116 MPa at a uniform strain of ε_u = 2,26% are applied to the numerical model. All joints have a chord diameter of d_0 = 120 mm. By varying the diameter of the braces d_1 and the wall thickness of the chord t_0 as well as the braces t_1, joints with diameter ratios of $0,2 \leq \beta \leq 1,0$, chord slenderness ratios of $10 \leq 2\gamma \leq 50$ and wall thickness ratios of $0,2 \leq \tau \leq 1,0$ are obtained. Additionally, joints with brace angles of θ_1 = 30, 45, 60, 75 and 90° and the influence of compressive loaded chords with chord utilisations of $0 \leq |n| = \sigma_{0,Ed}/f_{y0,m} \leq 0,8$ on the joint resistance are analysed.

4 Evaluation for joints made of circular hollow sections (CHS)

The evaluation takes account only of axially loaded CHS T, Y, X and K joints with gap of steels with a yield strength f_{y0}> 355 MPa failing by chord (face) failure or punching shear. The yield strength f_{y0} and the tensile strength of the chord $f_{u0}$and the section as well asthe joint dimensionsrequired for the calculation of the joint resistance have to be available for consider the joint. Joints with obviously recorded errors are not taken into account. Furthermore, tests for which brace failure has been observed by Puthli et al. (2010, 2014) are excluded since the ultimate loads from the tests equate the plastic resistances of the braces.Thus, member failure is governing.

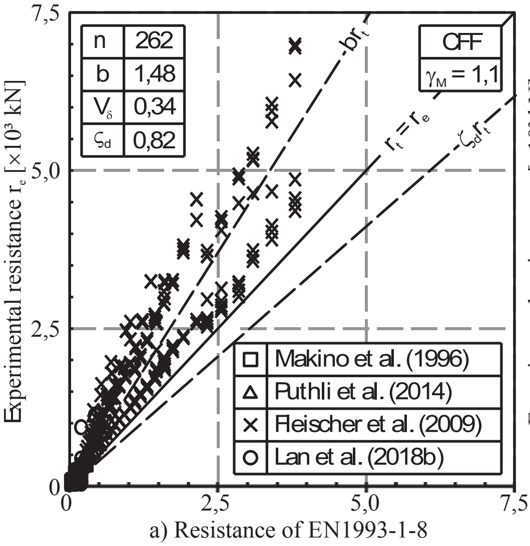
Additionally, joints with a minimum yield strength $f_{y0} \leq 355$ MPa are not considered in the evaluation. Mean valuesof the yield strengths $f_{y0,m}$are reduced to their minimum values f_{y0}= 0,882·$f_{y0,m}$ based on a coefficient of variation (CoV) of the yield strength of V_{fy} = 0,059 and a fractile factor of k = 2 corresponding to a probability of $P(f_{y0,m} \leq f_{y0})$ = 2,28% (Petersen 2001).

The joint resistances $r_t$for chord (face) failure and punching shear are calculatedwithmeasured dimensions and material propertiesand compared with the experimentally and numerically joint resistances r_e. Also, the material factors C_f are determined based on the measured yield strengths of the chords $f_{y0,m}$.By comparing the joint resistances r_t and the corresponding experimental and numerical resistances r_e, mean strength equations b·$r_t$are obtained by linear regression analyses.

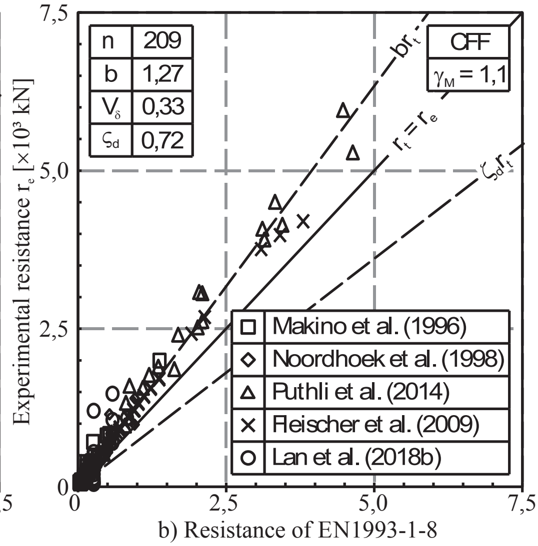
$$\boldsymbol{V_\delta = \sqrt{e^{s_\Delta^2} - 1}} \qquad (2)$$

