Template for comments and secretariat observations

Date: 2014-10-22	Document:	Project: EN 1993-1-8

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision
					KEY TO COMMENTATORS ES Afnor (Spain) FR Afnor (France) RO ASRO (Romania) GB BSI (UK) DE DIN (Germany) DS/DK DS (Denmark) BE NBN (Belgium) GR NQIS ELOT (Greece) PL PKN (Poland) FI SFS (Finland) SE SIS (Sweden) CZ UNMZ (Czech Republic)		Key to proposed procedure A: Accepted (PT) = this is e.g. the case if an editorial or similar obvious mistake has been discovered that can be dealt by the PT when drafting the new text without any big discussion in the WG or SC3 B: Accepted (WG) = This comment seems to be OK and relevant but needs further elaboration in WG, maybe also an acceptance vis SC3 decision C: Not accepted = in this case a reason should be given, e.g. damand against the policy of clarity and ease of use and reduction as expressed in Decision 3/2013 of TC250/SC3 D: To be clarified (WG) = maybe some comments cannot be easily answered and needs further considerations and discussions in WG E: Dealt with by Decision X of TC250/SC3 = as you know for a number of cases we had so called "Amendments" decided by SC3 but on hold in the "basket". We will not discuss them again

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							normally. F: Dealt with by current discussion in TC250/SC3/WG, see document y
Do any clauses require editorial or technical correction?							
ES1					"We support the revision of Eurocodes mentioned in N 1083 (Eurocode 3). Comments are submitted as requested by WGs and SC3.".		
FR1		1.2.4		Te/Ed	New standards for bolt have been published these last years and should be add.	Introduce EN 14399 parts 7, 8, 9 and 10 for preloaded bolts (DTI, HRC, HVP…). Introduce EN 15048 for non-preloaded bolts and EN 1090-4 for cold form elements.	A
FR2		2.2	Table 2.1	Te	The partial safety factor γ_{M7} is not used in practice. See comments on clause 3.6.1(2).	Delete partial safety factor γ_{M7} or clarify its intended use.	C
FR3		2.2	Table 2.1	Te	Values of γ_{M3} and $\gamma_{M3,ser}$ don't correspond to voted value initially in the standard.	Take γ_{M3} = 1,1 and $\gamma_{M3,ser}$ = 1,25.	E?
FR4		3.4.1	c)	Te	In category C, a bearing resistance equal to that use in category A has no sense because the behaviour is different. This calculation is very conservative.	Delete bearing resistance in category C and introduce appropriate edge limitations.	B
FR5		3.5	Table 3.3, 4)	Te	The nominal clearances given in EN 1090-2 for long slotted holes are quite short in practice. The nominal clearance for long slotted holes on the length could be improved from 1,5*d* to 2,5*d*.	For consistency, the definitions and dimensions of holes (normal round, oversize round, slotted) should be given in the present standard in relation with coefficient k_s of Table 3.6. A value of k_s should be specified for long slotted holes with clearance on the length between 1,5*d* to 2,5*d*.	B

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<table>
<tr><th>MB/ NC[1]</th><th>Line number
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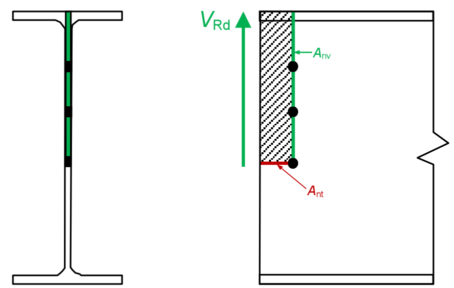
<tr><td>FR6</td><td></td><td>3.6.1</td><td>(2)</td><td>Te</td><td>The design preloaded $F_{p,Cd}$ is not used in practice. It should be deleted or its interest should be explained.</td><td>Delete this clause or clarify its intended use.</td><td>C design preload in several cases is important</td></tr>
<tr><td>FR7</td><td></td><td>3.6.1</td><td>(3)</td><td>Te</td><td>This clause is not clear, it is asked that thread should comply with EN 1090 but this standard gives no specific requirements about it.

It appears that the reduction of 0,85 should be applied when no suitability test has been done (see EN 15048 for example).</td><td></td><td>B</td></tr>
<tr><td>FR8</td><td></td><td>3.6.1</td><td>Table 3.4

Note 1</td><td>Te</td><td>The reduction factor given for bearing resistance of slotted holes, equal to 0,6, is too conservative for short slotted holes. This value is coherent for long slotted holes but not for short slotted holes. Experimental tests (Wald, 2002) show that a value of 0,8 could be used for short slotted holes such as for round oversize holes.</td><td>A reduction factor of 0,8 should be given for short slotted holes and round oversize holes.

See comments on §3.5.</td><td>B</td></tr>
<tr><td>FR9</td><td></td><td>3.10.2</td><td>(2)</td><td>Te</td><td>For the calculation of block shear in the web of an un-notched beam (see Figure 3.8), only the web thickness should be considered for the calculation of the net area subjected to shear.</td><td>Add the following figure :

</td><td>B</td></tr>
<tr><td>FR1
0</td><td></td><td>3.10.3</td><td>(2)</td><td>Te</td><td>The calculation of β leads to inconsistency because the addition of a bolt can lead to the decrease of the net section resistance. The net section resistance depends on the distance between the centres of the end fasteners.</td><td>Replace Table 3.8 by :
<table>
<tr><td rowspan="2">2 bolts</td><td>L_j</td><td>≤ 2,5 d_o</td><td>≥ 5,0 d_o</td></tr>
<tr><td>β_2</td><td>0,4</td><td>0,5</td></tr>
<tr><td rowspan="2">3 bolts or more</td><td>L_j</td><td>≤ 5,0 d_o</td><td>≥ 10,0 d_o</td></tr>
<tr><td>β_3</td><td>0,5</td><td>0,7</td></tr>
</table>
L_j : distance between the centres of the end</td><td>B</td></tr>
</table>

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						fasteners.	
FR1 1		5.1.5	Title	Te	The title is quite confusing because it refers to lattice girders in general whereas clause 5.1.5(1) refers to paragraph 7 which only refers to lattice girders composed of tube.	Change the title : "Global analysis of lattice girder made of hollow profile" Add a paragraph for other cases	B
FR1 2		6.2.2	(7)	Te	It is asked to consider the minimum of $F_{1,vb,Rd}$ and $F_{2,vb,Rd}$. The first resistance is always greater than the second.	Delete the first criterion $F_{1,vb,Rd}$.	A, maybe an additional hint on EC4 concerning allowable concrete pressure should be added
FR1 3		6.2.4.1	Table 6.2	Te	Effective lengths of circular pattern are used for mode 1. However, these failure modes can't develop in presence of prying effect, and mode 1 supposes that it's appearing. The plastic bending moment of mode 1 should be calculated with $l_{eff,2}$. The criterion for the development of prying effect is based on the elastic analysis whereas resistances of T-stubs are calculated with plastic analysis. It is a little contradiction that leads sometimes to a little decrease of the resistance of a T-stub when the thickness of the endplate increases.	Don't consider the equivalent length of circular pattern in failure mode 1.	B
FR1 4		6.2.6.1	(6) and (11)	Te	These clauses are very conservatives, tests made in INSA of Rennes (PhD of Loho, 2010) show that it is possible to consider the complete section of web stiffeners.	Delete clause (11) and add the possibility to consider all the section of stiffeners in the calculation of A_{vc}.	B
FR1 5		6.2.6.2	(6)	Te	This clause is very conservative, tests made in INSA of Rennes (PhD of Loho, 2010) show that it is possible to consider the complete section of web stiffeners.	Add the participation of complete section of web stiffeners.	B
FR1 6		6.2.7.1	(2)	Te	For section of class 4 in compression, it is surprising to have a criteria based on 5% of the plastic resistance. It should be better to consider the section	Replace "plastic resistance" by "section resistance".	A

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					resistance.		
FR1 7		6.2.7.1	(13)	Te	The simultaneous application of these loadings is hard to respect particularly in presence of beam splice with preloaded bolts. Moreover, in presence of bolted endplate, this clause is difficult to apply because there is actually no method in EN 1993-1-8 to determine the out-of-plane bending moment résistance.	Reduce the limit for bending moment from 25 % to 10%. Extend the range of application of the method for bolted endplate to bending in two directions and normal force.	B
FR1 8		6.2.8.3	Table 6.7	Te	Considering non symmetric baseplate leads to complex notation and calculation. A table with symmetric joints should be given.	Add a Table for symmetric joints.	B
FR1 9		6.3.1	(4)	Te	The methodology is applicable if the normal is not greater than 5% of the plastic resistance of the beam connected. In many practical applications, this limit is exceeded. A method should be given to calculate the stiffness for normal force greater than 5%. During these last years, methods have been developing to consider this phenomenon (Sokol et al, Eurosteel 2002, design of endplate joints subjected to moment and normal force).		B
FR2 0		6.3.2	Table 6.11	Te	Preloading of bolts increase the stiffness coefficient k_4, k_5, k_6 and k_{10}. Value considering this phenomenon should be proposed. It is indicated that k_1 is infinite in presence of stiffener. However, the type of stiffener is not indicated, it should be specified that diagonal stiffener have to be used.	Add values of stiffness coefficient considering the impact of preload in the bolts. Add that diagonal stiffener should be used.	B
RO1		3.6	3.6.1	te	Table 3.4 The shear resistance $F_{v,Rd}$ of a bolt (individual fastener) should be indicated for more than one shear plane Table 3.4. The thickness "t" for the bearing resistance $F_{b,Rd}$ should be defined as the		C not accepted because it is a matter of basic engineering knowledge

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					minimum sum of the thickness of the connected plates that tend to slip in the same sense		
RO2		6.2	6.2.6.5	te	Mathematical relations should be given in the code for α factors in figure 6.11.		F / B
GB1		1.5	Page 15	Ed	Symbol h_z is only used once, in Table 7.22 for T and Y joints, brace failure. Seems little point in this when it can be replaced by ($h_1 - t_1$) then the symbol h_z can then be deleted from list of symbols.	Delete symbol h_z and description. In Table 7.22 for T and Y joints, brace failure, h_z can then be replaced with ($h_1 - t_1$). This will be consistent with chord web yielding where ($h_1 - t_1$) is used.	C
GB2		1.5 (5)	Page 16	Te	Stress ratio for n and n_p are shown divided by γ_{M5}. This is incorrect as n and n_p are used in k_n and k_p which is used in the chord face failure equation which is also divided by γ_{M5}. Therefore, γ_{M5} is being applied twice.	Delete $/\gamma_{M5}$ from equations for n and n_p	A
GB3		1.5	Page 17 Figure 1.4	Ed	Figures (a), (b) and (c) chord force notation, $N_{0,Ed}$, $N_{p,Ed}$ and $M_{0,Ed}$ should use subscript.	Figures (a), (b) and (c), chord force notation should be formatted with subscript to read $N_{0,Ed}$, $N_{p,Ed}$ and $M_{0,Ed}$	A
GB4			Table 3.4	Ed	EN 14399 for pre-loadable bolts and prEN 15048 for The value of $k_2 = 0,63$ is obtained by taking 0,7 times the tension resistance $F_{t,Rd}$ – ie 0,63 = 0,7 x 0,9. The 0,7 factor should not have been applied to countersunk bolts. However, it should have been applied to rivets.non-preloadable bolts require countersunk bolts to have the same tension resistance as hexagon headed bolts. The factor of $k_2 = 0,63$ for countersunk bolts given iggestn Table 3.4 is a mistake.	Chairman of ECCS TC10 to check EN 14399 and EN 15048 and propose an amendment including the head of the bolt pulling through the plate.	F
GB5			Table 3.4	Tech	The note allows an inclined load on a bolt to be verified by considering separately the horizontal and vertical components of the forces on the bolt. In certain situations this is unsafe as an inclined load close to the edge of the plate will give a more critical result.	Change the text of note 3) to: ‘*Where the shear load applied to a bolt is neither parallel nor normal to the plate edge, the bearing resistance should be determined as follows:*	F

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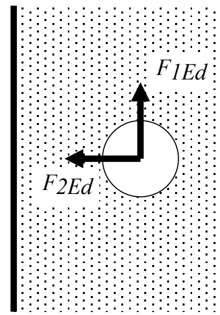
						The net force on the bolt should be resolved into the component parallel to the edge, F_{1Ed}, and that normal to the edge, F_{2Ed}. The bearing resistance parallel to the edge F_{1Rd} and that normal to the edge F_{2Rd} should be determined according to the expression in Table 3.4. *It should be verified that:* $\left(\frac{F_{1Ed}}{F_{1Ed}}\right)^2 + \left(\frac{F_{2Sd}}{F_{2Rd}}\right)^2 \leq 1.0$  Also, Table 3.4 should be redrafted to: a) Identify the specific equations with expression numbers Clarify the meaning of 'in the direction of load transfer' and 'perpendicular to the direction of load transfer' with a figure	

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GB6		3.5	(2)	te	EN 1993-1-9 does give maximum limits for spacing, edge and end distances	Reword as: For structures subject to fatigue minimum spacing, end and edge distances are given in EN 1993-1-9.	A
GB7		3.6.1	(4)	Te/ed	This clause is often read to mean 'bolts in oversized (i.e. non-clearance) holes have a shear strength of zero'. Why would the shear capacity of the bolt significantly change? We are allowed to assess its bearing strength in oversized or slotted holes from Note (1) on Table 3.4 – this is pointless if the bolt shear is to e taken as zero.	The design shear resistance $F_{v,Rd}$ given in Table 3.4 is applicable to bolts in normal, oversized and slotted holes as specified in 1.2.7 Reference Standards Group 7.	B
GB8			Table 4.2	Tech	*'Welding may be carried out within a length 5t either side of a cold-formed zone, see Table 4.2, provided that one of the following conditions is fulfilled: the r/t ratio satisfy the relevant value obtained from Table 4.2.'* This formulation does not specifically say that by fulfilling the r/t-ratios we can also weld directly in the cold-formed zones. This is totally not clear even by the figure in Table 4.2! We thoroughly studied all our documents on the preparation of EN 1993-1-8 and find out that there was a change in the stage 34 drafts between 30th April 2002 and 31st January 2003. The stage 34 dated 30th April 2002 used the following formulation (same as DIN 18800): *'Welding may be carried out in the cold-formed zones or within the adjacent width of 5t each side, see table 4.2, if one of the*	Proposed text: 4.14.1 In the bent areas and up to a distance of 5t from these areas (see figure in Table 4.2) welding may be carried out when one of the following conditions is fulfilled.	B

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					following conditions is fulfilled: - the r/t ratio satisfy the relevant value obtained from Table 4.2.' This formulation does clearly say that by fulfilling the r/t-ratios it is allowed also to weld in the cold-formed zones.		
GB9		4.14 (1)	Page 49	Ed/Te	The existing first sentence does not make it clear that welding may also be carried out in the bent area as well as the flat 5t area. *Welding may be carried out within a length 5t either side of a cold-formed zone, see Table 4.2, provided that one of the following conditions is fulfilled:*	Amend 4.14 (1) first sentence to; *In the bent areas and up to a distance of 5t from these areas (see figure in Table 4.2) welding may be carried out when one of the following conditions is fulfilled.*	B
GB10	(13-14)	6.2.7.1		te	Column splices have to be designed to transmit forces and moments between the connected parts (no minimum force/moment requirement is needed) and to maintain continuity of stiffness through the splice.	Propose minimum design stiffness requirements for the splice – see recent work by Girão Coelho et al. General expressions have been derived but need further simplification for inclusion in the code.	B / Frans
GB11		6.2.6		Tech	The 'Von Mises' criteria introduced through the 'beta' factor in the column web resistance evaluation is combined shear/tension can be a determining factor for the design moment of eave beam-column joints in industrial frame buildings. Consideration should be given to improving the design rule. It is possible that the basis of the present rule is somewhat too severe for 'extension' to the case of multiple bolt rows. Consideration should also be given to improving the design resistance of the column web panel in compression/shear. That the lower half of the web in compression – the part below the beam flange – is under much less shear than the upper half does not seem to have been considered. Furthermore, given that a factor 0,9 is also included on the shear resistance of the column web panel, the coefficient k_{wc} would	Interactions in the web panel are addressed in a simple way through the 0.9 factor. Considerations should be given to developing an approach based on the β factor that allows connections that are not coincident and are not the same size to be evaluated.	B

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					seem to be superfluous. For beam-column joints with an outer bolt row and a transverse column stiffener opposite the beam flange the 'beta' factor approach does not properly account for the contribution of that upper row (it is in another group) to shear in the column web panel. The 'beta' factor influence has a sense only when all bolt rows above the level under consideration are included. The method assumes that the connections are both coincident and the same size on opposite sides of the column. This assumption rapidly breaks down fro bolted connections when the connections are not coincident and are not the same size.		
GB1 2		7.2.1 (3)	Page 103	Te	Equations; $\sigma_{0,Ed} = \frac{N_{0,Ed}}{A_0} + \frac{M_{0,Ed}}{W_{el,0}}$ and $\sigma_{p,Ed} = \frac{N_{p,Ed}}{A_0} + \frac{M_{0,Ed}}{W_{el,0}}$ do not clarify what moments are to be considered, is it only in-plane moments or both planes? As moments from both planes combine at the corners of an SHS and similarly interact on a CHS, both planes should be considered. For a CHS the planes can be resolved.	Amend equation 7.1 to; *For SHS and RHS;* $\sigma_{0,Ed} = \frac{N_{0,Ed}}{A_0} + \frac{M_{ip,0,Ed}}{W_{el,ip,0}} + \frac{M_{op,0,Ed}}{W_{el,op,0}}$ $\sigma_{p,Ed} = \frac{N_{p,Ed}}{A_0} + \frac{M_{ip,0,Ed}}{W_{el,ip,0}} + \frac{M_{op,0,Ed}}{W_{el,op,0}}$ *Alternatively for CHS;* $\sigma_{0,Ed} = \frac{N_{0,Ed}}{A_0} + \sqrt{\left(\frac{M_{ip,0,Ed}}{W_{el,ip,0}}\right)^2 + \left(\frac{M_{op,0,Ed}}{W_{el,op,0}}\right)^2}$	B

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						$$\sigma_{p,Ed} = \frac{N_{p,Ed}}{A_0} + \sqrt{\left(\frac{M_{ip,0,Ed}}{W_{el,ip,0}}\right)^2 + \left(\frac{M_{op,0,Ed}}{W_{el,op,0}}\right)^2}$$	
GB1 3	12	7.2.1 (3)	Page 103	Te/Ed	Equation to calculate chord load $N_{p,Ed}$ is only applicable when brace and chord forces are balanced (in equilibrium). Experience shows forces are maximums and therefore not balanced, so rather than calculate $N_{p,Ed}$ the maximum chord forces are used. In this situation it is a case of deciding which of the chord forces are $N_{0,Ed}$ and $N_{p,Ed}$.	Add note after equation for $N_{p,Ed}$; *Alternatively, especially for unbalanced loads, $N_{0,Ed}$ is the most compressive chord axial load and $N_{p,Ed}$ is the least compressive chord axial load. Therefore $N_{0,Ed} \geq N_{p,Ed}$*	B
GB1 4			Page 108 Table 7.1	Ed	In the BS version the table has some incomplete limits which are present in other country versions. 'Chords, tension' limit is shown as; *$10 \leq d_0/t_0 \leq 50$ (generally), but:* 'Chords, compression' limit is shown as; *Class 1 or 2 and* *$10 \leq d_0/t_0 \leq 50$ (generally), but:*	Ensure BSI correct BS standard limits to; *Chords, tension;* *$10 \leq d_0/t_0 \leq 50$ (generally), but:* *$10 \leq d_0/t_0 \leq 40$ for X-joints* *Chords, compression;* *Class 1 or 2 and* *$10 \leq d_0/t_0 \leq 50$ (generally), but:* *$10 \leq d_0/t_0 \leq 40$ for X-joints* *$10 \leq d0/t0 \leq 50$ (generally), but:*	Is national problem
GB1 5			Page 108 Table 7.1	Te/Ed	Overlap limit 2 was amended in BS version for CEN corrigendum July 2009 on revision dated 28 February 2010 to; *$25\% \leq \lambda_{ov} \leq \lambda_{ov,lim}$, see 7.1.2 (6)* This is confusing as it can be interpreted as the	Amend; *$25\% \leq \lambda_{ov} \leq \lambda_{ov,lim}$, see 7.1.2 (6)* to *$25\% \leq \lambda_{ov}$*	B

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					overlap can not be above $\lambda_{ov,lim}$. The $\lambda_{ov,lim}$ value is the point at which above this value the brace to chord face shear should be checked. Therefore, it should not be included here, but as with other failure modes it should be shown as an application limit for the brace to chord shear check which should be shown as a failure mode in Table 7.2.	Also, include brace to chord shear failure check with an equation in Table 7.2.	
GB1 6		7.4.2 (2)	Page 108	Te	Equation 7.3 for brace member subject to combined bending and axial forces was revised to the CEN corrugendum to; $\frac{N_{i,Ed}}{N_{i,Rd}} + \left[\frac{M_{ip,i,Ed}}{M_{ip,i,Rd}}\right]^2 + \frac{\lvert M_{op,i,Ed}\rvert}{M_{op,i,Rd}} \leq 1,0$ As axial tension will be a negative value $N_{i,Ed}$ should also have the absolute vertical bars.	Amend equation 7.3 to; $\frac{\lvert N_{i,Ed}\rvert}{N_{i,Rd}} + \left[\frac{M_{ip,i,Ed}}{M_{ip,i,Rd}}\right]^2 + \frac{\lvert M_{op,i,Ed}\rvert}{M_{op,i,Rd}} \leq 1.0$	A
GB1 7			Page 109 Table 7.2	Te	Chord face failure – X joints It is advisable to check for chord shear between the braces when there is a gap between the brace toes. This check is not shown.	Add note; *For X-joints with cos $\theta_1 > \beta$, check also for chord shear failure (refer to equation in Table 7.7 for KK joint)*	A
GB1 8			Page 111 Table 7.4	Te	Punching shear failure was revised to the CEN corrugendum to; AC2) I or H sections with $\eta > 2$ (for axial compression and out-of-plane bending) and RHS sections: When $\eta > 2$ indicates the flanges are far apart but the equation only takes into account one flange while the 'All other cases' equation utilises both flanges. It appears $\eta > 2$ is incorrect and it should be $\eta \leq 2$.	Amend; *I or H sections with $\eta > 2$ …* to *I or H sections with $\eta \leq 2$ …*	A
GB1 9			Page 113 Table 7.5	Te/Ed	Punching shear failure – K and N gap joints and all T, X and Y joints;	Change subscript '1' to 'i' in all equations and validity limit for punching shear failure to read;	A

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					All subscripts to the brace are '1'. This is incorrect as it also applies to K and N joints which have two bracings, so brace subscripts should be 'i'. Punching shear failure - K and N gap joints and all T, X and Y joints When $d_1 \le d_0 - 2t_0$: $M_{ip,1,Rd} = \frac{f_{y0} t_0 d_1^2}{\sqrt{3}} \frac{1+3\sin\theta_1}{4\sin^2\theta_1} / \gamma_{M5}$ $M_{op,1,Rd} = \frac{f_{y0} t_0 d_1^2}{\sqrt{3}} \frac{3+\sin\theta_1}{4\sin^2\theta_1} / \gamma_{M5}$	*When $d_i \le d_0 - 2t_0$;* $M_{ip,i,Rd} = \frac{f_{y0}\, t_0\, d_i^2}{\ } \frac{1 + 3\sin\theta_i}{\ } /\gamma_{M5}$ $M_{op,i,Rd} = \frac{f_{y0}\, t_0\, d_i^2}{\ } \frac{3\sin\theta_i}{\ } /\gamma_{M5}$	
GB2 0			Page 114 Table 7.6	Te	Row 3 in table '*Members 1 and 3 are here in compression and member 2 is here in tension.*' Although the sketch shows a gap joint, this is applicable to overlap joints as well. A note is required to clarify this.	Add note in 1st column; *Valid for gap and overlap joints*	C
GB2 1		7.4.3 (2)	Page 115	Te	By referring to 7.4.2 the text implies the reduction factor μ is applied to moment resistances as well as axial resistance. CIDECT guidance implies μ should only be applied to axial forces. Also, for XX joints only axial forces are considered when calculating μ.	Confirm application of reduction factor in respect to forces and amend note.	B
GB2 2		7.4.3 (2)	Page 115	Te	The text implies the reduction factor μ should be applied to all failure modes in 7.4.2. Multiplanar effects would not punching shear therefore the reduction factor should only be applied to chord face failure and not punching shear failure.	Confirm application of reduction factor μ in respect to failure modes and amend note accordingly if required.	B
GB2 3			Page 115 Table 7.7	Te	KK joint; $V_{0,Ed}$ is not defined in section 1.5 or here.	Include in section 1.5 defined as; V_{Ed} Total design shear force	B
GB2 4			Page 116 Table 7.8	Te/Ed	Overlap limit was amended in BS version for CEN corrigendum July 2009 on revision dated 28 February 2010 to;	Amend; *25% $\le \lambda_{ov} \le \lambda_{ov,lim}$* to	B

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			(Also applies to Tables 7.1, 7.20 and 7.23)		$25\% \le \lambda_{ov} \le \lambda_{ov,lim.}$ [2)] This is confusing as it will be interpreted as the overlap can not be above $\lambda_{ov,lim}$ which it can. The upper limit is 100% although there the overlap can be slightly more to allow for welding. The $\lambda_{ov,lim}$ value is the point at which above this value the brace to chord shear should be checked. Therefore, it should not be included here, but as with other failure modes it should be shown as an application limit for the brace to chord shear check which should be shown as a failure mode in the relevant tables (Table 7.2, 7.10, 7.21 and 7.24).	*$25\% \le \lambda_{ov}$* or *$25\% \le \lambda_{ov} \le 100\%$* *But may be slightly above to allow for welding.* Also include brace to chord shear failure check with an equation in Table 7.2, 7.10, 7.21 and 7.24.	
GB2 5			Page 116 Table 7.8 (Also applies to Tables 7.20 and 7.23)		Note 2 was amended in BS version for CEN corrigendum July 2009 on revision dated 28 February 2010 to; *$\lambda_{ov,lim}$ = 60% if hidden seam not welded and 80% if hidden seam is welded. If the overlap exceeds $\lambda_{ov,lim}$ or if the braces are rectangular sections with $h_i < b_i$ and/or $h_j < b_j$, the connection between the braces and chord face has to be checked for shear.* This is confusing as it can be interpreted as the overlap can not be above $\lambda_{ov,lim}$. The $\lambda_{ov,lim}$ value is the point at which above this value the brace to chord shear should be checked. Therefore, it should not be included here, but as with other failure modes it should be shown as an application limit for the brace to chord shear check which should be shown as a failure mode in the relevant tables (Table 7.10, 7.21 and 7.24).	In addition to removing upper overlap limit $\lambda_{ov,lim}$ in the table (listed as separate item), suggest either; 1. Simply including a brace to chord face shear check equation as a failure mode in tables 7.10, 7.21 and 7.24 and then removing note 2. Or 2. Amend note 2 to read; *If $\lambda_{ov} \ge 60\%$ and hidden seam is not welded or $\lambda_{ov} \ge 80\%$ and hidden seam is welded or if rectangular section braces are $h_i < b_i$ or $h_j < b_j$ the connection between the braces and chord face has to be checked for shear.*	B
GB2 6			Page 116 Table 7.8	Te/Ed	'Gap or overlap' column, 'K overlap and N overlap' row; Overlapping to overlapped brace width parameter was changed in BS version for	Amend *$b_i/b_j \le 0.75$*	B

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					CEN corrigendum July 2009 on revision dated 28 February 2010 to; $b_i/b_j \leq 0{,}75$ This is incorrect, the previous version used ≥ which was correct.	to *$b_i/b_j \geq 0.75$*	
GB27			Page 118 Table 7.10	Te/Ed	Table 7.10 title is; *'Design axial resistances of welded joints between square or circular hollow section'* This is incomplete and inconsistent terminology with similar tables (use of circular hollow section instead of CHS).	Amend title to; *Design axial resistances of welded joints between SHS or CHS braces and SHS chords*	B
GB28			Page 118 Table 7.10	Te/Ed	Row 10 in table defines $b_{e,ov}$ for an SHS brace as; $b_{eff} = \frac{10}{b_0/t_0}\frac{f_{y0}t_0}{f_{yi}t_i}b_i$ but $b_{eff} \leq b_i$ And row 11 states replacing b_i and h_i with d_i. But better results can be obtained if additionally the constant value 10 is replaced with 12 for CHS braces (ref. CIDECT DG1 2nd Edition Table 4.3 for d_{ei}).	Included effective diameter equation for overlapping CHS brace; $d_{eff} = \frac{12}{d_0/t_0}\frac{f_{y0}t_0}{f_{yi}t_i}d_i$ but $\leq d_i$	B
GB29			Page 119 Table 7.11	Te	Row 3 in table, Note 2 starts; *'For 0.85 ≤ β ≤ 1.0 use linear interpolation….'* As failure modes exist for when $\beta \leq 0.85$ and β = 1.0 this statement is incorrect.	Amend Note 2 to start; *'For 0.85 < β < 1.0 use linear interpolation….'*	A
GB30			Page 120 Table 7.12	Te	Row 5, chord shear and row 12, chord shear area is defined as A_v, for consistency it should also include the subscript for the chord, '*0*'.	Amend A_v to $A_{v,0}$ in Chord Shear equation for $N_{i,Rd}$, $N_{0,Rd}$ and equation for A_v in functions list at end of Table 7.12. Also amend A_v where used elsewhere (section 1.5 (3), Table 7.21 and Table 7.24)	A
GB31			Page 121	Te	Transverse plate: Check also required for 'Plate effective width'.	Include in 'Transverse plate' check for 'Plate effective width';	A

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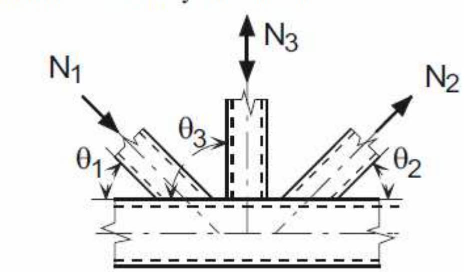
			Table 7.13			$N_{1,Rd} = f_{yi}\, t_1\, b_{eff} / \gamma_{M5}$ Note, this is applicable for all values of β.	
GB3 2		7.5.2.1 (5)	Page 122	Te	Equation 7.4 for brace member subject to combined bending and axial forces is; $\frac{N_{i,Ed}}{N_{i,Rd}} + \frac{M_{ip,i,Ed}}{M_{ip,i,Rd}} + \frac{M_{op,i,Ed}}{M_{op,i,Rd}} \le 1{,}0$ As axial tension will be a negative value and moments could be negative $N_{i,Ed}$, $M_{ip,i,Rd}$ and $M_{op,i,Rd}$ should be absolute values.	Amend equation 7.4 to; $\frac{\lvert N_{i,Ed}\rvert}{N_{i,Rd}} + \frac{\lvert M_{ip,i,Ed}\rvert}{M_{ip,i,Rd}} + \frac{\lvert M_{op,i,Ed}\rvert}{M_{op,i,Rd}} \le 1.0$	A
GB3 3			Page 124 Table 7.15	Te	Row 3, KT joint, 1st column. The member 1 is always in compression and member 2 is always in tension.  In the 1st equation in the 2nd column, brace 3 is included with brace 1 on the basis that brace 1 and brace 3 are in the same direction. As the equations are written, brace 3 can only be in compression.	Amend text to; *Members 1 and 3 are always in compression and brace 2 in tension.* Additionally, the sketch should be amended to show brace 3 in compression.	A
GB3 4			Page 124	Te	Row 3, KT joint.	Add note in 1st column;	A

