Structural Eurocodes

EN 1993 Design of Steel Structures

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Designers' guide to Eurocode 3: Design of Steel Structures (L Gardner and D A Nethercot) Thomas Telford

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EN 1993-1-4: Structures in stainless steel

EN 1993-1-5: Strength and stability of planar plated structures without transverse loading

EN 1993-1-6: Strength and stability of shell structures

EN 1993-1-7: Strength and stability of plate structures loaded transversally

EN 1993-1-8: Design of joints

EN 1993-1-9: Fatigue strength

EN 1993-1-10: Fracture toughness assessment

EN 1993-1-11: Design of structures with tension components made of steel

EN 1993-1-12: Use of high strength steels

Symbols

BS5950	A	x,y	r	Z	S	I	J	H
EC3	A	y,z	i	W_{el}	W_{pl}	I	I_t	I_w

BS5950	p_y	p_b	p_c	P	M	V	B,D	d	T,t
EC3	f_y	$\chi_{LT} f_y$	χf_y	N	M	V	B,h	d	t_f,t_w

Table 3.1: f_y f_u

Clause 3.2.6 $E = 210000\ N/mm^2$ $G \approx 81000\ N/mm^2$, $\nu = 0.3$
S275: $t < 40$ mm $f_y = 275\ N/mm^2$ $f_u = 430\ N/mm^2$
$40\ mm < t < 80\ mm$ $f_y = 255\ N/mm^2$ $f_u = 410\ N/mm^2$
Material properties taken from EN 10025-2, which gives additional values for different thicknesses.
So for S275 $16\ mm < t < 40\ mm$ $f_y = 265\ N/mm^2$. Table 3.1 is a simplification of EN 10025-2.

5.5 Classification of cross sections
Table 5.2 $\{\varepsilon = \sqrt{(235/f_y)}\}$ Class 1,2,3,4

5 Structural Analysis

5.1.1 (2) Calculation model and basic assumptions should reflect structural behaviour.

5.1.2 (2) Depending on joint behaviour – simple, continuous, semi-continuous

5.2.1 (3) First order analysis when $\alpha_{cr} = F_{cr}/F_{Ed} \geq$ 10 elastic analysis
15 plastic analysis

F_{cr}: Elastic critical buckling load $\quad$ F_{Ed}: Design load

5.2.1(4) For portal frames and beam-column plane frames, providing axial compression in beams/ rafters is not significant,

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right)$$

provided $\bar{\lambda} < 0.3\sqrt{\frac{Af_y}{N_{Ed}}}$

which translates as $N_{Ed}/N_{cr} \leq 0.09$

H_{Ed}: Design horizontal load at bottom of storey

V_{Ed}: Design vertical load at bottom of storey

h: Storey height

$\delta_{H,Ed}$: Horizontal displacement of top of storey relative to bottom of storey

5.2.2 Structural stability of frames

5.2.2 (3) b) Second order effects accounted for partially by global analysis and partially through individual member stability checks according to 6.3

5.2.2 (5) If $\alpha_{cr} \geq 3$, amplify all horizontal loads by

$$\frac{1}{1 - \frac{1}{\alpha_{cr}}}$$

If $\alpha_{cr} < 3$, second order analysis is necessary

Second order effects likely to arise in unbraced structures and in bracing design of braced structures

5.3 Imperfections

5.3.2 (3) a) Global initial sway imperfection: $\varphi = \varphi_0 * \alpha_h * \varphi_m$

φ_0: Initial value =1/200

α_h: Reduction factor for column height = $2/\sqrt{h}$ ($2/3 \leq \alpha_h \leq 1$)

(h: structure height)

φ_m: Reduction factor for number of columns in a row =

$$\sqrt{0.5\left(1 + \frac{1}{m}\right)}$$

Recommendation (SCI) - use φ_0 i.e let $\alpha_h = \varphi_m = 1.0$

5.3.2 (3) b) Relative initial local bow imperfection (Table 5.1)
5.3.4 (1) ..taken into account using checks in 6.3 provided

$$\bar{\lambda} < 0.5\sqrt{\frac{Af_y}{N_{Ed}}}$$

which translates as $N_{Ed}/N_{cr} \leq 0.25$

Use equivalent horizontal forces (EHF's) for all load combinations

6 Ultimate limit states

6.1 Material factors γ_M

γ_{M0} = 1.00 (cross-sections) γ_{M1} = 1.00 (Buckling)
γ_{M2} = 1.25 (Fracture) (1.1 in UK)