Taking account for the variation of the test results V_δ (Equation 2), derivedwith the error terms $\delta_i = r_{ei}/(b \cdot r_{ti})$ and the assumptionthat the variations $s_\Delta{}^2$ of $\Delta_i = \ln(\delta_i)$ are estimations of the basic populations $\sigma_\Delta{}^2$ (Fleischer et al. 2006)as well as fabrication tolerances (e.g. see Wardenier 1982), the mean strength equationsare converted to their characteristic valuer$_c$·b·r_t. Design strengths are obtained by partial safety factorsr$_c/\gamma_M$·b·r_t. Additionally, the use of minimum yield strengths and nominal dimensions has to considered (Lan et al. 2018a, Fleischer et al. 2006).Since mean and nominal dimensions correspond,only mean yield strengthshave to be converted, thus $\zeta_d \cdot r_t = r_c/\gamma_M \cdot b/0{,}882 \cdot r_t$. The comparisons for the joint resistances r_t obtained with EN 1993-1-8, prEN1993-1-8 andISO 14346resp. CIDECT Design Guide 1 for joints in- and outside the validity ranges and the experimental and numerical resistance $r_e$are given in Figures1a) – f).

Due to a reduction of the minimum wall thickness from 2,5 mm in EN 1993-1-8 to 1,5 mm in prEN 1993-1-8 results in deviating number of tests in- and outside the validity range. For ISO 14347 resp. CIDECT Design Guide 1 the number of tests is additionally reduced due to the limitation to a maximum yield strength of $f_y \leq 460$ MPa.



a) Resistance of EN1993-1-8
(joints inside validity range) r_t [×10? kN]



b) Resistance of EN1993-1-8
(joints outside validity range) r_e [×10? kN]

