Kommende ændringer til EN 1993-1-1 og -1-8 mm.

Eurocodes EC3 og EN1090 - Hvad er nyt og hvad er på vej

Carsten Munk Plum
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Formand for S-1993

For udførelsesstandarderne DS/EN 1090 er tidsplanerne:

Udgivet i år (kun stålrelevante dele):
DS/EN 1090-2 og DS/EN 1090-4

Vi mangler DS/EN 1090-1, men den er på vej i høring.
Det er ikke helt klart hvorledes de eksisterende certificeringer til CE-mærkning korresponderer med de nye regler.

Justeringer:

Alle krav i den gamle udgave af DS/EN 1090-2, som kun vedrørte tyndplader, er blevet overført til DS/EN 1090-4. Denne er endvidere suppleret med nogle yderligere krav.

En væsentlig ændring er, at EXC4 ikke umiddelbart indeholder en skærpelse i forhold til EXC3. En mulig skærpelse er betinget af projektspecifikke krav. Dette kunne tænkes at være relevant for udmattelsespåvirkede konstruktionsdele.
Desuden er der ingen ”default” udførelsesklasse mere (tidligere EXC2). Den skal specificeres i udførelsesdokumenterne.

Der er én skærpelse ved valg af EXC4. For svejsekoordinator kræves altid en ”omfattende viden”, hvor dette i EXC3 kun kræves for S420 til S700.

7.6.1 Routinemæssige godkendelseskriterier

Svejste komponenter skal opfylde kravene i pkt. 10 og 11.

Godkendelseskriterierne for svejsefejl skal være som følger med henvisning til EN ISO 5817, med undtagelse af "Forkert overgang mellem overflade af grundmetal og svejsesøm" (505) og "Mikrobindingsfejl" (401), der ikke skal tages i betragtning. Der skal tages hensyn til yderligere specificerede krav til svejsningens geometri og profil.

– EXC1 kvalitetsniveau D , men kvalitetsniveau C for ”utilstrækkeligt a-mål” (5213)

– EXC2 generelt kvalitetsniveau C med undtagelse af kvalitetsniveau D for ”Overløbning af svejsemetal" (506), "Tændsår" (601) og "Åben kraterpore" (2025)

– EXC3 kvalitetsniveau B

– EXC4 kvalitetsniveau B suppleret med yderligere opgavespecifikke krav, fx. aht. udmattelsespåvirkning

7.6.2 Godkendelseskrav mht. udmattelsespåvirkede konstruktionsdele

I DS/EN ISO 5817, anneks C, er anført yderligere specifikationer af grænseværdier for fejl afhængig af udmattelseskategori.

Sammenhængen mellem udmattelseskategori (se DS/EN 1993-1-9) og de i annekset anførte kvalitetsniveauer er:

Udmattelseskategori op til og med 63:	kvalitetsniveau C 63
Udmattelseskategori over 63 og op til og med 90:	kvalitetsniveau B 90
Udmattelseskategori over 90 og op til og med125:	kvalitetsniveau B 125

Der henvises i øvrigt til supplerende krav med hensyn til udførelse, som er anført i DS/EN 1993-1-9 og DS/EN 1993-2.

Det skal fremhæves, at ovennævnte gælder for udmattelsespåvirkede dele, hvor bæreevne/levetid er bestamt af kravene i DS/EN 1993-1-9.
Der vil i udmattelsespåvirkede konstruktioner være mange svejsesømme, som ikke er dimensioneret af udmattelseskriterierne. Disse bør man ikke udpege til særlige ekstra undersøgelser.

Tabel C.1 – Supplerende krav til tabel 1 for svejsninger påvirket af udmattelseslast

Nr.	Reference til ISO 6520-1	Fejlbetegnelse	t mm	Grænseværdier for fejl ved kvalitetsniveauer C 63[c]	B 90[c]	B 125
1.5	401	Mikrobindingsfejl	≥ 0,5	[a]	[a]	[a]
1.7	5011 5012	Kontinuert sidekærv Lokal sidekærv	> 3	[a]	[a]	Ikke tilladt
1.8	5013	Krympefuge (rodkærv)	> 3	[a]	[a]	Ikke tilladt
1.9	502	Overvulst (stumpsøm)	≥ 0,5	[a]	[a]	$h \leq$ 0,2 mm + 0,1 b, maks. 2 mm
1.10	503	Konveks sømoverflade (kantsøm)	≥ 0,5	[a]	[a]	[b]
1.11	504	Gennemløb	0,5 til 3	[a]	[a]	$h \leq$ 0,2 mm + 0,05 b
			> 3	[a]	[a]	$h \leq$ 0,2 mm + 0,05 b, men maks. 1 mm

[a] Samme værdier som angivet for hhv. kvalitetsniveau B og C i tabel 1.

[b] Ikke defineret.

[c] Værdier identiske med IIW-dok. XIII-2323-10. Værdierne er godkendt af IIW til en materialetykkelse på 10 mm og derover. Mindre materialetykkelser kan være gældende.

[d] Fejlgrænsen svarer til forholdet mellem summen af de forskellige poreområder og vurderingsområdet. Hvis afstanden mellem to poreområder er mindre end diameteren af det mindste poreområde, er en indhyllingskurve, der omgiver begge poreområder, relevant som ét fejlområde. Hvis afstanden mellem to porer er mindre end diameteren af en af de nærliggende porer, skal hele det pågældende område for de 2 porer udgøre summen af fejlområder.

Table 24 —Extent of routine supplementary NDT

Type of weld	Shop and site welds		
	EXC1	EXC2	EXC3 [a]
Transverse butt welds and partial penetration welds in butt joints:	0 % [b]	10 %	20 %
Transverse butt welds and partial penetration welds:			
— in cruciform joints	0 % [b]	10 %	20 %
— in T joints	0 %	5 %	10 %
Transverse fillet welds [c]:			
with $a > 12$mm or $t > 30$ mm	0 %	5 %	10 %
with $a \leq 12$mm and $t \leq 30$ mm	0 %	0 %	5 %
Full penetration longitudinal welds [d] between web and top flange of crane girders	0 %	10 %	20 %
Other longitudinal welds [d], welds to stiffeners and welds specified in the execution specification as being in compression	0 %	0 %	5 %

a For EXC4 the percentage extent shall be at least that given for EXC3.

b 10 % for such welds executed in steel ≥ S420.

c Terms *a* and *t* refer respectively to the throat thickness and the thickest material being joined.

d Longitudinal welds are those made parallel to the component axis. All others are considered as transverse welds.

Table L.1 — Guidance on a method for selection of weld inspection class

Level of fatigue utilization [a]	Consequences from failure of joint or component [c]	Stress in weld [b]	Weld Inspection Class (WIC)
High fatigue utilization	Substantial [b]	Welds with the direction of dynamic principal stress transverse to the weld (between 45° and 135°)	WIC5
		Welds with the direction of dynamic principal stress in the direction of the weld (between −45° and +45°)	WIC4
	Not substantial [c]	Welds with the direction of dynamic principal stress transverse to the weld (between 45° and 135°)	WIC3
		Welds with the direction of dynamic principal stress in the direction of the weld (between −45° and +45°)	WIC2
No fatigue (i.e. quasi-static) or **Low** fatigue utilization	Substantial [b]	Welds with high [d] tensile stresses transverse to weld	WIC5
		Welds with low tensile stresses transverse to weld and/or high [d] shear stresses	WIC4
	Not substantial [c]	For welds in EXC3 or EXC4 with high [d] tensile stresses transverse to weld	WIC3
		All other load-bearing welds except welds in EXC1	WIC2
		Welds in EXC1 and non-load-bearing welds	WIC1

a Low fatigue utilization means connection with calculated fatigue life longer than 4 times the required fatigue life.

b Substantial consequences means that the failure of the joint or member will entail:

- possible multiple loss of human life; and/or;
- significant pollution; and/or;
- major financial consequences.

c The consequences may be assessed as Not substantial if the structure has been provided with sufficient residual strength to meet specified accidental actions.