6.2.3 Tension

(1) $N_{Ed} \leq N_{t,Rd}$
(2) $N_{pl,Rd} = Af_y/\gamma_{M0}$ gross section
(3) $N_{u,Rd} = 0.9A_{net}f_u/\gamma_{M2}$ bolt holes

EN 1993-1-8: 4.13(2) For equal angle leg or unequal angle connected (by welding) by its larger leg $A_{eff} = A_{gross}$

EN 1993-1-8: 3.10.3(2) $N_{u,Rd}$ – formulae for resistance of angle connected through one leg using 1,2 or 3+ bolts

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6.2.4 Compression

(1) $N_{Ed} \leq N_{c,Rd}$

(2) $N_{c,Rd} = Af_y/\gamma_{M0}$ (class 1,2,3)

$A_{eff}f_y/\gamma_{M0}$ (class 4)

EN 1993-1-5 Clause 4.4:

A_{eff} is determined by excluding ineffective portion of section.

6.2.5 Bending moment

(1) $M_{Ed} \leq M_{c,Rd}$

(2) $M_{c,Rd} = M_{pl,Rd} = W_{pl} f_y / \gamma_{M0}$ (class 1,2)

$W_{el,min} f_y / \gamma_{M0}$ (class 3)

$W_{eff,min} f_y / \gamma_{M0}$ (class 4)

(4) Fastener holes in tension flange may be ignored provided:

$A_{f,net}\, 0.9 f_u / \gamma_{M2} \geq A_f\, f_y / \gamma_{M0}$

6.2.6 Shear

(1) $V_{Ed} \leq V_{c,Rd}$
(2) $V_{pl,Rd} = A_v(f_y/\sqrt{3})/\gamma_{M0}$
(3) I section $A_v = A - 2bt_f + (t_w + 2r)t_f$ but not less than $\eta h_w t_w$
(6) Shear buckling if $h_w/t_w > 72\varepsilon/\eta$
η may be conservatively taken as 1
EN1993-1-5 Clause 5.1 Note 2: $\eta = 1.2$ up to and including S460; otherwise 1

6.2.8 Bending and shear

(2) $V_{Ed1} \leq 0.5V_{pl,Rd}$ – no effect on $M_{c,Rd}$
(3) Reduced moment resistance: reduced design yield strength $(1\text{-}\rho)f_y$

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2$$

6.2.9 Bending and axial force

6.2.9.1 Class 1 and 2 cross-sections

(4) Axial force can be ignored:

yy axis: if $N_{Ed} \leq 0.25 N_{pl,Rd}$ **and** $N_{Ed} \leq 0.5 h_w t_w f_y / \gamma_{M0}$

zz axis: if $N_{Ed} \leq h_w t_w f_y / \gamma_{M0}$

(5) Values of reduced moment capacities: $M_{N,y,Rd}$, $M_{N,z,Rd}$

(6)

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}} \right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}} \right]^{\beta} \leq 1$$

Conservative alternative for all sections: 6.2.1 (7)

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + \frac{M_{z,Ed}}{M_{z,Rd}} \leq 1 \qquad \text{Eq 6.2}$$

As buckling is generally critical and will ‘override’ Eq. 6.2, the more exact equations in 6.2.9 need not generally be considered.

6.3 Buckling resistance of members

Elastic buckling theory (Euler)

$N_{cr} = \pi^2 EI/l^2$ for ideal strut

This value is modified by various imperfections:

Geometric imperfections (initial curvature)

Eccentricity of loading

Residual stresses (‘locked-in’ due to differential cooling)

Non-homogeneity of material properties

End restraints

These are taken into account in the Perry-Robertson formula

$$\sigma_{cr} = \frac{\sigma_y + (1+\eta)\sigma_E}{2} - \sqrt{\left\{((\sigma_y + (1+\eta)\sigma_E)/2)^2 - \sigma_y \sigma_E \right\}}$$

EC3 provides solution for this equation. Effectively $\sigma_{cr} = \chi f_y$

6.3.1 Uniform members in compression

6.3.1.1 Buckling resistance

(1) $N_{Ed} \leq N_{b,Rd}$

(3) $N_{b,Rd} =$ $\chi A f_y / \gamma_{M1}$ (class 1,2,3)

$\chi A_{eff} f_y / \gamma_{M1}$ (class 4)

6.3.1.2 Buckling curves

(1)

