RESISTANCE TABLES FOR STANDARDIZED JOINTS IN ACCORDANCE WITH EN 1993-1-8

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ABSTRACT

Publications containing resistance tables for joints in steel structures have been published in the last decades in various European countries. However, most of those publications provide joint resistances according to the design rules of national standards or according to the ENV version of Eurocode 3. The present paper refers to two new publications containing resistance tables in accordance with the new Eurocode 3 Part 1.8 (EN 1993-1-8). One of these two publications is devoted to joints with open sections including simple joints and moment resistant joints. One type of connection, traditionally used in Germany, but more and more also used in other European countries, is the end-plate connection with four bolts in a horizontal row. This connection type is not directly covered by Eurocode 3. The paper describes briefly the design model for this specific type of connection which is fully in accordance with the design methods of Eurocode 3. The second publication is of particular interest for practitioners because it is the first design tool dealing with hollow section joints in lattice structures. The paper discusses the practical impact of such resistance tables and it reports on some observations using the Eurocode 3 design rules for hollow section joints.

1. INTRODUCTION

Many papers have been published to report on the background of design models to determine the structural properties of joints in steel structures according to Eurocode 3 (EC3). Also design tools, for example resistance tables, were published. However, most of those publications provide joint resistances in accordance with the design rules of national standards or the ENV version of EC3. The present paper reports on some practical issues of two new publications (Weynand and Oerder, 2012 and Weynand *et al.*, 2011) containing resistance tables in accordance with the new EN 1993 Part 1.8.

The first publication is devoted to simple joints and moment resistant joints of open sections. One type of connection is the end-plate connection with four bolts in a horizontal row. This connection type is not directly covered by EC3. For the design of such connections it was necessary to extend the existing design model. In the present paper the extended model is briefly described. The model, which is fully in

accordance with the design methods of EC3, refers to the so-called T-stub approach. Therefore this approach is also summarised.

The second publication is the first design tool for the design of hollow section joints. These design tools, consisting of design resistance tables and a computer program, cover most commonly used types of joint in lattice structures.

2. MOMENT RESISTANT JOINTS OF OPEN SECTIONS

2.1 End plate connection with four bolts in a row

In Eurocode 3 Part 1-8 (EN 1993-1-8, 2005) rules for bolted end plate connections are regulated only for two bolts in each horizontal row. Especially when wide flange H-sections are used it is sometimes more economical to use four bolts in one row instead of two. The theoretical model, the so-called T-stub approach, on which the Eurocode 3 application rules are founded is general and can be potentially applied to connections with four bolts per horizontal row.

2.2 T-stub approach

Four bolts per row instead of two significantly influence the following components:
- end-plate in bending and
- column flange in bending.

The design rules for these components are founded on the so-called "T-stub approach". This approach substitutes a tensile part of the connection by a T-stub of appropriate effective length l_{eff}, with the flanged bolted to a presumably infinitely rigid foundation and subject to a uniformly distributed tension force acting in the web plate (Figure 1).
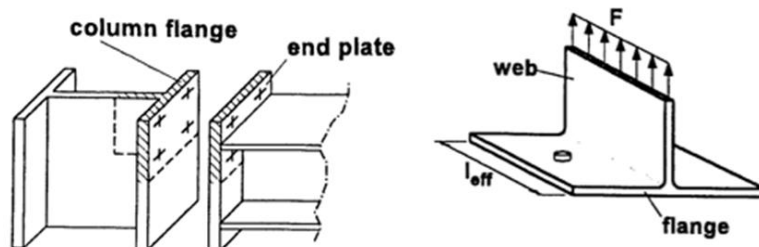

Figure 1. T-stub idealization and T-stub on a rigid foundation

In this approach, three different failure modes can appear:
1. Onset of a yield lines mechanism in the plate before the strength of the bolts is exhausted (Mode 1);
2. Mixed failure involving yield lines – but not a full plastic mechanism – in the plate and exhaustion of the bolt strength (Mode 2), and
3. Bolt fracture without prying forces, as a result of a very large stiffness of the plate (Mode 3).