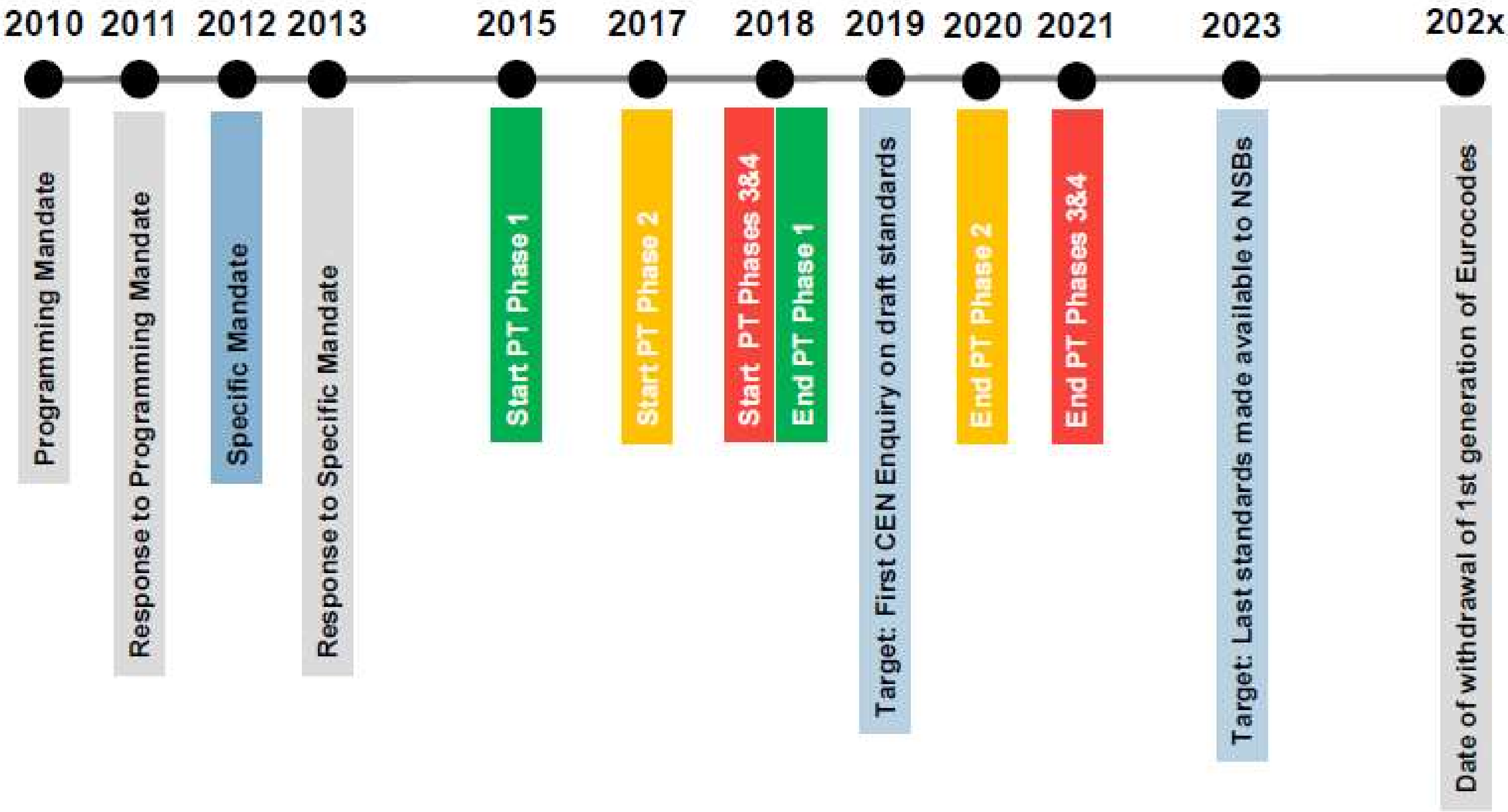
d High stresses are those that (quasi-)static stresse that exceed 50 % of the welds tensile or shear capacity, as appropriate. Low stresses conversely. Special consideration should also be given to the selection of WIC where the principal stress is in the through-thickness direction of the parent material.

Table L.2 — Percent extent of supplementary testing according to WIC

Weld Inspection Class (WIC)	Type of joint	RT	UT	MT/PT
WIC5	Full penetration in-line butt weld	10	100	100
	Full penetration T-butt weld	0	100	100
	Partial penetration welds with penetration depth greater than 12 mm	0	20	100
	Other partial penetration welds and all fillet welds	0	0	100
WIC4	Full penetration in-line butt weld	5	50	100
	Full penetration T-butt weld	0	50	100
	Partial penetration welds with penetration depth greater than 12 mm	0	10	100
	Other partial penetration welds and all fillet welds	0	0	100
WIC3	Full penetration in-line butt weld	0	20	20
	Full penetration T-butt weld	0	20	20
	Partial penetration welds with penetration depth greater than 12 mm	0	5	20
	Other partial penetration welds and all fillet welds	0	0	20
WIC2	Full penetration in-line butt weld	0	10	10
	Full penetration T-butt weld	0	10	10
	Partial penetration welds with penetration depth greater than 12 mm	0	5	5
	Other partial penetration welds and all fillet welds	0	0	5
WIC1	All joint types	0	0	0

supplementary NDT
supplementary non-destructive testing
NDT technique which is supplementary to visual examination (VT), e.g. magnetic particle (MT), penetrant (PT), eddy current (ET), ultrasonic (UT) or radiographic testing (RT)

Preliminary timeline for Eurocodes



CEN/TC 250 Meeting - Oslo – 24-25 May 2018 104

Final Document of prEN 1993-1-1

Date of document	2018-07-03
Expected action	Next Meeting
Due Date	2018-10-25

Background

Dear Member,

Please find attached the final document prEN 1993-1-1 from SC 3_PT 1. According to Decision 09/2017 taken by CEN/TC 250/SC 3 on 2017-10-12 a final technical approval should be agreed at the next SC 3 meeting in October. After this any technical amendments need to be discussed with WG 1 and decided by SC 3. An editorial improvement is still going on.

Table 5.1 — Nominal values of yield strength f_y and ultimate tensile strength f_u for structural steels conforming to the following standards: EN 10025, EN 10210 and EN 10219

Steel grade a)	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		$40 \text{ mm} < t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
S 235	235	360	215	360
S 275	275	390	245	370
S 355	355	490	325	470
S 420	420	510	390	490
S 460	460	540	410	510
S 500	500	580	450	580
S 550	550	600	500	600
S 600	600	650	550	650
S 620	620	700	560	660
S 690	690	770	630	710
a) Principal symbols in EN 10027-1.				

De markerede og nye stål vil ikke umiddelbart kunne antages at opfylde kravene til duktilitet – anvendelse til plastisk design.