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			Table 7.15		This method is only applicable for gap joints.	*Gap joints only*	
GB3 5			Page 125 Table 7.16	Te	Row 3, 2nd column, Welded knee joints, un-stiffened. *If 90° < $\theta \leq$ 180°:* If θ = 180° there is no knee, section is straight so you would not apply this.	Amend to; *If 90° < θ < 180°:*	A
GB3 6			Page 126 Table 7.17	Te	Row 4, flange plate reinforcement, tension loading. Flange plate thickness minimum limit; $t_p \geq 2t_1$ Can't find the basis for this limit. As t_p is included in the equation, if t_p is less it will be reflected in the joint capacity, so can't see reason for it.	Delete limit from table.	B
GB3 7			Page 126 Table 7.17	Te	Row 4, flange plate reinforcement, tension loading. Flange plate minimum length; $l_p \geq \frac{h_1}{\sin\theta_1} + \sqrt{b_p(b_p - b_1)}$ If brace is full width (or near) the square root term becomes zero, just leaving the reinforcing plate length the same as the h_1 dimension of the brace. This will not improve any of the failure modes. Referring back to ENV 1993-1-1:1992/A1:1994 Table K.21 there was a minimum limit for l_p of $l_p \geq 1.5\ h_1$	Reintroduce the minimum plate limit: *and* $l_p \geq 1.5\ h_1$ This will ensure any chord punching shear and deformation effects take place on the chord reinforcing plate.	A
GB3 8			Page 126 Table 7.17	Te	Row 6, flange plate reinforcement, compression loading. Flange plate minimum length; $l_p \geq \frac{h_1}{\sin\theta_1} + \sqrt{b_p(b_p - b_1)}$ If brace is full width (or near) the square root term becomes zero, just leaving the reinforcing plate	Reintroduce the minimum plate limit: *and* $l_p \geq 1.5\ h_1$ This will ensure any chord punching shear and deformation effects take place on the chord reinforcing plate.	A

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					length the same as the h_1 dimension of the brace. This will not improve any of the failure modes. Referring back to ENV 1993-1-1:1992/A1:1994 Table K.21 there was a minimum limit for l_p of $l_p \geq 1.5\, h_1$		
GB39			Page 126 Table 7.17	Te	Row 6, flange plate reinforcement, compression loading. Flange plate thickness minimum limit; $t_p \geq 2t_1$ Can't find the basis for this limit. As t_p is included in the equation, if t_p is less it will be reflected in the joint capacity, so can't see reason for it.	Delete limit from table.	B
GB40			Page 126 Table 7.17	Te	Row 8, side plate reinforcement for T, Y and X joints. Side plate minimum length; $l_p \geq 1.5\, h_1 / \sin\theta_1$ This is fine for T and Y joints but for X-joints it does not take into account the brace on the other side of the chord will be further along the chord depending on the brace angle. In this case the reinforcement plate will be too short.	Amend to; For T and Y joints: $l_p \geq 1.5\, h_1 / \sin\theta_1$ For X joints: $l_p \geq 1.5\, (h_1 / \sin\theta_1 + h_0 / \tan\theta_1)$	A
GB41			Page 126 Table 7.17	Te	Row 6, 2nd column, flange plate reinforcement, compression loading. The note mentions replacing t_0 with t_p but likewise f_{y0} could also be replaced with f_{yp} for the plate. Take $N_{1,\mathrm{Rd}}$ as the value of $N_{1,\mathrm{Rd}}$ for a T, X or Y joint from Table 7.11, but with $k_n = 1{,}0$ and t_0 replaced by t_p for chord face failure, brace failure and punching shear only.	Amend note to; *Take $N_{1,Rd}$ as the value of $N_{1,Rd}$ for a T, X or Y joint from Table 7.11, but with k_n = 1.0, t_0 replaced by t_p and f_{y0} replaced by f_{yp} for chord face failure, brace failure and punching shear only.*	B
GB42			Page 126 Table 7.17	Te	Row 8, side plate reinforcement, tension loading. Side plate thickness minimum limit; $t_p \geq 2t_1$	Delete limit from table.	A

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					Can't find the basis for this limit. As t_p is included in the amended equation, if t_p is less it will be reflected in the joint capacity, so can't see reason for it.		
GB4 3			Page 127 Table 7.18	Te	Row 3, flange plate reinforcement. Flange plate minimum thickness limit; $t_p \geq 2t_1$ and $2t_2$ This is not intended as a limit, this is the plate thickness required to develop the capacity of the brace members. As t_p is included in the equation, if t_p is less it will be reflected in the joint capacity, so suggest this limit is removed.	Delete limit from table.	B
GB4 4			Page 127 Table 7.18	Te	Row 3, flange plate reinforcement. The note mentions replacing t_0 with t_p but likewise f_{y0} could also be replaced with f_{yp} for the plate. Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N joint from Table 7.12, but with t_0 replaced by t_p for chord face failure, brace failure and punching shear only.	Amend note to; *Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N joint ramfrom Table 7.12, but with t_0 replaced by t_p and f_{y0} replaced by f_{yp} for chord face failure, brace failure and punching shear only.*	B
GB4 5			Page 127 Table 7.18	Te	Row 6, division plate reinforcement. The header could be misinterpreted as a method to use if the overlap (λ_{ov}) is less than the minimum 25%. *Reinforced by a division plate between the brace members because of insufficient overlap.*	Rename title to; *Reinforced by a division plate between the brace members to increase overlap effective width.*	B
GB4 6			Page 127 Table 7.18	Te	Row 6, division plate reinforcement. $t_p \geq 2t_1$ and $2t_2$ This is not intended as a limit, this is the plate thickness required to develop the capacity of the brace members. As t_p is included in the equation, if t_p is less it will be reflected in the joint capacity, so suggest this limit is removed.	Delete limit from table.	C
GB4		7.5.3 (2)	Page 128	Te	By referring to 7.5.2 the text implies the reduction	Confirm application of reduction factor μ in respect	B

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7					factor μ is applied to moment resistances as well as axial resistance. CIDECT guidance implies the μ should only be applied to axial forces. Also, for XX joints only axial forces are considered in calculating μ.	to forces and amend note.	
GB4 8			Page 128 Table 7.19	Te	KK joint; $V_{0,Ed}$ is not defined in section 1.5 or here.	Include in section 1.5 defined as; V_{Ed} Total design shear force	B
GB4 9			Page 129 Table 7.20	Te	5th column for h_i/b_i shows limit of 1.0 for 'T or Y' and 'K gap N gap'. This is incorrect for 'T or Y' as the limit is ≥ 0.5 but ≤ 2.0 (the same as for 'X').	Move horizontal divider bar in 5th column down one row so it is between 'T or Y' row and 'K gap N gap' row.	B
GB5 0			Page 129 Table 7.20	Te	7th column for b_i/b_j should include a minimum gap limit of $t_1 + t_2$ to ensure the brace toe is welded to the chord, in line with K joints with RHS chord.	Amend 7th column header by adding *Gap or overlap* above b_i/b_j and adding limit $g \geq t_1 + t_2$ row for 'K gap N gap'. Divider line between 'X' row and 'T or Y' row can then be moved down to be between 'T or Y' row and 'K gap N gap' row.	A
GB5 1			Page 129 Table 7.20	Te	7th column (b_i/b_j). Overlap limit was amended in BS version for CEN corrigendum July 2009 on revision dated 28 February 2010 to; AC2 $25\% \leq \lambda_{ov} \leq \lambda_{ov,lim.}$ 1) AC2 This is confusing as it will be interpreted as the overlap can not be above $\lambda_{ov,lim}$ which it can. The upper limit is 100% although there the overlap can be slightly more to allow for welding. The $\lambda_{ov,lim}$ value is the point at which above this value the brace to chord shear should be	Amend; $25\% \leq \lambda_{ov} \leq \lambda_{ov,lim}$ to $25\% \leq \lambda_{ov}$ or $25\% \leq \lambda_{ov} \leq 100\%$ *But may be slightly above to allow for welding.*	A

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					checked. Therefore, it should not be included here, but as with other failure modes it should be shown as an application limit for the brace to chord shear check which should be shown as a failure mode in Table 7.21.	Also include brace to chord shear failure check with an equation in Table 7.21.	
GB5 2			Page 129 Table 7.20	Te	Note 1 was amended in BS version for CEN corrigendum July 2009 on revision dated 28 February 2010 to; *$\lambda_{ov,lim}$ = 60% if hidden seam not welded and 80% if the hidden seam is welded. If the overlap exceeds $\lambda_{ov,lim}$ or if the braces are rectangular sections with $h_i < b_i$ and/or $h_j < b_j$, the connection between the braces and chord face has to be checked for shear.* This is confusing as it can be interpreted as the overlap can not be above $\lambda_{ov,lim}$. The $\lambda_{ov,lim}$ value is the point at which above this value the brace to chord shear should be checked. Therefore, it should not be included here, but as with other failure modes it should be shown as an application limit for the brace to chord shear check which should be shown as a failure mode in the relevant Table 7.21.	In addition to removing upper overlap limit $\lambda_{ov,lim}$ in the table (listed as separate item), suggest either; 1. simply including a brace to chord face shear check equation as a failure mode in tables 7.21 and then removing note 1. Or 2. Amend note 1 to read; *If $\lambda_{ov} \geq 60\%$ and hidden seam is not welded or $\lambda_{ov} \geq 80\%$ and hidden seam is welded or if rectangular section braces are $h_i < b_i$ or $h_j < b_j$ the connection between the braces and chord face has to be checked for shear.*	B
GB5 3			Page 130 Table 7.21	Te	Row 2, T, Y and X joints. Chord web failure is likely to occur for an X joint and this is not listed as a check.	Add equations for chord web yielding and chord shear check or add note to check these failure modes when *cos $\theta_1 > h_1/h_0$* using equation from K and N gap joints in Table 7.21.	B
GB5 4			Page 130 Table 7.21	Te	Row 7, K and N gap joints, chord web yielding equation, uses subscript '*1*' in $N_{1,Rd}$ and θ_1 which should be subscript '*i*' as there are two bracings.	Amend equation to; $N_{i,Rd} = \frac{f_{y0} t_w b_w}{[illegible]} / \gamma_{M5}$	A

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					AC2 Chord web yielding AC2 AC2 $N_{1,\mathrm{Rd}} = \frac{f_{y0} t_w b_w}{\sin\theta_1} / \gamma_{M5}$ AC2		
GB5 5			Page 130 Table 7.21	Te	Row 11, chord shear and row 18, chord shear area is defined as A_v, for consistency it should also include the subscript for the chord, *0*.	Amend A_v to $A_{v,0}$ in Chord Shear equation for $N_{i,Rd}$, $N_{0,Rd}$ and equation for A_v in functions list at end of Table 7.21. Also amend A_v where used elsewhere (section 1.5 (3), Table 7.12 and Table 7.21)	A
GB5 6			Page 130 Table 7.21	Te	Row 18, functions for p_{eff}: AC2 $p_{\mathrm{eff}} = t_w + 2r + 7 t_f f_{y0} / f_{yi}$ but for T, Y, X joints and K and N gap joints: $p_{\mathrm{eff}} \le b_i + h_i - 2t_i$ but for K and N overlap joints: $p_{\mathrm{eff}} \le b_i$ AC2 Note at end of table advising on simply replacing b_1 and h_1 with d_1 does not apply to p_{eff}.	Amend p_{eff} for T, Y, X joints and K and N gap joints to: $p_{eff} \le b_i + h_i - 2t_i$ (for RHS bracings) $p_{eff} \le 0.5\pi (d_i - t_i)$ (for CHS bracings)	B
GB5 7			Page 130 Table 7.21	Te	Row 19, function for $b_{e,ov}$: $b_{\mathrm{e,ov}} = \frac{10}{b_j / t_j} \frac{f_{yj} t_j}{f_{yi} t_i} b_i$ but $b_{\mathrm{e,ov}} \le b_i$ Note at end of table advising on simply replacing b_1 and h_1 with d_1 does not apply to $b_{e,ov}$.	Amend existing $b_{e,ov}$ by adding '*For RHS bracings*' before equation for $b_{e,ov}$. Add equation for CHS bracings; For CHS bracings; $b_{e,ov} = \frac{12}{\text{[illegible]}} \frac{f_{yj} t_j}{\text{[illegible]}} d_i$ but $\le d_i$	B
GB5 8		7.6 (8)	Page 131	Te	Equation for brace failure when chord stiffeners are used uses terms b_{eff} and $b_{eff,s}$. This is inconsistent with terms used in Table 7.21 which was changed to p_{eff} as it can be more than just the brace width.	Amend b_{eff} and $b_{eff,s}$ to p_{eff} and $p_{eff,s}$	A

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					$N_{i,Rd} = 2 f_{yi} t_i (b_{eff} + b_{eff,s}) / \gamma_{M5}$ where: $b_{eff} = t_w + 2r + 7 t_f f_{y0} / f_{yi}$ but $\leq b_i + h_i - 2t_i$ $b_{eff,s} = t_s + 2a + 7 t_f f_{y0} / f_{yi}$ but $\leq b_i + h_i - 2t_i$ $b_{eff} + b_{eff,s} \leq b_i + h_i - 2t_i$		
GB5 9			Page 131 Table 7.21	Ed	Row 1, 2nd column: Design resistance $[i = 1 \text{ or } 2, j = \text{overlapped brace}]$ As there is only one brace reference to '*i*' and '*j*' is incorrect.	Delete *[I = 1 or 2, j = overlapped brace]*	A
GB6 0			Page 131 Table 7.22	Te	Row 5, column 2. Replace symbol h_z with $(h_1 - t_1)$ for T and Y joints, brace failure.	Replace h_z with $(h_1 - t_1)$ in Table 7.22 for T and Y joints, brace failure. This will be consistent with chord web yielding where $(h_1 - t_1)$ is used. Symbol h_z can then be deleted from list of symbols in section 1.5 (3) as it is not used elsewhere.	C
GB6 1		7.7 (3)	Page 132	Te	This clause seems to refer to EN 1993-1-1 for determining $N_{0,Rd}$. But $N_{0,Rd}$ is given in Table 7.24.	Delete sentence; *Verification should be made according to EN 1993-1-1.*	A
GB6 2			Page 132 Table 7.23	Te	7th column (*Gap or overlap b_i/b_j*). Overlap limit was amended in BS version for CEN corrigendum July 2009 on revision dated 28 February 2010 to; AC2 $25\% \leq \lambda_{ov} \leq \lambda_{ov,lim}$ 2) AC2 This is confusing as it will be interpreted as the overlap can not be above $\lambda_{ov,lim}$ which it can. The upper limit is 100% although there the overlap can be slightly more to allow for welding. The $\lambda_{ov,lim}$ value is the point at which above this value the brace to chord shear should be checked. Therefore, it should not be included here, but as with other failure modes it should be	Amend; $25\% \leq \lambda_{ov} \leq \lambda_{ov,lim}$ to $25\% \leq \lambda_{ov}$ or $25\% \leq \lambda_{ov} \leq 100\%$ *But may be slightly above to allow for welding.* Also include brace to chord shear failure check	A

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					shown as an application limit for the brace to chord shear check which should be shown as a failure mode in Table 7.24.	with an equation in Table 7.24.	
GB63			Page 132 Table 7.23	Te	Note 2 was amended in BS version for CEN corrigendum July 2009 on revision dated 28 February 2010 to; *$\lambda_{ov,lim}$ = 60% if hidden seam not welded and 80% if the hidden seam is welded. If the overlap exceeds $\lambda_{ov,lim}$ or if the braces are rectangular sections with $h_i < b_i$ and/or $h_j < b_j$, the connection between the braces and chord face has to be checked for shear.* This is confusing as it can be interpreted as the overlap can not be above $\lambda_{ov,lim}$. The $\lambda_{ov,lim}$ value is the point at which above this value the brace to chord shear should be checked. Therefore, it should not be included here, but as with other failure modes it should be shown as an application limit for the brace to chord shear check which should be shown as a failure mode in the relevant Table 7.24.	In addition to removing upper overlap limit $\lambda_{ov,lim}$ in the table (listed as separate item), suggest either; 3. Simply including a brace to chord face shear check equation as a failure mode in tables 7.24 and then removing note 2. Or 4. Amend note 2 to read; *If $\lambda_{ov} \geq$ 60% and hidden seam is not welded or $\lambda_{ov} \geq$ 80% and hidden seam is welded or if rectangular section braces are $h_i < b_i$ or $h_j < b_j$ the connection between the braces and chord face has to be checked for shear.*	B
GB64			Page 132 Table 7.23	Te	7th column (*Gap or overlap b_i/b_j*). Validity limits are shown elsewhere in the table for CHS bracings but not for overlap.	Row 5 (*K overlap, N overlap*), column 7 (*Gap or overlap b_i/b_j*). Add; *$d_i / d_j \geq 0.75$*	B
GB65			Page 133 Table 7.24	Te	Row 3, column 1, K and N gap joints. The sketch uses subscripts *i* and *j* for overlap joints, for gap joints it should use subscripts *1* and *2*.	Amend subscript *i* to *1* and *j* to *2*.	A
GB66			Page 133 Table 7.24	Te	Row 4, column 2. '*Chord failure*' title is inconsistent as it is '*Chord shear*' in previous tables.	Amend '*Chord failure*' title to '*Chord shear*'.	A
GB67			Page 133 Table 7.24	Te	Row 5, chord shear and row 12, chord shear area is defined as A_v, for consistency it should also include the subscript for the chord, *0*.	Amend A_v to $A_{v,0}$ in Chord Shear equation for $N_{i,Rd}$, $N_{0,Rd}$ and equation for A_v in functions list at end of Table 7.24. Also amend A_v where used elsewhere (section 1.5 (3), Table 7.12 and Table	A

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						7.21)	
GB68			Page 133 Table 7.24	Te	Row 12, column 1: $V_{\mathrm{pl,Rd}} = \frac{f_{y0} A_v}{\sqrt{3}} / \gamma_{M5}$ Equation for $V_{pl,Rd}$ includes the partial safety factor γ_{M5} which should not be applied here as it is a joint partial safety factor. I believe it should be γ_{M0} as shown in EN 1993-1-1 626 (2).	Amend γ_{M5} to γ_{M0} in equation for $V_{pl,Rd}$.	C
GB69			Page 133 Table 7.24	Te	Row 12, column 2, function for $b_{e,ov}$: $b_{\mathrm{e,ov}} = \frac{10}{b_j / t_j} \frac{f_{yj} t_j}{f_{yi} t_i} b_i$ Note at end of table advising on simply replacing b_1 and h_1 with d_1 does not apply to $b_{e,ov}$.	Amend existing $b_{e,ov}$ by adding '*For RHS bracings*' before equation for $b_{e,ov}$. Add equation for CHS bracings; For CHS bracings; $b_{e,ov} = \frac{12}{\text{[illegible]}} \frac{f_{yj} t_j}{\text{[illegible]}} d_i \quad but \ \leq d_i$	B
DE1				general	As there was not enough time for the German mirror group to handle all national comments on the revision of this Part of EC 3 more comments will be sent to TC 250 and SC 3 until the middle of December 2014.		
DE2			all	general	The revision process of EN 1993-1-8 should include: a) the correction of mistakes, b) the elimination of inconsistencies and c) the amendment of readability. Evaluation of the experience with the application of the existing Eurocode 3-1-8 in the different CEN member states as the basis for the identification of the main issues		

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					for the revision is necessary. The standard should be improved in user friendliness by applying principles of mechanics. Where empirical approaches cannot be avoided, they have to be labelled as such. The Eurocode should be state of the art and not the state of science.		
DE3		1.4.2 to 1.4.5		technical	Subclauses 1.4.2 to 1.4.5 are textbook knowledge.	Consider to delete these clauses.	A
DE4		1.5	(3) to (6)	technical	We would recommend to move chapter 6 into Annex.	Paragraph (3) to (6) could be deleted, if chapter 6 was moved into Annex.	B
DE5		2.3	(2)	technical	Check the delimination for considering deformation and rigidity conditions of joints.	Consider to add a paragraph: (2) The qualities of the joints concerning her moment-rotation-characteristic features should correspond to the assumption of the working load method.	B
DE6		2.5		technical	This clause is the interface between analysis of structures and the design of joints. It is aimed to find an understandable and clear allocation of the rules to the suitable parts.	If chapter 5, 6 and 7 were moved into Annex some of their clauses could be moved into clause 2.5 (see below).	B
DE7		2.5	(2)	technical	The limit of implementation for framework with compact construction or very rigid girts would be helpfully.	Detailed information will be supplied until the middle of December 2014.	B
DE8		2.7		technical		Consider to add the former subclause 6.2.2 (4) after this clause (NEW 2.8).	B

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DE9		2.7		technical	After this clause a new clause 2.9 "Load distribution and load-sharing concept" should be added.	Consider to add three new subclauses: 2.9.1 Distribution of Loads by bending stress 2.9.2 Distribution of Loads under the subjection of pressure and tension 2.9.3 Distribution of Loads (Former subclause 6.2.5 (3)	B
DE10		3.6.1	(4)	technical	What about over-large holes?	Consider to add a clause for over-large holes.	D
DE11		3.6.1	(14)	technical	It is textbook knowledge.	Consider to delete the clause.	B
DE12		3.6	Table 3.4	technical	The results for combined shear and tension are conservative. Is background information available?		D
DE13		3.6.2		technical	*"Injection bolts"* are special-purpose building elements and are not used regularly.	Consider to move into Annex.	B
DE14		3.6.3		technical	Anchor bolts	Consider to add the former subclause 6.2.2 (7) in a new clause (NEW 3.6.3).	B
DE15		3.10.3	(1)	technical	clarify	Consider to add a technical drawing to clarify what is meant, because angles connected by one leg can be either unsymmetrical members or symmetrical members.	D
DE16		3.10.3	(2)	technical	Check formular (3.12) and (3.13). Comparative calculations with elematary		D

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					engineering calculation methods produce other results. Is background information available?		
DE17		4.2		Editorial	The content "Welding consumables" is already covered by EN 1090.	Consider to delete the clause.	D
DE18		4.3.2.1		Editorial	clarify	Consider to add technical drawings to clarify the general rules.	B
DE19		4.4		Editorial	clarify	Consider to add technical drawings to clarify the rule.	B
DE20		4.5.3	Table 4.1	technical	Does the correlation factor β_w depend on the standard?	The rule could be simplified by omitting the steel standards.	B
DE21		4.5.3		technical	The "*Simplified method for design resistance of fillet weld*" (Subclause 4.5.3.3) is the more general method.	We recommend to put subclause 4.5.3.3 ahead of subclause 4.5.3.2	A
DE22		**5.**		technical	Clauses for *"semi-continuous joints"* are special rules however not suitable for ordinary analysis of steel constructions.	For condensed basic rules we would recommend to move chapter 5 into Annex. Bases and assumption for global analysis are also necessary. That's why we recommend to move some of the subclauses into chapter 2. (see above: e.g. subclause 5.1.5)	B
DE23		**6.**		technical		We would recommend to move chapter 6 into Annex. Bases and assumption for global analysis we recommend to move into chapter 2. (see	B

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						above: subclause 6.4.1; 6.2.2(2))	
DE2 4		**7.**		technical		We would recommend to move chapter 7 into Annex.	B
DS/ DK1		GE		GE	Please consider and respond to the following 6 questions using the generic comment template provided. 1. Do any clauses require editorial or technical correction? 2. Which clauses would benefit from improvements in clarity? 3. Where should the scope of the EN be extended? 4. Where could the EN by shortened? 5. Are there any clauses whose application leads to uneconomic construction? 6. Are there any clauses whose application necessitates excessive design effort?	As the mandate M/515 has been agreed and presently is awaiting the tender process by NEN and the financial agreement between the EC Commission and CEN we find pinpointing specific clauses for improvements, clarifications etc. is a work that should be undertaken in the SC, WG and PT. Therefore, the following comments are primarily from the National Annex and other selected comments. There will be additional comments from Denmark through the future work in SC, WG and PT.	
BE1				Ge	Belgium is not able to upload its comment on time. We will communicate our complete comments before 2015-01-15 at the latest.		
GR1		1.2		ed	Compliance with EN 1998	Comply with the provisions and requirements of EN 1998 relating to the strength and stiffness of the connections.	D
GR2		3.4.2	Table 3.2	ed	The minimum and maximum distances in various cases missing.	Correct/clarify this clause	C: comment not understandable
GR3		3.4.2	Fig. 3.1b	ed	The limits are in contradiction with the text of paragraph 5 of Table 3.2. If spacing p_2 has the minimum value 1,2d_o the distance L must be greater than 2,4 d_o. If spacing p_2 is greater than	Correct	C: not necessary

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					2,4 d_o then the distance L can be smaller than 2,4 d_o.		
GR4		3.5	Figure 3.1	te	The distance p_1 should be included in Figure 3.1b with staggered spacing of fasteners. The distance e_1 should also be added in Figure 3.1c, and clarify if max e_1 refers to the upper or lower row of fasteners.	Provide improved figures.	D
GR5		3.5	Table 3.3	te	Minimum spacing for $p_{1,0}$ and $p_{1,i}$ is missing.	Include the minimum spacing for these cases.	C: table shows everything
GR6		3.5	Table 3.3	te	Maximum spacing for $p_{1,0}$ and $p_{1,i}$ in the non-exposed case, and in compression, is unlimited or the same as p_1? If the latter is true, then it appears that for the $p_{1,i}$ the exposed case provides a more relaxed limit than the non-exposed one, which is strange.	Divide the non-exposed case in tension and compression sub-cases.	C: intensions is not clear
GR7		3.6.1	Table 3.4	te	The bearing resistance of staggered bolts is limited due to k_1 factor. As result unreasonable values are obtained. The reason is the definition of distance p_2 of the fig. 3.1 .	Clarify the distance p_2 which must be used for the calculations	D
GR8		3.6.1	Table 3.4	te	The influence on the bearing resistance of the factor k_1 is significant for small edge distances e_2. This is in contradiction to resent reaserch.	Correct/clarify this clause	D
GR9		6.2.6.1	Par.2	te	For the web-panel shear strength the assumption of similar beam depths is enforced, without quantifying a criterion for such similarity.	Provide a criterion to determine if double beam joints have similar depths or not.	D
GR1 0		6.2.6.1	Par.5	te	This clause is very short and gives no suggestion how the $V_{wp,Rd}$ is affected when a diagonal stiffener is present.	Elaborate on the procedure to follow for the verification of the diagonal stiffener and how $V_{wp,Rd}$ is affected. Take into account that many different diagonal stiffener configurations are available in practice.	B
GR1		6.2.6.2	Table 6.3	te	The symbols ω_1 and ω_2 are not described.	Describe the meaning of indices 1 and 2 in the	C: description not

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1						text.	necessary
GR1 2		6.2.6.4 6.2.6.5	Table 6.4 Table 6.5 Table 6.6	te	Hiding the yield line background behind the values in these tables is problematic and error prone. The learning curve for using these tables is very steep.	The learning curve could be shortened if figures were available for enough cases, showing the possible yield line patterns and for each bolt row the cases to be considered.	D
GR1 3		6.2.6.6	Figure 6.11	te	The analytical expressions for the curves of α are required for programming the methodology in a connection design software.	Include an analytical procedure for the estimation of α.	B
GR1 4		7.5.2.1	Tables 7.10 -7.11	te	The formula for the design resistance for T,Y and X joints leads to unsafe results.	Correct/clarify	D
PL1	3.5	Table 3.2		te	For connections category E subjected to fatigue loads the criterion should be $\gamma_{Ff} F_{t,Ed} \leq F_{p,Cd} / \gamma_{Mf}$ with bolt stress range $0{,}2\gamma_{Ff}\sigma_E \leq \Delta\sigma_R / \gamma_{Mf}$	Add NOTE: For connections category E subjected to fatigue loads the criterion should be $\gamma_{Ff} F_{t,Ed} \leq F_{p,Cd} / \gamma_{Mf}$ with bolt stress range $0{,}2\gamma_{Ff}\sigma_E \leq \Delta\sigma_R / \gamma_{Mf}$	D
PL2	6.2	Table 6.2		te	Criterion when prying forces should be taken into account in the first row of Table 6.2: $L_b \leq \frac{8{,}8 m^3 A_s}{\Sigma \ell_{eff,1} t_f^3}$ should be replace by: $t_f < 1{,}5d\sqrt{\frac{m f_{u,b}}{l_{eff} f_y}}$ Prying forces criterion $L_b \leq L_b$* supported by base plate tests should be in paragraph 6.2.6.12. The equation to calculate value of prying force is necessary to check stress range in the bolts. (see attached below **Background 1**)	Change criterion $L_b \leq L_b$* by $t_f < 1{,}5d\sqrt{\frac{m f_{u,b}}{l_{eff} f_y}}$ Transmit L_b and equation L_b* to paragraph 6.2.6.12 Add: Q is the prying force in T-stub row (for one direct bending) given by $\Sigma Q = \Sigma F_{t,Rd} - F_{T,Rd}$ **NOTE 3:** Criterion for anchor bolts of one direct bending base plates see 6.2.6.12 Proposal for changed Table 6.2 in attached below **"Proposal A"**	B
PL3		6.2	Table 6.5	ed	Description of distance e_1 under this Table is wrong	Should be: "distance to free end of column" like under table 6.4	A
PL4		6.2	Table 6.7	te	Information in this Table and referred drawings (fig.) are not consistent. Following changes should be	Table 6.7 should be changed: see attached below	B

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					considered: - Boundary values should be weak (with „ < ” not “ ≤ “) to avoid “zero” in denominator of formulas and ∞ when e = 0, - signs of calculated values of moment resistance are in some cases negative, what is not consistent with fig. 6.18 and formula in last row in the table.	**Proposal B**	
PL5		6.3	Table 6.11	ed	l_{eff} for k_{15} reference is wrong	Should be: 6.2.6.5	A
PL6		6.3	Table 6.12	te	Following changes should be considered: - Boundary values for N and e should be weak (with „ < ” not “ ≤ “) to avoid “zero” in denominator of formulas and ∞ when e = 0, - results of calculation of stiffness in table 6.12 (sign of stiffness) is sometimes negative, what is disconformities with physics,	Table 6.12 should be changed similar to Table 6.7 change accordingly	B
PL7		6.2	**6.2.7.2 (9) (6.26)**	Te	In multiply bolt row connections with semi rigid end-plates bolt forces exceeds ultimate tension resistance and ultimate moment resistance to design moment resistance ratio is very low. (see attached below **Background 2)**	Modify text equation (6.26) and add additional text as in attached below **Proposal C**	B
FI1				General	In these Finnish comments line number has not been given mainly due to the following reasons: -CEN has not defined how the line number should be calculated ***from the beginning or from the end of the standard ***form the top or the bottom of the page ***from the beginning of section, clause or subclause -We assume that people giving answers to these comments are clever enough to		

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					understand if the reference is made for example to clause 1.2.3.4(5)		
FI2		1.2.4		te/ed	For example EN 14399-1:2002 is dated in list of reference standards and in EN 1090 is not dated. Should both standards EN 1993-1-8 and EN 1090-2 have similar normative references?		C: dated or not dated is intended
FI3		1.2.6		te	Rivets could be deleted from EN 1993-1-8. The general aim in the revision on EN 1993 (including all Eurocodes) is the reduction of the number on NDP`s.	Delete the clause.	C: why delete rivets?
FI4		1.5		ed	$d_{o,t}$ and $d_{o,v}$ are not used in EN 1993-1-8.	Delete $d_{o,t}$ and $d_{o,v}$.	D
FI5		1.5		ed	(5) The stress ratios used in section 7 are defined as follows: *n* is the ratio (σ0,Ed / *f*y0) / γM5 (used for RHS chords); *n*p is the ratio (σp,Ed / *f*y0) / γM5 (used for CHS chords);	n = (σ0,Ed / *f*y0) / γM5 is wrong and should be written as σ0,Ed / (*f*y0 / γM5).	C

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					$\sigma_{0,Ed}$ is the maximum compressive stress in the chord at a joint; $\sigma_{p,Ed}$ is the value of $\sigma_{0,Ed}$ excluding the stress due to the components parallel to the chord axis of the axial forces in the braces at that joint, see Figure 1.4. $n = (\sigma_{0,Ed} / f_{y0}) / \gamma_{M5}$ is wrong and should be written as $\sigma_{0,Ed} / (f_{y0} / \gamma_{M5})$.		
FI6		2.2(2)		te	Delete γ_{M7}. See also comment to 3.6.1(2).	Delete γ_{M7}.	C
FI7		1.3(b)	Figure	ed	In the figure 1.3(b) the lines of the right hand side of p and q shall be in the same line.	Correct the figure 1.3(b)	B
FI8		2.2(2)		te	Requirements for quality level of welds by using γ_{M2} = 1,25 should be given in this clause more in details (or in clause 4.1(3). See also comments to 4.1(3). Proposal on the right hand side is based on the proposal of the revision on Finnish NA SFS- EN 1993-1-8.	In calculating the design resistance of the welds by using the partial safety factor γ_{M2} = 1,25 the precondition is, that the quality level of welds is at least C according to SFS-EN ISO 5817. In the execution class EXC2 "Overlap" (506) and "Stray arc" (601) may be allowed as quality level D, if there is not other harm. "End crater pipe" (2025) is not allowed in the effective length of the weld in any weld quality class.	B