The design resistance of a T-stub flange depending on the failure mode is given in the following:

Mode 1: $F_{Rd,1} = \dfrac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)}$

Mode 2: $F_{Rd,2} = \dfrac{2M_{pl,2,Rd} + n\sum F_{t,Rd}}{m+n}$

Mode 3: $F_{Rd,3} = \sum F_{t,Rd}$

with:

m	defined in Figure 2;
n	$= e_{min}$ as defined in Figure 2, but $n \le 1,25m$
e_w	$d_w / 4$ (see Figure 2);

$\sum F_{t,Rd}$ is the sum of the design resistances of the bolts connecting the T-stub to the foundation;

$M_{pl,1,Rd} = 0,25 l_{eff,1} t_f^2 f_y / \gamma_{M0}$;

$M_{pl,2,Rd} = 0,25 l_{eff,2} t_f^2 f_y / \gamma_{M0}$;

t_f, f_y the thickness and yield strength of the T-stub flange;

$l_{eff,1}$ minimum effective length associated to circular or non-circular patterns (see 2.3.3);

$l_{eff,2}$ minimum effective length associated to non-circular patterns (see 2.3.3).

As these formulas are only valid for joints with two bolts per row, new formulas for four bolts per row will be derived in 2.3.
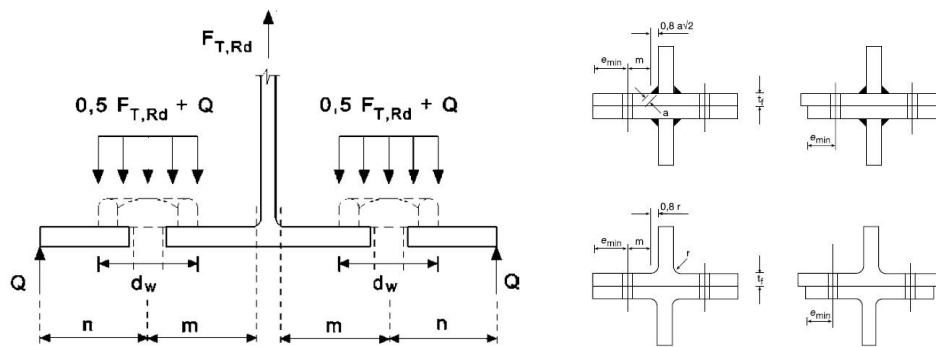

Figure 2. T-stub parameters (with two bolts)

Also the derivation of the effective lengths for joints with four bolts in a row is requested. Tables with analytical formulas are given in Eurocode 3 (EN 1993-1-8 (2005), for end-plate or column flange with two bolts per row. These effective lengths will be different if the considered bolt row is an external or inner one. Examples for formulae to predict these effective lengths are reported in 2.3.3.

2.3   Extension of the T-stub approach to connections with four bolts per row
2.3.1 General
To extend the T-stub approach to connections with four bolts in a row, both the basic formulae checking all possible failure modes as well as the determination of the effective length need to be adopted. Preliminary studies have been reported by Weynand *et al.*, 2008. A more complete description of the extend design model was present by Demonceau *et al.*, 2011.

2.3.2 Possible failure modes for T-stub with four bolts
For both, T-stubs with two and T-stubs with four bolts in a row, three failure modes can appear, which are explained in the following.

Mode 1: This failure mode represents a yielding line within the T-stub in the zone between the web and the bolts which are closest to the web, as illustrated in Figure 3. The resistance of this failure mode is calculated with the same formula as for a T-stub with two bolts:

$$F_{Rd,1} = \frac{(8n - 2e_w)M_{pl,1,Rd}}{2mn - e_w(m+n)}$$

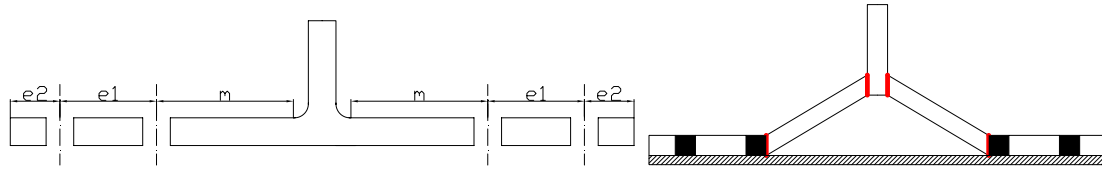with $n = e_1 + e_2$ (see Figure 3) but $n \leq 1,25m$, $n_1 = e_1$, $n_2 = e_2$ but $n_2 \leq 1,25m + n_1$.