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \overline{\lambda}^2}} \quad \text{(but} \leq 1\text{)} \qquad \varphi = 0{,}5\left[1 + \alpha(\overline{\lambda}^2 - 0.2) + \overline{\lambda}^2\right]$$

$$\overline{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} \quad \text{(class 1,2,3)} \qquad \overline{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} \quad \text{(class 4)}$$

α: imperfection factor (Table 6.1) (Table 6.2)

N_{cr}: Elastic critical buckling load

χ (chi) can also be obtained from Figure 6.4

(4) If $\overline{\lambda} \leq 0.2$ ($N_{Ed} \leq 0.04 N_{cr}$) buckling effects can be ignored

6.3.1.3 Slenderness for flexural buckling

$f_{cr} = N_{cr}/A = \pi^2 EI/Al^2 = \pi^2 EAi^2/Al^2 = \pi^2 E/\lambda^2$

Letting $f_{cr} = f_y$ limiting slenderness $\lambda_1 = \pi\,(E/f_y)^{0.5} = 93.9\varepsilon$

$\lambda = L_{cr}/i_z$

$$\bar{\lambda} = \lambda/\lambda_1$$ *: non-dimensional slenderness*

(ratio of actual slenderness to slenderness at boundary between yielding and elastic buckling) $= \pi(E/f_{cr})^{0.5}/\,(\pi(Ef_y)^{0.5}) = (f_y/f_{cr})^{0.5}$

$$= \sqrt{\frac{Af_y}{N_{cr}}}$$

6.3.2 Uniform members in bending

Beam behaviour analogous to column behaviour

Beam subject to equal moments at each end

$$M_{cr} = \frac{\pi^2 EI_z}{L_{cr}^{\;2}} \left[\frac{I_w}{I_z} + \frac{L_{cr}^{\;2} GI_t}{\pi^2 EI_z} \right]^{0.5}$$

This is expanded to:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L_{cr}^{\;2}} \left[\left(\frac{k}{k_w} \right)^2 \frac{I_w}{I_z} + \frac{L_{cr}^{\;2} GI_t}{\pi^2 EI_z} + C_2 z_g^{\;2} \right]^{0.5} - C_2 z_g$$

for other load situations.

M_{cr}: Elastic critical buckling load

C_1, C_2, k, k_w: constants
z_g: Eccentricity of load relative to centroid of beam (destabilizing load)
C_1: depends on moment diagram
k, k_w: depend on support conditions (generally 1)
C_2: does not apply when $z_g = 0$ (non destabilizing loads)

NCCI SN003a-EN-EU and NCCI SN006a-EN-EU (cantilevers) provide information on calculation of M_{cr}.
Program to calculate M_{cr} can be downloaded from
http://www.cticm.eu

The results for buckling and lateral torsional buckling are similar to those in BS5950 (Annex C – buckling *(exact)*, Annex B – lateral torsional buckling)

6.3.2.1 Buckling resistance

(1) $M_{Ed} \leq M_{b,Rd}$

(3) $M_{b,Rd} = \chi_{LT} W_y f_y / \gamma_{M1}$

$W_y = W_{pl,y}$ (class 1,2) $W_y = W_{el,y}$ (class 3) $W_y = W_{eff,y}$ (class 4)

6.3.2.2 Lateral torsional buckling curves – General case

(1)

$$\chi_{LT} = \frac{1}{\varphi_{LT} + \sqrt{\varphi_{LT}^{2} - \bar{\lambda}_{LT}^{2}}} \qquad \text{(but} \le 1\text{)}$$

$$\varphi_{LT} = 0,5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT}^{2} - 0.2\right) + \bar{\lambda}_{LT}^{2}\right]$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

α_{LT}: imperfection factor (Table 6.3) (Table 6.4)

(3) χ_{LT} can also be obtained from Figure 6.4

(4) If $\bar{\lambda}_{LT} \le 0.4$ $(M_{Ed} \le 0.16 M_{cr})$ buckling effects can be ignored

NCCI SN002a: Simplified assessment of

$$\bar{\lambda}_{LT} = \frac{1}{\sqrt{C_1}} 0.9 \frac{\lambda_z}{\lambda_1}$$ where $\lambda_z = L_{cr}/i_z$, $\lambda_1 = \pi\sqrt{(E/f_y)}$