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					It is strongly proposed close co-operation between CEN/TC250/SC3 and CEN/TC135 in questions dealing with structural safety.	In the execution class EXC1 weld quality class should fulfil at least the requirements for the execution class EXC2 given in the clause 7.6 of EN 1090-2.	
FI9		3.1.1(3)		te	Note and NDP should be deleted. "Market forces" will decide which grades are used in various countries. There is no need for this kind of information in standard. The general aim in the revision on EN 1993 (including all Eurocodes) is the reduction of the number on NDP`s.	Delete the Note.	C: there are national differences
FI10		3.4.2(1)		te	Generally NDP should also include recommended value, but it is missing from this Note. In the Finnish National Annex it is stated: "The preload in this case should be 0,70 f_{ub} A_s. In this case bolted connections should be controlled at least as non-preloaded connections." The general aim in the revision on EN	"The preload in this case should be 0,70 f_{ub} A_s. In this case bolted connections should be controlled at least as non-preloaded connections." or give at least recommended value.	B

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					1993 (including all Eurocodes) is the reduction of the number on NDP`s.		
FI11		3.5		te	Rivets could be deleted from EN 1993-1-8. The general aim in the revision on EN 1993 (including all Eurocodes) is the reduction of the number on NDP`s.	Delete rivets from EN 1993-1-8.	C: why delete
FI12		3.6.1(2)	formula (3.1)	te	The formula (3.1) is in contradiction to the formula (3.7), which is used in formulas (3.6), (3.8a) and (3.8b). Formula (3.1) may be deleted, because it is not used. Also γ_{M7} should be deleted. See also comment to 2.2(2)	Delete formula (3.1) and make reference to the formula (3.7).	D
FI13		3.6.1(5)		T	Are the rules of EN 1993-1-8 also applicable to bolts, when diameter is less than M12? See also rules in table 8.4 of EN 1993-1-3, where M6 is allowed, but with certain limitations.		D
FI14		Table 3.4		te	Note 1: “in slotted holes, where the longitudinal axis of the slotted hole is perpendicular to the direction of the	Coefficient of 0.6 should only be applied, when slot length/bolt diameter is less or equal than 3,5.	B

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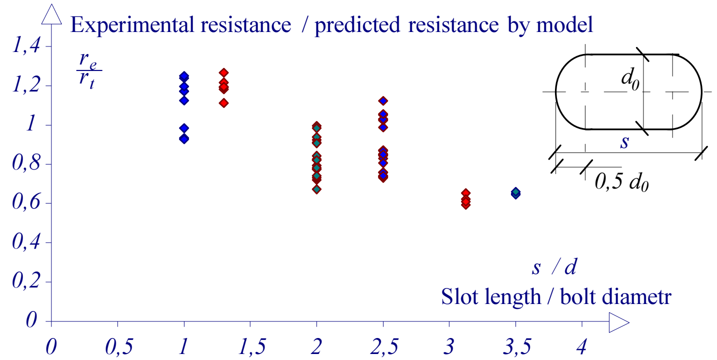

					force transfer, is 0,6 times the bearing resistance for bolts in round, normal holes.” Coefficient of 0.6 should only be applied, when		

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					slot length/bolt diameter is limited on about 3,5. See: "Design of structural connections to Eurocode 3. Frequently asked questions. Building Research Establisment Ltd., Watford, September 2003., 140 p. " See the figure below:		
FI15		3.6.2.2(4)		te/ed	The wording "Wenn keine Injektion vorhanden wäre" in the German translation is not used in the original English version. Translations in CEN-official languages should be equal.	Translations in CEN-official languages should be equal.	C: it is obvious that future translations should be improved
FI16		3.9.1(1)		ed	A note or a clause should be added: In the formula (3.6) either γ_{M3} or $\gamma_{M3,ser}$ is used depending of the category of the bolted connections according to the table 2.1 and the table 3.2.	Add: In the formula (3.6) either γ_{M3} or $\gamma_{M3,ser}$ is used depending of the category of the bolted connections according to the table 2.1 and the table 3.2.	D
FI17		3.9.1(1), Table 3.6		ed	Change "bolts in normal holes" into "bolts in normal <u>round</u> holes".	Add: Change "bolts in normal holes" into "bolts in normal <u>round</u> holes".	A

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FI18		3.10.3(2)	formula (3.11)	te	$N_{u.Rd} = \frac{2{,}0(e_2 - 0{,}5d_0)t\ f_u}{\gamma_{M2}}$; …(3.11) For example case a) in figure 3.9 Steel: S355, f_u = 510 N/mm^2, f_y = 355 N/mm^2 L60x6 is connected with one M20 bolt, strength class 8.8 e_2 = 30 mm, e_1 = 30 mm, d_0 = 22 mm Concentric tension load resistance: $N_{u.Rd}$ = 2,0x(30 – 0,5x22)x6x510/1,25 = 93,0 kN If block shear is taken into account: $V_{eff,1,Rd} = f_u A_{nt} / \gamma_{M2} + (1/\sqrt{3}) f_y A_{nv} / \gamma_{M0}$ ... (3.9) A_{nt} = (e2-0,5d0)t = (30 – 0,5x22)x6 = 114 mm^2 A_{nv} = (e1-0,5d0)t = (30-0,5x22)x6 = 114 mm^2 $V_{eff,1,Rd}$ = 510x114/1,25 + (1/ √3) x 355x114/1,0 = 69,9 kN Minimum bearing resistance in one shear plane connections: $F_{b,Rd} \leq 1{,}5\ f_u\ d\ t / \gamma_{M2}$ …(3.2)	Make reference between clauses 3.10.2. 3.10.3 and 3.6.1(10) saying which clauses should be checked in the case of angle sections.	B

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					= 1,5x510x20x6/1,25 = 73,44 kN Formula (3.11) gives much bigger values than (3.9) and (3.2). Probably rules given in 3.10.3(2) cover also connections in one shear plane (see clause 3.6.1(10)) Probably rules given in 3.10.3(2) cover also connections in one shear plane (see clause 3.6.1(10). The key question is if rule according to clause 3.6.1(10) should also be applied in the case if angle section is one of the two connected members. See the attached article from Kulak.		
FI19		3.10.2		te	Clause 3.10.2 does not cover the following situation:	Proposal for interaction. $\frac{F_{Ed,V}}{V_{eff,Rd,V}} + \frac{F_{Ed.H}}{V_{eff,Rd,H}} \leq 1$	F

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					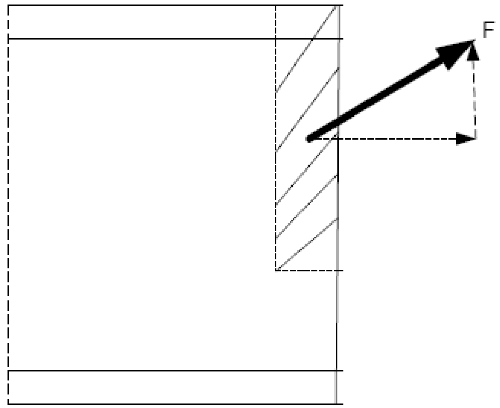 Proposal is on the right hand side, if better interaction formula does not exist.		
FI20		4.1(3)		te	Is the weld quality class D acceptable from the structural safety point of view? See two examples below.	Weld quality D should not be used at all. (or give appropriate limitation of the use of weld quality class D)	B

Example 1/ Copy of EN ISO 5817

No.	ISO 6520-1 reference	Imperfection designation	Remarks	t mm	Limits for imperfections for quality levels D	C	B
1.7	5011 5012	Continuous undercut	Smooth transition is required. This is not regarded as a systematic	0,5 to 3	Short imperfections: $h \leq 0{,}2\ t$	Short imperfections: $h \leq 0{,}1\ t$	Not permitted

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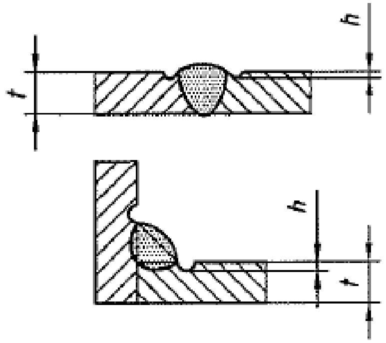
		Intermittent undercut	imperfection. 		> 3 h ≤ 0,2 t, but max. 1 mm	h ≤ 0,1 t, but max. 0,5 mm	h ≤ 0,05 t, but max. 0,5 mm
FI21		4.1(3)		te	Weld quality level D (numbers 5011 and 5012 above): If we assume t = 3 mm, we get h = 0,2* 3 = 0,6 mm Real thickness of the plate is then 3-0,6 = 2,4 mm If there is tension in the welded connection then the resistance is 100*2.6/3 = 80 % of the original nominal tension resistance. If there is bending about the plate in the welded connection then the resistance is $100*(2.6/3)^3$ = 51,2 % of the original nominal bending resistance. Is the intention that γ_{M2} = 1,25 should	Weld quality D should not be used at all. (or give appropriate limitation of the use of weld quality D)	B

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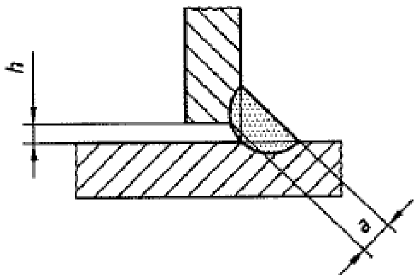
					cover also these cases? Proposal: Weld quality D should not be used at all. Compare EN 1090-2 where it is allowed in EXC1.		

Example 2: Copy of EN ISO 5817

No.	ISO 6520-1 reference	Imperfection designation	Remarks	t mm	Limits for imperfections for quality levels		
					D	C	B
3.3	617	Incorrect root gap for fillet welds	The limitation of Clause 5 as regards systematic imperfection does not apply.	0,5 to 3	h ≤ 0,5 mm + 0,1 a	h ≤ 0,3 mm + 0,1 a	h ≤ 0,2 mm + 0,1 a
				> 3	h ≤ 1 mm + 0,3 a, but max. 4 mm	h ≤ 0,5 mm + 0,2 a, but max. 3 mm	h ≤ 0,5 mm + 0,1 a, but max. 2 mm



MB/ NC[1]	Line number	Clause/ Subclause	Paragraph/ Figure/ Table/	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision
FI22		4.1(3)		te	Weld quality level D (number 617 above): If we assume a = 4 mm, we get h = 1 + 0,3* 4 = 2,2 mm	Weld quality D should not be used at all. (or give appropriate limitation of the use of weld quality D)	B

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					In this case throat thickness will decrease about 39 % compared to the original throat thickness 4 mm. Is the intention that γ_{M2} = 1,25 should cover also this case? Proposal: Weld quality D should not be used at all. Compare EN 1090-2 where it is allowed in EXC1.		
FI23		4.3.1(2)		ed	See Finnish comment to 4.5.2 and 4.5.2(1)		B
FI24		4.5.2 and 4.5.2(1) and 4.5.2(2)		ed/te	In EN 12345 the wording “effective throat thickness” has a different meaning than in EN 1993-1-8. In EN 12345 equal leg angle is assumed, but in EN 1993-1-8 an un-equal leg angle is assumed. **Note:** EN 12345 is now EN ISO 17659 (but we have not checked, if the definitions of EN ISO 17659 are same as in EN 12345). Harmonisation of terminology and		B

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					definitions between welding standards and EN 1993-1-8 is needed.		
FI25		4.5.2(1)	Fig. 4.3	ed/te	According to EN 12345 throat thickness given in the figure 4.3 is called "design throat thickness". **Note:** EN 12345 is now EN ISO 17659. (but we have not checked, if the definitions of EN ISO 17659 are same as in EN 12345) Harmonisation of terminology and definitions between welding standards and EN 1993-1-8 is needed.		B
FI26		4.5.2(3)	Including Fig. 4.3 and Fig. 4.4	ed/te	"a <u>deep</u> penetration fillet weld" is not defined in EN 1993-1-8 and not in EN 1090-2 and not in welding standards like EN 12345. When a penetration is <u>deep</u> and when it is not <u>deep?</u> How much penetration should be (in millimetres or in %) to be called <u>deep</u> penetration. There seems not to be any case, where	Replace " a <u>deep</u> penetration" by "penetration".	C: each fillet weld has some penetration. A deep penetration is a controlled process defined parameter

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					deep penetration should really be used. Our understanding is that there is no need to define in details, what is deep penetration. **Note:** EN 12345 is now EN ISO 17659 (but we have not checked, if the definitions of EN ISO 17659 are same as in EN 12345) Harmonisation of terminology and definitions between welding standards and EN 1993-1-8 is needed.		
FI27		4.5.2 and 4.5.2(1) and 4.5.2(2) and 4.5.2(3)	Terminol ogy	te/ed	Below is copy of the definitions of ISO/TR 25901-1 (still draft) for information. Our understanding is: - report will probably be accepted before the end of this year - at the moment we do not know, if deep penetration will be defined more in details in ISO/TR 25901-1 (for example how many %) than given in the table, probably not and hopefully not - Finnish proposal is as given above to clause 4.5.2(3), if that is not accepted, then the right place to define deep		C: a matter of EN1090

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					penetration in details is standard EN 1090-2 as proposed by CEN/TC250/SC3 during the conversion from ENV to EN, but for some reason CEN/TC135 has not given any definitions so far		

Table: Definitions according to ISO/TR 25901-1 (still proposal at the time when these comments are written)

throat thickness	thickness of a fillet weld	gorge	épaisseur d'une soudure d'angle		
design throat thickness	throat thickness specified by the designer	gorge théorique (soudures d'angle) ; épaisseur théorique (soudures bout à bout)	gorge/épaisseur spécifiée par le concepteur NOTE Voir fig. 13 de l'ISO 17659	Sollnahtdicke	vom Konstrukteur festgelegte Nahtdicke
effective throat thickness	design value of the height of the largest triangle that can be inscribed in the section of a fillet weld <u>NOTE See 1 in Figure 1b.</u>	gorge efficace (soudures d'angle) ; épaisseur efficace (soudures bout à bout)	pour les soudures d'angle, hauteur du plus grand triangle pouvant être inscrit dans la section de la soudure pour les soudures bout à bout, distance de la surface de la pièce à la partie inférieure de la pénétration	wirksame Nahtdicke	Abmessung, die für die Kraftübertragung maßgebend ist, abhängig von der Ausführung der Naht und vom Einbrand
deep penetration	nominal or effective throat thickness to				

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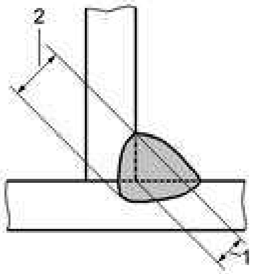
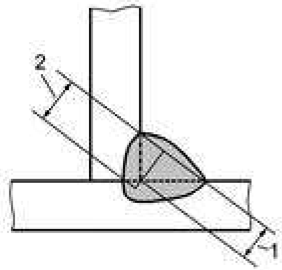
throat thickness	which a certain amount of fusion penetration is added <u>NOTE See 2 in Figure 1.</u>				
nominal throat thickness	design value of the height of the largest isosceles triangle that can be inscribed in the section of a fillet weld <u>NOTE See 1 in Figure 1a.</u>				



Key

1 nominal throat thickness

2 deep penetration throat thickness



Key

1 effective throat thickness

2 deep penetration throat thickness

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Figure 1 a) Nominal throat thickness						**Figure 1b) Effective throat thickness**	
FI28		4.5.2(1)… (3)		te/ed	The key point in EN 1993-1-8 should be that the designer says what is <u>the needed throat thickness based on his calculations</u> (if he/she has made calculations) independent whether penetration or ("deep penetration") is used or not. When (not if) different wording and definitions are used by designers and "welding people", then to outcome may be that people do not understand each other at all and make safety related mistakes.		C. see above
FI29		4.5.3.1		te	In clause 7.3 of standard EN 1090-2 various welding processes are given. Are all design rules given in EN 1993-1-8 valid if the welding is made by all possible welding processes given in clause 7.3 of standard EN 1090-2. If not, then all acceptable welding		C. all metal arc welding processes are covered

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					processes from EN 1993-1-8 point of view should be given in EN 1993-1-8.		
FI30		4.5.3.2(6)	Table	te	The steel grades (and β_W- values) given in table 4.1 should be checked based on the final choice of material standards and steel grades covered by various part of EN 1993. Standards EN 10149-2 and EN 10149-3 should be added to EN 1993-1-1 and EN 1993-1-8, which means that β_W- values to steel grades according to SEN 10149-2 and EN 10149-3 should also be given in table 4.1. Also steel grade S450 according to EN 10025 is missing from the table 4.1. Also some steel grades mentioned in EN 1993-5 are missing from table 4.1. In the Finish National Annex guidance is given as follows: β_W- values to steel grades according to SFS-EN 10149-2 and SFS-EN 10149-3 should be determined based on yield strength as for steels according to SFS-EN 10025.		B

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FI31		4.7.2(1) and 4.7.2(2)		te/ed	In 4.7.2(1) it is stated: "(1) The design resistance of a partial penetration butt weld should be determined using the method for a deep penetration fillet weld given in 4.5.2(3)." - Is it clear, what has to be done, when "deep penetration" is not defined? - Is it clear, what it meant by "a partial penetration butt weld" In 4.7.2(2) it is stated: "(2) The throat thickness of a partial penetration butt weld should not be greater than the depth of penetration that can be consistently achieved, see 4.5.2(3)." - Is it clear, what has to be done, when "deep penetration" is not defined? - Is it clear, what it meant by "a partial penetration butt weld" See Finnish comments to 4.5.2.		B

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FI32		5.2.1(2)	Note	te	Additional information is not given in the Finnish National Annex. Therefore, Finland may also accept, if the note (NDP) is deleted. The general aim in the revision on EN 1993 (including all Eurocodes) is the reduction of the number on NDP`s.	Delete the note.	B
FI33		6.2.7.2(9)	Note	te	No further information is given in the Finnish National Annex. The recommended equation is used in the Finnish National Annex. Therefore, Finland may also accept, if this note (NDP) is deleted. The general aim in the revision on EN 1993 (including all Eurocodes) is the reduction of the number on NDP`s.	Delete the note.	B
FI34		6.2.6.11(2)		te	Is corrigenda correct compared to the original text? Background document is needed.		D
FI35		7.1.2(6)		te	AC of EN 1993-1-8 contains the following addition: *Paragraph* "(6)", *add to the text:*	Proposal for the text, see below.	B

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					"If the overlap exceeds $\lambda_{ov,lim.}$=60% in case the hidden seam of the overlapped brace is not welded or $\lambda_{ov,lim.}$=80% in case the hidden seam of the overlapped brace is welded or if the braces are rectangular sections with $h_i < b_i$ and/or $h_j < b_j$, the connection between the braces and the chord face should be checked for shear.". Also rules how this checking for shear should be done should be added.		

Proposal as addition to the clause 7.1.2(6) (numbering is based the proposal for the revision of Finnish National Annex of EN 1993-1-8).

2. Overlapped joints of structural hollow sections

2.1 General

(1) In overlapped joints welding or not welding of the hidden seam of bracing members shall be shown in execution specification.

(2) When following the recommendations given in this chapter 2, the nominal yield strength of hot and cold formed hollow sections shall not be more than 460 N/mm^2, when hollow section is encompassed as an end product. When the nominal yield strength is more than 355 N/mm^2, the strength values given in this chapter shall be multiplied by value 0,9.

(3) In Clause No 7.1.2(6) of standard SFS-EN 1993-1-8 according the additional rule of AC/2009 an additional checking shall be done for strength overlapped joints:

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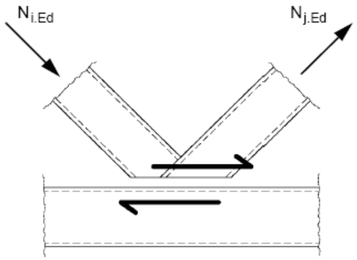
When $\lambda_{ov} > \lambda_{ov.lim}$ (circular or square or rectangular bracing members) or when rectangular bracing members $h_i < b_i$ or $h_j < b_j$, shearing failure of bracing members in direction of longitudinal axis of chord members shall be checked. (see picture 2.1).



Picture 2.1 Overlapped joint. Bracing members shearing off from chord in overlapped K joints.

(4) Limit value of overlapping $\lambda_{ov.lim}$ is defined in following way:

- $\lambda_{ov.lim}$ = 60 %, when the hidden seam of overlapped brace member is NOT welded to chord.
- $\lambda_{ov.lim}$ = 80 %, when the hidden seam of overlapped brace member is welded to chord.

(5) In Standard SFS-EN 1993-1-8 it is not given rules for checking the resistance of above mentioned shear failure mode. In their absence, the following criteria mentioned in 2.2 -2.3 should be used.

(6) Depending on is the chord member made of hollow section or I-profile, rules given in table 2.1 should be used.

2.2 Circular hollow sections as bracing members

(1) Shearing resistance of bracing members shall be checked in a following way:

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

When: 60 % < λ_{ov} < 100 %, when the hidden seam overlapped brace member is NOT welded to chord.
or: 80 % < λ_{ov} < 100 %, when the hidden seam overlapped brace member is welded to chord.

The following rule should be checked:

$$N_{i.Ed}\cos\theta_i + N_{j.Ed}\cos\theta_j \leq N_{s.Rd} \qquad (2.1)$$

where:

$$N_{s.Rd} = \frac{\pi}{4}\cdot\left[\frac{f_{ui}}{\sqrt{3}}\cdot\frac{\left[\left(\frac{100-\lambda_{ov}}{100}\right)\cdot 2d_i + d_{eff.i}\right]\cdot t_i}{\sin\theta_i} + \frac{f_{uj}}{\sqrt{3}}\cdot\frac{\left(2d_j + c_s d_{eff.j}\right)\cdot t_j}{\sin\theta_j}\right]\cdot\frac{1}{\gamma_{M5}} \qquad (2.2)$$

When λ_{ov} = 100 % the following rule should be checked:

$$N_{i.Ed}\cos\theta_i + N_{j.Ed}\cos\theta_j \leq N_{s.Rd} \qquad (2.3)$$

where:

$$N_{s.Rd} = \frac{\pi}{4}\cdot\frac{f_{uj}}{\sqrt{3}}\cdot\frac{\left(3d_j + d_{eff.j}\right)\cdot t_j}{\sin\theta_j}\cdot\frac{1}{\gamma_{M5}} \qquad (2.4)$$

(2) In clauses (2.1) - (2.4) sub index i means overlapping and sub index j means overlapped bracing member mentioned in standard EN 1993-1-8.

Other notations are defined in following way:

f_u Nominal ultimate strength of bracing member

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

c_s An effective shearing area coefficient:

$c_s = 1$, when hidden seam of overlapped bracing member is not welded into chord

$c_s = 2$, when hidden seam of overlapped bracing member is not welded into chord

d_{eff} An effective diameter according to table 2.1.

All other notations are according to standard EN 1993-1-8.

2.3 Square or rectangular hollow sections as bracing members

(1) Shearing resistance of bracing members shall be checked in a following way:

When: 60 % < λ_{ov} < 100 %, when seamed side of overlapped bracing member is not welded into chord
or: 80 % < λ_{ov} < 100 %, when seamed side of overlapped bracing member is not welded into chord
or: $h_i < b_i$ ja $\lambda_{ov} < 100$ %
or: $h_j < b_j$ ja $\lambda_{ov} < 100$ %

the following rule should be checked:

$$N_{i.Ed}\cos\theta_i + N_{j.Ed}\cos\theta_j \le N_{s.Rd} \qquad (2.5)$$

where:

$$N_{s.Rd} = \left[\frac{f_{ui}}{\sqrt{3}} \cdot \frac{\left[\left(\frac{100-\lambda_{ov}}{100}\right)\cdot 2h_i + b_{eff.i}\right]\cdot t_i}{\sin\theta_i} + \frac{f_{uj}}{\sqrt{3}} \cdot \frac{\left(2h_j + c_s b_{eff.j}\right)\cdot t_j}{\sin\theta_j}\right]\cdot\frac{1}{\gamma_{M5}} \qquad (2.6)$$

When: λ_{ov} = 100 % the following rule should be checked:

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$$N_{i.Ed}\cos\theta_i + N_{j.Ed}\cos\theta_j \le N_{s.Rd} \qquad (2.7)$$

where:

$$N_{s.Rd} = \frac{f_{uj}}{\sqrt{3}} \cdot \frac{(2h_j + b_j + b_{eff.j}) \cdot t_j}{\sin\theta_j} \cdot \frac{1}{\gamma_{M5}} \qquad (2.8)$$

(2) In clauses (2.5) - (2.8) subindex i means overlapping and and subindex j overlapped bracing member according to standard EN 1993-1-8.

Other notations are defined in a following way:

f_u The nominal ultimate strength of bracing member
c_s An effective cross section area coefficient:
$c_s = 1$, when hidden seam of overlapped bracing member is not welded into chord
$c_s = 2$, when hidden seam side of overlapped bracing member is welded into chord
b_{eff} An effective width according to table 2.1.

All other notations are according to standard SFS-EN 1993-1-8.

Table 2.1 Shear resistance of brace members in overlapped joints. Effective width of bracing members.

		Bracing members	
		Circular (CHS)	Square or rectangular (RHS)
	Circular	Overlapping CHS-brace to CHS-chord: $d_{eff.i} = \frac{12}{d_0/t_0} \cdot \frac{f_{y0} \cdot t_0}{f_{yi} \cdot t_i} \cdot d_i \quad but \le d_i$	–

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	(CHS)	Overlapped CHS-brace to CHS-chord: $d_{eff.j} = \frac{12}{d_0/t_0}\cdot\frac{f_{y0}\cdot t_0}{f_{yj}\cdot t_j}\cdot d_j \quad but \le d_j$	–
Chord	Square or rectangular (RHS)	Overlapping CHS-brace to RHS-chord: $d_{eff.i} = \frac{10}{d_0/t_0}\cdot\frac{f_{y0}\cdot t_0}{f_{yi}\cdot t_i}\cdot d_i \quad but \le d_i$	Overlapping RHS-brace to RHS-chord: $b_{eff.i} = \frac{10}{b_0/t_0}\cdot\frac{f_{y0}\cdot t_0}{f_{yi}\cdot t_i}\cdot b_i \quad but \le b_i$
		Overlapped CHS-brace to RHS-chord: $d_{eff.j} = \frac{10}{d_0/t_0}\cdot\frac{f_{y0}\cdot t_0}{f_{yj}\cdot t_j}\cdot d_j \quad but \le d_j$	Overlapped RHS-brace to RHS-chord: $b_{eff.j} = \frac{10}{b_0/t_0}\cdot\frac{f_{y0}\cdot t_0}{f_{yj}\cdot t_j}\cdot b_j \quad but \le b_j$
	I-profile	Overlapping CHS-brace to I-chord: $d_{eff.i} = t_w + 2r + 7t_0\cdot\frac{f_{y0}}{f_{yi}} \quad but \le d_i$	Overlapping RHS-brace to I-chord: $b_{eff.i} = t_w + 2r + 7t_0\cdot\frac{f_{y0}}{f_{yi}} \quad but \le b_i$
		Overlapped CHS-brace to I-chord: $d_{eff.j} = t_w + 2r + 7t_0\cdot\frac{f_{y0}}{f_{yj}} \quad but \le d_j$	Overlapped RHS-brace to I-chord: $b_{eff.j} = t_w + 2r + 7t_0\cdot\frac{f_{y0}}{f_{yj}} \quad but \le b_j$

MB/ NC	Line number	Clause/ Subclause	Paragraph/ Figure/ Table/	Type of comment	Comments	Proposed change	Proposed procedure / decision
FI36		7.3.1(6)		te	When designing welded joints of tubular members following EN rules, the welds should be made as strong as the joined members. This entails large welds and large fabrication costs, especially in cases where the actions of the joints are		B

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					not large. Typically, for example in trusses all the joints are not under maximum actions, so smaller weld sizes may be used in these joints and fabrication costs reduced accordingly. Clause EN 1998-1-8, 7.3.1(6) allows the use of smaller weld sizes. The clause states: *"The criterion given in 7.3.1(4) may be waived where a smaller weld size can be justified both with regard to resistance and with regard to deformation capacity and rotation capacity, taking account of the possibility that only part of its length is effective."* However, this rule cannot be used since the rules stating how to fulfil these requirements are missing. This enforces the use of full strength welds. Detailed rules should be given in standard including following requirements: • Justification of the resistance of smaller welds, • with regard to deformation capacity, • and rotation capacity,		

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					• taking into account the possibility that only part of its length is effective. If detailed rules are not possible to give, then one possibility could be to delete whole clause, because it may be dangerous		
FI37		7.5.2.1(8)	Table 7.16	te	Second case in the table 7.16: * Requirement for cross-sections class should be given also for this case. Third case in the table 7.16: * Requirement for cross-sections class should be given also for this case.		B
FI38		7.4.2(2)	Table 7.2	ed	In the figures of the table 7.2 N_1 should be changed to $N_{1,Ed}$, etc. This remark is valid also for other relevant tables in section 7.		C
FI39		7.4.2	Table 7.2	te	Add the following text: If the overlap exceeds $\lambda_{ov,lim.}$=60% in	If the overlap exceeds $\lambda_{ov,lim.}$=60% in case the hidden seam of the overlapped brace is not welded or	A

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					case the hidden seam of the overlapped brace is not welded or $\lambda_{ov,lim.}$=80% in case the hidden seam of the overlapped brace is welded or if the braces are rectangular sections with $h_i < b_i$ and/or $h_j < b_j$, the connection between the braces and the chord face should be checked for shear. See also Finnish comment to 7.1.2(6).	$\lambda_{ov,lim.}$=80% in case the hidden seam of the overlapped brace is welded or if the braces are rectangular sections with $h_i < b_i$ and/or $h_j < b_j$, the connection between the braces and the chord face should be checked for shear.	
FI40		7.4.2	Table 7.3	te	Limits $t_i/d_0 \leq 0,2$ are given in ENV 1993 Annex KK. However we have not found these limitations from CIDECT guidelines and not from the book of J.A. Packer. Compare table 7.13, where these kind of limits are given. Should be checked during the revision.		A
FI41		7.7(1)		te	Should there be some limitations (minimum and maximum values) for the ratio h_0/b_0 for channel sections?		B

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FI42		7.7(2)		ed/te	The clause should be: For joints which are not within the limits of table 7.23 all failure modes according to 7.2.2 should be checked. In additions, the secondary moments in the joint caused by their rotational stiffness should be taken into account. See also the wording used in 7.4.1(3), 7.5.1(3) and 7.6(3). The wording "bending stiffness" should be changed into "rotational stiffness" as in clauses 7.4.1(3), 7.5.1(3) and 7.6(3).	Change to: For joints which are not within the limits of table 7.23 all failure modes according to 7.2.2 should be checked. In additions, the secondary moments in the joint caused by their rotational stiffness should be taken into account.	A
FI43		7.6	Table 7.20	te	Add the following text: If the overlap exceeds $\lambda_{ov,lim.=}$60% in case the hidden seam of the overlapped brace is not welded or $\lambda_{ov,lim.=}$80% in case the hidden seam of the overlapped brace is welded or if the braces are rectangular sections with $h_i < b_i$ and/or $h_j < b_j$, the connection between the braces and the chord face should be	If the overlap exceeds $\lambda_{ov,lim.=}$60% in case the hidden seam of the overlapped brace is not welded or $\lambda_{ov,lim.=}$80% in case the hidden seam of the overlapped brace is welded or if the braces are rectangular sections with $h_i < b_i$ and/or $h_j < b_j$, the connection between the braces and the chord face should be checked for shear.	A