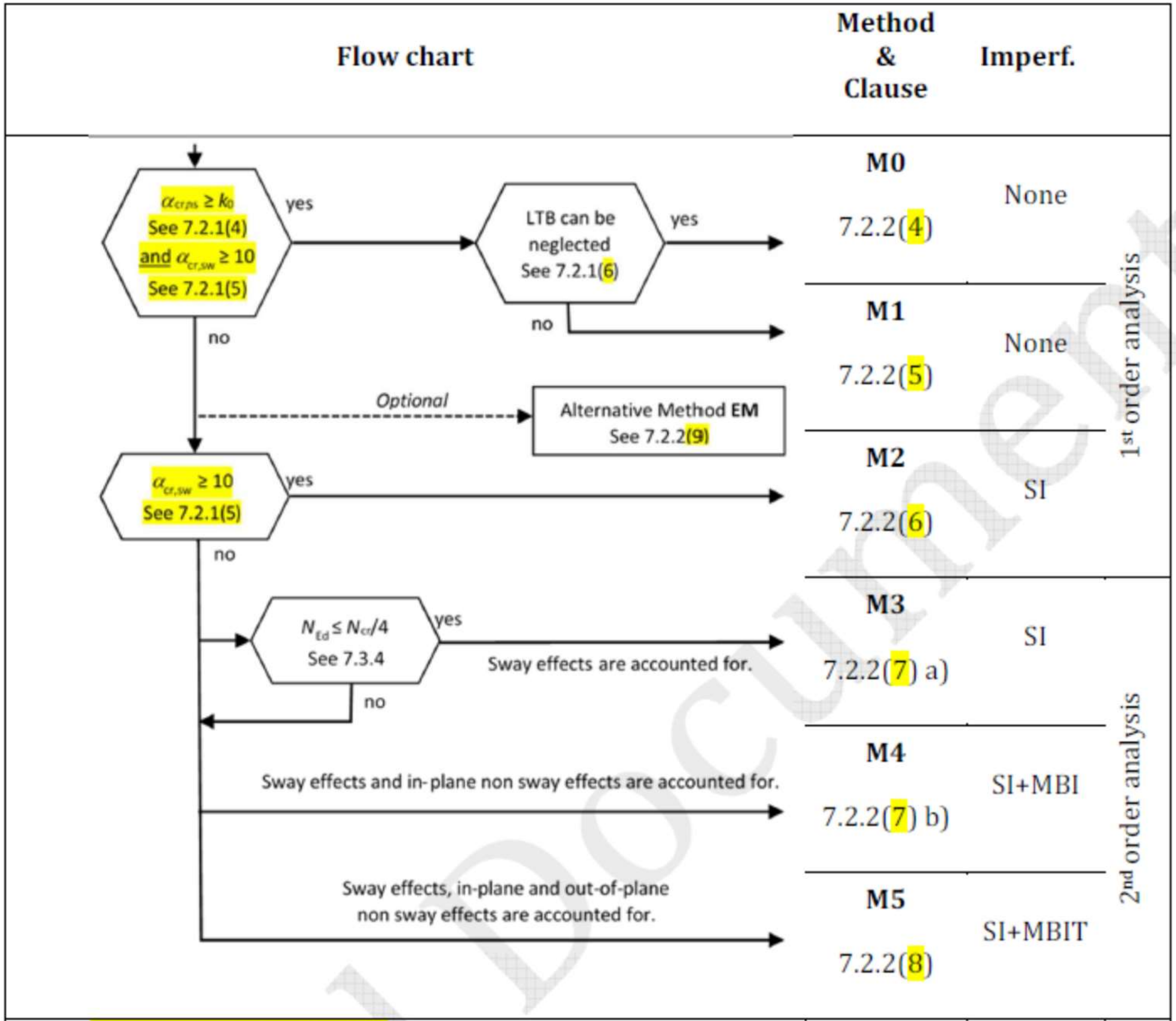
Ny note om træk vinkelret på pladetykkelsen

(3) The selection of material quality Class should be in accordance with Table 5.3.

NOTE Particular care should be given to welded beam to column connections and welded end plates with tension in the through-thickness direction.

Table 5.3 — Choice of quality Class according to EN 10164

Target design value Z_{Ed} according to EN 1993-1-10	**Quality Class according to EN 10164 (Available design Z-value Z_{Rd})**
$Z_{Ed} \leq 10$	—
$10 < Z_{Ed} \leq 20$	Z 15
$20 < Z_{Ed} \leq 30$	Z 25
$Z_{Ed} > 30$	Z 35



Keys:

LTB	Lateral torsional buckling
SI	Sway imperfection
MBI	Member bow imperfection (in-plane)
MBIT	Member bow imperfection including torsional effects (in-plane and out-of-plane)

Figure 7.3 — Methods of structural analysis applicable to ultimate limit state design checks

Eksempel på beskrivelse af beregningsprincip

(6) Method M2:

If second order effects due to member buckling may not be neglected (i.e. the criterion in 7.2.1(4) is not satisfied) but second order effects due to in-plane, global (sway) buckling may be neglected (i.e. the criterion in 7.2.1(5) is satisfied):

— the verification of the cross-sectional resistance according to 8.2 may be based on first order internal forces and moments;

— the in-plane and out-of-plane verification of the buckling resistance of individual members according to 8.3 is required and may be based on first order internal forces and moments considering appropriate non-sway buckling lengths and corresponding bending moment diagrams;

— the global analysis may neglect equivalent bow imperfections but should consider global (sway) imperfections;

— the buckling length may conservatively be assumed to be equal to the system length.

Ny norm for anvendelse af FEM

7.4 Methods of analysis considering material non-linearities

7.4.1 General

(1) The internal forces and moments may be determined using either

a) Elastic global analysis or

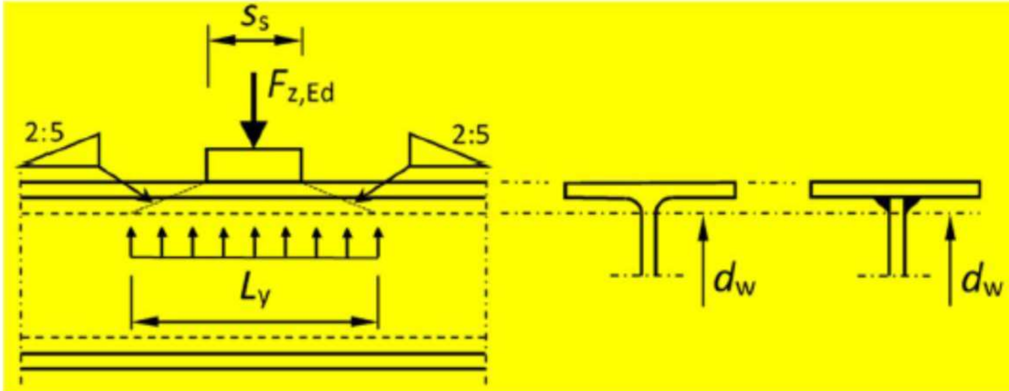
b) Plastic global analysis

NOTE For finite element model (FEM) analysis, see EN 1993-1-14.

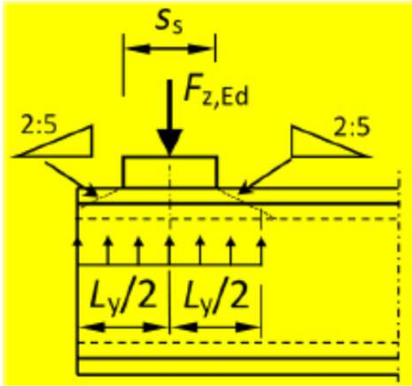
(2) Elastic global analysis may be used in all cases.