Figure 3. Failure mode 1 – Yield line plastic mechanism

Mode 2: In this failure mode two yielding lines appear. One in the T-stub web and one in the bolts closest to the web, see Figure 4. The formula for T-stubs with two bolts cannot be directly extended to the present configuration, because this failure mode is influenced by the presence of the "outer" bolts, i.e. the bolts which are the closest to the flange boundaries.

Therefore to cover this failure mode new formulas were developed. Two different situations are taken into account for T-stubs with four bolts:

- Failure mode "2p": this failure mode corresponds to a non-circular pattern of the yielding lines along with the existence of prying forces Q at the T-stub boundaries:

$$F_{Rd,2,p} = \frac{2M_{pl,2,Rd} + \dfrac{\sum F_{t,Rd}}{2}\left(\dfrac{n_1^2 + 2n_2^2 + 2n_1 n_2}{n_1 + n_2}\right)}{(m + n_1 + n_2)}$$

- Failure mode "2np": this failure mode corresponds to a circular or non-circular pattern for the yielding lines without prying forces at the T-stub boundaries:

$$F_{Rd,2,np} = \frac{2M_{pl,1,Rd} + \dfrac{\sum F_{t,Rd}}{2}n_1}{(m + n_1)}$$

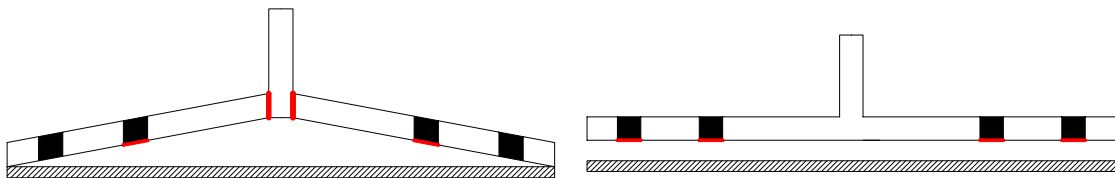Finally: $F_{Rd,2} = \min(F_{Rd,2,p}; F_{Rd,2,np})$

Figure 4. Mode 2 (yield lines + bolt failure) and mode 3 (failure of the 4 bolts)

Mode 3: This failure mode is similar to the one of the T-stub with two bolts. In order to derive these formulae, the T-stub flange is assumed to be a rigid body (i.e. these formulas are derived from a rigid-plastic theory). The latter may be fully justified for T-stubs with 2 bolts per row while it could possibly lead to unsafe results for T-stub with 4 bolts. When the bolts which are closest to the web reach $F_{t,Rd}$, due to

the flexibility of the T-stub flange, the axial load in the other bolts could be estimated to at least 80% of $F_{t,Rd}$ (see Petersen, 1993 and Demonceau, 2011). Because of that the formula for the resistance is equal to:

$$F_{Rd,3} = \frac{\sum F_{t,Rd}}{2}(1+0.8) = 0,9 \sum F_{t,Rd}$$

2.3.3 Determination of the effective lengths

Because of the presence of four bolts per row new formulas for the determination of the effective lengths need to be developed. Finally, there are quite a lot of formulae to check all possible yield line pattern. The present paper does not give enough room to present the complete set of formulae, However, as an example, the formulae for effective lengths of circular and non-circular patterns of an external bolt row in the extended part of an end-plate are given hereafter.