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					checked for shear. See also Finnish comment to 7.1.2(6).		
FI44		7.7	Table 7.23	te	Add the following text: If the overlap exceeds $\lambda_{ov,lim.=}$60% in case the hidden seam of the overlapped brace is not welded or $\lambda_{ov,lim.=}$80% in case the hidden seam of the overlapped brace is welded or if the braces are rectangular sections with $h_i < b_i$ and/or $h_j < b_j$, the connection between the braces and the chord face should be checked for shear. See also Finnish comment to 7.1.2(6).	If the overlap exceeds $\lambda_{ov,lim.=}$60% in case the hidden seam of the overlapped brace is not welded or $\lambda_{ov,lim.=}$80% in case the hidden seam of the overlapped brace is welded or if the braces are rectangular sections with $h_i < b_i$ and/or $h_j < b_j$, the connection between the braces and the chord face should be checked for shear.	A
SE1		2.2(2)		te	Delete γ_{M7}. See also comment to 3.6.1(2).	Delete γ_{M7}.	C
SE2		3.6.1(2)	formula	te	The formula (3.1) is in contradiction to	Delete formula (3.1) and make reference	B

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			(3.1)		the formula (3.7), which is used in formulas (3.6), (3.8a) and (3.8b). Formula (3.1) may be deleted, because it is not used. Also γ_{M7} should be deleted. See also comment to 2.2(2)	to the formula (3.7).	
SE3		3.6.1(5)		T	Are the rules of EN 1993-1-8 also applicable to bolts, when diameter is less than M12? See also rules in table 8.4 of EN 1993-1-3, where M6 is allowed, but with certain limitations.		See FI13
SE4		3.9.1(1)		ed	A note or a clause should be added: In the formula (3.6) either γM3 or γM3,ser is used depending of the category of the bolted connections according to the table 2.1 and the table 3.2. The expression "bolts in normal holes" is used in table 3.6. The corresponding expression in EN 1090-2 is "bolts in normal rounded holes".	Add: In the formula (3.6) either γ_{M3} or $\gamma_{M3,ser}$ is used depending of the category of the bolted connections according to the table 2.1 and the table 3.2. Change to "bolts in normal rounded holes" in table 3.6.	See FI16

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SE5		3.10.3(2)	formula (3.11)	te	$N_{u.Rd} = \frac{2{,}0(e_2 - 0{,}5d_0)t\ f_u}{\gamma_{M2}}$; …(3.11) For example case a) in figure 3.9 Steel: S355, f_u = 510 N/mm², f_y = 355 N/mm² L60x6 is connected with one M20 bolt, strength class 8.8 e_2 = 30 mm, e_1 = 30 mm, d_0 = 22 mm Concentric tension load resistance: $N_{u.Rd}$ = 2,0x(30 – 0,5x22)x6x510/1,25 = <u>93,0 kN</u> If block shear is taken into account: $V_{eff,1,Rd} = f_u A_{nt} / \gamma_{M2} + (1 / \sqrt{3}) f_y A_{nv} / \gamma_{M0}$ ... (3.9) A_{nt} = (e2-0,5d0)t = (30 – 0,5x22)x6 = 114 mm² A_{nv} = (e1-0,5d0)t = (30-0,5x22)x6 = 114 mm² $V_{eff,1,Rd}$ = 510x114/1,25 + (1/ √3) x 355x114/1,0 = <u>69,9 kN</u> Minimum bearing resistance in one shear plane connections: $F_{b,Rd} \leq 1{,}5\ f_u\ d\ t / \gamma_{M2}$ …(3.2)	Make reference between clauses 3.10.2. 3.10.3 and 3.6.1(10) saying which clauses should be checked in the case of angle sections.	See FI18

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					= 1,5x510x20x6/1,25 = 73,44 kN Formula (3.11) gives much bigger values than (3.9) and (3.2). Probably rules given in 3.10.3(2) cover also connections in one shear plane (see clause 3.6.1(10). The key question is if rule according to clause 3.6.1(10) should also be applied in the case if angle section is one of the two connected members.		
SE6		4.5.2(3)	Including Fig. 4.3 and Fig. 4.4	ed/te		Replace “ a deep penetration” by “penetration”.	See FI26
SE7		4.5.3.2(6)	Table	te	Standards EN 10149-2 and EN 10149-3 should be added to EN 1993-1-1 and EN 1993-1-8, which means that β_W- values to steel grades according to SEN 10149-2 and EN 10149-3 should also be given in table 4.1. Also steel grade S450 according to EN 10025 is missing from the table 4.1. Also some steel grades mentioned in EN	4.5.3.2(6)	See FI30

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<table>
<tr><th>MB/ NC[1]</th><th>Line number (e.g. 17)</th><th>Clause/ Subclause (e.g. 3.1)</th><th>Paragraph/ Figure/ Table/ (e.g. Table 1)</th><th>Type of comment[2]</th><th>Comments</th><th>Proposed change</th><th>Proposed procedure / decision</th></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td>1993-5 are missing from table 4.1.

In the Swedish NA for EN 1993-1-1 it is stated that the following should be added to EN 1993-1-8, table 4.1.

Tabell E-2 Korrelationsfaktor β_w för kälsvetsar
<table>
<tr><th colspan="2">Standard och stålsort</th><th>Faktor β_w</th></tr>
<tr><th>SS-EN 10149-2</th><th>SS-EN 10149-3</th><th></th></tr>
<tr><td></td><td>S 260NC</td><td>0,85</td></tr>
<tr><td>S 315MC S 355MC</td><td>S 315NC S 355NC</td><td>0,9</td></tr>
<tr><td>S 420MC S 460MC</td><td>S420NC</td><td>1,0</td></tr>
</table></td><td></td><td></td></tr>
<tr><td>SE8</td><td></td><td>7.3.1(6)</td><td></td><td>te</td><td>When designing welded joints of tubular members following EN rules, the welds should be made as strong as the joined members. This entails large welds and large fabrication costs, especially in cases where the actions of the joints are not large. Typically, for example in trusses all the joints are not under maximum actions, so smaller weld sizes may be used in these joints and fabrication costs reduced accordingly.

Clause EN 1998-1-8, 7.3.1(6) allows the use of smaller weld sizes. The clause states:

“The criterion given in 7.3.1(4) may be</td><td></td><td>See FI36</td></tr>
</table>

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					waived where a smaller weld size can be justified both with regard to resistance and with regard to deformation capacity and rotation capacity, taking account of the possibility that only part of its length is effective." However, this rule cannot be used since the rules stating how to fulfil these requirements are missing. This enforces the use of full strength welds. Rules should be given in standard including following requirements: • Justification of the resistance of smaller welds, • with regard to deformation capacity, • and rotation capacity, • taking into account the possibility that only part of its length is effective.		
CZ1				ge	Missing criterion for limit for strain e (if any more complicate detail is calculated with nonlinear FEM analysis, limit of e should be governing)	Give limit strain e – for example e= 5 %	B, maybe better defined in 1.1
CZ2		3.10.3	Table 3.8	te	Joint of the frame battened member from two angles, connected by one leg - not clear whether this clause is applicable	Make more clear explanation.	C: explanations seems to clear enough

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CZ3		3.5; 3.6.1	Table 3.3; Table 3.4	te	Table 3.3, Note (5): "For staggered rows of fasteners a minimum line spacing of p_2 = 1,2d_0 may be used, provided that the minimum distance, *L*, between any two fasteners is greater or equal than 2,4d_0, see Figure 1.3b)": Considering the note (5) and the formula for calculation the coefficient k_1 in Table 3.4: $k_1 = 1{,}4\frac{p_2}{d_0} - 1{,}7 = 1{,}4\frac{1{,}2d_0}{d_0} - 1{,}7 = -0{,}02$	To consider the minimum distance, *L*, defined in the Note (5) in the calculation of k_1 instead of p_2: $k_1 = 1{,}4\frac{L}{d_0} - 1{,}7 = 1{,}4\frac{2{,}4d_0}{d_0} - 1{,}7 = 1{,}66 = \frac{2}{3}\cdot 2{,}5$	B
Which clauses would benefit from improvements in clarity?							
FR2 1		3.6.1	Table 3.4	Te	Values of k_s are given in Table 3.6 for slotted holes in the longitudinal direction. However, Table 3.4 gives no information on the bearing resistance in the longitudinal direction.	Clarify the bearing resistance of slotted holes.	B
FR2 2		4.12	(2)	Te	No information is given to consider the effect of eccentricity.		C: rejected, basic knowledge
FR2 3		6.2.6.1	(5)	Te	Requirements should be given concerning design and shear resistance in presence of diagonal stiffeners.		D
FR2 4		6.2.6.5	Figure 6.11	Te	An analytical formulation should be given for α such as an abacus can't be used automatically.	The Green Book (Publication SCI P398) gives a formulation.	C: is already dealt above
FR2 5		6.2.6.7	(1)	Te	This clause is only applicable when flanges are symmetric.	Clarifications should be given for non-symmetric flanges.	D
FR2 6		6.2.6.12	(3)	Te	This clause refers to EN 1992-1-1 to determine the design bond resistance. However, this standard only gives information on bond resistance of steel bar according to EN 10080 which are not commonly used in steel structures.	Formulations of ENV 1992 for plain bars should be given.	D
FR2 7		6.2.6.12	(5)	Te	These limitations create big problems in practice. These limitations are normally given for	Improve the limit to 360 N/mm^2 and revise Figure 6.14a.	D

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					completely threaded bars. It should be indicated.	Suppress the sentence : "The anchorage length should be calculated in accordance with EN 1992-1-1" And give a method to determine this resistance.	
FR28		6.2.7.2	(2) to (8)	Ed	These clauses are not easy to understand and should be clarified.		B
FR29		6.2.8.1	(5)	Te	It is asked to determine the bearing resistance according to EN 1992. However, EN 1992-1-1 gives no rules about it.	Add a note : prEN 1992-4 is under UAP-enquiry and should give information about concrete check.	D
GB70			Page 14	Te/Ed	A_v is defined as chord shear area. For consistency it should also include the subscript for the chord, *0*.	Amend A_v to $A_{v,0}$ here and where used elsewhere (Table 7.12, Table 7.21 and Table 7.21)	A
GB71		1.5	Page 16 Figure 1.4	Te	Chord forces $N_{0,Ed}$ and $N_{p,Ed}$ are shown and tension in figures (a) and (c) but compression in (b). Irrelevant difference cause confusion. The chord force is mainly relevant to a reduction in chord face failure when the chord is in compression. Therefore the standard diagram usually shows compression chord forces. Although the brace in (a) could be in tension or compression the standard diagram shows it in compression. Figure (c) $N_{0,Ed}$ and $N_{p,Ed}$ are shown at wrong ends.	Change figures; (a) Change force arrow for N_i, $N_{0,Ed}$ and $N_{p,Ed}$ to compression (c) Change force arrow for $N_{0,Ed}$ and $N_{p,Ed}$ to compression. Swop $N_{0,Ed}$ and $N_{p,Ed}$ around so $N_{0,Ed}$ is on the right and $N_{p,Ed}$ is on the left.	C
GB72			Table 5.1	Tech	This table relates the type of joint model (pinned, rigid, semi-rigid) to the method of global analysis. Some designers find this difficult to interpret and consideration should be given to improving the clarity of the table.		C
GB73		1.5 (6) 7.1.2 (6)	Page 16, 101,	Te	$\lambda_{ov,lim}$ is the point at which brace to chord shear should be checked, it is not a maximum overlap limit. It use suggests it is a maximum overlap	Amend; $25\% \leq \lambda_{ov} \leq \lambda_{ov,lim}$	A

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		Table 7.1, Table 7.8, Table 7.20, Table 7.23	108, 116, 129 and 132		limit. Therefore, it should not be included here, but as with other failure modes it should be shown as an application limit for the brace to chord shear check which should be shown as a failure mode in the relevant tables (Table 7.2, 7.10, 7.21 and 7.24).	to *25%* $\leq \lambda_{ov}$ or *25%* $\leq \lambda_{ov} \leq$ *100%* *But may be slightly above to allow for welding.* Also include brace to chord shear failure check with an equation in Table 7.2, 7.10, 7.21 and 7.24.	
GB7 4		1.5	Page 17 Figure 1.4	Ed	Figure (b) bracings are on the underside when figures (a) and (c) bracings are shown above the chord. The standard orientation is with bracings above the chord. As it is, the most obvious difference between (b) and (c) is location of bracings rather than one being a gap and the other an overlap. This is confusing.	Flip figure (b) so bracings are on top of chord.	C
GB7 5		3.5	Table 3.3	ed	It would be helpful to draw the designer's attention to the different minimum values for structures subject to fatigue – the consequence of the reference in (2) is not always noted in practice and the Table is used for fatigue structures.	In the heading to column 2, append: (For structures subject to fatigue, see EN 1993-1-9)	D
GB7 6		3.5	Figure 3.1	ed/te	The minimum limits should be removed from the figure, which, according to its title, only gives symbols. (And also because the figure is referred to by EN 1993-1-9, which does not necessarily have the same minimum limits in b.).). Requirements should not be duplicated in a Eurocode.	Delete the limiting values in b), c) and d).	B
GB7 7		4.7.2		te	Pat Pen Butt welds designed as a deep penetration FW	As an alternative, the strength can be calculated using the yield strength of the weaker part joined over a reduced depth equal to the effective throat thickness.	B

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Template for comments and secretariat observations

Date: 2014-10-22	Document:	Project: EN 1993-1-8

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision
GB78		6.2.7.1 (14)		te	What is meant by ‘material’? does this include bolts and splice plates? How does this apply to a simple end plate bearing splice, located close to a point of restraint e.g. a column splice within a floor? No net uplift due to BM and 4 bolts within end plate to cater for tie forces / location.	Remove the clause, unnecessary and confusing. What does it achieve, if the connection is in bearing then why do other elements need to carry 25% of the load?	D
GB79	(1-2)	6.4.2		te	An expression for evaluation of the available joint rotation needs to be included in the code.		D
GB80		7.1.2 (6)	Page 101	Ed	Second paragraph was modified as part of the CEN corrigendum to; *If the overlap exceeds $\lambda_{ov,lim}$ =60% in case the hidden seam of the overlapped brace is not welded or $\lambda_{ov,lim}$ =80% in case the hidden seam of the overlapped brace is welded or if the braces are rectangular sections with $h_i < b_i$ and/or $h_j < b_j$, the connection between the braces and the chord face should be checked for shear.* This does not read good English.	Amend text to; *If the overlap exceeds $\lambda_{ov,lim}$ =60% ~~in case~~ when the hidden seam of the overlapped brace is not welded or $\lambda_{ov,lim}$ =80% ~~in case~~ when the hidden seam of the overlapped brace is welded or if the braces are rectangular sections with $h_i < b_i$ and/or $h_j < b_j$, the connection between the braces and the chord face should be checked for shear.*	A
GB81		7.3.1 (2)	Page 107	Ed	Clause states overlap joints only require hidden part to be welded if components perpendicular to chord differ by more than 20%. However, welding may be required to increase the brace to chord shear resistance.	Add note; *However, welding may be required to increase the brace to chord shear resistance.*	B
GB82			Page 110 Table 7.3	Ed	Punching shear failure equation; $\sigma_{max} t_1 = (N_{Ed} / A + M_{Ed} / W_{el}) t_1 \leq 2 t_0 (f_{y0} / \sqrt{3}) / \gamma_{M5}$ Terms in first set of brackets refer to brace 1 (plate), but this is not clear. Requires clarification.	Amend terms in first set of brackets to; $\sigma_{max}\ t_1$ = $(N_{1,Ed} / A_1 + M_{1,Ed} / W_{el,1})\ t_1 \leq 2t_0\ (f_{y0} / \sqrt{3}) / \gamma_{M5}$	A
GB83			Page 111 Table 7.4	Te	Punching shear failure was revised to the CEN corrugendum to;	Clarification required.	B

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					AC2 I or H sections with $\eta > 2$ (for axial compression and out-of-plane bending) and RHS sections: The term *'for axial compression and out-of-plane bending'* could be clarified. If you only have axial brace loading do you use *'All other cases'*, or does it mean both equations are based on axial and out-of-plane bending? If this is the intended meaning why is in-plane bending not considered?		
GB8 4		7.4.3 (2)	Page 115	Te	The text implies the reduction factor μ should be applied to all failure modes in 7.4.2. Multiplanar effects would not affect punching shear therefore the reduction factor should only be applied to chord face failure and not punching shear failure.	Confirm application of reduction factor μ in respect to failure modes and amend note.	B
GB8 5		7.5.3 (2)	Page 129	Te	The text implies the reduction factor μ should be applied to all failure modes in 7.5.2. Multiplanar effects would not affect punching shear therefore the reduction factor should only be applied to chord face failure.	Confirm application of reduction factor in respect to failure modes and amend note.	B
GB8 6				Tech	Consideration should be given to replacing the rule for the design of fillet welds given in EC3: Part 1.8 with the method given in EC3: Part 1.12.		See FI36
DS/ DK2		3.10.2		te	The design rules for block tearing are too simple and very limited.	The rules should be changed to 1 describe the yield line circumference around the bolts (crossing through or outside the holes) 2 define the strength to be used for the shear and normal stresses along the circumference (yield line) When using the above principles it will be possible to design the block tearing for tension, shear, moment etc.	D
DS/ DK3		4.1	2(P)	te	The required quality level for welding "C" according to EN 25817 (EN ISO 5817) should be deleted.	Delete the requirement to class "C" or ad a comment that class "D" should only be used for a welding with and supplementary safety factor on	B

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						the bearing capacity (for example a factor 1.2).	
DS/ DK4		4.2	Table 4.2	te	It should be clarified which direction of welding this paragraph is valid for. It is an open question if this table been coordinated with the requirement connected to EN 1993-1-10, table 2.1.	It should be clarified which direction of welding this paragraph is valid for.	D
GR1 5		3.6.1 (5)		te	The directive of paragraph which leads to reduction of the shear resistance of the bolts opposes to the logical conclusion that the reduction regards the bearing design resistance.	Clarify this clause	B
GR1 6		3.6.1 (5)		ed	It must be clearly defined that the formula (3.2) is complementary to the formulas of table 3.4.	Clarify this clause. For example : “The design bearing resistance $F_{b,Rd}$ of table 3.4 for each bolt should be limited to……”	B
GR1 7		3.10.2		te	The design block tearing resistance distinct concentric and eccentric loading. It is not clear if the formulas can be used in case of interaction of axial and shear force.	Clarify this clause	D
GR1 8		3.10.3		te	The title “ Angles connected by one leg and other unsymmetrically connected members in tension” is in not covered by the text of the paragraph. Formulas concerns only angle cleats. The part “other unsymmetrically” remains unclear without reference or figure.	Clarify the term “other” and provide the guidelines for the design.	C
GR1 9		3.13.2	Fig. 3.11	te	The formula for the bending moment is provided only for simple pinned connection (3 plates). There is no reference or recommendation for more complex layouts.	Provide the guidelines for the design.	C

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GR20		6.1.3	Table 6.1 (20)	te	There are no guidelines for the calculation of hunched connections, especially for seismic zones.	Provide the guidelines for the design.	D
GR21		6.2.6.1	Par. 2	te	The resistance for a double-sided joint is given only for configurations with similar beam depths without any definition of similarity.	Provide criterion for this restriction.	B
GR22		6.2.6.1	Par. 5	te	According to the paragraph when diagonal web stiffeners are used the design shear resistance of a column web should be determined according to EN 1993-1-1. This leads to the conclusion that the diagonal web stiffeners have negligible influence.	Clarify this clause	D
GR23		6.2.6.4	Tables 6.4 6.5 , 6.6	te	The effective length p (see fig. 6.9) for bolt-rows considered as part of group has exact value only for equally spaced bolt rows. The general case is $p=(p_{i-1} + p_i)/2$	Generalize fig. 6.9 in order to cover all geometries.	D
GR24		6.2.6.4	Tables 6.4 6.5 , 6.6	te	Figures of the yield lines of beams flange and end plate helps to understand the meaning of the referred tables.	Provide figures similar to annex J .	D
PL8	3.10.2(2) i (3)			te	Lack of precise distinction of axially and non-axially loaded connection	To clarify	D
PL9							
PL10		6.3	Table 6.11	te	k_{10} value should be different for prestressed and not prestressed connections. similar to base plate connections. **Generally** distinguish should be introduced between prestressed and not prestressed end-plate connection	It should be clarify	B Frans

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PL1 1		6.2	**6.2.6.1(5)**	te	NOTE for 6.2.6.1(5) what does it mean „to have similar depth" ?	change end of the sentence to: " .. …the two beams are assumed to have depth value the greatest or mean of beams depths".	D
PL1 2		6.2	**6.2.6.12 (3)**	te	Add criterion for prying force in base plates	Transmit L_b and equation L_b* from Table 6.2	D
FI45				General	In these Finnish comments line number has not been given mainly due to the following reasons: -CEN has not defined how the line number should be calculated ***from the beginning or from the end of the standard ***form the top or the bottom of the page ***from the beginning of section, clause or subclause -We assume that people giving answers to these comments are clever enough to understand if the reference is made for example to clause 1.2.3.4(5)		
FI46				General	General Finnish comments to **all Parts** of EN 1993.		
FI47		General		te/ed	General Finnish comments to all parts of EN 1993: Informative annexes should not be used at all in the revised EN 1993, because some users think that informative	Change informative annexes into normative annex or delete informative annexes	

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					annexes need not to be followed at all. - it is prosed, that informative annexes are changed into normative annexes (including NDP`s as needed) or deleted - generally standardization means “to agree on something”, in most of the cases informative annexes contain issues on which agreement has not been achieved and therefore informative annexes have almost nothing to do with standardization – in order to avoid any misunderstanding most of the informative annexes are very useful also from the practical point of view and also from the point of view of writing National Annexes - see Finnish comments to various parts of EN 1993		
FI48		General		te/ed	B-rules (for buildings) should be avoided as far as possible, most of those rules are more general	Delete “B” from B-rules. B-rules should be applicable also for other structures than buildings.	
FI49		General		te/ed	P-rules should not be given in EN 1993 at all, all needed P-rules are possible to give in EN 1990 and/or EN 1991 in	Delete all P-rules from EN 1993 and check that EN 1990 covers all needed P-rules	

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					general form independent of material		
FI50		General		te/ed	The design (service) life of the structure should be determined separately for each project and given in the execution specification and not in National Annexes due to the following reasons: a) the owner of the structure should have the right to determine the design life of his property (structure) b) therefore this issue does not belong to National Annex and not to the authority - of course authorities could have a right to give some minimum values	Change all rules for the design life on such a way that the determination of the design life time belong preferably to the owner of the structures, not to National Annex and the authority	
FI51		General		te/ed	Various parts of EN 1993 include some guidance of the design life of the structures, which is not bad at all, but: a) On the other hand EN 1990 gives some guidance for the choice of the design life and therefore all guidance should be collected into one place, that is: in EN 1990. b) The present rules and recommendations in various part of EN 1993 are different – they should be	a) All rules dealing with the design life time should be given only in one place b) At least various parts of EN 1993 should be harmonized between each other.	

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					harmonized if some rules remains in the revised EN 1993 – see comments for various parts of EN 1993 and proposals above c) Also the wording should be same: “The design (service) life” or “The design life”		
Where should the scope of the EN be extended?							
FR30		6.2.2	(5)	Te	In clause 6.2.2(5) and in EN 1993-1-8 generally, it is supposed that the shear key is welded to the baseplate, whereas this design is not commonly used. Generally, the baseplate bears on an embedded plate where a shear key is welded :	Design should be given for this type of baseplate.	C

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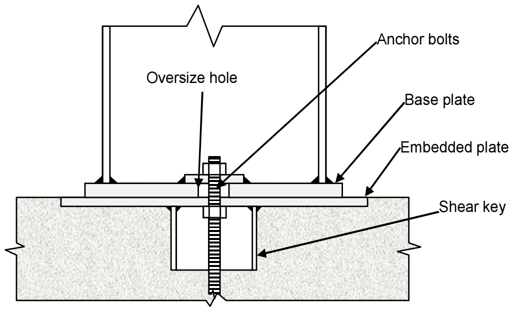
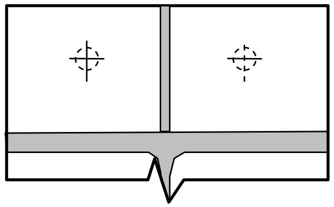
							
FR3 1		6.2.4.1	Table 6.2	Te	This design method can only be used with connections with two bolts per row.	Information should be given in presence of 4 bolts per row. The last research (Demonceau, Latour, Prinz, Kozlowski…) can be used.	B
FR3 2		6.2.6.1	(13)	Te	A limit is given concerning the width b_s. However, a common practice is to use plug weld.	Requirements should be given on the distance between plug welds to exceed this limit.	D
FR3 3		6.2.6.5	Table 6.6	Te	Table 6.6 gives no information about outside bolt row with stiffener whereas it is a common practice. 	Complementary information should be given about outside bolt row with stiffener.	B
FR3 4		6.2.6.12	(6)	Te	EN 1993-1-8 gives no information about tension resistance of anchor bolt provided with washer plate. EN 1992-1-1 gives no more information.	Add a methodology to calculate this type of anchorage.	B
FR3		6.2.7.1	(3)	Te	The method proposed is conservative. Existing	The design method proposed by Sokol et al	D

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5					methods permit to determine the bending resistances such as in the method for baseplates.	(Eurosteel 2002, design of endplate joints subjected to moment and normal force) permit to increase the bending resistance.	
GR2 5				ge	Biaxial bending is not covered neither for beam to column connection nor for column bases.	Provide rules for the design.	C
GR2 6				ge	Bolted flange plate connections between hollow sections are not covered.	Provide rules, similar to beam to column connections, for the design.	C
GR2 7		3.4	Table 3.2	te	The design ultimate resistance of the net cross-section at holes for fasteners is also one of the design criteria of the connections.	A comment should be added in the Table for all types of bolted connections in order to take into account the design ultimate resistance ($N_{u,Rd}$, see §6.2.2.2 and §6.2.2.5, EC3-1.1) of the net cross-section at holes for fasteners.	C
GR2 8		3.4		te		Reference to §6.5.5 of EC8-1 should be made, regarding the design rules for connections in dissipative zones	D
GR2 9		3.10.2		te	The described method for block tearing verification assumes a single force applied either concentrically or eccentrically. In many cases however, a secondary force is present (e.g a tensile force in figure 3.8 top).	Provide suggestions on how the method should be applied when both shear and tensile forces are present.	B
GR3 0		5.3	Par. 4	te	It is currently not possible to implement this suggestion, as the methodology available in chapter 6, hardwires the shear panel stiffness and strength as rotational stiffness and strength.	Provide the required clauses in chapter 6 that elaborate on the calculation of $S_{j,ini}$ and $M_{j,rd}$, excluding the contribution of web panel. Also, provide detailed modeling options about the separate modeling of web panel.	B
GR3 1		6.1.3	Table 6.1	te	Components for bolted joints with 4 bolts per row are not available.	Include an appropriate methodology for bolted joints with 4 bolts per row.	B
GR3 2		6.1.3	Table 6.1	te	Components for the web cleats are not available.	A component web cleat in bending should be included.	D

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GR33		6.1.3	Table 6.1	te	Components for weak-axis beam-to column joints are not available	Components for weak-axis beam-to column joints should be included.	D
GR34		6.2.6.1		ge	For the web-panel shear strength the assumption of similar beam depths is required. No suggestion is given for dissimilar beams.	Provide suggestions for the characterization of the web panel shear strength in case of dissimilar beams.	D
GR35		6.2.7		ge	The calculation of the beam-to-column joint resistance under beam weak-axis bending is not available or commented upon.	Provide suggestions for the characterization of the joint resistance under weak-axis bending and biaxial bending.	C
GR36		6.2.7.1	Par. 2,3	te	The limit of 5% to the axial force is significantly low. The interaction formula leads to the same result for negative and positive values of axial force. Also the axial resistance $N_{j.Rd}$ is not defined.	Provide rules for the calculation of the axial , resistance and expand the existing rules for axial forces bigger than 5% of plastic resistance of the beam.	D
GR37		6.2.8		ge	The verification of column bases under column biaxial bending is not available or commented upon.	Provide suggestions for the characterization of the column base resistance under weak-axis bending and biaxial bending.	C
GR38		6.2.8		ge	It is common for column bases to get stiffeners in positions not covered, and also to add a second row of bolts in the extended zone, parallel to column flanges. For these cases no equivalent T-stub lengths are available in Tables 6.4-6.6.	Provide proper equivalent T-stub lengths for the mentioned cases.	D
GR39		6.2.8.3		te	For T-stubs 1,3 of Figure 6.19 it is possible that some bolts are well outside the beam flange width (towards the edges of the plate). For such a case, no equivalent T-stub length can be obtained from the available Tables 6.4-6.6.	Provide proper equivalent T-stub lengths for bolts, not in the vicinity of a base column flange.	D
GR40		6.2.8.3		te	The use of stiffeners and more than two anchor columns at base plate connections is common. The use of tables 6.4, 6.5 and 6.6 for the determination of effective lengths of the T-stubs is limiting the design options.	Provide tables for the determination of effective lengths of the T-stubs specially for base plates.	D

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GR4 1		6.4.1	Par. 2	te	The use of high strength steel is often.	Provide rules which are covering high strength steel.	C
PL1 3		6.2	**6.2.6.11(2)**	te	prying effect in relatively thin base plates usually are significant and should not be omitted	Detale item „(2)" and corrigendum	C
FI52				General	In these Finnish comments line number has not been given mainly due to the following reasons: -CEN has not defined how the line number should be calculated ***from the beginning or from the end of the standard ***form the top or the bottom of the page ***from the beginning of section, clause or subclause -We assume that people giving answers to these comments are clever enough to understand if the reference is made for example to clause 1.2.3.4(5)		
FI53				General	General Finnish comments to **all Parts** of EN 1993.		
CZ4		3.1.1	Table 3.1	ge	Bolts 12.9 are missing	Extend scope of EN by bolts 12.9	D
CZ5		5.1 (6.3)		ge	Approximate values of rotational rigidity $S_{j,ini}$ which can be used for global analysis are missing	Extend scope of EN by these values	D
CZ6		6.2.4		ge	Method for setting of equivalent T-stub in tension for 4 bolts in row is missing	Extend scope of EN by this method	D
CZ7		6.2.6.12	(3)	ge	Link to EN 1992 is given for values of cohesiveness in concrete	Extend this paragraph by cohesiveness in concrete not to need to study EN 1992	B
CZ8		6.2.6		ge	Component anchoring bolt in shear is missing	Extend this paragraph	D

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Template for comments and secretariat observations

Date: 2014-10-22	Document:	Project: EN 1993-1-8

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision
CZ9		6.2.8		ge	Anchor plate of CHS, RHS with bolts in corners (not aligned by flanges and stiffeners). Setting of l_{eff} of T-stub. Failure mode determination.	Extend this paragraph	D

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MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision
Where could the EN by shortened?							
FR3 6		3.6.2		Ed	Injection bolts are not used in practice. Paragraph 3.6.2 is not important.	Delete paragraph 3.6.2. Reference could be made to ECCS requirement n°79.	D
FR3 7		6.2.4.1	Table 6.2	Te	Method 2 (alternative method) doesn't really increase the resistance.	Delete Method 2.	B
GB8 7					Bearing and buckling checks on a fabricated box section. This is not specifically covered in part 1 and 8. Bearing and buckling is hidden away in part 5, but should be referenced in either 1 or 8. According to P363 section 9.2 (page A31) box sections are not covered. P363 uses P202 method for rolled section but no guidance given for fabricated section and working through formulae given falls under the 'excessive design effort' heading. The formulae in part 6 of 1993-1-5 also require excessive design effort for a simple beam compared to BS5950.		C
GB8 8		5.2.2.1 (1)	Page 54	Te	Classification by stiffness. Rules are given for joints connecting H or I sections but 'note' says *'…joints connecting hollow sections are not given in this standard.'*	Provide advice on stiffness classification for hollow section joints, especially T and X joints with moments.	B
GB8 9			Page 123 Table 7.14	Te	Moment resistance is only valid for T and X joints. There is no advice for K or N joints.	Provide advice for moments out of plane for K and N joints.	C
GB9 0			Page 124 Table 7.15	Te	Row 3, KT joint. This method is only applicable for gap joints. Majority of KT joints are overlap joints but no information is given.	Suggest adding extra case for overlap KT joints. *Where brace 3 is the overlapped brace, check brace 1 and 3 as an overlap K-joint, then check brace 2 and 3 as an overlap K-joint.* *Overlapped brace 3 joint resistance based on the*	B