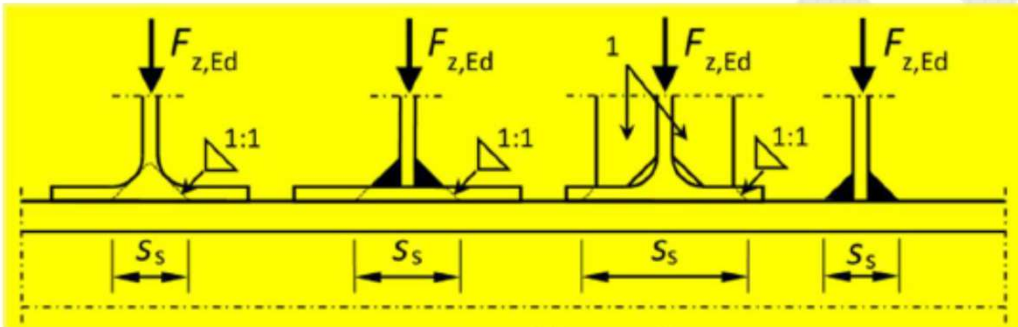
Koncentreret tryk på flange overført fra EN 1993-1-5 til EN 1993-1-1

Case 1



Case 2
End zone





Nye regler for torsionsudknækning

8.3.1.4 Buckling reduction factors for torsional and torsional-flexural buckling

(1) For members with open cross-sections the resistance of the member to either torsional or torsional-flexural buckling may be less than its resistance to flexural buckling. This is relevant for members with doubly symmetric cross-sections in combination with intermediate lateral restraints, as well as for cruciform and asymmetric cross-sections, e.g. channel, L, T, and I- or H- sections with single symmetry.

$$\chi_{TF} = \frac{1}{\Phi_{TF} + \sqrt{\Phi_{TF}^2 - \bar{\lambda}_{TF}^2}} \quad \text{but } \chi_{TF} \leq 1{,}0 \qquad (8.73)$$

where:

$$\phi_{TF} = 0{,}5\left[1 + \left(\frac{\bar{\lambda}_{TF}}{\bar{\lambda}_z}\right)^2 . \alpha_{TF}(\bar{\lambda}_z - 0{,}2) + \bar{\lambda}_{TF}^2\right] \qquad (8.74)$$

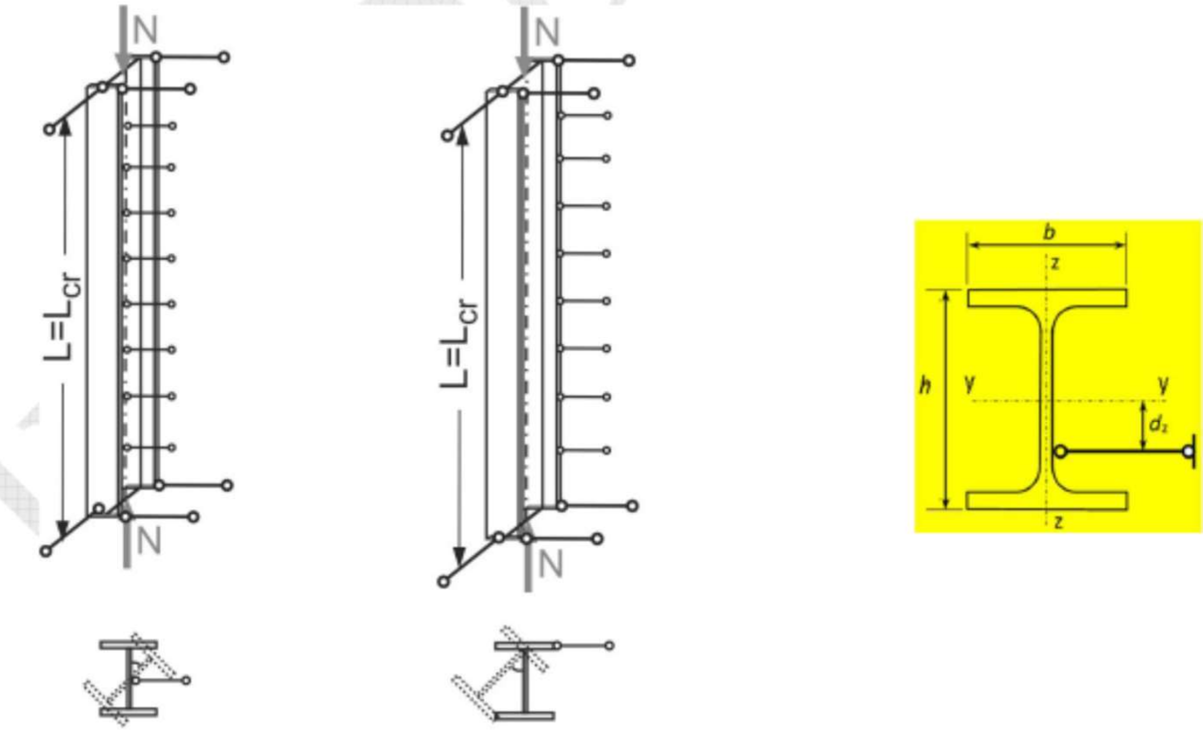


Figure 8.6 — Torsional-flexural buckling of laterally braced doubly symmetric sections - Geometric definitions

For kipning skal i fremtiden benyttes den almindelige søjlekurve, men med et afskæringsniveau ved slankheden 0.4 (anbefalet værdi).

Dette komme til at give et spring i reduktionsfaktoren ved 0.4.

8.3.2.3 Buckling reduction factors χ_{LT} for lateral torsional buckling

(1) Verification of lateral torsional buckling resistance may be neglected when the relative slenderness for lateral torsional buckling satisfies $\bar{\lambda}_{\mathrm{LT}} \leq \bar{\lambda}_{\mathrm{LT,0}}$ or when the design value of the bending moment satisfies: $M_{\mathrm{Ed}} \leq \bar{\lambda}^2_{\mathrm{LT,0}}\, M_{\mathrm{cr}}$.

NOTE 1 Limitations of validity concerning the beam depth or h/b ratio can be set by the National Annex for use in a country.

NOTE 2 $\bar{\lambda}_{\mathrm{LT,0}}$=0,4 if end-fork support conditions are assumed for the calculation of M_{cr} between lateral supports unless the National Annex gives a different value for use in a country.

(2) In general cases of prismatic members with arbitrary boundary conditions, the buckling reduction factor χ_{LT} may be determined by using the buckling formulae and curves of Formula (8.71) and Figure 8.5. The relative slenderness for lateral torsional buckling $\bar{\lambda}_{\mathrm{LT}}$ should be used for the relative slenderness $\bar{\lambda}$, and the selection of buckling curves as specified in Table 8.4 should apply.

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For dobbeltsymmetrisk tværsnit kan et mere kompliceret udtryk benyttes, som sandsynligvis vil give større bæreevne.

(3) For doubly symmetric I- and H- sections and fork boundary conditions at both ends, the buckling reduction factor χ_{LT} may be taken as:

$$\chi_{LT} = \frac{f_M}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - f_M \, \bar{\lambda}_{LT}^2}} \quad \text{but } \chi_{LT} \leq 1{,}0 \tag{8.79}$$

where:

$$\phi_{LT} = 0{,}5\left[1 + f_M\left(\left(\frac{\bar{\lambda}_{LT}}{\bar{\lambda}_z}\right)^2 \alpha_{LT}(\bar{\lambda}_z - 0{,}2) + \bar{\lambda}_{LT}^2\right)\right] \tag{8.80}$$

α_{LT} is the imperfection factor taken from Table 8.5;

$\bar{\lambda}_{LT}$ is the relative slenderness for lateral torsional buckling, as defined in 8.3.2.2;

$\bar{\lambda}_z$ is the corresponding relative slenderness for weak axis flexural buckling, as defined in 8.3.1.2, with the buckling length $L_{cr,z}$, taken as the distance between the discrete lateral restraints;

f_M is a factor that accounts for the effect of the bending moment distribution between discrete lateral restraints. It may conservatively be taken as 1,0 in cases that cannot be approximated by the diagrams in Table 8.6.