Ψ (= M_1/M_2)	**C_1 (Designers' Guide)**	**C_1 (SN003a)**
1	1.000	1.00
.75	1.141	1.14
.5	1.323	1.31
.25	1.563	1.62
0	1.879	1.77
-.25	2.281	2.05
-.5	2.704	2.33
-.75	2.927	2.57
-1	2.752	2.55
pinned, udl	1.132	1.127
pinned, central P	1.365	1.348

$$C_1 = 1.88 - 1.4\psi + 0.5\,\psi^2 \; (C_1 \leq 2.70)$$

6.3.2.3 Lateral torsional buckling curves for rolled sections or equivalent welded sections

(1) For members in bending (beams)

$$\chi_{LT} = \frac{1}{\varphi_{LT} + \sqrt{\varphi_{LT}^{\ 2} - \beta \bar{\lambda}_{LT}^{\ 2}}}$$

$$\text{but} \leq 1 \text{ and} \leq \frac{1}{\bar{\lambda}_{LT}^{\ 2}}$$

$$\varphi_{LT} = 0{,}5\left[1 + \alpha_{LT}(\bar{\lambda}_{LT}^{\ 2} - \bar{\lambda}_{LT,0}) + \beta \bar{\lambda}_{LT}^{\ 2}\right]$$

$$\bar{\lambda}_{LT,0} = 0.4$$

$$\beta = 0.75$$

α_{LT}: imperfection factor (Table 6.3) (Table 6.5)

(2) $\chi_{LT,mod} = \chi_{LT}/f \quad (\leq 1)$

$$f = 1 - 0.5(1 - k_c\left[1 - 2.0(\bar{\lambda}_{LT} - 0.8)^2\right] \qquad (\leq 1)$$

k_c: Table 6.6

6.3.2.3 used for rolled sections of 'standard' dimensions
6.3.2.2 can be used for all sections including plate girders (bigger than 'standard' sections), castellated and cellular beams

(4) Members which are subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \qquad (6.61)$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \qquad (6.62)$$

where N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$ are the design values of the compression force and the maximum moments about the y-y and z-z axis along the member, respectively

$\Delta M_{y,Ed}$, $\Delta M_{z,Ed}$ are the moments due to the shift of the centroidal axis according to 6.2.9.3 for class 4 sections, see Table 6.7,

χ_y and χ_z are the reduction factors due to flexural buckling from 6.3.1

χ_{LT} is the reduction factor due to lateral torsional buckling from 6.3.2

k_{yy}, k_{yz}, k_{zy}, k_{zz} are the interaction factors

Table 6.7: Values for $N_{Rk} = f_y A_i$, $M_{i,Rk} = f_y W_i$ and $\Delta M_{i,Ed}$

Class	1	2	3	4
A_i	A	A	A	A_{eff}
W_y	$W_{pl,y}$	$W_{pl,y}$	$W_{el,y}$	$W_{eff,y}$
W_z	$W_{pl,z}$	$W_{pl,z}$	$W_{el,z}$	$W_{eff,z}$
$\Delta M_{y,Ed}$	0	0	0	$e_{N,y} N_{Ed}$
$\Delta M_{z,Ed}$	0	0	0	$e_{N,z} N_{Ed}$

NOTE For members not susceptible to torsional deformation χ_{LT} would be $\chi_{LT} = 1,0$.

(5) The interaction factors k_{yy} , k_{yz} , k_{zy} , k_{zz} depend on the method which is chosen.

NOTE 1 The interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} have been derived from two alternative approaches. Values of these factors may be obtained from Annex A (alternative method 1) or from Annex B (alternative method 2).

NOTE 2 The National Annex may give a choice from alternative method 1 or alternative method 2.

NOTE 3 For simplicity verifications may be performed in the elastic range only.

6.3.3 Uniform members in bending and compression

For Class 1,2,3 sections, Equations 6.61 and 6.62 reduce to:

$$\frac{N_{Ed}}{N_{b,y,Rd}} + k_{yy}\frac{M_{y,Ed}}{M_{b,Rd}} + k_{yz}\frac{M_{Z,Ed}}{M_{c,z,Rd}} \leq 1 \qquad \text{Eq. 6.61}$$

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy}\frac{M_{y,Ed}}{M_{b,Rd}} + k_{zz}\frac{M_{Z,Ed}}{M_{c,z,Rd}} \leq 1 \qquad \text{Eq.6.62}$$

k values in Annex A, Annex B.
Annex A is more precise (French-Belgian)
Annex B is easier to use (Austrian-German)