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						efficiency ratio of the overlapping bracing to the overlapped bracing, i.e.: $N_{i,Rd} = N_{i,Rd} \frac{A_j f_{yj}}{\text{[illegible]}}$ *Where $N_{i,Rd}$ is the lowest joint resistance of overlapping bracing 1 or 2.*	
DE2 5				general	First chapters in the beginning which are repeated in every Eurocode part should be presented only once in EN 1990.	Consider to remove chapters “Background to the Eurocode programme”, “Status and field of application of Eurocodes”, “National standards implementing Eurocodes” and “Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products”, which should be presented only in EN 1990.	D
DE2 6		3.6.2		technical	*“Injection bolts”* are special-purpose building elements and are not used regularly.	Consider to move into Annex.	D
DE2 7		**5.**		technical	Clauses for *“semi-continuous joints”* are special rules however not suitable for ordinary analysis of steel constructions.	For condensed basic rules we would recommend to move chapter 5 into Annex. Bases and assumption for global analysis we recommend to move into chapter 2 (see above).	D
DE2 8		**6.**		technical	Rules for *“Structural joints connecting H or I sections”* are special self-contained rules.	For condensed basic rules we would recommend to move chapter 6 into Annex.	B
DE2 9		**7.**		technical	Rules for *“Hollow section joints”* are special self-contained rules.	For condensed basic rules we would recommend to move chapter 7 into Annex.	B
DS/ DK5		6		ed	Many of the design rules in this chapter are more text book like. They could be deleted, as the		D

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					general rules for connections using bolts and welding are sufficient to design joints.		
DS/ DK6		7		ed	These paragraphs could be deleted and reference made to the literature from CTICM.		B
GR4 2		4.5.3.3		te	A simplified method for design resistance of fillet weld is provided, alternatively to the one presented in §4.5.3.2.	If alternative methods are provided they should be accompanied by clear criteria for selecting one method over the other.	C
FI54				General	In these Finnish comments line number has not been given mainly due to the following reasons: -CEN has not defined how the line number should be calculated ***from the beginning or from the end of the standard ***form the top or the bottom of the page ***from the beginning of section, clause or subclause -We assume that people giving answers to these comments are clever enough to understand if the reference is made for example to clause 1.2.3.4(5)		
FI55				General	General Finnish comments to **all Parts** of EN 1993.		
FI56		Foreword and first general pages		ed	a) Background to the Eurocode programme b) Status and field of application of Eurocodes c) National Standards implementing	a) Delete. It is enough that this kind of information is given only ones in EN 1990. b) Delete. It is enough that this kind of information is given only ones in EN 1990.	D

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					Eurocodes d) Links between Eurocodes and harmonized technical specifications (ENs and ETAs) for products	c) Delete. It is enough that this kind of information is given only ones in EN 1990. d) Delete. It is enough that this kind of information is given only ones in EN 1990.	
FI57		General		te/ed	Design based on FE-methods: a) Annex C of EN 1993-1-5 contains some detailed rules, which is a good start for steel structures b) Also some other parts of EN 1993 contains some rules for FE-methods c) EN 1990 should contain basic principles and basic rules for the FE-based design d) All detailed rules for FE-based design of steel structures should be collected into one place, preferable as annex into EN 1993-1-1	All detailed rules for FE-based design of steel structures should be collected into one place, preferable as annex into EN 1993-1-1.	B
FI58		General		te/ed	Design based on testing: a) At present various rules for the design based on testing are given in various places at least as follows:	a) Annex A of EN 1993-1-3 and Annex D of EN 1990 should be checked so that they are not in contradiction b) All details for design based on testing	B

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					* Annex D of EN 1990 ***Specific comments to Annex D of EN 1990 1) Rules and process given in Annex D of EN 1990 do not take into account different safety levels between brittle and tough failure modes. Compare this differentiation in some parts of EN 1993, where γ_{M0} =1,0 and γ_{M2} = 1,25 are given. 2) Basic principles or rules should be given in Annex D of EN 1990 for brittle/tough failure modes. * EN 1993-1-3, EN 1993-3-2, EN 1993-5 *** Some basic rules in Annex D of EN 1990 and in Annex A of EN 1993-1-3 are in contradiction: the basic question is: which document should be followed in practice. ***If there are in additions some rules in ETAG guidelines, which are different, then the question is: Which rules should be followed? *** Annex A of EN 1993-1-3 and Annex A of EN 1993-5 contain much	should be collected only into one document to be included into EN 1990 c) If proposal b) above is not acceptable then at least all rules concerning the design based on testing of steel structures should be collected into one document – preferable as annex into EN 1993-1-1. d) Basic rules for FE-based design should be given in EN 1990 e) Detailed rules for FE-based design of steel structures should be given in one place as annex to EN 1993-1-1.	

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					repetition , which should not be allowed in the revised Eurocode-system		
SE9				ed	Foreword and first general pages		
Are there any clauses whose application leads to uneconomic construction?							
FR3 8		3.6.1	Table 3.4	Te	Due to the reduction factor α_d for edge bolts, the bearing resistance is very conservative comparatively to the French practice.		D
FR3 9		3.13.2	(2)	Te	The distribution of contact pressure, between the pin and connected parts, proposed have no sense when the bearing resistance is not critical. In these case, the contact pressure should applied at the edge of the connected parts, thus the bending moment is reduced.	The SCI proposition (see AD366 of NSC april 2012) tends in this direction and propose the following distribution :	D

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						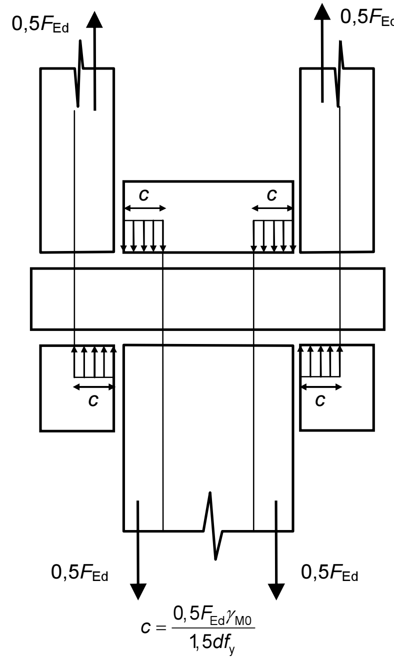	
FR40		6.2.2	(6)	Te	The friction coefficient is clearly lower than that used in France in common practice.	Replace by : For sand-mortar : $C_{f,d}$ = 0,30	D
GB91	4.10			te	The requirements in this clause assume that the plate is fully yielded and independent of the force applied. The unstiffened flange will not fail if the applied force (not the plate capacity) is less than the bending resistance of the outstand of the flange. As written the flange requires stiffening in many, many more cases than would be the case if the design were based on the applied loading. The plate should be design for the applied force not its plastic capacity.	Change the clause to allow the plate to be design for the applied force.	B

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GB92	6.2.7.1	(13)		te	For column splices, not prepared for full contact in bearing, this clause requires the splice to be design for a moment equal to not less than 25% of the weaker section about both axes. This limit is very onerous and results in a significant increase in the number and size of the bolts compared to current practice. Rather than introducing an arbitrary limit it would be better to say the 'splice should provide continuity of stiffness'. Additional rules are required for cap and base-plate connections.		Frans/B
GB93	6.2.7.1			Tech	This clause gives recommendations for the design of beam splices and requires the beam web to take its share of the moment. Current practice is to take the moment on the flanges and design the web for shear only. This clause should be changed to allow this approach.		D
FI59				General	In these Finnish comments line number has not been given mainly due to the following reasons: -CEN has not defined how the line number should be calculated ***from the beginning or from the end of the standard ***form the top or the bottom of the page ***from the beginning of section, clause or subclause -We assume that people giving answers to these comments are clever enough to understand if the reference is made for example to clause 1.2.3.4(5)		

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FI60				General	General Finnish comments to **all Parts** of EN 1993.		
FI61				General	See Finnish technical/editorial comments to each part of EN 1993 The question of economy/un-economy is not correct or is misleading. The real question is: Are the rules correct or not independent if they lead to economic or un-economic structures.		
Are there any clauses whose application necessitates excessive design effort?							
FR41		5.2.2.5	(2)	Te	The check of stiffness of baseplate of equation (5.2 d) is hard and needs important reinforcement in comparison to common practice. In the meantime, for baseplate the stiffness depends on the loading (e_N for example), the classification can change in function of the loading and the evolution of the project. Thus the check of a baseplate is more complex. Finally, there is no criterion to prove that a baseplate is pinned in elastic analysis.		D
DE30		**5.**		technical	Rules for *"semi-continuous joints"* necessitates an excessive design effort.	For condensed basic rules we would recommend to move chapter 5 into Annex.	C
FI62				General	In these Finnish comments line number		

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MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision
					has not been given mainly due to the following reasons: -CEN has not defined how the line number should be calculated ***from the beginning or from the end of the standard ***form the top or the bottom of the page ***from the beginning of section, clause or subclause -We assume that people giving answers to these comments are clever enough to understand if the reference is made for example to clause 1.2.3.4(5)		
FI63				General	General Finnish comments to **all Parts** of EN 1993.		
FI64				General	See Finnish technical/editorial comments to each part of EN 1993 The question is not correct or is misleading. The real question is: Are the rules correct or not independent if they lead to excessive design effort or not.		

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POLISH PROPOSALS FOR CORRECTIONS OF EN 1993-1-8

A. PROPOSAL A Correction of Table 6.2

Table 6.2 correction

	Prying forces may develop if: $t_f < 1{,}5d\sqrt{\dfrac{m\, f_{u,b}}{l_{eff}\, f_y}}$ [3)]		No prying forces
Mode 1	Method 1	Method 2 (alternative method)	$F_{T,1\text{-}2,Rd} = \dfrac{2M_{pl,1,Rd}}{m}$
Without backing plates	$F_{T,1,Rd} = \dfrac{4M_{pl,1,Rd}}{m}$	$F_{T,1,Rd} = \dfrac{(8n-2e_w)M_{pl,1,Rd}}{2mn-e_w(m+n)}$	
With backing plates	$F_{T,1,Rd} = \dfrac{4M_{pl,1,Rd}+2M_{bp,Rd}}{m}$	$F_{T,1,Rd} = \dfrac{(8n-2e_w)M_{pl,1,Rd}+4nM_{bp,Rd}}{2mn-e_w(m+n)}$	
Mode 2	$F_{T,2,Rd} = \dfrac{2M_{pl,2,Rd}+n\Sigma F_{t,Rd}}{m+n}$		
Mode 3	$F_{T,3,Rd} = \Sigma F_{t,Rd}$		

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

Mode 1: Complete yielding of the flange
Mode 2: Bolt failure with yielding of the flange
Mode 3: Bolt failure
L_b is - the bolt elongation length, taken equal to the grip length (total thickness of material and washers), plus half the sum of the height of the bolt head and the height of the nut or
- the anchor bolt elongation length, taken equal to the sum of 8 times the nominal bolt diameter, the grout layer, the plate thickness, the washer and half the height of the nut

$L_b{}^* = \frac{8{,}8 m^3 A_s}{\Sigma l_{eff,1} t_f^3}$ in red transmit to 6.2.6.12

$F_{T,Rd}$ is the design tension resistance of a T-stub flange
Q is the prying force in T-stub row (for one direct bending) given by $\Sigma Q = \Sigma F_{t,Rd} - F_{T,Rd}$
$M_{pl,1,Rd} = 0{,}25\ \Sigma\ l_{eff,1} t_f^2\ f_y / \gamma_{M0}$
$M_{pl,2,Rd} = 0{,}25\ \Sigma\ l_{eff,2} t_f^2\ f_y / \gamma_{M0}$
$M_{bp,Rd} = 0{,}25\ \Sigma\ l_{eff,1} t_{bp}^2\ f_{y,bp} / \gamma_{M0}$
$n = e_{min}$ but $n \leq 1{,}25m$
$F_{t,Rd}$ is the design tension resistance of a bolt, see Table 3.4;
$\Sigma F_{t,Rd}$ is the total value of $F_{t,Rd}$ for all the bolts in the T-stub;
$\Sigma \ell_{eff,1}$ is the value of $\Sigma \ell_{eff}$ for mode 1;
$\Sigma \ell_{eff,2}$ is the value of $\Sigma \ell_{eff}$ for mode 2;
e_{min} , m and tf are as indicated in Figure 6.2.
$f_{y,bp}$ is the yield strength of the backing plates;
t_{bp} is the thickness of the backing plates;
$e_w = d_w / 4$;
d_w is the diameter of the washer, or the width across points of the bolt head or nut, as relevant.
NOTE 1: In bolted beam-to-column joints or beam splices it may be assumed that prying forces will develop.
NOTE 2: In method 2, the force applied to the T-stub flange by a bolt is assumed to be uniformly distributed under the washer, the bolt head or the nut, as appropriate, see figure, instead of concentrated at the centre-line of the bolt. This assumption leads to a higher value for mode 1, but
leaves the values for $F_{T,1-2,Rd}$ and modes 2 and 3 unchanged.
NOTE 3: Criterion for anchor bolts of one direct bending base plates see 6.2.6.12

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B. **PROPOSAL B:** Correction of Table 6.7

Information in this table and referred drawings (fig.) are not consistent. Following changes should be considered:

- Boundary values should be weak (with „ < ” not “ ≤ “) to avoid “zero” in denominator of formulas and ∞ when e = 0,
- signs of calculated values of moment resistance are in some cases negative, what is not consistent with fig. 6.18 and formula in last row in the table.

Below is proposal to changes in Table 6.7

Tablica 6.7: Design moment resistance of column bases $M_{j,Rd}$ correction

Loading	*Lever arm z*	Design moment resistance $M_{j,Rd}$	
Left side In tension Right side In compression (fig 6.18(d))	$z = z_{T,l} + z_{C,r}$	$N_{Ed} > 0$ and $e > z_{T,l}$	$N_{Ed} < 0$ and $e < -z_{C,r}$
		The smaller of: $\frac{F_{T,1,Rd}\, z}{z_{C,r}/e+1}$ i $\frac{-F_{C,r,Rd}\, z}{z_{T,1}/e-1}$	

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<table>
<tr><td rowspan="2">Left side In tension
Right side In tension
(Fig. 6.18(b))</td><td rowspan="2">$z = z_{T,l} + z_{T,r}$</td><td>$N_{Ed} > 0$ and
$0 < e < z_{T,l}$</td><td>$N_{Ed} > 0$ and
$-z_{T,r} < e \le 0$</td></tr>
<tr><td>The smaller of:
$\frac{F_{T,l,Rd}\ z}{z_{T,r}\ /e+1}$ $\frac{F_{T,r,Rd}\ z}{z_{T,l}\ /e-1}$</td><td>The smaller of:
$\frac{-F_{T,l,Rd}\ z}{z_{T,r}\ /e+1}$ $\frac{F_{T,l,Rd}\ z}{z_{T,l}\ /e-1}$</td></tr>
<tr><td rowspan="2">left side In compression
Right side In tension
(Fig. 6.18(c))</td><td rowspan="2">$z = z_{C,l} + z_{T,r}$</td><td>$N_{Ed} > 0$ and $e \le -z_{T,r}$</td><td>$N_{Ed} \le 0$ and $e > z_{C,l}$</td></tr>
<tr><td colspan="2">The smaller of:
$\frac{-F_{C,l,Rd}\ z}{z_{T,r}\ /e+1}$ $\frac{-F_{T,r,Rd}\ z}{z_{C,l}\ /e-1}$</td></tr>
<tr><td rowspan="2">left side In compression
Right side In compression
(Fig. 6.18(a))</td><td rowspan="2">$z = z_{C,l} + z_{C,r}$</td><td>$N_{Ed} \le 0$ and
$0 < e < z_{C,l}$</td><td>$N_{Ed} \le 0$ and
$-z_{C,r} < e \le 0$</td></tr>
<tr><td>The smaller of:
$\frac{-F_{C,l,Rd}\ z}{z_{C,r}\ /e+1}$ i $\frac{F_{C,r,Rd}\ z}{z_{C,l}\ /e-1}$</td><td>The smaller of:
$\frac{-F_{C,l,Rd}\ z}{z_{C,r}\ /e+1}$ i $\frac{-F_{C,r,Rd}\ z}{z_{C,l}\ /e-1}$</td></tr>
<tr><td colspan="4">M_{Ed} >is clockwise, $N_{Ed} > 0$ is tension
$e = \frac{M_{Ed}}{N_{Ed}} = \frac{M_{Rd}}{N_{Rd}}$ In red to cancel</td></tr>
</table>

C. **PROPOSAL C** Modification 6.2.7.2. (9) (see attached **Background**)

Replace equation (6.26) as follows:

$$F_{tr,Rd} \le \frac{F_{t(x+1),Rd}\ h_r}{h_x} \qquad (6.26),$$

where $F_{t(x+1),Rd}$ – effective design tension resistance of next inner bolt row (nearer to the compression centre than row (x)).

h_r is the distance from bolt-row (r) to the centre of compression.

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h_x is the distance from bolt-row (x) to the centre of compression

Add followed items:

a) For full strength connections of built-up beams with the cross section of Class 3, the sum of effective design tension resistance of bolt-rows adjacent to tension flange should be bigger than the design resistance of tension beam flange, and for partial strength joints - not lower than the design force in tension flange. Exceptions from this rule are possible provided that design is verified by testing.

b) Design bending resistance of built-up beams with the cross section of Class 4 should be determined assuming the effective design tension resistance of bolt-rows adjacent to tension flange only.

Backgrounds to the Polish proposal for rev EN 1993-1-8

Back. 1

Formula in the first row in table 6.2, for boundary when prying forces are to be taken into account:

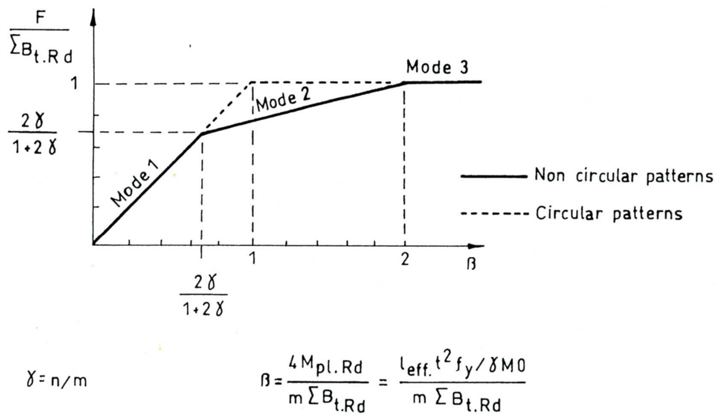
$$L_b \leq \frac{8{,}8 m^3 A_s}{\Sigma \ell_{eff,1} t_f^{\,3}}$$

is for column bases!! and not for beam-to-column connections !

For b-t-c connections should be:

Prying forces are active in Mode 1 and 2, what means when β<2

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision



(b) Design resistance

$$\beta = \frac{4M_{pl,Rd}}{m\Sigma F_{t,Rd}} = \frac{l_{eff} t^2 f_y}{m\Sigma F_{t,Rd}} = 2$$

$$t \le \sqrt{\frac{2m\Sigma F_{t,Rd}}{l_{eff}\, f_y}} = 2\sqrt{\frac{mF_{t,Rd}}{l_{eff}\, f_y}}$$ substituting $F_{t,Rd} = 0{,}72\frac{\pi d^2}{4} f_{u,b} = 0{,}565 d^2 f_{u,b}$

Boundary for prying forces in two-directorial bending is $t \le 1{,}5d\sqrt{\frac{mf_{u,b}}{l_{eff} f_y}}$

(when one-directional bending prying is always active).

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

Background 2 for rev EN 1993-1-8/6.2.7.2 (9)

1. Results of experimental tests

Following experimental test results show that real forces in bolt rows can exceed resign resistance of the bolt, what can be dangerous.

1.1 Sumner E. Unified Design of Extended End-Plate Moment Connections Subject to Cyclic loading. Dissertation submitted to the Faculty of the Virginia Polytechnic Institute, 2003.

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

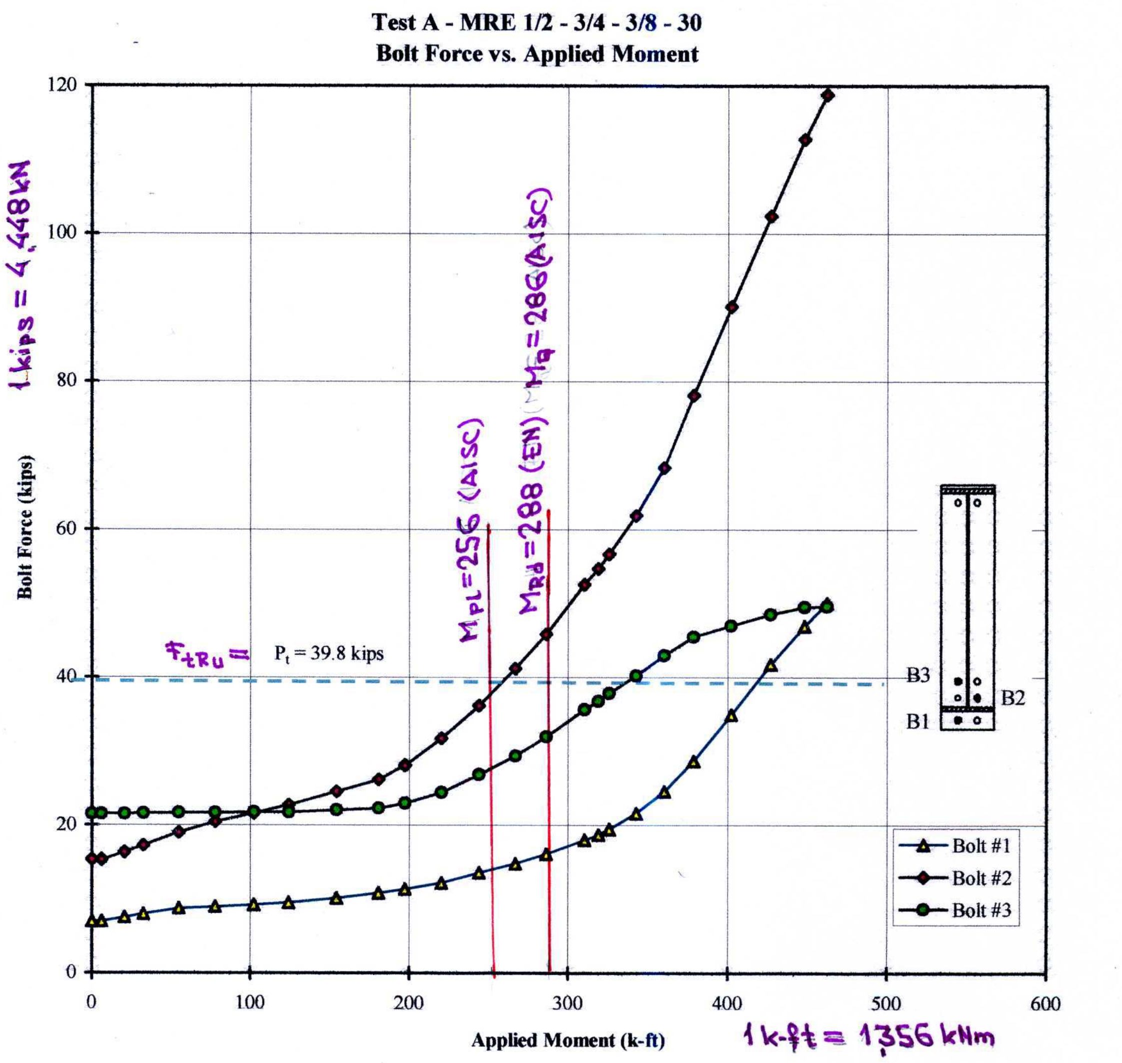


FIGURE 3.22: TYPICAL BOLT FORCE VS. APPLIED MOMENT RESPONSE FOR A THIN PLATE SPECIMEN (TEST A-MRE 1/2-3/4-3/8-30)

Beam splices data: End plates t_p=1/2"

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

f_y=62,0ksi=427MPa Bolts ¾" A325

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MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

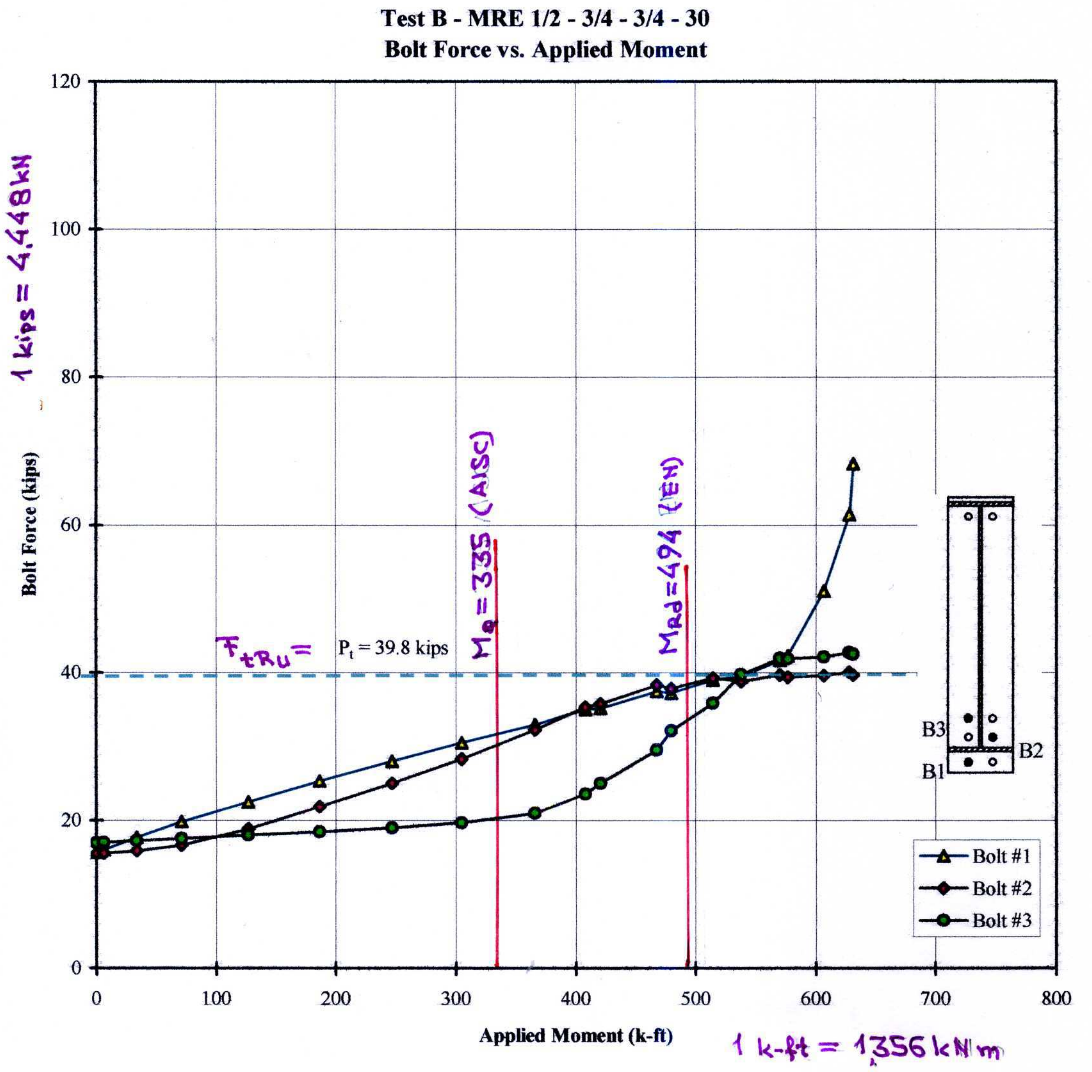

FIGURE 3.26: TYPICAL BOLT FORCE VS. APPLIED MOMENT RESPONSE FOR A THICK PLATE SPECIMEN (TEST B-MRE 1/2-3/4-3/4-30)

nments from the ISO/CS editing unit are identified by **)

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

Beam splices data: End plates t_p=1/2” f_y=62,3ksi=430MPa Bolts ¾” A325

1.2 Katula L. Bolted end-plate joints for crane brackets and beam-to-beam connections PhD Dissertation. Budapest. 2007.

Beam splices data:

End plates TB5 t_p=12mm; TB9 t_p=15mm; TB13 t_p=20mm; TC t_p=20mm; Steel S355

Bolts M20 Class 8.8 (for recalculated bending capacity γ_{M2}=1,00)

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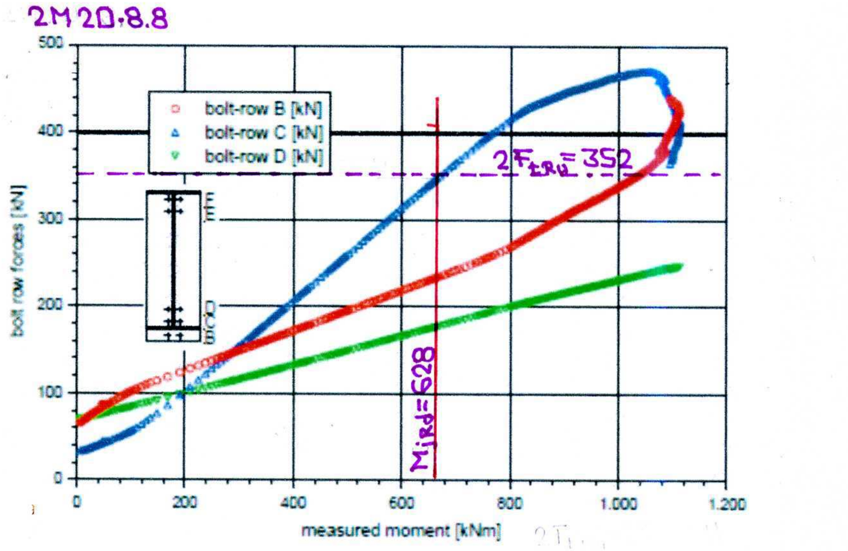

Fig. 3.64 Moment vs. bolt-row force diagrams, test TB5. $t_p=12\,mm$

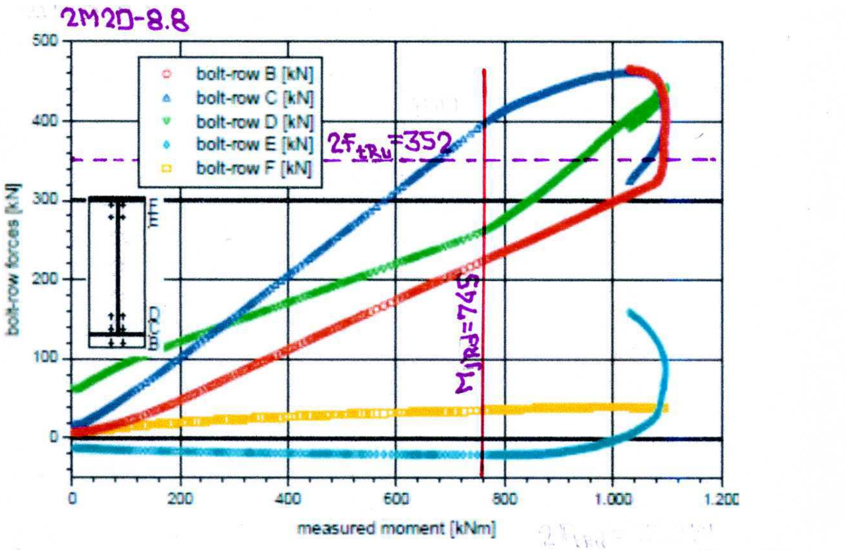

Fig. 3.67 Moment vs. bolt-row force diagrams, test TB9. $t_p=15\,mm$

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

2M20-8.8

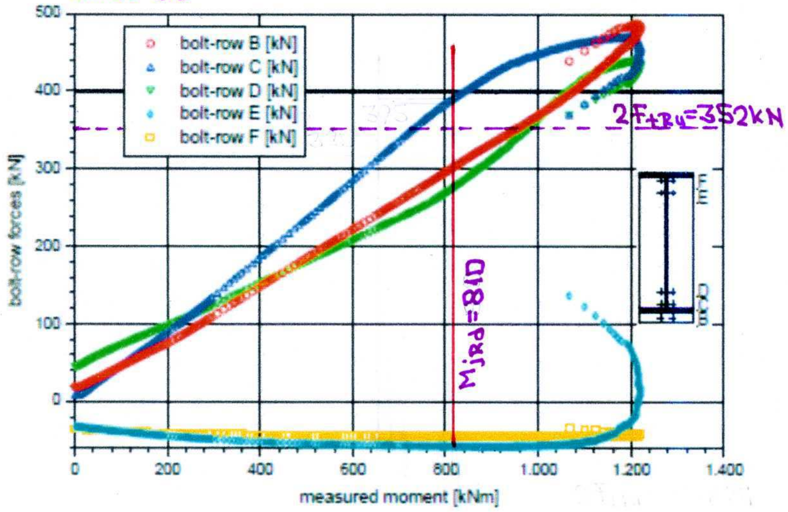

Fig. 3.71 Moment vs. bolt-row force diagrams, test TB13. t_p = 20mm

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

Table 3.3 Summary of test results.

end-plate type	speci men	achieved load level [kN] ([kNm])	M_{Ed} / $M_{c,Rd}$ [%]	bolt-row that failed first	measured maximum bolt force F_i^{max} [kN]	measured maximum bolt force ΔF_i^{max} [kN]	calculated design moment resistance $M_{j,Rd}$ [kNm] ([%])	re-calculated design moment resistance $M^*_{j,Rd}$ [kNm] ([%])
I	TB2	503 (1,509)	60.2	bolt-row C	263	253	929 (61.6)	1,108 (73.4)
	TB6	545 (1,635)	65.3	bolt-row C	241	236	1,044 (63.9)	1,248 (76.3)
	TB10	536 (1,608)	64.2	bolt-row C	239	214	1,062 (66.0)	1,397 (86.9)
II	TB3	504 (1,512)	60.4	bolt-row C	242	223	1,121 (-)	1,298 (-)
	TB7	550 (1,650)	65.9	bolt-row C	256	233	1,453 (-)	1,675 (-)
	TB11	567 (1,701)	67.9	bolt-row C	227	208	1,469 (-)	1,869 (-)
III	TB4	495 (1,485)	59.3	bolt-row C	244	226	1,022 (68.8)	1,137 (76.6)
	TB8	623 (1,869)	74.6	bolt-row C	230	226	1,225 (65.5)	1,462 (78.2)
	TB12	538 (1,614)*	64.4	bolt-row C	235	230	1,341 (83.1)	1,649 (102.2)
IV	TB5	368 (1,104)	44.1	bolt-row C	239	226	628 (56.9)	752 (68.1)
	TB9	365 (1,095)	43.7	bolt-row C	234	226	745 (68.0)	888 (81.1)
	TB13	407 (1,221)	48.7	bolt-row C	244	239	810 (66.3)	983 (80.5)

* Premature failure in the threads of the bolts.