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Table 8.5 — Imperfection factor α_{LT} for lateral torsional buckling of doubly symmetric I- and H-sections

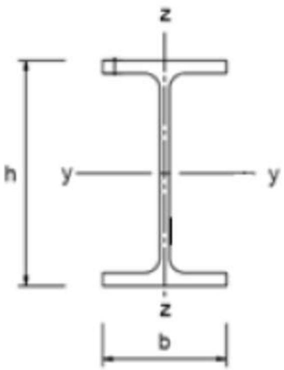
Cross-section		Limits		α_{LT}
Rolled I-sections		$h/b > 1,2$	$t_f \leq 40$ mm	$0,12\sqrt{\frac{W_{el,y}}{W_{el,z}}}$ but: $\alpha_{LT} \leq 0,34$
			$t_f > 40$ mm	$0,16\sqrt{\frac{W_{el,y}}{W_{el,z}}}$ but: $\alpha_{LT} \leq 0,49$
		$h/b \leq 1,2$	-	$0,16\sqrt{\frac{W_{el,y}}{W_{el,z}}}$ but: $\alpha_{LT} \leq 0,49$

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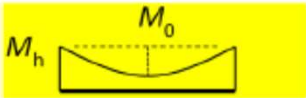
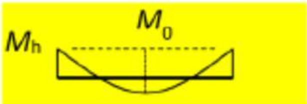
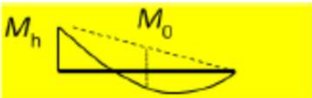
Table 8.6 — Factors f_M and k_c

Load case	Factor f_M	Factor k_c
M = uniform 	1,00	1,00
 -1 ≤ ψ ≤ +1	$1{,}25 - 0{,}1\,\psi - 0{,}15\psi^2$	$\dfrac{1}{1{,}33 - 0{,}33\psi}$
	1,05	0,94
 	For $0 \le \dfrac{M_0}{M_h} < 2{,}0$: $1{,}0 + 1{,}35\dfrac{M_0}{M_h} - 0{,}33\left(\dfrac{M_0}{M_h}\right)^3$ For $\dfrac{M_0}{M_h} \ge 2$: 1,05	$\dfrac{M_0}{M_h} < 1{,}0$: 1,00 $\dfrac{M_0}{M_h} \ge 1{,}0$: 0,90
	For $0 \le \dfrac{M_0}{M_h} < 1{,}47$: $1{,}25 + 0{,}5\left(\dfrac{M_0}{M_h}\right)^2 - 0{,}275\left(\dfrac{M_0}{M_h}\right)^4$ For $\dfrac{M_0}{M_h} \ge 1{,}47$: 1,05	$\dfrac{M_0}{M_h} < 0{,}5$: 0,75 $\dfrac{M_0}{M_h} \ge 0{,}5$: 0,91

Kombinationsformler for tryk og to-akset bøjning er uændrede og der benyttes den gamle metode 2 til bestemmelse af interaktions-konstanter.

Det er fortsat tilladt at benytte den gamle metode 1, idet den bliver beskrevet i en TS: ***CEN TS EN 1993-1-101***.

Der vil også blive udgivet en TS, som vil angive ”lærebogsstof” i form af formeludtryk for elastisk kritisk søjlelast henholdsvis elastisk kritisk kipningslast: ***CEN TS EN 1993-1-103***.

Det præcise indhold i disse to TS-ere er endnu ikke blevet præsenteret.

Nyheder i **normative** annekser:

I anneks B er en udglatning af overgangen mellem klasse 2 og 3 tværsnit med hensyn til bøjningspåvirkede tværsnit blevet indført.

I anneks C præsenteres nye kombinationsudtryk for elementer påvirket af tryk, to-akset bøjning samt **torsion.**

I anneks D præsenteres nye udtryk for virkning af sidefastholdelse og vridningsfastholdelse fra trapezpladebeklædning på bjælker. Trapez beklædning skal være udført i henhold til EN 1993-1-3.

I anneks E er anført værdier for middelværdi, variationskoefficient, karakteristisk værdi og designværdi for henholdsvis stålmaterialer og tværsnitsgeometri. De anførte værdier er blevet benyttet til at kalibrere partialkoefficienterne på bæreevnen af tværsnit og elementer (stabilitet).

Det skal nævnes, at for stålmaterialerne er værdierne baseret på **mange** prøvninger af **europæisk** producerede materialer.

prEN 1993-1-8:2005 (E) –

CEN/TC 250

Date: 2018-07-27

EN 1993-1-8:2005

CEN/TC 250

Secretariat: BSI

Eurocode 3 — Design of steel structures — Part 1-8: Design of joints

When the resultant force on a bolt or a rivet is neither parallel nor perpendicular to the plate edge, the design bearing resistance should satisfy the following:

$$\left[\frac{F_{\mathrm{b,Ed}}^{(1)}}{F_{\mathrm{b,Rd}}^{(1)}}\right]^2 + \left[\frac{F_{\mathrm{b,Ed}}^{(2)}}{F_{\mathrm{b,Rd}}^{(2)}}\right]^2 \leq 1{,}0 \qquad (5.5)$$

where the force is resolved into orthogonal components, see Figure 5.7. The design bearing resistances in the orthogonal directions should be determined from Table 5.7.

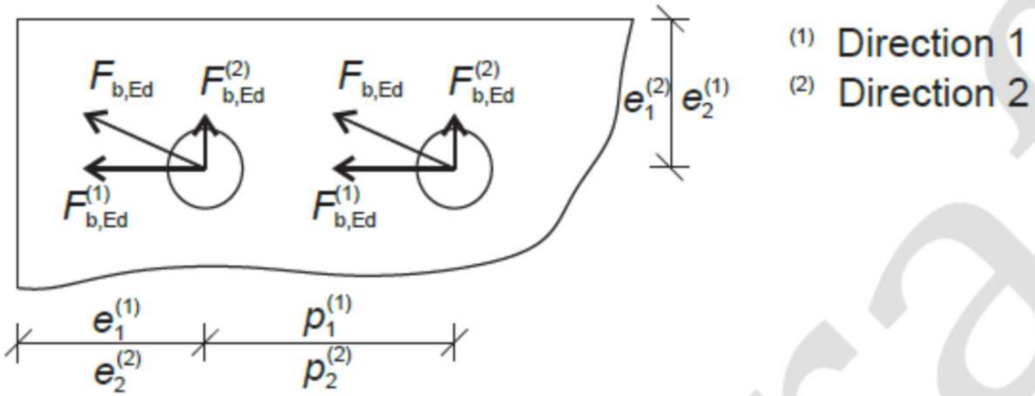
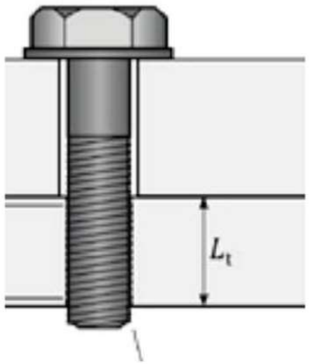
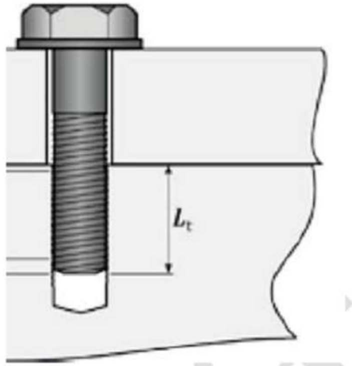


Figure 5.7 — Bearing force in two directions



a) Bolt in threaded through hole



b) Bolt in threaded blind hole

5.7.4 Bolts in threaded holes

(1) The minimum ratio of thread engagement length L_t to bolt diameter d for bolts used in threaded blind holes or threaded through holes should satisfy the values given in Table 5.8.