For non-circular patterns $l_{eff,nc} = \min(l_{eff,20}; l_{eff,21}; ... l_{eff,29})$ with $l_{eff,20}$ to $l_{eff,29}$ defined in Figure 5. For circular patterns $l_{eff,c} = \min(l_{eff,14}; l_{eff,15}; ... l_{eff,19})$ with $l_{eff,14}$ to $l_{eff,19}$ defined in Figure 6.
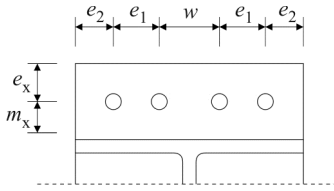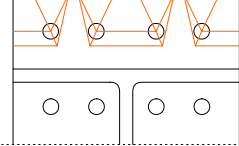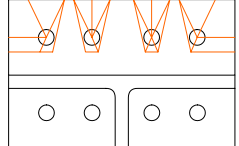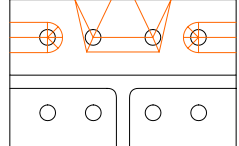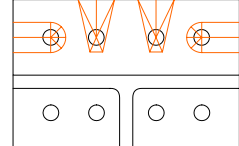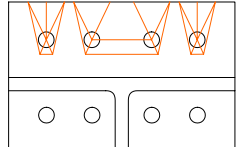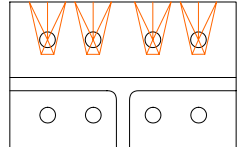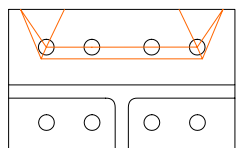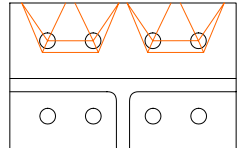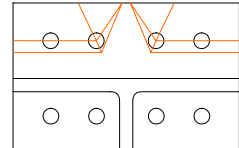

$$l_{eff,20} = 4m_x + 1,25e_x + 0,5w + e_2$$

$$l_{eff,21} = 6m_x + 1,875e_x + e_2$$

$$l_{eff,22} = 2m_x + 0,625e_x + 0,5w + \pi m_x + e_2$$

$$l_{eff,23} = 4m_x + 1,25e_x + \pi m_x + 2e_2$$

$$l_{eff,24} = 6m_x + 1,875e_x + 0,5w$$

$$l_{eff,25} = 8m_x + 2,5e_x$$

$$l_{eff,26} = 0,5b_p$$

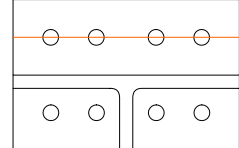$$l_{eff,27} = 2m_x + 0,625e_x + (\,$$

$$l_{eff,28} = 4m_x + 1,25e_x + e_1$$

$$l_{eff,29} = 2m_x + 0,625e_x + e_1 + e_2$$

Figure 5. Non-circular yielding patterns for the external bolt row

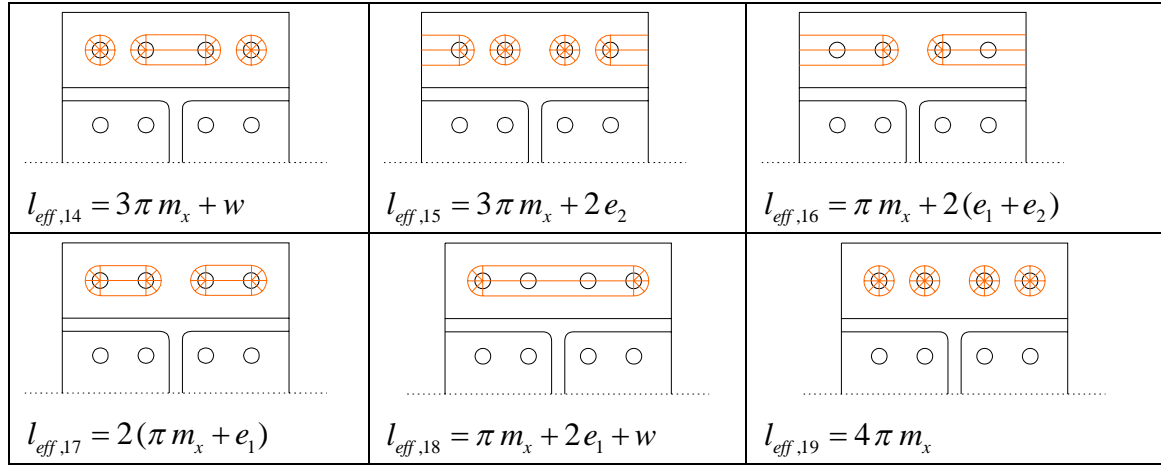| $l_{eff,14} = 3\pi m_x + w$ | $l_{eff,15} = 3\pi m_x + 2e_2$ | $l_{eff,16} = \pi m_x + 2(e_1 + e_2)$ |
| $l_{eff,17} = 2(\pi m_x + e_1)$ | $l_{eff,18} = \pi m_x + 2e_1 + w$ | $l_{eff,19} = 4\pi m_x$ |