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Table 3.4 Comparison of the re-calculated and measured bolt forces.

	test TB5 [t_{ep} = 12 mm]		
	re-calculated bolt-row forces $M^*_{j,Rd}$ = 752 kNm	measured bolt-row forces for M = 752 kNm	measured bolt-row forces for M* = 1,060 kNm
bolt-row B	210.5 kN	238.0 kN	309.5 kN
bolt-row C	305.5 kN	394.3 kN	451.7 kN
bolt-row D	326.4 kN	192.3 kN	226.3 kN
∑ bolt-row forces	842 kN	823 kN	988 kN
	test TB9 [t_{ep} = 15 mm]		
	re-calculated bolt-row forces $M^*_{j,Rd}$ = 888 kNm	measured bolt-row forces for M = 888 kNm	measured bolt-row forces for M* = 1,040 kNm
bolt-row B	301.5 kN	262.8 kN	310.6 kN
bolt-row C	330.2 kN	439.9 kN	461.7 kN
bolt-row D	360.3 kN	320.2 kN	409.1 kN
∑ bolt-row forces	992 kN	1,023 kN	1,181 kN
	test TB13 [t_{ep} = 20 mm]		
	re-calculated bolt-row forces $M^*_{j,Rd}$ = 983 kNm	measured bolt-row forces for M = 983 kNm	measured bolt-row forces for M* = 1,190 kNm
bolt-row B	350.2 kN	362.4 kN	458.4 kN
bolt-row C	381.5 kN	444.4 kN	471.0 kN
bolt-row D	430.7 kN	354.2 kN	437.4 kN
∑ bolt-row forces	1,162 kN	1,161 kN	1,367 kN

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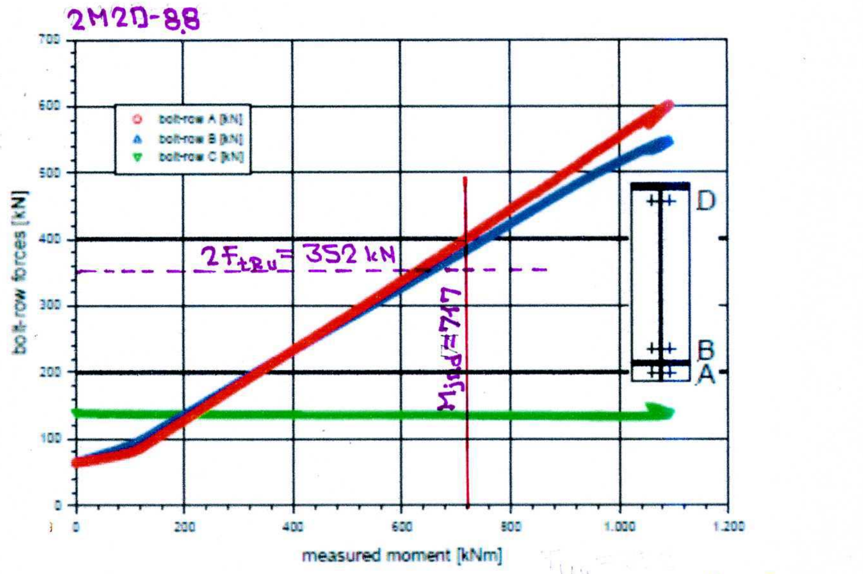

Fig. 3.129 Moment vs. bolt-row force diagrams, test TC. $t_p = 20$ mm

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MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

Table 3.11 Summary of test results.

test	achieved load level [kN] ([kNm])	M_{Ed} / $M_{c,Rd}$ [%]	bolt-row that failed first	measured maximum bolt force F_i^{max} [kN]	measured maximum bolt force ΔF_i^{max} [kN]	calculated design moment resistance $M_{j,Rd}$ [kNm] ([%])	re-calculated design moment resistance $M^*_{j,Rd}$ [kNm] ([%])
TA	291 (873)	40.8	bolt-row B	277	244	560 (64.1)	655 (75.0)
TB	300 (900)	42.1	bolt-row B	291	282	649 (72.1)	757 (84.1)
TC	364 (1,092)	51.1	bolt-row A	314	274	717 (65.7)	913 (83.6)
TD	462 (1,385)	64.7	bolt-row A	295	271	902 (65.1)	1,053 (76.0)
TE	626 (1,878)	65.4	bolt-row A	306	254	1,393 (74.2)	1,679 (89.4)
TF	634 (1,902)	66.2	bolt-row A	305	258	1,490 (78.3)	1,759 (92.5)

Table 3.13 Comparison of the re-calculated and measured bolt forces.

	test TC		
	re-calculated bolt-row forces $M^*_{j,Rd}$ = 913 kNm	measured bolt-row forces for M = 913 kNm	measured bolt-row forces for M* = 1,080 kNm
bolt-row A	504.0 kN	506.3 kN	594.2 kN
bolt-row B	544.2 kN	480.2 kN	545.8 kN
Σ bolt-row forces	1,048 kN	987 kN	1,140 kN

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MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

1.3 Steurer A. Das Tragverhalten und Rotationsvermögen geschraubter Stirnplattenverbindungen. Institut für Baustatik und Konstruktion Eidgenössiche Technische Hochschule Zürich. 1999.

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

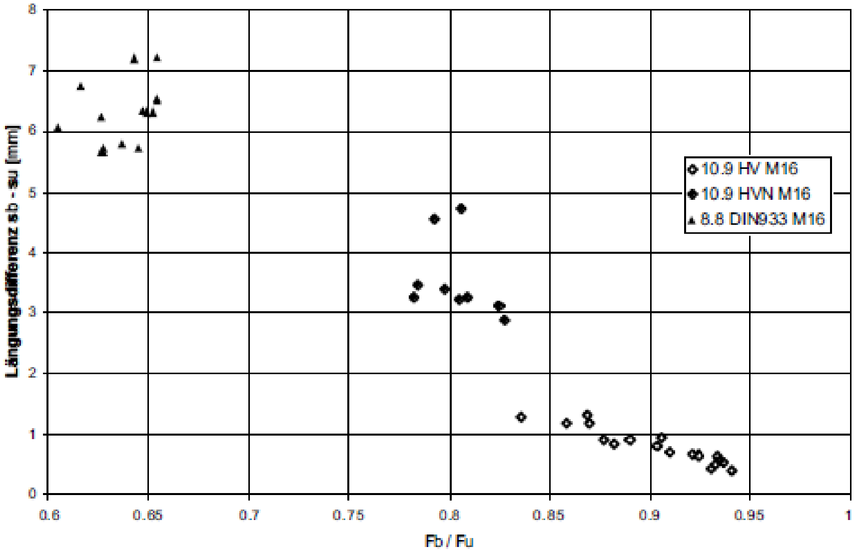


Abb. 3.77: Absolute Grösse der "überplastischen" Reserve s_b-s_u bei der Schraube M16 mit und ohne Schaft bei den Festigkeitsklassen 10.9 und 8.8 in Abhängigkeit des Verhältnisses der Bruchlast F_b zum Tragwiderstand F_u

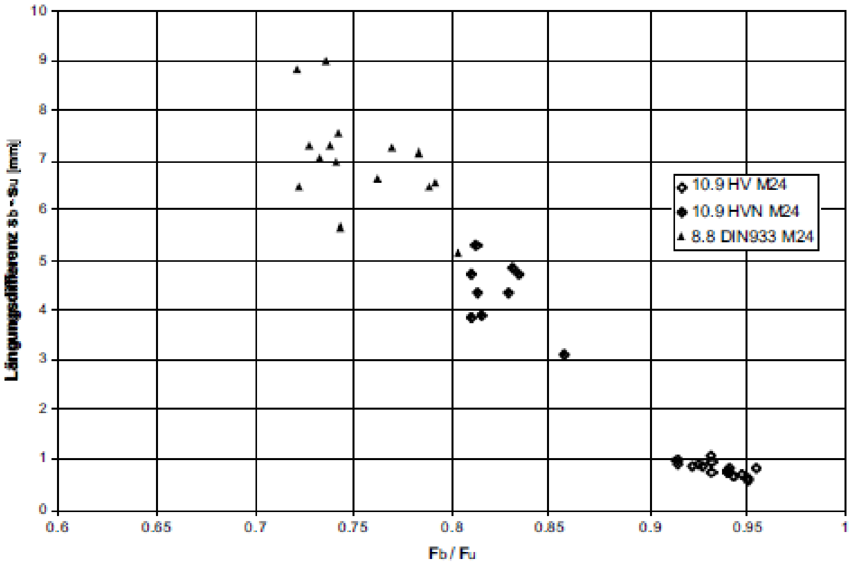


Abb. 3.78: Absolute Grösse der "überplastischen" Reserve s_b-s_u bei der Schraube M24 mit und ohne Schaft bei den Festigkeitsklassen 10.9 und 8.8 in Abhängigkeit des Verhältnisses der Bruch-

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

Festigkeits-klasse	Schraubenform	Schraubentyp	Festwert "Überplastischer" Verformungsanteil
10.9	Schraube **mit** Schaft	DIN 6914 (HV-Schraube DIN 931	1 mm (2 mm)[a]
	Schraube **ohne** Schaft	"HVN" (DIN 6914) DIN 933	3 mm
8.8	Schraube **mit** Schaft	DIN 931	2 mm (3.5 mm)[a]
	Schraube **ohne** Schaft	DIN 933	5 mm

[a] Falls freie Gewindelänge ≥ 1.0·d (d: Schraubendurchmesser)

Tab. 3.11. Festwerte für die näherungsweise Erfassung der "überplastischen" Verformungs-anteile im Fall der verformungsgesteuerten Beanspruchung

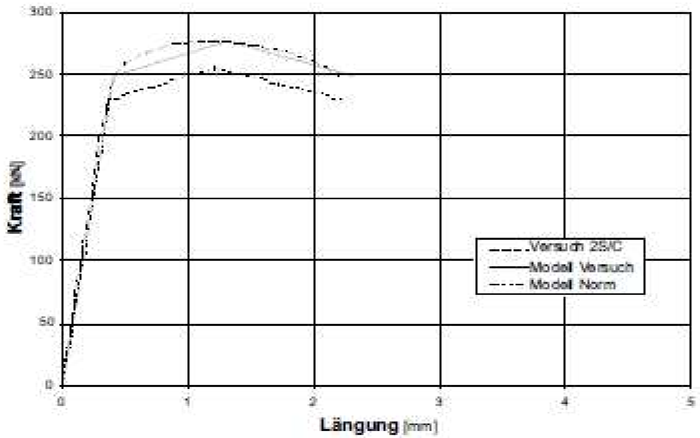


Abb. 3.79: Last-Verformungskurve der im Versuch geprüften HV-Schraube DIN 6914 M20x110 der Festigkeitsklasse 10.9 mit einer Klemmdicke von 81 mm im Vergleich zum bilinearen Verlauf im vorliegenden ergänzt durch den "überplastischen" Bereich auf dem "Niveau Versuch" bzw. auf dem "Niveau Norm"

Anhand der bereits im vorangehenden Abschnitt verwendeten Beispiele der herkömmlichen HV-Schraube der Festigkeitsklasse 10.9 und der verformungsgünstigen Schraube ohne Schaft (DIN 933) der Festigkeitsklasse 8.8 wird die Anwendung der Erweiterung mit dem "überplasti-

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

2. FEA analisis of multiple bolt rows end-plate moment splices

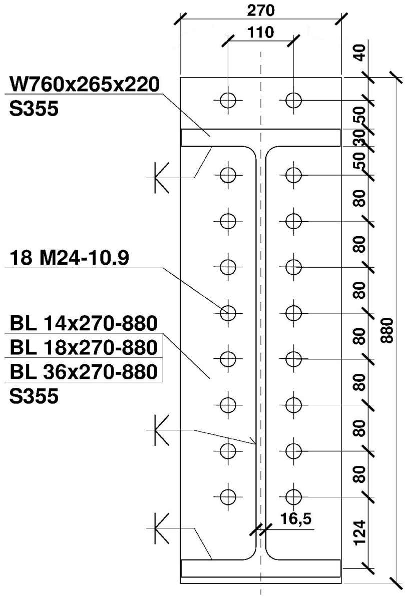
Pawel Kawecki "Moment resistance of multi-row bolted composed beam splices" PhD Rzeszow University of Technology, Poland (in progress).

2.1 FEA model characteristic

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

Investigated extended and-plate splice of W760x265x220 beam have end-plate thickness t_p=14; 18; 24 and 36mm and one row extended and 8 or 3 internal bolt rows between beam flanges. Steel S355 and bolts M24-10.9 class are assumed. Design and ultimate moment resistances of the splice are calculated according to EN 1993-1-8/6.2.7.2(9). The same limit states of design moment resistance and ultimate moment (for plastic end-plate resistance or ultimate bolt resistance) are analyzed in 9 FEA models not prestressed connections in ABAQUS program. As the results bolt forces distribution in all rows at the design moment resistances of the connections are presented.

Beam cross section characteristic W_y=7143cm^3 M_{Rd}=7143x0,355=2535 kNm



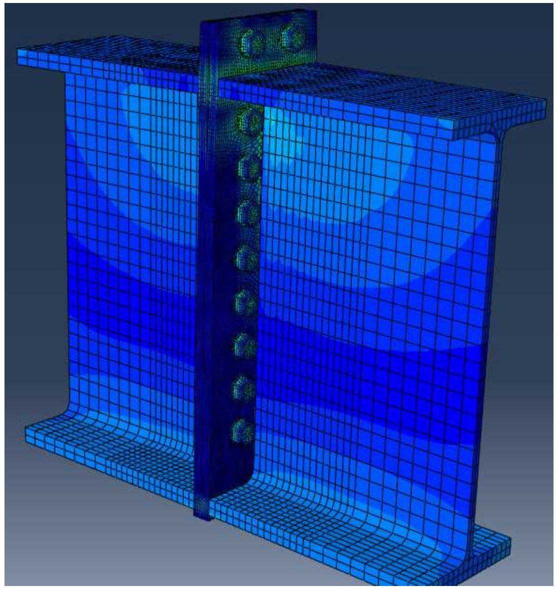


Fig.2.1. Schema and MES model of investigated beam splice (it is partial strength bolted connection of beam loaded in elastic range)

2.2 Validation of FEA modelling

The adequacy of FEA model was checked on Sumners test B-MRE 1/2-3/4-3/4-30.

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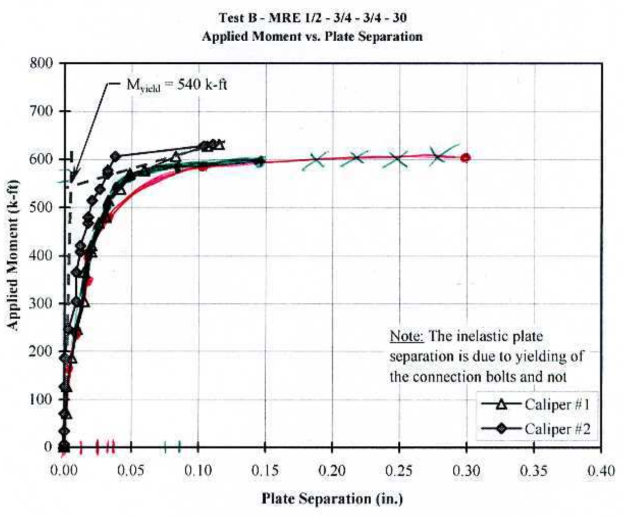

FIGURE 3.25: TYPICAL APPLIED MOMENT VS. END-PLATE SEPARATION RESPONSE FOR A THICK PLATE SPECIMEN (TEST B-MRE 1/2-3/4-3/4-30)

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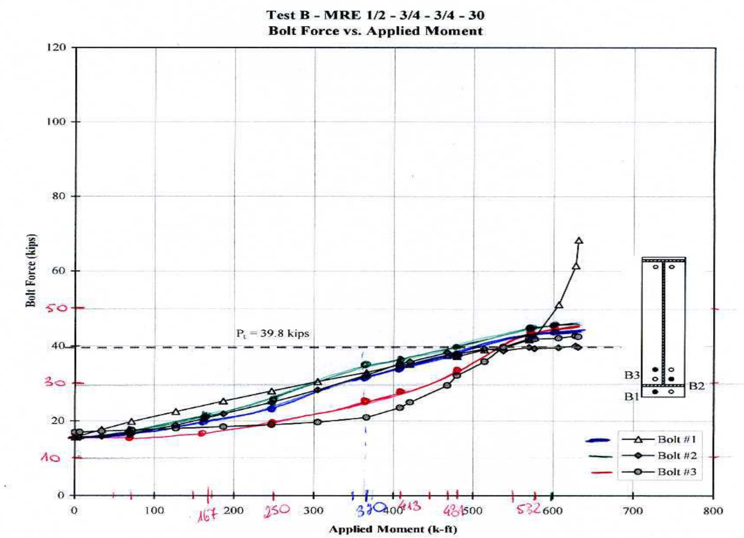

FIGURE 3.26: TYPICAL BOLT FORCE VS. APPLIED MOMENT RESPONSE FOR A THICK PLATE SPECIMEN (TEST B-MRE 1/2-3/4-3/4-30)

2.3 Results of calculations and FEA analysis

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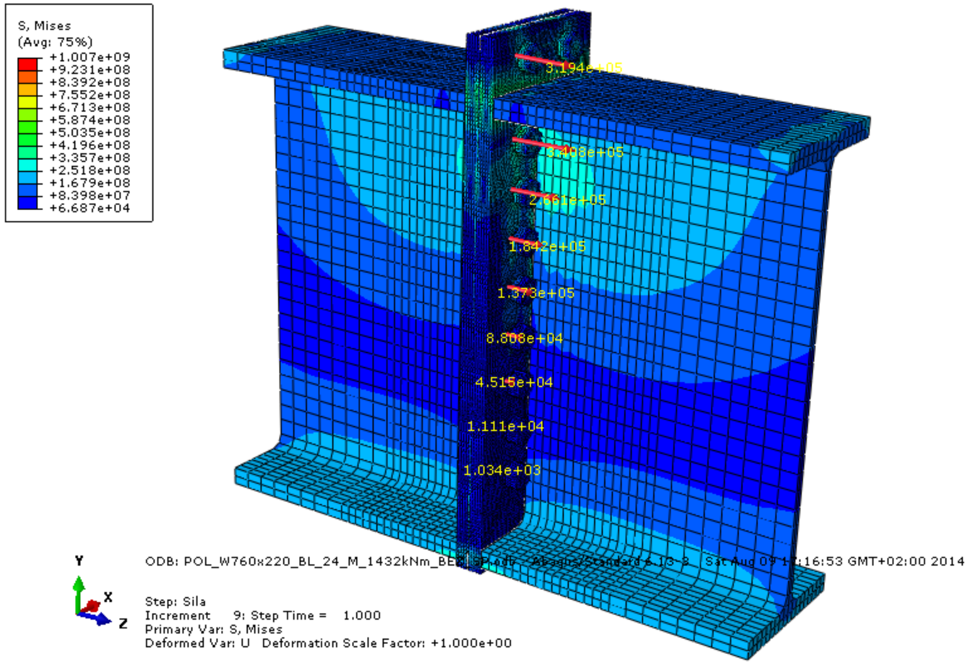


MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

Fig. 2.2 Result of FEA analysis of splice type 1/8 with end-plate thickness 24mm

Comparison of tests and calculations results Table 2.1

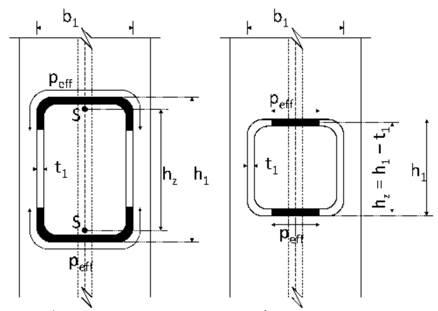
Type [1)]	1/3	1/8	1/3	1/8	1/1	1/3	1/8	1/8 [9)]	1/3	1/8
t_p [2)] (mm)	14		18		24				36	
M_{jT} [6)] (kNm)	627	862	898	1272	701	1131	1477	1317	1104	1396
Mode [3)]	Mode 1		Mode 1 and 2		Mode 2 and 3				Mode 3	
M_{jRd} [4)] (kNm)	627	862	898	1272	701	1131	1477	1317	1104	1396
M_{jRu} [5)] (kNm)	627	862	973	1361	926	1481	1963	1722	1534	1940
M_{jRu}/M_{jRd}	1,00	1,00	**1,08**	**1,07**	1,32	1,31	1,32	1,31	1,39	1,39
M_{jTu} [7)]	845[8)]	1050[8)]	1200	1500	850	1407	1550	1550	1700	1840
M_{jTu}/M_{jRd} [10)]	1,35	1,22	1,34	1,18	1,21	1,24	1,05	1,18	1,54	1,32
F_{t1e} (kN)	189	238	249	**312**	212	**301**	**320**	**313**	242	**313**
F_{t1i} (kN)	227	**272**	**287**	**325**	**299**	**316**	**341**	**324**	**307**	**324**
F_{t2i} (kN)	127	172	145	229		171	**266**	221	145	250
F_{t3i} (kN)	121	159	143	209		152	184	151	102	129
F_{t4i} (kN)		132		170			137	109		87
F_{t5i} (kN)		99		123			88	69		52
F_{t6i} (kN)		65		75			45	34		20
F_{t7i} (kN)		33		33			11	7		15
F_{t8i} (kN)		8		4			1	1		12
F_{t1i}/F_{tRd}	0,89	**1,07**	**1,13**	**1,28**	**1,18**	**1,24**	**1,34**	**1,27**	**1,21**	**1,27**
F_{t1i}/F_{tRu}	0,64	0,77	0,81	0,92	0,85	0,90	0,97	0,92	0,87	0,92

1) Type of splice – number of external bolt rows/number of internal bolt rows

2) t_p – end plate thickness in mm,

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision

3) Failure mode of bolt rows according to EN 1993-1-8/Table.6.2
4) M_{jRd} – the design moment resistance of splice according to EN 1993-1-8/6.2.7.2(9) for $F_{tRd}=0{,}72f_{ub}A_s=254kN$
5) M_{jRu} – the ultimate moment resistance of splice according to EN 1993-1-8/6.2.7.2(9) for $F_{tRu}=f_{ub}A_s=353kN$
6) M_{jT} – the bending moment in FEA model = M_{jRd}
7) M_{jTu}– the ultimate moment in FEA model when $F_{t1i}= F_{tRu}=353kN$
8) M_{jTpl} – the ultimate moment in FEA model for plasitified end-plate
9) For the design moment resistance of splice according proposal for formula (6.26)(yellow market)
10) lowest values of M_{jTu}/M_{jRd} – green market

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision
Do any clauses require editorial or technical correction?							
DE 31		1.5 (3)		ed	In 1.5 (3), A_v is generally defined as the shear area of the chord. However, to avoid misinterpretation, a definition for $A_{V,0}$ should specifically also be added in 1.5 (3) to define the shear area of the chord in the gap region. This value is dependent on the gap size "g", as can be noted typically in Table 7.12	Add the following symbol and definition in 1.5 (3): $A_{V,0}$ the effective shear area of the chord in the gap region	A
DE 32		1.5 (3)		ed	In the July 2009 Corrigendum to EN 1993-1-8, h_z is defined as: "The distance between centres of gravity of the effective width **parts** of a rectangular section beam connected to a I or H section column" For clarity, "parts" should be replaced by "areas" It is also necessary to present figures to clarify 2 situations for determining h_z which is needed in Table 7.22 for determining in-plane design resistance (brace failure). The equation for determining p_{eff} in the figure can be found in Table 7.22.	The text in the definition for h_z should read: The distance between centres of gravity of the effective width areas of a rectangular section beam connected to a I or H section column. The following figures should also be added to the definition for h_z :  a) $p_{eff} > b_1$ b) $p_{eff} < b_1$	A
DE 33		1.5 (3)		ed	Definition for p_{eff} is missing.	p_{eff} is the effective width for a brace member to an I or H section chord	A

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Template for comments and secretariat observations

Date: 2014-10-22	Document:	Project: EN 1993-1-8

MB/ NC[1]	Line number (e.g. 17)	Clause/ Subclause (e.g. 3.1)	Paragraph/ Figure/ Table/ (e.g. Table 1)	Type of comment[2]	Comments	Proposed change	Proposed procedure / decision
DE 34		1.5	Figure 1.3 (b)	ed	The definition of overlap is not clear from the figure	Figure 1.3 (b) should be redrawn more clearly, to understand what “p” and “q” are.	A
DE 35		1.5 (4)		ed	To avoid mistakes, it should additionally be stated that bending stresses should be ignored when determining $\sigma_{0,Ed}$ and $\sigma_{p,Ed}$ if the eccentricity is within the limits of 5.1.5 (5)	$\sigma_{0,Ed}$ is the maximum compressive stress in the chord at a joint, ignoring bending stresses when joint eccentricities are within the limits of 5.1.5 (5) $\sigma_{p,Ed}$ is the value of $\sigma_{0,Ed}$ excluding the stress due to the components parallel to the chord axis of the axial forces in the braces at that joint, see Figure 1.4. Ignore bending stresses when joint eccentricities are within the limits of 5.1.5 (5)	B
DE 36		1.5 (5)		te	The factor γ_{M5} should not be applied to $\sigma_{0,Ed}$ /f_{y0} and $\sigma_{p,Ed}$ /f_{y0}. This is because this ratio represents utilized chord capacity, which reduces joint strength as given in the equations in Table 7.2 to Table 7.5, Table 7.10, and Table 7.11 to Table 7.14. The factor γ_{M5} is used instead in these joint strength equations.	The text to 1.5 (5) should read: *n* is the ratio $\sigma_{0,Ed}$ /f_{y0} (used for RHS chords) n_p is the ratio $\sigma_{p,Ed}$ /f_{y0} (used for CHS chords)	A
DE 37		4.14 (1)	4.14 (1) and Table 4.2	ed	Welding in the cold-formed corners of RHS produced to EN 10219 is permissible providing the criteria of the July 2009 Corrigendum to EN 1993-1-8 are fulfilled. Despite this, there have been misunderstandings in the interpretation of Clause 4.14 for welding in cold-formed zones. The text and note in Clause 4.14 should be corrected to make clear that, for the conditions specified, welding in the cold-formed corners and the adjacent zones is permitted. Thus, if the conditions in Table 4.2 are met, which is the case for some EN 10219 products, welding in the corners and adjacent cold-formed zones is automatically permitted. For other EN10219 products which do not meet the geometric conditions in Table 2, but satisfy the chemical analysis in the proposed change, welding in the corners and adjacent cold-formed zones, such as shown in Figure 2, is permitted.	The text in 4.14 (1) should read: Welding may be carried out in the cold formed zones (in the cold bent corners and up to an influence distance of 5t from these corners as shown in the figure in Table 4.2), provided the following condition is fulfilled: – The cold-formed zones are normalized after cold-forming but before welding. – The inside corner-to-thickness ratio r/t satisfies the relevant value obtained from Table 4.2 For clarity, the title to Table 4.2 should read: Table 4.2: Conditions for welding cold-formed corners and adjacent material Further, the NOTE in Table 4.2 should be presented more clearly as follows: For cold formed hollow sections according to EN 10219 which do not satisfy the inside corner-to-thickness (r/t) limits in this Table, welding in the cold formed corners and adjacent distances of 5t from the corners is also permitted if the following requirements are satisfied:: – The thickness ≤ 12,5 mm – The steel is Aluminium killed	B Remove this sentence to cover comment BE6 from Belgium

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						– The quality is J2H, K2H, MH, MLH, NH or NLH – The chemical analysis meets the following limits: C ≤ 0,18 %, P ≤ 0,020 % and S ≤ 0,012 %. In other cases, welding in this area is only allowed if it can be shown by tests that welding can be permitted for that particular application.	
DE 38		5.1.5 (9)	Table 5.3	ed	Table 5.3 creates misinterpretation as presented. It should be clarified that bending moment due to eccentricity for the compression chord is only for compression chord design and not for stresses at the joint.	In the last column, 3rd row, after “Yes”, add the sub-sentence “, only for design of the compression chords”	A
DE 39		7.1.1 (2)		ed	Since the resistance is sometimes related to the chord member and shear resistance is also considered, the clause should be extended accordingly	7.1.1 (2) should read: The static design resistances of the joints are expressed in terms of maximum design axial, shear and/or moment resistances for the brace or chord members.	A
DE 40		7.1.2 (2)		ed	The July 2009 Corrigendum to EN 1993-1-8 added some words at the end of the sentence, which made the sentence unclear. Delete “The compression elements of”	7.1.2 (2) should read: The members should satisfy the requirements for Class 1 or Class 2 given in EN 1993-1-1 for the condition of axial compression.	A
DE 41		7.1.2	Figure 7.1	ed	Not all types of joints using hollow sections are shown here, so that the title “Types of joints…” should be altered to “Typical joints…”	The title to Figure 7.1 should read: Typical hollow section joints.	A
DE 42		7.2.1 (3)	Equations (7.1) and (7.2)	te	To include the effects of possible out-of-plane moments and to clarify that the chord connecting face is governing, the text and equations should be updated as proposed in the adjacent column here.	7.2.1 (3) should read as follows: (3) The stresses $\sigma_{0,Ed}$ and $\sigma_{p,Ed}$ at the joint at the chord connecting face should be determined from: For RHS chords: $\sigma_{0,Ed} = \frac{\lvert N_{0,Ed} \rvert}{A_0} + \frac{\lvert M_{ip,0,Ed} \rvert}{W_{ip,el,0}} + \frac{\lvert M_{op,0,Ed} \rvert}{W_{op,el,0}} \leq f_{y0} / \gamma_{M5}$ …(7.1) For CHS chords:	B