NOTE The National Annex can specify more precise design methods for the calculation of the minimum thread engagement length L_t.

Table 5.8 — Minimum thread engagement lengths L_t to bolt diameter d ratio for bolts M12 to M36

Steel grade/Bolt property	L_t/d for steel of grade:		
	S235	S355	≥ S460
4.6	1,00	1,00	1,00
5.6	1,02	1,00	1,00
8.8	1,34	1,11	1,06
10.9	1,58	1,26	1,19

Der skal altid tages hensyn til ”modholdskræfter”.

Faste regler for hvordan man bestemmer forskydningsbæreevnen af en gruppe af bolte.

5.7.6 Prying forces

(1) Fasteners required to carry an applied tensile force should be designed to resist the additional force due to prying action, where this may occur.

NOTE The rules given in 8.3 on the equivalent T-stub in tension implicitly account for prying forces.

5.8 Design resistance of a group of fasteners loaded in bearing and shear

(1) The design resistance of a group of fasteners ΣF_{Rd} should be taken as follows:

a) if the design shear resistance $F_{v,Rd}$ of each individual fastener is greater than or equal to its design bearing resistance $F_{b,Rd}$ then ΣF_{Rd} should be taken as the sum of the design bearing resistances of the individual fasteners;

b) otherwise, ΣF_{Rd} should be taken as the number of fasteners multiplied by the smallest design resistance of any individual fasteners.

5.10 Design block tearing resistance

(1) Block tearing should be avoided.

NOTE: Block tearing consists of failure in shear at the line of bolt holes on the shear face of the bolt group, accompanied by tensile rupture along the line of bolt holes on the tension face of the bolt group, see Figures 5.13 and 5.14.

(3) For a bolt group where the tension stress on the tension area is uniform, see Figure 5.13, the design block tearing resistance $V_{\mathrm{eff,1,Rd}}$ should be obtained from:

$$V_{\mathrm{eff,1,Rd}} = \left[A_{\mathrm{nt}} f_{\mathrm{u}} + \min\left(\frac{A_{\mathrm{gv}} f_y}{\sqrt{3}}; \frac{A_{\mathrm{nv}} f_u}{\sqrt{3}}\right)\right] / \gamma_{\mathrm{M2}} \qquad (5.13)$$

where

A_{nt} net area subjected to tension;

A_{gv} gross area subjected to shear;

A_{nv} net area subjected to shear.

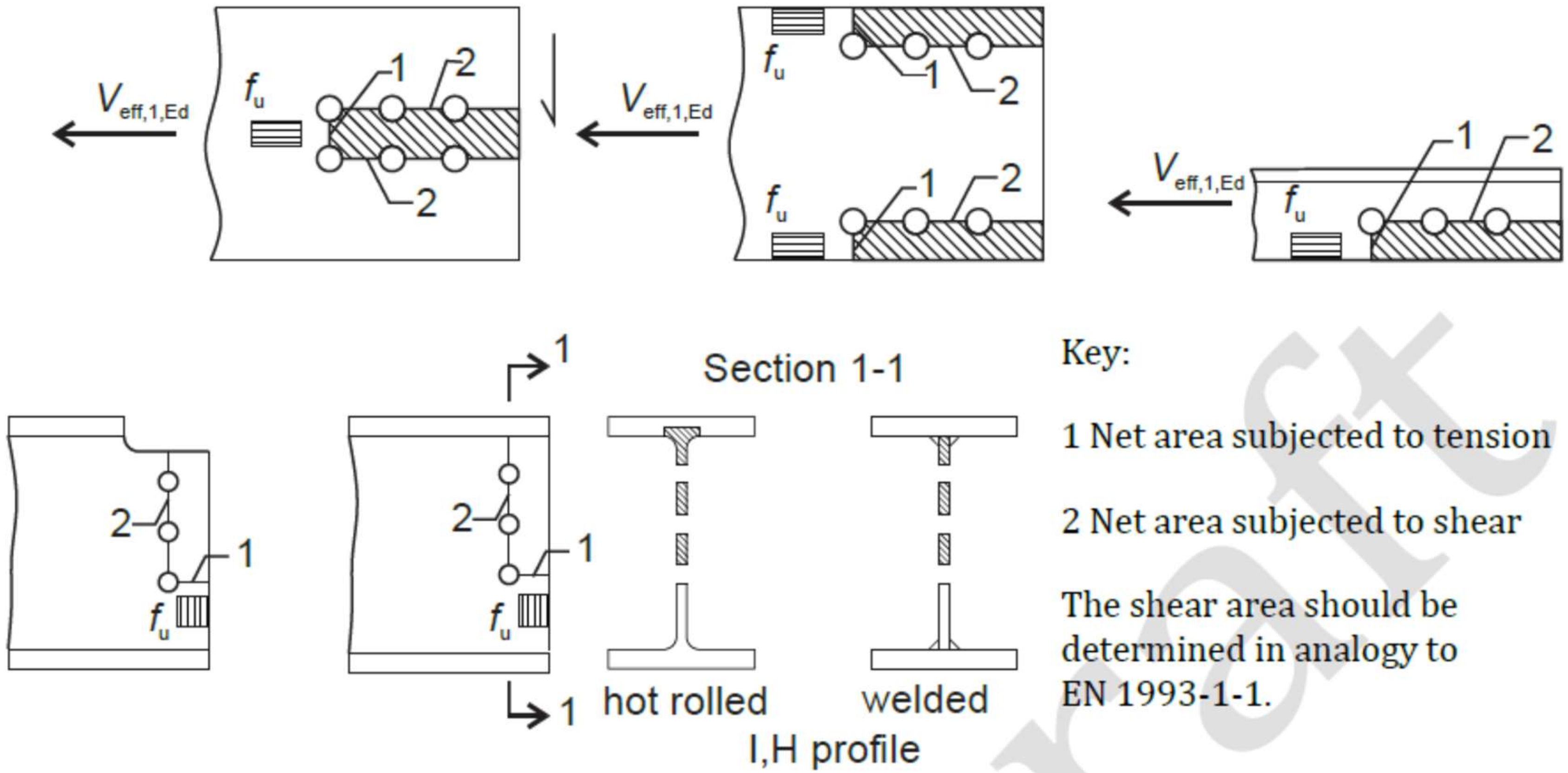
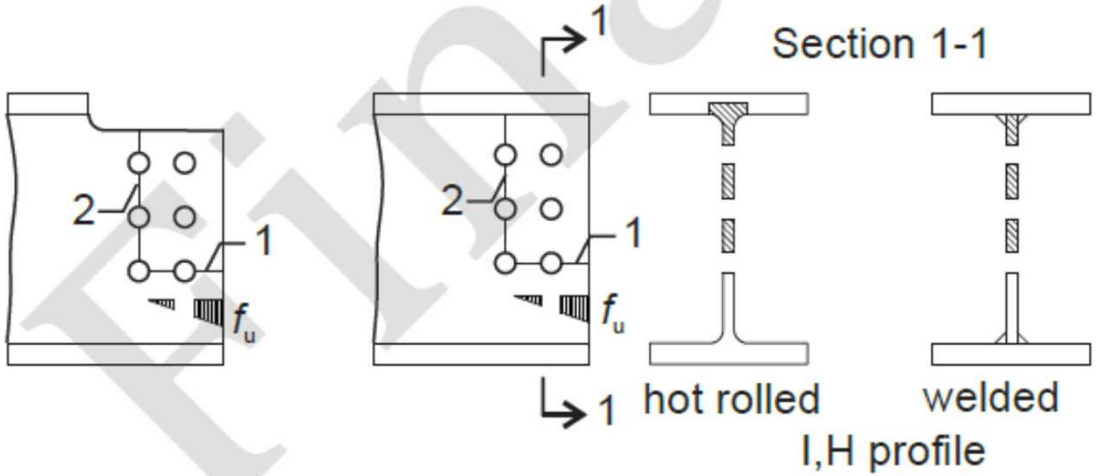