Figure 6. Circular yielding patterns for the external bolt row

3.   SIMPLE JOINTS OF OPEN SECTIONS

3.1   Double web cleated connections

The ECCS publication "European Recommendations for the Design of Simple Joints in Steel Structures" (Jaspart *et al.*, 2009) provides detailed rules for the design of simple joints. However, explicit rules are given only for header plate connections and for fin plate connections respectively. Detailed formulae for connections with web cleats are not given. They must be deduced from those rules given for header plate and fin plate connections. Following this approach, this is quite clear and easy for most of the components which can be identified for a web cleated connection. However, the transfer of the component "header plate in bending" need some further considerations.

For the check "header plate in bending", the header plate is modelled as a simple single-span beam with elastic stress distribution over the cross section. This model is no more valid for web cleats. The eccentricity of the bolts connected to the supporting member lead to a moment, see Figure 7, resulting in horizontal compression forces the in the upper part and horizontal tension forces in the lower part of the cleat. It is assumed that the compression forces will be carried by contact of the cleats to the web of the beam (Figure 8). There would be two assumptions to carry the tension forces:

- additional (horizontal) forces in the bolts on the bolts connected to the supporting member (Figure 9)
- tension forces in the bolts on the beam side (supported member).
  However this assumption leads in many practical case to a significant reduction of the shear resistance of the bolts when the tension-shear interaction is taken into account. Therefore, this approach is only followed for cleats with long legs, see section 3.2.

The cleats and the resulting moment rotate around the centre point of the compression zone. An elastic distribution at the resulting horizontal forces is assumed. So the horizontal forces in the bolts increase linear (Figure 9).

Because of the short distance from the compression zone to the supporting element side, the horizontal forces are directly absorbed by the bolts at the supporting element side. Torsion in the cleat and additional tying forces in the bolts on the supported element side do not arise.
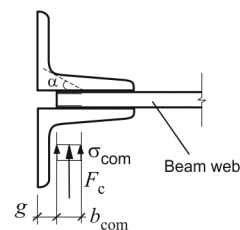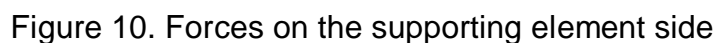
Figure 7. Forces at supporting member side   Figure 8. Compression zone

Figure 9. Forces on the supporting element side

3.2   Double web cleated connections with long legs

A compression zone directly at the beginning of the cleat does not exist. Because of that the cleat can rotate more freely and the moment rotates around the centre point of the bolt pattern on the supporting element side. So the resulting forces will appear as shown in Figure 10

Figure 10. Forces on the supporting element side

It cannot be assumed that the resulting horizontal forces get diverted directly into the bolts at the supporting element side. The bolts at the supported element side have to take these forces, so the load capacities of these bolts have to be reduced.

At web cleats with one bolt on the supporting element side the torsional moment has to be taken by the torsional stiffness of the cleat.

4. HOLLOW SECTION JOINTS

To facilitate the use of hollow sections, an initiative has been launched to develop new design tools for the daily design practice resulting in a new publication (Weynand *et al.*, 2011). It will enable the engineer to perform design checks of joints connecting hollow sections in an easy, safe and economic manner.

Two different kinds of design tools are provided. A set of design resistance tables for standardised joints with predefined dimensions are published in a book. The resistance tables have been checked and approved by the German building authorities. In addition, software for the verification of joints with individual dimensions is available. Both, the book and the software, are available in English and German language.

With these design tools it is possible to determine the structural characteristics of hollow section joints under static loading. Joints made in typical lattice girder steel constructions and simple frames made of hollow sections are covered. The design tools for hollow section joints are strictly based on the design rules given in EC 3 including its latest corrigenda (ECCS TC 10, 2011) and the German National Annexes. Beside the design resistance, also the relevant failure mode is given.