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						$$\sigma_{p,Ed} = \frac{\lvert N_{p,Ed} \rvert}{A_0} + \left[\left(\frac{M_{ip,0,Ed}}{W_{el,0}}\right)^2 + \left(\frac{M_{op,0,Ed}}{W_{el,0}}\right)^2\right]^{0,5}$$ ≤ fy0 / γM5 …(7.2) where: $$N_{p,Ed} = N_{0,Ed} - \sum_{i>0} (N_{i,Ed}\cos\theta_i)$$	
DE 43		7.2.2 (1)		te	Chord distortional failure and shear between braces and chord face are not yet included in the list of failure modes.	Add the following failure modes to 7.2.2 (1): g) RHS Chord distortional failure under out-of-plane moment loading. h) Shear between brace(s) and the chord face in overlap joints.	B
DE 44		7.2.2 (2) and 7.2.2 (4)	Figure 7.2 and Figure 7.4	te	Failure mode (h) should be added to 7.2.2 (2) and 7.2.2 (4). Also, a figure illustrating shear between brace(s) and the chord face in overlap joints, (failure mode h) is required for Figure 7.2 and Figure 7.4	Add in 7.2.2 (2) and 7.2.2 (4), after (a) to (f): and (h) In Figure 7.2 and Figure 7.4: Add a figure depicting Mode h	A
DE 45		7.2.2 (3)	Figure 7.3	te	Failure modes (g) and (h) should be added to 7.2.2 (3) Also, figures illustrating failure modes g and h are required in Figure 7.3	Add in 7.2.2 (2) and 7.2.2 (4), after (a) to (f): and (h) In Figure 7.3: add 2 figures, depicting Modes g and h	A
DE 46		7.3.1 (2)		ed	The 2nd sentence reads: However in partially overlapping joints the hidden part of the connection need not be welded, provided that the axial forces in the brace members are such that their components perpendicular to the axis of the chord do not differ by more than 20%. This is confusing; 20% of what? See Proposed change	The 2nd sentence should read: However in partially overlapping joints the hidden part of the connection need not be welded, provided that the axial forces in the brace members are such that their components perpendicular to the axis of the chord do not differ by more than 20% of the higher value.	A
DE 47		7.4.1	Table 7.1	te	Correction to braces in compression	In the last column of the table, 5th row, after “Class 1 or Class 2, add:	B

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						and di / ti ≤ 50	
DE 48		7.4.2	Table 7.2	ed	To cover shear failure for X joints with small angles, the title "X joints" in the third row should be provided with an asterisk *) and a note should be added at the bottom of the Table to check also for chord shear failure between the braces, as proposed here.	Add an asterisk to the title "X-joints" in the third row as follows: Chord face failure - X joints*) And add a box at the bottom of Table 7.2 with a note as follows: *) For X joints, with cos $\theta_1 > \beta$, check also for chord shear failure in between the braces. See the last right hand box in Table 7.7 for the procedure	B
DE 49		7.4.2	Table 7.2	ed	The July 2009 Corrigendum to EN 1993-1-8 introduced a sentence in Clause 7.1.2 (6) to check for shear also for certain cases of overlap between braces. For consistency with the other tables, a note should therefore also be added at the end of Table 7.2 to determine when to check for shear.	The title for K and N gap or overlap joints should be provided with a double asterisk *) to refer to the note to follow. Also, add a note at the bottom of Table 7.2 as follows: **) If the overlap $\lambda_{ov} > \lambda_{ov,\ lim.}$, check also for shear of the overlapped braces from the chord face; also see Table 7.1.	B
DE 50		7.4.2	Table 7.2, Table 7.3, Table 7.4 and Table 7.5	ed	To avoid confusion with all other recommendations and codes where tension is designated as positive, put the ratio n_p between absolute signs. (It would have been better to designate tension also as positive in EC 3 but that is probably not feasible anymore).	In Table 7.2, Table 7.3, Table 7.4 and Table 7.5, the last row for the chord compression effect should be modified to: For $n_p > 0$ (compression) $k_p = 1 - 0{,}3\|n_p\| - 0{,}3n_p^2$ but $k_p \leq 1{,}0$	B
DE 51		7.4.2	Table 7.2	ed	The failures in the chord cover cross-sectional failures in general, so that the word "face" in the 1st, 3rd and 5th rows should be deleted.	In the first, third and fifth rows, delete the word "face", to read: Chord failure	A
DE 52		7.4.2	Table 7.3	ed	In the 7th row, the symbols are not correct. N_{Ed} and M_{Ed} should read: $N_{1,Ed}$ and $M_{1,Ed}$. *In the proposed change, everything in row 7 is repeated, with the corrections included, to avoid confusion*	The 7th row giving the equations for punching shear should be replaced by the following: $\sigma_{max.}t_1 = (N_{1,Ed}/A_1 + M_{1,Ed}/W_{el,1})t_1 \leq 2t_0(f_{y0}/\sqrt{3})/\gamma_{M5}$ Note: If the in-plane and/or out-of-plane moments in the brace are present, both have to be considered.	B

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DE 53		7.4.2	Table 7.3 and Table 7.4	te	A lower limit should be given to η to avoid excessive deformation of the chord face.	In the last row, left column, change the limit $\eta = \frac{h_1}{d_0} \leq 4$ to: $1 \leq \eta = \frac{h_1}{d_0} \leq 4$	A
DE 54		7.4.2	Table 7.4	ed	Table 7.1 only gives the validity range between CHS chord and CHS braces. Here, the validity range between CHS chord and RHS, I or H Section braces should be given.	In the last row, left column, additional limits should be given as follows: RHS braces in tension: $b_1/t_1 \leq 35$ and $h_1/t_1 \leq 35$ RHS braces in compression: class 1 or 2 and limits as for tension I or H section braces in compression: class 1 or 2	B
DE 55		7.4.2	Table 7.4	te	In the 7th row after the figures, with the equations for punching shear, there is a mistake in the η limit. The limit η > 2 should read: η ≤ 2. Also, the symbols are not correct. N_{Ed} and M_{Ed} should read: $N_{1,Ed}$ and $M_{1,Ed}$. In addition, a note is necessary as a reminder to include out-of-plane moments also, if present *In the proposed change, everything in row 7 is repeated, with the corrections included, to avoid confusion*	The 7th row giving the equations for punching shear should be replaced by the following: I or H Sections with $\eta \leq 2$ (for axial compression and out-of-plane bending) and RHS Sections: $\sigma_{max.}t_1 = (N_{1,Ed}/A_1 + M_{1,Ed}/W_{el,1})t_1 \leq t_0(f_{y0}/\sqrt{3})/\gamma_{M5}$ All other cases: $\sigma_{max.}t_1 = (N_{1,Ed}/A_1 + M_{1,Ed}/W_{el,1})t_1 \leq 2t_0(f_{y0}/\sqrt{3})/\gamma_{M5}$ Where t_1 is the flange or wall thickness of the I-, H-, or RHS section brace. Note: If the in-plane and/or out-of-plane moments in the brace are present, both have to be considered.	B
DE		7.4.2	Table 7.6	ed	It is not clear what Design Criteria are implied in the	Change text in first row, right box, to:	B

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56					top row, right hand column of Table 7.6. In all cases, it is for "chord face failure", which should be added. Also, a clarification is necessary to indicate that punching shear should be checked individually for each brace footprint on the chord.	Design criteria for chord face failure[1)] Add a note at the bottom of the table: 1) Check each brace individually for punching shear	
DE 57		7.4.3	Table 7.6 and Table 7.7	ed	$N_{0,Ed}$ and $V_{0,Ed}$ are not defined anywhere, so should be defined in both tables. Also, it should be shown how to calculate $N_{pl,0,Rd}$, $V_{pl,0,Rd}$ and the shear area $A_{V.0}$ for the CHS chord.	For clarification, in the last row right box for KK gap joints in one plane (Table 7.6) and for multi-planar KK gap joints in 2 different planes to each other (Table 7.7), add the following after the interaction equation: where: $N_{0,Ed}$ = design axial force in the gap; $N_{pl,0,Rd} = \frac{f_{y0} A_0}{\gamma_{M0}} = f_{y0} \pi (d_0 - t_0) t_0 / \gamma_{M0}$ $V_{0,Ed}$ = design shear force in the gap; $V_{pl,0,Rd} = \frac{(f_{y0} / \sqrt{3}) A_{V.0}}{\gamma_{M0}} = \frac{f_{y0}}{\sqrt{3}} \frac{2A_0}{\pi} \frac{1}{\gamma_{M0}}$	B
DE 58		7.4.3	Table 7.7	ed	It is not made clear, that the reduction factor μ pertains to the chord failure criterion, so the title in the top right hand box should be expanded accordingly	Change text in right hand top row to: Reduction factor μ for the chord failure criterion.	B
DE 59		7.5.1 (1)		ed	The statement "hollow section brace members" in the second line of 7.5.1 (1) is misleading, since plates and open sections are also implied. "Hollow section" should be deleted.	In the second line of 7.5.1 (1), replace "… hollow section brace members …" with: " …brace members …"	B

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DE 60		7.5.1	Table 7.8	te	Comments: The Table 7.8 currently reads as follows, where there are a number of errors and modifications are required, as will be described after the existing table copied		B

Table 7.8: Range of validity for welded joints between CHS or RHS brace members and RHS chord members

Type of joint	Joint parameters [$i = 1$ or 2, j = overlapped brace] b_i/b_0 or d_i/b_0	b_i/t_i and h_i/t_i or d_i/t_i Compression	b_i/t_i and h_i/t_i or d_i/t_i Tension	h_0/b_0 and h_i/b_i	b_0/t_0 and h_0/t_0	Gap or overlap b_i/b_j
T, Y or X	$b_i/b_0 \geq 0,25$	$b_i/t_i \leq 35$ and $h_i/t_i \leq 35$ and [AC2⟩ Class 1 or 2 ⟨AC2]	$b_i/t_i \leq 35$ and $h_i/t_i \leq 35$	$\geq 0,5$ but $\leq 2,0$	≤ 35 and [AC2⟩ Class 1 or 2 ⟨AC2]	–
K gap N gap	$b_i/b_0 \geq 0,35$ and $\geq 0,1 + 0,01\, b_0/t_0$				≤ 35 and [AC2⟩ Class 1 or 2 ⟨AC2]	$g/b_0 \geq 0,5(1-\beta)$ but $\leq 1,5(1-\beta)$ 1) and as a minimum $g \geq t_1 + t_2$
K overlap N overlap	$b_i/b_0 \geq 0,25$	Class 1			[AC2⟩ Class 1 or 2 ⟨AC2]	[AC2⟩ $25\% \leq \lambda_{ov} \leq \lambda_{ov,lim.}$ 2) $b_i/b_j \leq 0,75$ ⟨AC2]
Circular brace member	$d_i/b_0 \geq 0,4$ but $\leq 0,8$	Class 1	$d_i/t_i \leq 50$	As above but with d_i replacing b_i and d_j replacing b_j.		

1) If $g/b_0 > 1,5(1-\beta)$ and [AC2⟩ $g > t_1 + t_2$ ⟨AC2] treat the joint as two separate T or Y joints.

2) [AC2⟩ $\lambda_{ov,lim.}$ = 60% if the hidden seam is not welded and 80% if the hidden seam is welded. If the overlap exceeds $\lambda_{ov,lim.}$ or if the braces are rectangular sections with $h_i < b_i$ and/or $h_j < b_j$, the connection between the braces and chord face has to be checked for shear. ⟨AC2]

below.

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					Based on reanalysis for the IIW 2009 Recommendations, Class 1 and Class 2 can be changed to Class 1 or Class 2. In the 5th row, 7th column, below “gap or overlap” the error $b_i/b_j \leq 0{,}75$ should be changed to: $b_i/b_j \geq 0{,}75$. Based on recent analyses the lower b_i/b_0 limit for T, Y, X and Kgap joints, K gap joints can be taken the same (simplification), being: $b_i/b_0 \geq 0{,}1 + 0{,}01 b_0/t_0$ but $\geq 0{,}25$. For circular brace members similar lower limits (missing in the current table 7.8) can be used as for RHS braces, i.e. $0{,}25 \leq d_i/b_0 \leq 0{,}80$ and $d_i/b_0 \geq 0{,}1 + 0{,}01 b_0/t_0$. In note 1) at the bottom it should be indicated that the chord shear check still applies. For K and N overlap joints after $b_i/b_0 \geq 0{,}25$ for circular braces, add $d_i/b_0 \geq 0{,}25$ (missing in current version). For a better understanding give the second note 2) in separate items, as shown below in a reordered table. The corrections are reordered in a compact and more readable format in the following new Table 7.8: The table has been carefully prepared to avoid repetition and prevent any misinterpretation.		

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					Proposed change: (see below)		

Proposed change:

Table 7.8: Range of validity for welded joints between CHS or RHS brace members and RHS chord members

<table>
<tr><td colspan="5">Joint parameters (For T, Y and X joints: i = 1 for the brace;
for K, KT and N joints with gap: i = 1 (compression brace), 2 (tension brace), 3 (vertical brace);
for overlap joints: i = overlapping brace, j = overlapped brace)</td></tr>
<tr><td colspan="2">Chords</td><td colspan="2">Braces</td><td rowspan="2">Gap or overlap</td></tr>
<tr><td>General</td><td>Compression</td><td>General</td><td>Compression</td></tr>
<tr><td rowspan="3">$b_0/t_0 \leq 35$
$h_0/t_0 \leq 35$</td><td rowspan="3">Class 1 or 2 and general limits</td><td rowspan="2">$0{,}5 \leq h_i/b_i \leq 2{,}0$
$b_i/t_i \leq 35$
$h_i/t_i \leq 35$
$d_i/t_i \leq 50$</td><td rowspan="2">Class 1 or 2 and general limits</td><td>$0{,}5(1-\beta) \leq g/b_0 \leq 1{,}5(1-\beta)$ [1)]
But as a minimum
$g \geq t_1 + t_2$</td></tr>
<tr><td>$25\% \leq \lambda_{ov} \leq 100\%$ [2)]
$b_i/b_j \geq 0{,}75$ $d_i/d_j \geq 0{,}75$</td></tr>
<tr><td colspan="3">For T, Y, X und K gap joints:
$b_i/b_0 \geq 0{,}1 + 0{,}01 b_0/t_0$ but $\geq 0{,}25$
$d_i/b_0 \geq 0{,}1 + 0{,}01 b_0/t_0$ and $0{,}25 \leq d_i/b_0 \leq 0{,}80$
For K and N overlap joints:
$b_i/b_0 \geq 0{,}25$ and $d_i/b_0 \geq 0{,}25$</td></tr>
<tr><td colspan="5">1) For $g/b_0 > 1{,}5(1-\beta)$ and $g > t_1 + t_2$, check the joint as two separate T or Y joints, but with an additional chord shear check in the gap.

2) If $\lambda_{ov.} \geq 60\%$ and the hidden seam of the overlapped brace is not welded,
or $\lambda_{ov} \geq 80\%$ and the hidden seam of the overlapped brace is welded,
or the braces are rectangular hollow sections with $h_i < b_i$ and/or $h_j < b_j$,
the connection between braces and chord has to be checked for shear.</td></tr>
</table>

Note [2)] above also needs to replace the relevant notes in Tables 7.20 and 7.23.

Also, in Tables 7.20 and 7.23, $25\% \leq \lambda_{ov} \leq \lambda_{ov,lim}$ should be replaced by $25\% \leq \lambda_{ov} \leq 100\%$

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			Tables 7.20 and 7.23				
DE 61		7.5.2.1 (4)		ed	In the first sentence, “CHS or RHS” should be deleted, since plates and open sections should also be considered	In the first sentence, delete: CHS or RHS	B
DE 62		7.5.2.1	Table 7.10	ed	The title is not complete. The end of the title should have the following words added: “braces and square hollow section chords”	The title should read: Table 7.10: Design axial resistances of welded joints between square or circular hollow section braces and square hollow section chords	B
DE 63		7.5.2.1, 7.6 and 7.7	Table 7.10, Table 7.21 and Table 7.24	ed	The overlap is given in %. Therefore, in the equation for Brace failure for 25% ≤ λ_{ov} < 50% for $N_{i,Rd}$, λ_{ov} / 50 should be replaced by λ_{ov} / 50%	In the box below the box: “Brace failure 25% ≤ λ_{ov} < 50%” in the equation for $N_{i,Rd}$, λ_{ov} / 50 should be replaced by: λ_{ov} / 50%	B
DE 64		7.5.2.1	Table 7.10	te	In the 8th row of the first column, the parameters “d_{eff}“ and “$d_{e,ov}$” for the effective width for circular braces is missing. In the 9th and 10th rows left column, respectively, expressions for “d_{eff} “ and “$d_{e,ov}$” for effective widths with circular braces are missing. In the 2nd last row, mention should be made that for circular braces, expressions for “d_{eff} “ and “$d_{e,ov}$” should be used instead of those for “b_{eff} “ and “$b_{e,ov}$”	In the 8th row, first column, add d_{eff} and $d_{e,ov}$ The 8th row thus reads: Parameters b_{eff}, d_{eff}, $b_{e,ov}$, $d_{e,ov}$ and k_n In the 9h row, left column, add: $d_{eff} = \left(\frac{12}{d_0/t_0}\right)\left(\frac{f_{y0}t_0}{f_{yi}t_i}\right)d_i$ but $d_{eff} \le d_i$ In the 10th row, left column, add: $d_{e,ov} = \left(\frac{12}{d_j/t_j}\right)\left(\frac{f_{yj}t_j}{f_{yi}t_i}\right)d_i$ but $d_{e,ov} \le d_i$ The 2nd last row should read: For circular braces, multiply the above resistances by $\pi/4$, replace b_1 and h_1 by d_1, replace b_2 and h_2 by d_2 and replace b_{eff} and $b_{e,ov}$ by d_{eff} and $d_{e,ov}$ respectively	B
DE 65		7.5.2.1	Table	ed	To maintain consistency with the other Tables, an additional note should be made for when to check	A second note to check for shear between the braces and the chord, as mentioned in Table 7.8, should be added as follows, at the	B

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			7.10		for shear between the braces and the chord.	bottom of the table: **) If the overlap λ_{ov} exceeds $\lambda_{ov,lim}$,the connection between the braces and chord should be checked for shear Also see Table 7.8	
DE 66		7.5.2.1	Table 7.10	ed	It does not say what to do for T, Y and X joints when β > 0,85	Also add a third note to consider what to do when β > 0,85: ***) If β > 0,85, use Table 7.11 or 7.12	B
DE 67		7.5.2.1	Table 7,10, Table 7.11, Table 7.12 and Table 7.14	ed	To avoid confusion with all other recommendations and codes where tension is designated as positive, put the ratio "n" between absolute signs. (It would have been better to designate tension also as positive in EC 3 but that is probably not feasible anymore). For compression in the chord, k_n should not be taken lower than 0,3, being the minimum for k_m in table 7.13 for longitudinal plates. This results in the following box for k_n:	The last right hand box, the k_n value for compression should be replaced by: For n in compression: $k_n = 1{,}3 - \frac{0{,}4\,\lvert n \rvert}{\beta}$ but ≤ 1,0 and $k_n \geq 0{,}3$	B
DE 68		7.5.2.1	Table 7.11	ed	The reference to note [1)] should also be given in the 1st row, right box, since side wall shear resistance should always be checked for small angles.	In 1st row, right box, change "Design resistance" to: Design resistance[1)]	A
DE 69		7.5.2.1	Table 7.11	ed	In the 4th row, right, "buckling[1)]" should be replaced by "failure", since not only compression (therefore buckling), but also tension is considered.	The 4th row, right box should read: Chord side wall failure[1)] β = 1,0 [2)]	B
DE 70		7.5.2.1	Table 7.11	ed	By putting note [1)] in the first row, right box, the note can be shortened and explained that g = ∞ should be used in Table 7.12 to calculate side wall shear resistance.	The note [1)] should read: [1)] For X joints with cos $\theta_1 > h_1/h_0$,check the design shear resistance of the chord side walls using table 7.12, assuming that g = ∞ (or α = 0)	B
DE 71		7.5.2.1	Table 7.11	te	The design resistances for chord failure are given for β ≤ 0,85 and β = 1. The note [2)] should therefore be changed from 0,85 ≤ β ≤ 1,0 into 0,85 < β < 1,0.	Note [2)] should read: [2)] For 0,85 < β <1,0 use linear interpolation between the resistance for chord face failure at β =0,85 and the governing value for chord side wall failure at β = 1,0 (side wall buckling or chord shear).	B

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DE 72		7.5.2.1	Table 7.11 and Table 7.12	ed	In the 11th row last column, mention should be made that for circular braces, expressions for "d_{eff} " and "$d_{e,p}$" should be used instead of those for "b_{eff} " and "$b_{e,p}$" In the 12th and 13th rows, right column, respectively, expressions for "d_{eff} " and "$d_{e,p}$" for effective widths with circular braces are missing.	The 11nd row last column should read: For circular braces, multiply the above resistances by $\pi/4$, replace b_1 and h_1 by d_1, replace b_2 and h_2 by d_2 and replace b_{eff} and $b_{e,p}$ by d_{eff} and $d_{e,p}$ respectively In the 12th row, right column, add: $d_{eff} = \left(\frac{12}{d_0/t_0}\right)\left(\frac{f_{y0} t_0}{f_{yi} t_i}\right) d_i \ \text{ but } d_{eff} \le d_i$ In the 13th row, right column, add: $d_{e,p} = \left(\frac{12}{d_0/t_0}\right) d_i \ \text{ but } d_{e,p} \le d_i$	B
DE 73		7.5.2.1	Table 7.12	ed	As mentioned in the comment above to 1.5 (3), Av should be replaced by Av,0, to represent shear area in the gap region of a K joint.	In the 5th row, right box, change all references to Av by Av,0, to give: $N_{i,Rd} = \frac{f_{y0} A_{v,0}}{\sqrt{3}\sin\theta_i} / \gamma_{M5}$ $N_{0,Rd} = \left[(A_0 - A_{v,0}) f_{y0} + A_{v,0} f_{y0} \sqrt{1 - (\frac{V_{0,Ed}}{V_{pl.0,Rd}})^2}\right] / \gamma_{M5}$ Also, in the last row, left box, change Av in the first equation to Av,0 as follows: $A_{v,0} = (2h_0 + \alpha\, b_0) t_0$	B
DE 74		7.5.2.1	Table 7.13	ed	The figures only show T joints. So, for consistency and to specify that the formulas also apply to X joints, the headings in the first, third and 5th row left boxes should include "T and X joints".	In the 1st, 3rd and 5th row left boxes, change the titles to: T and X joints - transverse plate T and X joints - longitudinal plate T and X joints - I or H section	B

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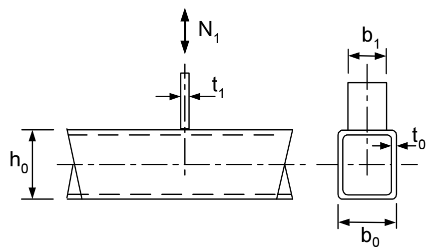
DE 75		7.5.2.1	Table 7.13	te	The check for "Plate failure" was erroneously deleted after the July 2009 Corrigendum to EN 1993-1-8 was published. So this and other (general) changes in the table only pertaining to the transverse plate are first listed below: : 1. For consistency with the other tables, the first row right box should read "Design resistance". 2. Only 1 brace exists, so change index "I" into "1". 3. In the July 2009 Corrigendum to EN 1993-1-8, the check for "Plate failure" was deleted by error and shown again below. 4. For consistency with Table 7.11, the 3rd row right box should be changed from "Chord side wall crushing (for $b_1 \geq b_0 - 2t_0$)" into "Chord side wall failure (for β= 1,0)"		B

Proposed change to the **top part of Table 7.13** pertaining to Transverse plate:

T- and X- joints – transverse plates	**Design resistance**
	Plate failure (for all β)
	$N_{1,Rd} = f_{y1} t_1 b_{eff} / \gamma_{M5}$ (*)
	Chord face failure (for $\beta \leq 0{,}85$)
	$N_{1,Rd} = k_n f_{y0} t_0^2 \left[\frac{2+2{,}8\beta}{\sqrt{1-0.9\beta}}\right]/\gamma_{M5}$
	Chord punching shear: (for $b_1 \leq b_0 - 2t_0$)
	$N_{1,Rd} = \frac{f_{y0}}{\sqrt{3}} t_0 (2t_1 + 2b_{e,p})/\gamma_{M5}$
	Chord side-wall failure (for $\beta = 1{,}0$)
	$N_{1,Rd} = k_n f_{y0} t_0 (2t_1 + 10t_0)/\gamma_{M5}$ For 0,85 < β <1,0 use linear interpolation between the resistance for chord face failure at β =0,85 and the resistance for chord side wall failure at β = 1,0.

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DE 76		7.5.2.1	Table 7.13	ed	The range of validity is not consistent and should be better specified. After "in addition to the limits given in Table 7.8" change limits into: $0{,}4 \le b_1/b_0 \le 1{,}0$; $1 \le h_1/b_0 \le 4$; delete: $b_0/t_0 \le 30$ and show $\theta_1 = 90^\circ$ in all the figures, as in Table 7.14.	Thus, the 7th and 8th row, left box, should read: Range of validity In addition to the limits given in Table 7.8: $0{,}4 \le b_1/b_0 \le 1{,}0$ $1 \le h_1/b_0 \le 4$ and $\theta_1 = 90^\circ$	A
DE 77		7.5.2.1	Table 7.13	ed	To avoid confusion with all other codes and recommendations where tension is designated as positive, put the ratio "n" between absolute signs. (It would have been better to designate tension also as positive in EC 3 but that is probably not feasible anymore).	For consistency with Tables 7.10, 7.11 and 7.12 add or change the following at the end of the table, in the right box: For k_n, see Table 7.11 For n compression: $k_m = 1{,}3(1 - \|n\|)$ but $\le 1{,}0$	B
DE 78		7.5.2.1	Table 7.14	te	For extending the validity to S 460 and consistency with Table 7.11, the 5th row right box should be amended for in-plane moments. The reduction factor k_n appears to have been erroneously omitted. See Table 7.11 and Table 7.13	Replace the 5th row, right box of Table 7.14 with: $M_{ip,1,Rd} = 0{,}5 k_n f_b t_0 (h_1 + 5t_0)^2 / \gamma_{M5}$ $f_b = f_{y0}$ for T joints $f_b = 0{,}8\chi f_{y0}$ for X joints	B
DE 79		7.5.2.1	Table 7.14	te	For extending the validity to S 460 and consistency with Table 7.11, the 11h row right box should be amended for out-of-plane moments. Here too, the reduction factor k_n appears to have been erroneously omitted. See Table 7.11 and Table 7.13	Replace the 11th row, right box of Table 7.14 with: $M_{op,1,Rd} = k_n f_b t_0 (b_0 - t_0)(h_1 + 5t_0)/\gamma_{M5}$ $f_b = \chi f_{y0}$ for T joints $f_b = 0{,}8\chi f_{y0}$ for X joints	B
DE 80		7.5.2.1	Table 7.14	te	For consistency with Tables 7.11 and 7.13, change in 2 places: "Chord side wall crushing $0{,}85 < \beta \le 1{,}0$" into "Chord side wall failure $\beta = 1{,}0$" and add a note to interpolate between the resistance for chord face failure at $\beta = 0{,}85$ and the resistance for chord side wall failure at $\beta = 1{,}0$.	Replace the 4th and 10th row, right boxes with: Chord side wall failure $\beta = 1{,}0$ Below this add: For $0{,}85 < \beta < 1{,}0$, use linear interpolation between the resistance for chord face failure at $\beta = 0{,}85$ and the resistance for chord side wall failure at $\beta = 1{,}0$.	A
DE 81		7.5.2.1	Table	ed	In the figures in the 3rd row left box, index "1" is missing.	In the 3rd row, left box, replace θ in the figures by θ_1 and for the	B

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			7.14		Add a note to prevent distortional buckling for X joints. Also add a note at the end of the table for the range of validity.	lower brace in the X joint, replace M_{ip} by $M_{ip,1}$ Add 2 notes at the end of the table: **) For X joints, distortional failure should be prevented ***) Range of validity: Same as in Table 7.8, but with θ_1 = 90	
DE 82		7.5.2.1	Table 7.15	ed	For consistency with Table 7.6, the braces 1 and 3 in the figure for KT joints in the 3rd row left box should be shown in compression (arrows pointed to the joint) and brace 2 in tension. The arrow depicting N_3 should only show compression (arrow pointed towards the joint), as otherwise the corresponding design criteria is incorrect.	In the 3rd row, left box, N_3 in the figure should only show compression, with the arrow pointing downwards.	A
DE 83		7.5.2.1	Table 7.15	ed	The type of design criteria should be given. Chord face failure should be mentioned. Also, a note is required to check at each brace for chord punching shear	The first row right box should read: Design criteria for chord face failure Add a note at the bottom of the Table 7.15: *) Check at each brace for chord punching shear	B
DE 84		7.5.2.1	Table 7.15	ed	For clarity and consistency with Table 7.12 change A_V into $A_{V,0}$ and replace the interaction equation in the last row right box as given in the proposed change.	Change A_V into $A_{V,0}$ to represent shear area in the gap region, for the following equations. These equations should replace the equations at the end of the last row right box. $V_{0,Ed} \leq V_{pl,0,Rd} = \frac{(f_{y0}/\sqrt{3})A_{V.0}}{\gamma_{M0}}$ $N_{0,Ed} \leq N_{pl,0,Rd} = \frac{f_{y0} A_0}{\gamma_{M0}}$ $N_{0,Rd} = \left[(A_0 - A_{V,0})f_{y0} + A_{V,0} f_{y0}\sqrt{1-\left(\frac{V_{0,Ed}}{V_{pl,0,Rd}}\right)^2}\right]/\gamma_{M5}$ Where: $N_{0,Ed}$ = Design axial force in the gap $V_{0,Ed}$ = Design shear force in the gap	B
DE 85		7.5.2..1	Table 7.16	ed	For cranked chord joints where $N_{i,Rd}$ is the value of $N_{i,Rd}$ for a K or N overlap joint from Table 7.12.should be taken from Table 7.10,	In the last row, right box, the sentence should read: where $N_{i,Rd}$ is the value of $N_{i,Rd}$ for a K or N overlap joint from Table 7.10.	A

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					not Table 7.12		
DE 86		7.5.2.2	Table 7.17	ed	In the 2nd row, the text after “plates” should be deleted, as the relevant boxes specifically refer to the failure criteria to be checked.	In the 2nd row, delete the text after “plates” (correction). The text should read: Reinforced with flange plates	B
DE 87		7.5.2.2	Table 7.17	ed	In 4th row, right box, delete the last equation and add for clarification, the text in the proposed changes.	In 4th row, right box, delete the last equation for $N_{1,Rd}$ and add the following text for clarification: For brace tension loading, use for chord failure, brace failure and punching shear the resistances of Table 7.11 with k_n = 1,0, but: - f_{y0} is replaced by: f_{yp} - t_0 is replaced by: t_p - β is replaced by: $\beta_p = b_1/b_p$ - η is replaced by: $\eta_p = h_1/b_p$	B
DE 88		7.5.2.2	Table 7.17	ed	In 6th row, right box, delete the text (last 4 lines) and specify more clearly:	The last 4 lines in the 6th row, right box should be replaced by: For brace compression loading, use for chord failure, brace failure and punching shear the resistances of Table 7.11 with k_n = 1,0, but: - f_{y0} is replaced by: the lowest of f_{y0} and f_{yp} - t_0 is replaced by: $t_0 + t_p$ - β is replaced by: $\beta_p = b_1/b_p$ - η is replaced by: $\eta_p = h_1/b_p$	B
DE 89		7.5.2.2	Table 7.17	ed	In the 7th row, delete the text after “plates”	In the 7th row, delete the last part of the text, so that it should read: Reinforced with side plates	B
DE 90		7.5.2.2	Table 7.17	ed	In the 8th (last) row, right box, delete text “Take $N_{1,Rd}$ as the value…..” and specify more clearly:	In the 8th (last) row, right box, the last 4 lines of text should be deleted and replaced for clarification by: Use for chord face failure, brace failure and punching shear the resistances of Table 7.11 but for chord side wall failure and chord shear failure:	B