Figure 5.13 — Examples of block tearing, where the tension stress on the tension area is uniform

(4) For a bolt group where the tension stress on the tension area is non-uniform, see Figure 5.14, the design block shear tearing resistance $V_{\mathrm{eff,2,Rd}}$ should be obtained from:

$$V_{\mathrm{eff,2,Rd}} = \left[0{,}5A_{\mathrm{nt}}f_{\mathrm{u}} + \min\left(\frac{A_{\mathrm{gv}}f_y}{\sqrt{3}}; \frac{A_{\mathrm{nv}}f_u}{\sqrt{3}}\right)\right] / \gamma_{\mathrm{M2}} \quad (5.14)$$



Key:

1 Net area subjected to tension

2 Net area subjected to shear

The shear area should be determined in analogy to EN 1993-1-1.

Figure 5.14 — Examples of block tearing, where the tension stress on the tension area is non-uniform

6.3.2.1 Angles between the fusion faces

(1) Fillet welds may be used for connecting parts where the fusion faces form an angle of between 60° and 120°, see Figure 6.1.

(2) End returns should be indicated on the drawings.

(3) For eccentricity of single-sided fillet welds, see 6.12.

NOTE: For angles smaller than 60° and greater than 120°, see Figure 6.1, the resistance of fillet welds can be determined by testing in accordance with EN 1990 Annex D: Design by testing.

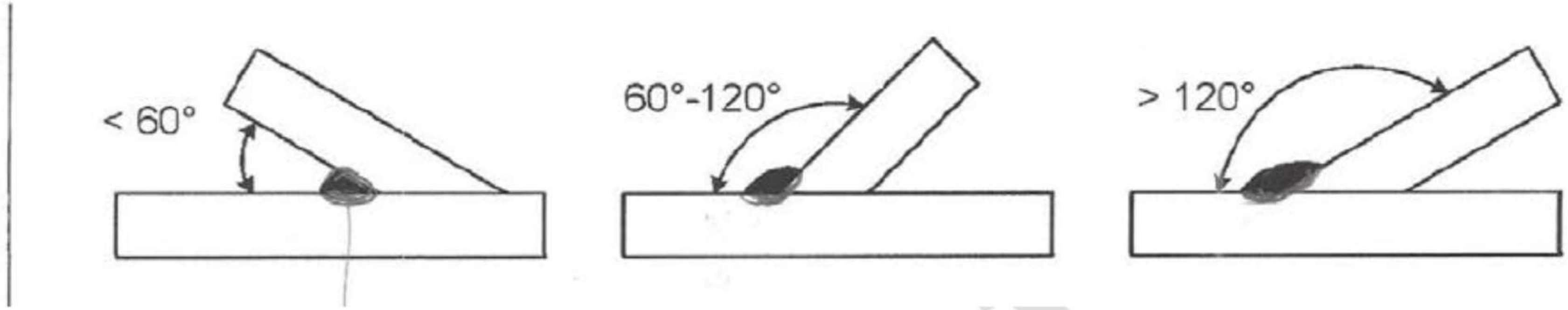


Figure 6.1 — Angle between attached plates *Ed. Show the fusion line, welds in grey*

6.4 Welds with packings

(1) Where two parts connected by welding are separated by packing having a thickness not greater than the leg size of the weld necessary to transmit the design force, see Figure 6.4a):

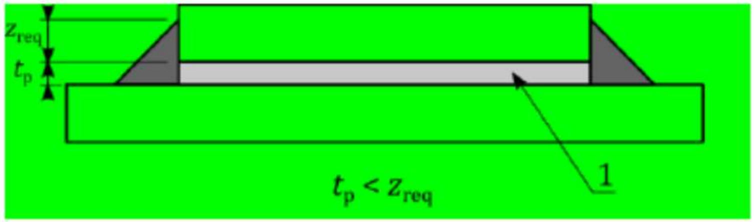
a) the packing should be trimmed flush with the edges of the outer connected part, and;

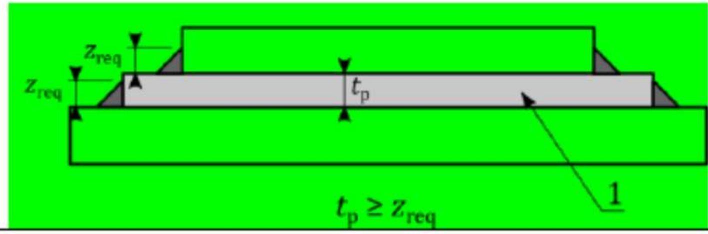
b) the required leg size should be increased by the thickness of the packing.

(2) Where two parts connected by welding are separated by packing having a thickness equal to, or greater than, the leg size of the weld necessary to transmit the design force, see Figure 6.4b):

a) the packing should extend beyond the edges of the outer connected part, and;

b) the packing should be connected to each of the parts by welds capable of transmitting the design force without overstressing the packing.





1 Packing plate

a) packing having a thickness not greater than the leg size of the weld	b) packing having a thickness equal to, or greater than, the leg size of the weld

Figure 6.4 — Welds with packings

(5) The design resistance of a fillet weld should be taken as sufficient if the following are both satisfied:

$$\sqrt{\sigma_\perp^2 + 3\left(\tau_\perp^2 + \tau_\parallel^2\right)} \leq \frac{f_u}{\beta_w \gamma_{M2}} \quad \text{and} \quad \sigma_\perp \leq \frac{0{,}9 f_u}{\gamma_{M2}} \tag{6.1}$$

where

f_u nominal ultimate tensile strength of the part joined, which is of lower strength grade;

β_w appropriate correlation factor from Table 6.1.

(6) Welds between parts with different material strength grades should be designed using the properties of the material with the lower strength grade.

(7) Alternatively, the design resistance of a fillet weld in connections of steel grades equal to or greater than S460, and with different parent and filler metal strength, should be taken as sufficient if the following is satisfied:

$$\sqrt{\sigma_\perp^2 + 3\left(\tau_\perp^2 + \tau_\parallel^2\right)} \leq \frac{0{,}25 f_{u,PM} + 0{,}75 f_{u,FM}}{\beta_{w,mod} \gamma_{M2}} \tag{6.2}$$

where

$f_{u,PM}$ nominal ultimate tensile strength of the parent metal (weaker part joined);

$f_{u,FM}$ nominal ultimate tensile strength of the filler metal according to Table 6.2, and according to EN ISO 2560, EN ISO 16834 and EN 18276;

$\beta_{w,mod}$ modified correlation factor that depends on the filler metal strength from Table 6.2.