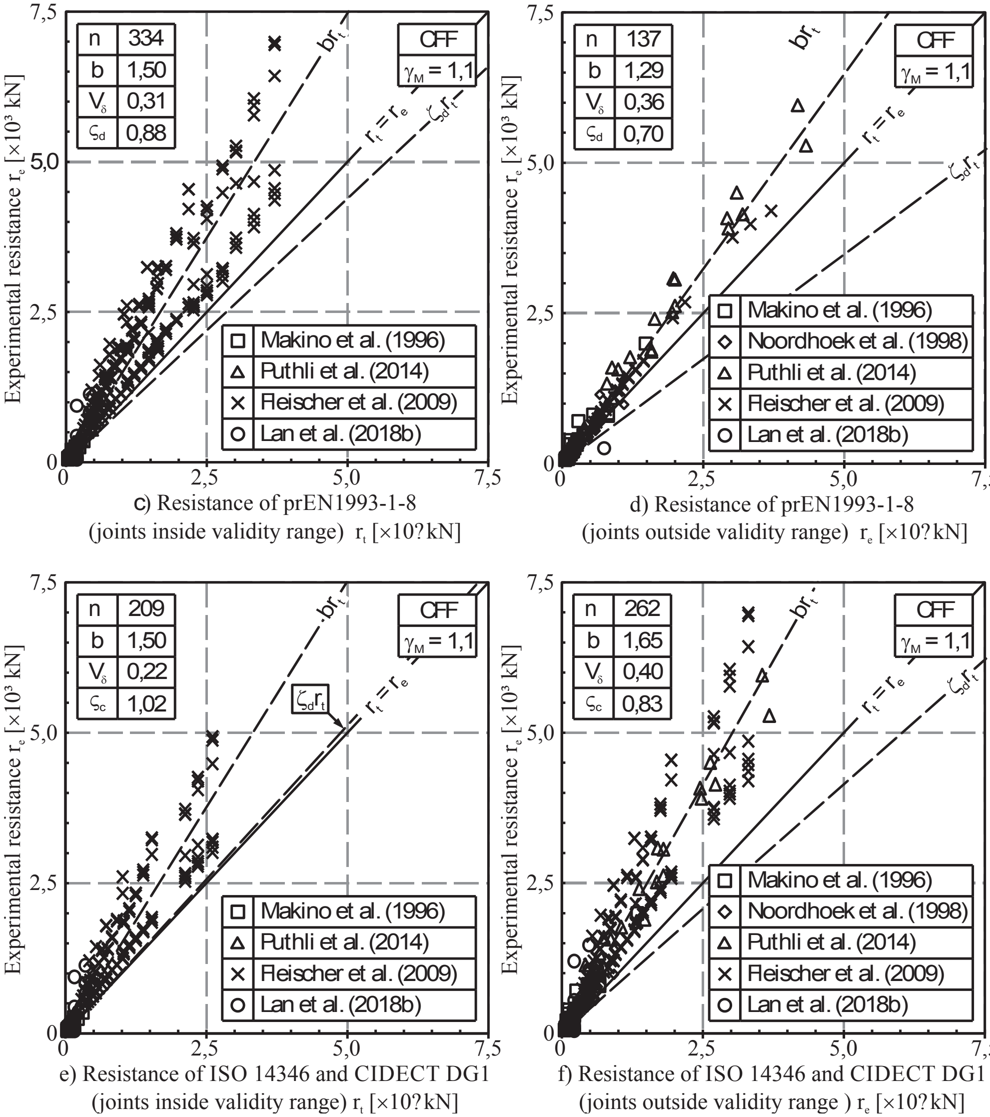


Figure 1 Comparison of CFF design resistances r_t of EN 1993-1-8, prEN 1993-1-8 and ISO 14346 resp. CIDECT Design Guide 1 and ultimate loads of tests $r_e$for joints in- and outside the validity ranges

Punching shear failure is observed in theexperimental investigations only for one X and three T joints with brace anglesof θ_1 = 90°, all loaded in tension and all without an additional chord load ($k_p = Q_f = 1,0$). The measured dimensions and material properties of the chords and the resulting joint parameters, the failureloads $N_{1,max}$ of the tests and the design resistances calculated with measured dimensions and material properties was well as their ratios to the failure loads are given in Table3.

Table 3 Tests failed by punching shear acc. to EN 1993-1-8, prEN 1993-1-8 and ISO 14346 resp. CIDECT Design Guide 1

	$b_0 \times t_0$	β	γ	τ	f_{y0}	$0,8f_{u0}$	$N_{1,max}$	Design resistance $N_{1,Rd}$ EN	prEN	ISO CIDECT	Ratio $N_{1,max}/N_{1,Rd}$ EN	prEN	ISO CIDECT
					MPa	MPa	kN	kN	kN	kN			
T[1]	114,3 × 4,7	0,53	12,2	0,81	431	377	229	223	175	176	1,02	1,30	1,30
T[1]	114,3 × 4,7	0,67	12,2	0,85	431	377	254	281	220	221	0,91	1,15	1,15
T[1]	318,5 × 4,5	0,44	35,4	0,98	415	427	350	473[3]	426[3]	428[3]	0,74	0,82	0,82
X[2]	325 × 15	0,55	10,8	0,58	734	642	2389	2845[3]	2487[3]	2499[3]	0,84	0,96	0,96

Annotation: [1] Makino et al (1996).
[2] Puthli et al. (2010, 2014).
[3] Outside validity range.

5 Conclusions and future work

It is observed, that inside the validity ranges the design resistances for chord (face) failure according to EN 1993-1-8 and prEN 1993-1-8 overestimate the joint resistances about 18% (Figure 1a) resp. 12% (Figure 1c), whereas the design resistances according to ISO 14346 resp. CIDECT Design Guide 1 only marginally underestimate the design resistances about 2% (Figures 1e). For joint resistances according to ISO 14346/CIDECT Design Guide 1, the lowest variation of test results V_δ= 22%is obtained additionally. This isconsiderably lower as for design resistances according to EN 1993-1-8 (V_δ = 34%) and prEN1993-1-8 (V_δ = 31%).

For joints outside the validity rangesthe design resistances generally overestimate the joint resistances about approximately 30 % for design resistances according to EN 1993-1-8 and prEN 1993-1-8 (Figure 1b and c) and only about 17 % for joint resistances according to ISO 14346 resp. CIDECT Design Guide 1(Figure 1f). For all evaluations outside the validity ranges highvariations of the test results $V_\delta$are determined.

For punching shear, two test results inside and two test results outside the validity ranges are available.Hence, a statistical evaluation is not possible. For the joints inside the validity ranges the design resistances of EN 1993-1-8 are equal to or even higher than the failure loads. However, the design resistances of prEN 1993-1-8 and ISO 14346 resp. CIDECT Design Guide 1 are smaller than the failure loads. For joints outside the validity ranges the design resistances are always higher than the failure loads, but for EN 1993-1-8 the determined design resistances are even higher (Table 3).

In the future new research for joints of CHSin high strength steel (e.g. Qu et al. 2018, Lee et al. 2017, Lan et al. 2018c) will be included and the investigations will be extended to joints of RHS, overlapped joints and joint loaded by IPB. Additionally, it will be investigated if the different deformation limitsexplain the high variations of the test results. For axially loaded CHS T and X joints the applicability of the 3% deformation criterion is well investigated. However, for K and N joints with gap the determination of joint resistances for CFF no systematic investigations exist, especially for joints made of high strength steels.

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