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						- f_{y0} is replaced by the lowest of f_{y0} and f_{yp} - t_0 is replaced by $(t_0 + t_p)$	
DE 91		7.5.2.2	Table 7.17	ed	Circular braces are not considered in the expressions. For circular braces, b_1 and h_1 should be replaced by d_1	Add a note: For circular braces, replace b_1 and h_1 by d_1 in the expressions above.	A
DE 92		7.5.2.2	Table 7.17 and Table 7.18	ed	The index “o” in all the figures is incorrect. It should be replaced by “0” (zero) everywhere.	For consistency in all the figures in Table 7.17,: Change index “o” into “0”	A
DE 93		7.5.2.2	Table 7.18	ed	In the 2nd row, the text after “plates” should be deleted, as the relevant boxes specifically refer to the failure criteria to be checked.	Replace the text in the 2nd row with: “Reinforced with flange plates $\beta_p \geq 0{,}85$”	B
DE 94		7.5.2.2	Table 7.18	ed	In the 3rd row, right box, delete text (not the equations) and specify more clearly as in the proposed change.	In the 3rd row, right box, delete text (not the equations) and specify more clearly, as follows: Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N gap joint from Table 7.12 for chord face failure, brace failure and punching shear, but: - f_{y0} is replaced by: f_{yp} - t_0 is replaced by: t_p - β is replaced by: $\beta_p = b_1/b_p$ - η is replaced by: $\eta_p = h_1/b_p$	B
DE 95		7.5.2.2	Table 7.18	ed	In the 4th row, the text should be shortened as in the 2nd row.	Shorten the text in the 4th row to:: Reinforced with a pair of side plates.	B
DE 96		7.5.2.2	Table 7.18	ed	Delete text in the 5th row right (not the equation) and specify the text more clearly as in the proposed change.	In the 5th row, right box, delete the text (not the equations) and specify more clearly, as follows: Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N gap joint from Table 7.12 but for chord shear failure: - f_{y0} is replaced by: the lowest of f_{y0} and f_{yp} - t_0 is replaced by: $t_0 + t_p$	B

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DE 97		7.5.2.2	Table 7.18	ed	The text in the 6th row is incorrect. The division plate is to allow ease of welding and only slightly increases the resistance, because of substituting the thickness of the division plate for that of the overlapped plate in the equations. Also, the overlap may not be less than 25% to maintain ductile behaviour. The text should be as in the proposed change.	The text in the 6th row should read: Division plate between the brace members for overlap joints, for overlap greater than 25%.	A
DE 98		7.5.2.2	Table 7.18	ed	The text in the 7th row right box has some errors. It now reads: Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N overlap joint from Table 7.12 with $\lambda_{ov} < 80\%$, but with b_j t_j and f_{yj} replaced by b_p t_p and f_{yp} in the expression for $b_{e,ov}$ given in Table 7.10. This should be replaced as in the proposed change.	Replace the text (last 4 lines) in the 7th (last) row with: Take $N_{i,Rd}$ as the value of $N_{i,Rd}$ for a K or N overlap joint from Table 7.10 with $\lambda_{ov} > 25\%$, but with b_j t_j and f_{yj} replaced by b_p t_p and f_{yp} in the expression for $b_{e,ov}$ given in Table 7.10.	A
DE 99		7.5.2.2	Table 7.18	ed	Circular braces are not considered in the expressions. For circular braces, b_1 and h_1 should be replaced by d_1 and, b_2 and h_2 should be replaced by d_2	Add a note: For circular braces, replace b_1 and h_1 by d_1 and b_2 and h_2 by d_2 in the expressions above.	A
DE 100		7.5.3	Table 7.19	ed	The failure criterion is missing in the 1st row, right box	Add in 1st row, right box, after μ "for chord face failure", thus it should read: Reduction factor μ for the chord face failure criterion	B
DE 101		7.5.3	Table 7.19	ed	For clarity and consistency with Table 7.12 change A_V into $A_{V,0}$ and replace the interaction equation in the last row right box as given in the proposed change, as for Table 7.15. For consistency with proposed changes to Table 7.15, the equation in the last row should be replaced with formulas to determine the plastic resistances.	Change A_V into $A_{V,0}$ to represent shear area in the gap region, for the following equations. These equations should replace the equations at the end of the last row right box. $$\frac{\sin(\varphi/2)}{2} V_{0,Rd} \leq V_{pl,0,Rd} = \frac{(f_{y0}/\sqrt{3}) A_{V.0}}{\gamma_{M0}}$$ $$N_{0,Ed} \leq N_{pl,0,Rd} = \frac{f_{y0} A_0}{\gamma_{M0}}$$	B

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						$= \frac{f_{y0}}{\sqrt{3}}(0{,}5A_0)\frac{1}{\lambda_{M0}}$ for a RHS chord with $h_0 = b_0$ $N_{0,Rd} = \left[(A_0 - A_{v,0})f_{y0} + A_{v,0}f_{y0}\sqrt{1-(\frac{\sin(\varphi/2)\cdot V_{0,Ed}}{V_{pl.0,Rd}})^2}\right] / \gamma_{M5}$ $N_{0,Ed}$ = design axial force in the gap $V_{0,Ed}$ = resulting design shear force in the gap	
DE 102		7.6	Table 7.20	ed	For circular braces, the limiting ratio between overlapping braces is missing.	The 2nd row, last box, should read: b_i / b_j or d_i / d_j	A
DE 103		7.6	Table 7.20	ed	A note should be added at the bottom of the table for the minimum gap size to allow welding.	Add a note at the bottom of the table: For K and N joints with gap, $g \geq t_1 + t_2$	A
DE 104		7.6	Table 7.20	te	The restriction of h_i / b_i = 1,0 for T, Y, K gap and N gap joints can be relaxed to " ≥ 0,5 but ≤ 2,0" if a check for local yielding in the brace (brace effective width check) for all joints between RHS braces and I section chords is carried out.	In the 4th row, 5th column of Table 7.20: Delete "1,0" and add: ≥ 0,5 but ≤ 2,0	B
DE 105		7.6	Table 7.21	ed	In the 7th row, middle box, replace index 1 by index I for $N_{i,Rd}$ as both braces i=1 (compression) and i=2 (tension) should be checked.	In the 7th row, middle box, replace index 1 by index I, as follows: $N_{i,Rd} = \frac{f_{y0} t_w b_w}{\sin\theta_i} / \gamma_{M5}$	B
DE 106		7.6	Table 7.21	ed	The figure in the 5th row left box for K and N joints partly encroaches on the box below.	For clarity, move Figure for K and N gap joints upwards in the box.	A
DE 107		7.6	Table	ed	The value for p_{eff} for CHS braces is missing. See proposed change.	For CHS braces, add in the box for p_{eff}, for T, Y, X and K and N gap joints after $p_{eff} \leq b_i + h_i - 2t_i$:	B

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			7.21			or $p_{eff} \leq 0{,}5\pi(d_i - t_i)$ and add after $p_{eff} \leq b_i$ for K and N overlap joints: or $p_{eff} \leq d_i$	
DE 108		7.6	Table 7.21	ed	A note to check chord shear failure for X joints with a small included angle θ_1 is missing. See proposed change.	Add at the bottom of the table, a note for X joints: (**) For X joints with $\cos\theta_1 > h_1 / h_0$, check also for chord shear failure in between the braces	B
DE 109		7.6	Table 7.21	ed	In the middle box (for $b_{e,ov}$) above the last box for notes, an expression for overlap between CHS braces is missing. See proposed change.	In the middle box (for $b_{e,ov}$) above the last box for notes, an expression for overlap between CHS braces should be added: $d_{e,ov} = \left(\frac{12}{d_j/t_j}\right)\left(\frac{f_{yj}t_j}{f_{yi}t_i}\right)d_i$ but $d_{e,ov} \leq d_i$ In summary, the text in the two central boxes above the comments box should look as follows:	B

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						$p_{eff} = t_w + 2r + 7t_0 \frac{f_{y0}}{f_{yi}}$ But for T, Y, X and K and N joints with gap : $p_{eff} \leq b_i+h_i-2t_i$ or $p_{eff} \leq 0{,}5\pi(d_i-t_i)$ But for K and N overlap joints: $p_{eff} \leq b_i$ or $p_{eff} \leq d_i$ $b_{e,ov} = \left(\frac{10}{b_j/t_j}\right)\left(\frac{f_{yj}t_j}{f_{yi}t_i}\right)b_i$ but $b_{e,ov} \leq b_i$ $d_{e,ov} = \left(\frac{12}{d_j/t_j}\right)\left(\frac{f_{yj}t_j}{f_{yi}t_i}\right)d_i$ but $d_{e,ov} \leq d_i$	
DE 110		7.6	Table 7.21	ed	For consistency with other tables, the indices “o“ and “f“ in the figures should be converted to “0“ (zero). Also for consistency in all equations, t_f should be changed to t_0	In all the figures, for consistency, all indices with “o” and “f” should be replaced by “0” (zero). In all equations, change t_f to t_0	A
DE 111		7.6	Table 7.21	ed	For consistency with Table 7.12 for chord shear, A_v should be replaced by $A_{v,0}$, to imply shear area in the gap region of K joints.	Replace “A_v“ by $A_{v,0}$ in the 1st equation in the 8th row, left box, as well as 3 times in the 11th row, right box.	A
DE 112		7.6	Table 7.21	ed	For consistency with Table 7.12 for chord shear, V_{Ed} should be replaced by $V_{0,Ed}$, to imply shear area in the gap region of K joints	Replace V_{Ed} by $V_{0,Ed}$, in the 11th row, right box, in the equation for $N_{0,Rd}$	A
DE 113		7.6	Table 7.21	ed	The note for CHS braces (last row) needs to be extended for overlap braces and including $d_{e,ov}$	In the last row, the note for CHS braces should read: For CHS braces multiply the above resistances for brace failure by $\pi/4$ and replace both b_1 and h_1 by d_1, both b_2 and h_2 by d_2, both b_i and h_i by d_i, both b_j and h_j by d_j, except for chord shear, and replace $b_{e,ov}$ by $d_{e,ov}$.	B
DE 114		7.6 (8)		ed	For consistency with Table 7.21, the index “f” should be converted to “0” (zero) in two places.	In 2 places in clause 7.6.8, replace t_f by t_0 .	A
		7.6	Table 7.22	ed	For consistency with Table 7.21, the index “f” should be converted to “0” (zero) in 3 places.	In 3 places, replace t_f by t_0 .	A

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Template for comments and secretariat observations

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DE 115		7.7	Table 7.23	ed	To consider also CHS braces, d_i / b_0 and d_i / d_j should be added after b_i / b_0 and b_i / b_j respectively.	In the 2nd row 2nd box, add d_i / b_0 In the 2nd row last box, add d_i / d_j	A
DE 116		7.7	Table 7.24	ed	For K and N joints, the limits for the ratio between CHS brace diameters is missing.	In the 4th row 7th (last) column, add: $d_i/d_j \geq 0{,}75$	B
DE 117		7.7	Table 7.24	ed	The indices used in the figures are not In accordance with Clause 1.5 (4). By definition, for K and N joints with gap, index “1” is for compression chord and index “2” for tension chord. On the other hand, index “ i ” is for the overlapping brace and index “ j ” for the overlapped brace. Therefore, the first figure (3rd row, left box) should be amended.	In the 3rd row, left box, for K and N joints with gap: Replace the indices “ i “ by “ 1 “ and “j” by “ 2 “	A
DE 118		7.7	Table 7.24	te	The effective width for CHS braces is missing.	In the 12th row, right box, after the equation for $b_{e,ov}$ for RHS braces, add: $$d_{e,ov} = \left(\frac{12}{\mathrm{d_j/t_j}}\right)\left(\frac{f_{yj}t_j}{f_{yi}t_i}\right)d_i \quad \text{but} \leq d_i$$	B
DE 119		7.7	Table 7.24	te	For consistency with other tables and clarity, the title “Chord failure“ should be reworded more precisely as proposed.	In the 4th row, right box, replace “Chord failure” by: Chord shear failure	A
DE1 20		7.7	Table 7.24	te	For consistency with Table 7.10 and Table 7.12 for chord shear, A_v should be replaced by $A_{v,0}$, to imply shear area in the gap region of K joints.	Replace “A_v“ by $A_{v,0}$ 3 times in the 5th row, right box, as well as 2 times in the 6th row, left box.	A
DE 121		7.7	Table 7.24	ed	For consistency with Table 7.10 and Table 7.12 for chord shear, V_{Ed} should be replaced by $V_{0,Ed}$, to imply shear area in the gap region of K joints	Replace V_{Ed} by $V_{0,Ed}$, in the 5th row, right box, in the equation for $N_{0,Rd}$	A

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DE 122		7.7	Table 7.24	ed	The note for CHS braces (last row) needs to be extended for overlap braces and including $d_{e,ov}$ Also, chord failure replaced by chord shear.	In the 2nd last row, the note for CHS braces should read: For CHS braces multiply the above resistances for brace failure by $\pi/4$ and replace both b_1 and h_1 by d_1, both b_2 and h_2 by d_2, both b_i and h_i by d_i, both b_j and h_j by d_j, except for chord shear, and replace $b_{e,ov}$ by $d_{e,ov}$.	B

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Do any clauses require editorial or technical correction?							
BE1		1.5(1)			The symbols $d_{o,t}$ and $d_{o,v}$ are not used.	These symbols should be deleted.	B
BE2		3.4.1(1)	Table 3.2	te	It is not logical to check the bearing resistance and the net section of shear connections category C. However, if it is decided to maintain the verification of the bearing resistance, a verification of the shear resistance of the bolts should be added. In this case the design applied load should be reduced for both checks).		B
BE3		3.6.1(3)		ed	The references to EN 1090 are not correct.	Reference should be made to EN 14399.	B
BE4		3.9.1(1)	Table 3.7 Note 4	ed	The expression “painted surface treatments” is too specific and should be enlarged.	Remove “painted surface treatments” and replace by “protection against corrosion”.	B
BE5		4.5.3.3(3)	(4.4)	te	It is strange that $f_{vw.d}$ for steel S420 is lower than $f_{vw.d}$ for steels S355 and S460?	Recent test results are available and should be evaluated.	D
BE6		4.14	Table 4.2	te	Clause 4.14 is related to the weldability of cold formed zones. Table 4.2 shall always be applied for tubes according to EN 10219. For tubes according to EN 10210 one should distinguish between hot-rolled seamless tubes, normalized tubes (for which table 4.2 is not applicable) and stress relieved tubes (for which table 4.2 is applicable). The corrigendum to clause 4.14 gives information about which tubes according to EN 10219 can be assumed to satisfy the limits mentioned in table 4.2. There is no logical relation between EN 1993-1-8 and the product standards EN 10210 and EN 10219.	A coordination between CEN/TC250/SC3 and ECISS/TC10 is necessary. If an exception to clause 4.14 is accepted for tubes according to EN 10219, we would expect a similar exception for stress relieved tubes according to EN 10210.	B

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BE7		5.2.2.5(1)	Figure 5.4	Te	Intellectually speaking, it is nice to have a stiffness boundary for rigid joints and a second one for pinned joints. Practically speaking, one can wonder who will ever use the stiffness boundary for nominally pinned joints: • The joints which could be concerned use fin plates, header plates, web cleats …. A laboratory test is the only way nowadays (except few models published in the literature) to evaluate the rotational stiffness of these joints. And Part 1-8 provides no recommendations for such joints. So why to give a boundary which has, in fact, to be compared to a value (the actual stiffness of the joint) which is unknown! • This value (0,5 *EI/L*) has no clear and well-established background, besides its "non useful" character. • In ECCS publication n°126, it has been shown that the above-mentioned connections (assumed to be nominally pinned by all the users, whatever the country), would be in reality semi-rigid, if their actual stiffness would be compared to 0,5 *EI/L*. And in this publication, it has been demonstrated that it is not at all a problem to consider these joints as pinned (even if they are semi—rigid according to Fig. 5.4) as long as they possess a sufficient ductility. And the publication provides guidelines to ensure this "sufficient rigidity".	Remove the "pinned" stiffness boundary from Part 1-8 and introduce the definition of nominally pinned joints according to ECCS n°126.	B
BE8		5.2.2		Te	A joint is rigid if $S_{j,ini}$ is higher than a given boundary ($25\ EI/L$ or $8\ EI/L$). So, in fact, the designer performs the frame analysis by referring to $S_{j,ini} = \infty$ while, in reality $S_{j,ini} \neq \infty$. This is not a problem: the plastification criterion ensures that the "mistake" is small in terms of displacements,	Introduce stiffness classification for semi-rigid joints in complement to the one suggested for rigid joints according http://hdl.handle.net/2268/30506, page 110.	B

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					internal forces, failure loads of the structure. If a designer wants to benefit from semi-rigid joints, he can select a joint, evaluate its actual stiffness and analyse the structure accordingly. Let's assume that the fabricator, for any good reason, wants to modify the detailing of the joint; in fact, as long as the actual stiffness of the joint which is realised on site is "not so far" from the value used initially by the designer, it will also not be a problem, for the same reason that the one expressed just above. In http://hdl.handle.net/2268/30506, boundaries indicating "how much" the stiffness of the actual joint may defer from the one used in the design without affecting too much the structural response have been derived (similar background that for 25EI/L or 8EI/L for rigid joints). These boundaries should be introduced in Part 1-8.		
BE9		5.2.3		Te	Part 1-8 is mainly written to cover joints subjected to bending moments. But it also deals with joints subjected to M+N+V. So why to restrict the concept of partial/full strength joints to joints subjected to bending moments? The definition of partial/full strength joints should therefore be extended to all loading situations. How? Two possibilities exist: • speak in terms of loading on the joints and not only in terms of moments • keep the present formulation but refer to $M_{j,N\text{-}V,Rd}$ bending resistance (bending resistance in presence of a possible axial and/or shear force).	Extend the scope of 5.2.3 to any loading situation. BE would suggest to adopt the second proposal expressed in the box at the left side of the present box.	D
BE10		6.2.4.1(5)		te	A formula how to calculate the values of the prying forces is missing. This is useful in view of the application of clause 6.2.2(2).	Such a formula exists in the literature and should be added.	B
BE11		6.4.2(1) and	(1)	te	The limit of 69ε has been kept in the ENV to EN	69ε ➔ 72ε/η together with a single definition of	B

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		6.2.6.1(1)			conversion process of Part 1-8 while it has been changed into $72\varepsilon/\eta$ in Part 1-1. Should not the consistency between both parts be kept? On the other hand, in Part 1-1 the web height is the distance between the flanges while, in Part 1-8, dc is the clear depth of the column web!	the web height?	
BE12		6.2.8.3		Te	First of all, it should be stated that this table is there to cover cases with proportional loading. This being, it may be said (bottom of Table 6.7) that $e=M_{j,N,Ed}/N_{j,M,Ed} = M_{j,N,Rd}/N_{j,M,Rd.}$	Amend 6.2.8.3(1) accordingly and define $M_{j,N,Rd}$ and $N_{j,M,Rd}$	B
BE13		6.2.8.3	Table 6.7 and 6.2.8.3(1)	Te	In the legend, speak about $M_{j,N,Rd}$ instead of $M_{j,,Rd}$ (and the same for N resistance and applied force)	$M_{j,,Rd}$ → $M_{j,N,Rd}$ (or M_{Rd} at the botton of Table 6.7) N_{Rd} → $N_{j,M,Rd}$ (bottom of Table 6.7) N_{Ed} → $N_{j,M,Ed}$ (Table 6.7) M_{Ed} → $M_{j,N,Ed}$ (bottom of Table 6.7)	B
BE14		6.2.8.3	Table 6.7	Te	The definitions of the sign for N_{Ed} ($N_{j,M,Ed}$) and M_{Ed} ($M_{j,N,Ed}$) should be kept as they are. But the formulae contained in Table 6.7 are wrong!	Formulae in Table 6.7 to be corrected (contact J.P. Jaspart who re-derived exact formulae in preparing the ECCS Manual on Joints in Steel Structures)	B
BE15		6.3.2	Table 6.11 (components "bolts in shear" and "bolts in bearing")	Te	Similarly to what has been done in the corrigendum to Table 6.2 (for resistance), n_b should be defined as the number of bolts rows, but with two bolts per row!	n_b is the number of bolt-rows (with two bolts per row).	B
BE16		6.1.3	Table 6.1	Te	In the last column of the Table, it is referred to "rotation capacity". In reality, this has no sense for a component (and the Table refers to "components"). One should so prefer the term "deformation capacity" as a component is subjected to tension, compression or shear forces. In this last column, references are made (when any) to section 6.4 where … recommendations are related to the **rotation capacity of joints** and not to the **deformation capacity of**	Remove the last column in Table 6.1 as the latter refers to components and not to joints. State somewhere in the document that nowadays provisions are only provided in section 6.4 for the rotation capacity of joints in bending.	B

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					components!!!		
BE17		6.2.6.4	Tables 6.4, 6.5, 6.6		A strict application of these tables may lead to unconservative results in the case of non equidistant bolt rows.	Add this note.	B
BE18		6.2.6.4	Table 6.5 – Box "individual bolt-row / non-circular patterns / end bolt-row adjacent to a stiffener"		**FIRST COMMENT** In Table 6.5, e_1 is used. e_1 is the distance between the considered bolt-row and the top of the column. In the corrigenda, we thought that e_1 would be defined in Figure 6.9. But now when we refer to the latest version of the corrigenda we have (EN 1993-1-8:2005/AC dated July 2009), it appears that e_1 is defined as "the distance from the centre of the fasteners in the end row to the adjacent stiffener of the column flange measured in the direction of the axis of the column profile (see row 1 and row 4 in Figure 6.9)". This is not at all correct. So where does this "correction" come from? In fact, e_1, as it is defined for Table 6.4, in the corrigenda, is exact. And the same definition applies for Table 6.5 **SECOND COMMENT** In Table 6.5, in the box defined before, the value of l_{eff} is not correct. In fact, one should have the following: l_{eff} = the smaller of " $\alpha * m$ " and "$e_1 + \alpha * m - (2m + 0,625e)$"	Good definition of e_1 to be given in Figure 6.9 and in the corrigenda Make correction of formula in Table 6.5 Box "individual bolt-row / non-circular patterns / end bolt-row adjacent to a stiffener" ➔ l_{eff} = the smaller of "$\alpha * m$" and "$e_1 + \alpha * m - (2m + 0,625e)$"	B
BE19		6.2.6.7	(1),(2) and (3)	Te	BE has questions about EN 1993-1-8 6.2.6.7(1) on "Beam flange and web in compression". And more especially on the following sentence: *If the height of the beam including the haunch exceeds 600 mm the contribution of the beam web to the design compression resistance should be limited*	The proposal from BE would be to apply Formula 6.21 without any limitation to 600 mm beam depth. For haunches, the formula 6.21 should be adapted to account for the haunch inclination.	D

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					to 20%. As far as one knows: • In different countries, and more particularly in UK, it was usual to take as a reference the resistance of the flange only (and also by assuming that the flange was fully efficient, i.e. class 1 or class 2). This resistance may write: $A_f.f_y$ • In UK, this resistance was increasing to account for strain hardening (20%) and for a contribution of the web, so leading to a resistance of: $1{,}4.A_f.f_y$. One will not refer here to the alternative method which consists in taking the area of the flange and of a part of the web, modifying the position of the compression centre and increasing the resistance by 20% only to account for strain hardening. Anyway, in the green books, it was (is?) clearly stated that the flange should belong to class 1 or 2. • When writing EN1993 Part 1-8, the formula which is presently proposed to the designers was suggested (Formula 6.21). Long discussions took place about that in TC10 but finally it was agreed that: ○ the UK approach and the Part 1-8 formulae were leading to similar results for class 1 or class 2 sections; ○ the UK rule was not fitted for class 3 or class 4 profiles (especially by the flange); ○ the 1-8 rule was more adequate, as it explicitly integrates the reduction of the resistance resulting from a partial efficiency of the web and/or the flange (in fact, one had initially worked on this formula at Liège to satisfy the request of a fabricator		

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					who only fabricates very slender built-up welded sections. Now a sentence has been added in the code to restrict the general rule for profiles lower than 600 mm. Clarifications would be required: • One may assume that the "20%" originates from UK . Is it right? • One may assume that the meaning of the sentence is the following: one starts by evaluating the design resistance of the flange only, and then one increases the result by 20%. These 20% could be those representing the contribution of the web. It seems that the "other 20% " introduced in UK to account strain hardening are not applied. Probably because of the fact that high profiles could not be class 1 or class 2 and therefore that strain hardening is not likely to develop. Is it a good interpretation? • For profiles where the flange in compression is class 4, should the design resistance of the flange only be evaluated as $A_{eff}.f_y$ instead of $A_f.f_y$? Should also the 20% be applied in this case? This being, one has difficulties to see where this "600 mm" rule comes from. In fact, the classes of the sections and the height of the sections are not directly related parameters. On the other side, why, for a class 1 section with a height of more than 600 mm, could not the user refer to the general rule (this is just a particular case, but one could mention many others)? Finally, how should EN1993-1-8 6.2.6.7(1) be applied for a profile higher than 600 mm: do we have to consider the minimum value respectively obtained by the general rule and the 20% rule, or does the general rule is limited to profiles with a height		

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					lower than 600 mm? Besides that, no background seems to be available for cases where haunches are used. In particular, the inclination of the haunch seems to have no influence on $F_{c,fb,Rd}$, what looks strange!		
BE20		6.4.1(3)		Te	Let's assume, for instance, a single sided joint beam-to-column joint (case a in 5.2.3.3(2)) where bending resistance of the column cross-section $M_{c,pl.Rd}$ (with due account of the possible axial force) is lower than the bending resistance of the beam $M_{b,pl,Rd}$. Then a full strength joint is such that $M_{j.Rd}$ is higher or equal to $M_{c,pl.Rd}$; and not higher or equal to the resistance of the connected member (here, $M_{b,pl,Rd}$), as indicated in 6.4.1(3). So a joint does not require any rotational ductility if its resistance is higher or equal to 1,2 times the full strength resistance as defined in 5.2.3.3.	Replace "the design plastic resistance of the cross section of the connected member" by "the full-strength resistance of the joint", as defined in 5.2.3.3. Or, as an alternative, write in Clause 6.4.1(3" "…… the design resistance of the cross-section of the connected members". The first way is preferred. In these proposals above, it has to be pointed out that • no specific reference is made to the loading of the joint (bending/ bending or axial force) • no specific reference is made to the class of the cross-section by speaking about "design resistance" and not about "plastic design resistance).	B
BE21		6.4.1(3), 6.2.3(5) and 1.4.4		Te	The rotation capacity of a joint has not to be checked when the condition specified in Clause 6.4.1(3) is satisfied. In 6.2.3(5), for a partial-strength joint under bending in which rotation capacity is required, the conditions to design the welds are specified. Let's assume a particular case in which $M_{j,Rd}$ equals $M_{pl,Rd}$. If one would like to see the hinge to form in the beam, then the welds should resist to $M_{pl,Rd}$ (according 6.2.3(5)) while the resistance of the joint should be increased to 1,2 $M_{pl,Rd}$ (according 6.4.1(3)). This means that different conditions apply to the welds and to the joints.	Consistency to be realised between clauses 6.4.3(3), 6.2.3(5) and 1.4.4.	B

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					And therefore that "the welds" are not part of "the joint". But if ones refers to Clause 1.4.4, the welds are included in the definition of the joint (one of the components used to transfer the forces between the connected members – component 19 in Table 6.1)).		
BE22		6.2.3(4)		Te	In this clause, it is stated that: *In all joints, the sizes of the welds should be such that the design moment resistance of the joint* $M_{j,Rd}$ *is always limited by the design resistance of its other basic components, and not by the design resistance of the welds.* This looks surprising. Should we need rotational ductility, for sure! But if not, why the joint resistance could not be limited by the resistance of the weld. Obviously, to respect 6.2.3(4) is a wise way to design and a sort of extra safety to avoid brittle failure in the case, for instance, of an not foreseen overloading of the structure or an approximation (well understandable) in the elastic distribution of forces within the structure. Then it is possible to benefit from plasticity and from related plastic redistributions. But it is surprising that the code imposes it; in BE, fabricators are reluctant to apply this clause "for all joints" and would require some more "flexibility" in using it. But why, then, not to do the same for bolted connections and avoid the development of Mode 3 failure modes in T-stub components who would limit the resistance of a joint.	Clause 6.2.3(4) should be somewhat relaxed by allowing some exceptions to its use : • first of all by writing " … is always limited by the design resistance of at least one of its basic components" • but also in other cases (elastic frame design process, for instance, where the designer could take the responsibility to associate the design resistance of the joint to the one of the welds). The first bullet point would allow to establish consistence with clause 5.9(5). The second bullet point would require to relax also the first sentence of clause 4.9(4). If 6.2.3(4) is kept, a similar rule should then be added to avoid Mode 3 failure modes in T-stub components (when these ones limit the global resistance of the joint only, obviously).	D
BE23		6.2.3(4) and 6.4.1(3)		Te	Again (as specified here above – comments to 6.4.1(3), 6.2.3(5) and 1.4.4)), the designer could feel an inconsistency between the two clauses if	Definition of "joints" and "welds" to be revisited (see two boxes above).	D

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					the "joint" includes the "welds".		
BE24		6.2.6.2	Formulae (6.13a) and (6.13b)	Te	In accordance with Part 1-5 (Formula (4.2) in this Part), BE would suggest to replace, in Formula (6.13b), the factor 0,2 by 0,22. As a consequence, when selecting Formula (6.13a) or formula (6.13b), the plate slenderness should be compared to 0,673 and not to 0,72.	Replace 0,72 by 0,673 in both formulae. Replace 0,2 by 0,22 in Formula (6.13b).	B
Which clauses would benefit from improvements in clarity?							
BE25		7.	Whole chapter 7	Ge/Te	Under the umbrella of CIDECT, the contents of Chapter 7 has been "converted" into a component format completely similar to the one for open sections in Chapter 6. It is now possible to merge Chapters 6 and 7 into a single unique format, without changing the results got by means of the present formulae included in Chapter 7. This would lead: • to an easier application of Chapter 7 (through a unique design approach common to the whole Part 1-8) • To extend de facto the scope of Part 1-8 to all joints between I, H or tubular sections.	Apply results of CIDECT research 16F ➔ rewording of Chapter 7 and integration of Chapter 6 and 7 into a unique chapter with a similar background and type of application. And, as a direct result, an extension of the scope of Part 1-8. If CEN decides not to make these changes, then reference should be anyway made to the CIDECT research report 16F where mistakes actually contained in the present version of Chapter 7 of Part 1-8 are listed (these ones would be automatically corrected in the case of CEN would decide to reword Chapter 7 and merge it with Chapter 6 (as proposed above).	D
Where should the scope of the EN be extended?							

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BE26		6.1.3	Table 6.1	Ge/Te	Many new components could be added to the list so as to cover an even wider scope of application. Amongst them : circular bolted end plates , U-channels, column web plate in beam-to-column weak axis joints, hammerheads, endplates with four bolts per row, bolted joints of beams bended about the weak axis, bolted end plates of tubular sections. This material has been validated through various recent European researches. ECCS TC10 would be the right place to agree on this material which is nowadays at disposal.		D
BE27		Part 1-8	Part 1-8	Ge/Te	In the ECCS publication n° 126 devoted to the design of simple joints, European design recommendations validated in ECCS TC10 are presented. This material should be made available to designers through Part 1-8.		D

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Where could the EN by shortened?							
Are there any clauses whose application leads to uneconomic construction?							
Are there any clauses whose application necessitates excessive design effort?							

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