Table 6.1 — Correlation factor β_w for fillet welds

Standard and steel grade				Correlation factor β_w
EN 10025	EN 10210-1	EN 10219-1	EN 10149-2	
S 235 S 235 W	S 235 H	S 235 H		0,80
S 275 S 275 N/NL S 275 M/ML	S 275 H S 275 NH/NLH	S 275 H S 275 NH/NLH S 275 MH/MLH		0,85
S 355 S 355 N/NL S 355 M/ML S 355 W	S 355 H S 355 NH/NLH	S 355 H S 355 NH/NLH S 355 MH/MLH		0,90
S 420 N/NL S 420 M/ML	S 420 NH/NLH	S 420 NH/NLH S 420 MH/MLH		0,88
S 450				1,05
S 460 N/NL S 460 M/ML S 460 Q/QL/QL1	S 460 NH/NLH	S 460 NH/NLH S 460 MH/MLH		0,85
S 500 Q/QL/QL1			S 500 MC	0,90
S 550 Q/QL/QL1			S 550 MC	0,95
S 620 Q/QL/QL1			S 600 MC	1,05
S 690 Q/QL/QL1			S 650 MC S 700 MC	1,10

Table 6.2 — Ultimate strength of filler metals $f_{u,FM}$ and modified correlation factor $\beta_{w,mod}$

Filler metal strength class	42	46	69	89
Ultimate strength $f_{u,FM}$ [N/mm²]	500	530	770	940
Correlation factor $\beta_{w,mod}$ [-]	0,89	0,85	1,09	1,19
For filler metals different to those given in Table 6.2 the correlation factor should be taken conservatively according to the given values.				

6.14 Welding in cold-formed zones

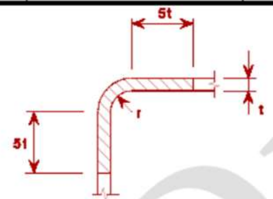
(2) For cold formed hollow sections according to EN 10219 which do not satisfy the inside corner-to-thickness (r/t) limits in Table 6.3, welding in the cold formed corners and adjacent distances of 5t from the corners may be carried out if the following are satisfied:

- thickness $t \leq 12{,}5$ mm;
- impact toughness class MLH, QLH or QL1H
- S ≤ 0,006%, P ≤ 0,015%
- inclusion modification treatment (CaSi, Ca or similar) is mandatory
- additionally for MLH steels: Nb ≥ 0,005% and C ≤ 0,10%
- additionally for QLH & QL1H steels: Mo ≥ 0,05% or P ≤ 0,012%

In other cases, welding in this area may be carried out only if it can be shown by tests that welding can be permitted for that particular application.

Table 6.3 — Conditions for welding cold-formed corners and adjacent material

r/t	**Strain due to cold forming (%)**	**Maximum thickness (mm)**		
		Generally		Fully killed Aluminium-killed steel (Al ≥ 0,02 %)
		Predominantly static loading	Where fatigue predominates	
≥ 25	≤ 2	any	any	any
≥ 10	≤ 5	any	16	any
≥ 3,0	≤ 14	24	12	24
≥ 2,0	≤ 20	12	10	12
≥ 1,5	≤ 25	8	8	10
≥ 1,0	≤ 33	4	4	6



Der er anført mange begrænsninger for anvendelse af stålmaterialer med styrke større end S460. Dette gælder især anvendelse af lokal plasticitet i samlinger – også for anvendelse af global plastisk analyse.

For steels with grades greater than S460, if plastic global analysis is used (see 7.1.3 and 7.1.4), joints should be full-strength. If elastic global analysis is used, partial-strength joints may be used, provided that their resistance exceeds the internal forces and moments acting on the joint. For steels with grades greater than S460, the resistance of joints should be determined based on elastic distribution of internal forces in the joint.

Indhold i **normative** annekser:

I anneks A er anført de konstruktive egenskaber af grundkomponenterne i samlinger. Her kan bl.a. findes de elastiske egenskaber, som kan benyttes i FEM til at modellere elasticitet af forbindelser.

I anneks B er anført regler for momentstive forbindelser mellem bjælker og søjler. Der er informationer om styrke, stivhed og rotationskapacitet af samlingerne.

I anneks C er anført regler for simple, boltede forbindelser mellem søjler og bjælker af I- eller H-profiler til at sikre at forbindelserne kan betragtes som rotationsslappe (hængsel).

Anneks D indeholder regler for søjlefødder på fundamenter.

Anneks C:

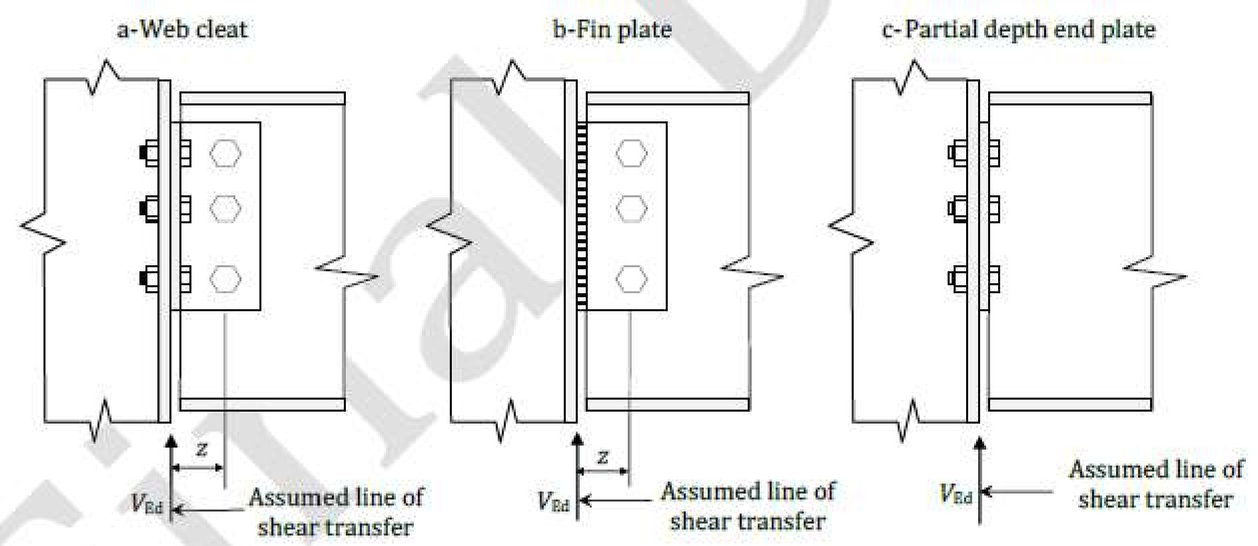


Figure C.2 — Assumed line of shear transfer and eccentricity z

Betingelser (bl.a.):

Forskydningsbæreevne af bolte langs søjleflange reduceres med faktor 0.8

Forskydningsbæreevne af bruttotværsnit af plade/L-jern divideres med 1.27

Bøjningsbæreevne af plade/L-jern baseres på elasticitet

Rotationskapacitet sikret bl.a. ved:

- hulrandsbæreevne < overklipningsbæreevne for bolte
- minimumskrav til størrelse af kantsømme