4.1 Observations using the Eurocode 3 rules

Some observations resulting from an intensive and systematic use of the design rules of EN 1993-1-8 chapter 7 (hollow section joints) while developing/checking the design tools are presented hereafter.

4.1.1 T, X, and Y joints between RHS or CHS braces and RHS chords

In Table 7.11 of EN 1993-1-8 design axial resistances of welded T, X and Y joints between RHS or CHS braces an RHS chords are provided, see Figure 11.
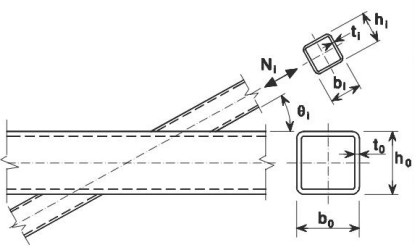
Type of joint	Design resistance $[i = 1]$	
	Chord face failure	$\beta \leq 0,85$
	$N_{i,Rd} = \dfrac{k_n f_{y0} t_0^2}{(1-\beta)\sin\theta_1}\left(\dfrac{2\eta}{\sin\theta_1} + 4\sqrt{1-\beta}\right)/\gamma_{M5}$	
	Chord side wall buckling [1]	$\beta = 1,0$ [2]
	$N_{i,Rd} = \dfrac{f_b t_0}{\sin\theta_i}\left(\dfrac{2h_i}{\sin\theta_1} + 10t_0\right)/\gamma_{M5}$	
	Brace failure	$\beta \geq 0,85$
	$N_{i,Rd} = f_{yi} t_i (2h_i - 4t_i + 2b_{eff})/\gamma_{M5}$	
	Punching shear	$0,85 \leq \beta \leq (1 - 1/\gamma)$
	$N_{i,Rd} = \dfrac{f_{y0} t_0}{\sqrt{3}\sin\theta_1}\left(\dfrac{2h_i}{\sin\theta_1} + 2b_{e,p}\right)/\gamma_{M5}$	

Figure 11. Design axial resistance of welded T, X and Y joints between RHS or CHS braces an RHS chords

For this type of joint different failure modes have to be checked. They are dependent on the width ratio $\beta = b_1/b_0$. For $\beta \leq 0.85$ the failure mode "chord face failure" is governing, while for $\beta \geq 0.85$ other failure modes have to be checked. To show the effect of the switch from one failure mode to another, Figure 12 shows re-

sistance values for an Y joints (chord 150x150x20 mm, brace 120x120x12.5 mm, S 355, $\Theta = 37°$, $N_{0,Ed} = 0$), where b_1 varies between 120 mm and 135 mm, so that β varies between 0.8 to 0.9. Relevant failure modes are indicated. A significant drop of the resistance becomes obvious at the point $\beta = 0,85$ as in the first case failure mode "chord face failure" and in the latter "brace failure" is governing.
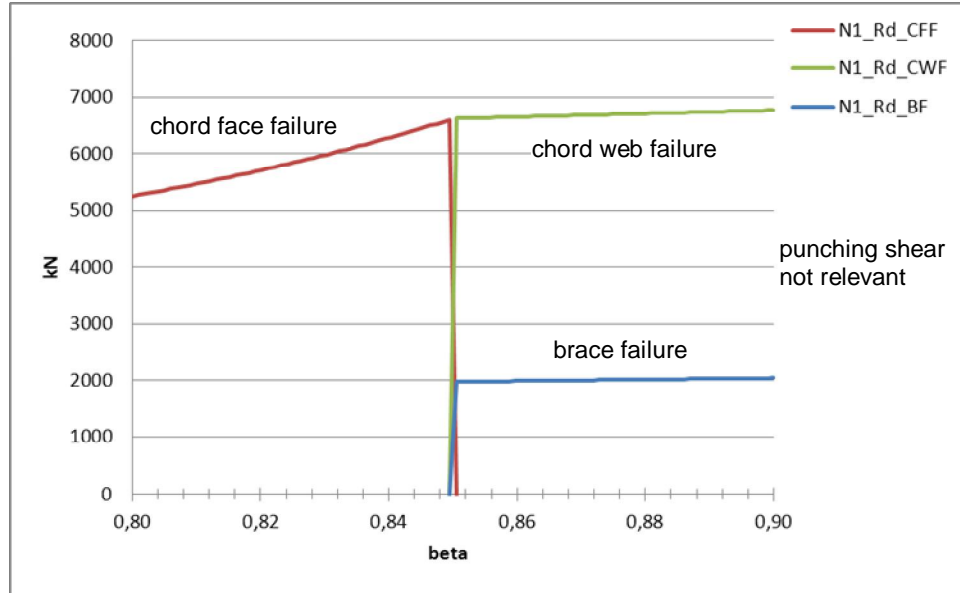

Figure 12. Design resistances

4.1.2 T joints between transverse plates to RHS members

Desing rules for T joints between transverse plates and RHS members can ve found in EN 1993-1-8 Table 7.13. The latest corrigendum proposed by ECCS TC 10 and CIDECT (ECCS TC 10, 2011) is already taken in account in the calculations (Note: A check of chord face failure had been added).

For this type of joint similar steps (drops od increases) in the resistance function as shown in section 4.1.1 can be found. Again, different failure modes have to be checked and they are dependent on the width ratio $\beta = b_1/b_0$. To show the effect of the switch from one failure mode to another, Figure 13 shows resistance values for an T joints (chord 300x300x20 mm, plate 240x25 mm, S 355, $N_{0,Ed} = 7800\,\mathrm{kN}$), where b_1 varies between 240 mm and 270 mm, so that β varies between 0.8 to 0.9. Relevant failure modes are indicated. In this specific configuration the resistance curve shows two steps. The first step is an sudden increase when the governing failure mode "chord face failure" changes to "brace failure", the second step is drop from "punching shear failure" to "chord web failure". However, in this particular example, the second step is not relevant, because the governing failure mode is "brace failure".

4.1.3 Summary

These examples indicate that the limiting conditions of β in tables 7.11 and 7.13 in EN 1993-1-8 seem not to reflect the mechanical behaviour of such joints. A "step" in curve of a resistance over a specific joint parameter should not occur. It

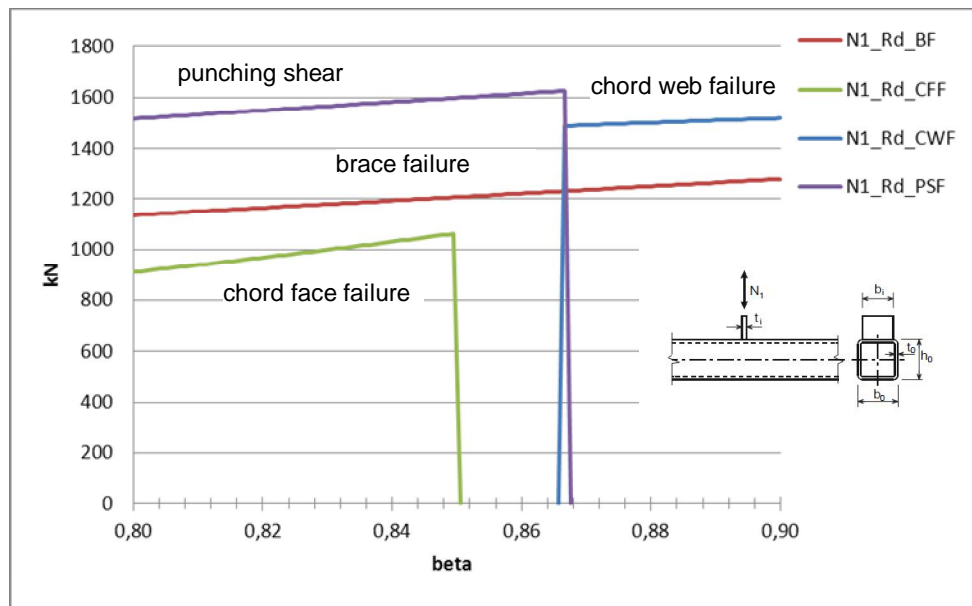would be desirable if, in a future revision of EN 1993-1-8, such inconsistencies could eliminated.

Figure 13. Design resistance

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