Table B.3:
If ψ =1 (uniform moment in column – no lateral load on column)
$C_{my} = C_{mz} = C_{mLT} = 1$
If ψ =0 (M varies from 0 to M_{max} – no lateral load on column) $C_{my} = C_{mz} = C_{mLT} = 0.6$
Members susceptible to torsional deformations:
(Table B.2/ Table B.1)

(Table B.2/ Table B.1) $\bar{\lambda}_z < 0.4$ (I sections)

$$k_{zy} = MAX\left[\left[1 - \frac{0.1\bar{\lambda}_z}{(C_{mLT} - 0.25)}\frac{N_{Ed}}{N_{b,z,Rd}}\right], \left[1 - \frac{0.1}{(C_{mLT} - .25)}\frac{N_{Ed}}{N_{b,z,Rd}}\right]\right]$$

$$k_{yy} = MIN\left[C_{my}\left[1 + (\bar{\lambda}_y - 0.2)\frac{N_{Ed}}{N_{b,y,Rd}}\right], C_{my}\left[1 + 0.8\frac{N_{Ed}}{N_{b,yRd}}\right]\right]$$

$$k_{zz} = MIN\left[C_{mz}\left[1 + (2\bar{\lambda}_z - 0.6)\frac{N_{Ed}}{N_{b,z,Rd}}\right], C_{mz}\left[1 + 1.4\frac{N_{Ed}}{N_{b,z,Rd}}\right]\right]$$

$k_{yz} = 0.6k_{zz}$ $\quad$ $k_{zy} = 0.6k_{yy}$

For columns in simple construction N generally dominates, UC sections are less likely to buckle about yy axis → Equation 6.62 likely to be critical.

As N dominates, k values can be chosen conservatively

Access Steel (NCCI SN048b-EN-GB) suggests k_{zy}= 1, k_{zz}= 1.5 (conservatively)

$$\frac{N_{Ed}}{N_{b,z,Rd}} + \frac{M_{y,Ed}}{M_{b,Rd}} + 1.5\frac{M_{Z,Ed}}{M_{c,z,Rd}} \leq 1$$

7 Serviceability limit states

Deflections to be agreed with client – BS5950 values proposed in UK National Annex

Guidance is also provided in NCCI SN034a-EN-EU

Cantilever: Length/180

Beams carrying plaster or other brittle finish: Span/360

Other beams: Span/200

Horizontal deflection limits also provided

EN 1993-1-8 Design of Joints

Table 2.1 Partial safety factors for joints

Bolts, Plates in bearing, Welds $\gamma_{M2} = 1.25$ (UK: 1.5 for grade 4.6)

Slip resistance at sls $\gamma_{M3,ser} = 1.1$

Preload of high strength bolts $\gamma_{M7} = 1.1$

2.5 Design Assumptions (1) Joints should be designed on the basis of a realistic assumption of the distribution of internal forces and moments. *Identify a load path through the joint and check 'all links in chain'.*

2.7 Eccentricity at connections

Table 3.1 Bolt strength f_y, f_u

3 Connections made with bolts, rivets or pins

3.1.2 Preloaded bolts

Table 3.2

Bearing: $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$

Slip (sls): $F_{v,Ed,ser} \leq F_{v,Rd,ser}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$

(use grade 8.8 or 10.9)

Tension: Include prying action

Table 3.3 End, edge distances (e) Spacings (p)

(d_0 – bolt hole diameter)

e_1, p_1: Parallel to load e_2, p_2: Perpendicular to load

3.6.1 (2) Preload in bolts: $F_{p,Cd} = 0.7\ f_{ub}\ A_s / \gamma_{M7}$

(10) Single lap joints with one bolt row

(12) Bolts through packing

Table 3.4 Design resistance

Shear resistance $F_{v,Rd} = \alpha_v\ f_{ub}\ A_s/\ \gamma_{M2}$ $\alpha_v = 0.6$ or 0.5

Bearing resistance $F_{b,Rd} = k_1\ a_b\ f_u\ d\ t/\ \gamma_{M2}$

$a_b = \text{Min.}\ (a_d, f_{ub}/f_u, 1)$ end bolts $a_d = e_1/3d_o$

inner bolts $a_d = p_1/3d_o - 0.25$

$k_1 = \text{Min.}\ (2.8\ e_2/d_o - 1.7, 2.5)$ edge bolts

$k_1 = \text{Min.}\ (1.4\ e_2/d_o - 1.7, 2.5)$ inner bolts

Tension & Combined Shear and Tension also included

3.8 Long joints

3.9 Design slip resistance $F_{s,Rd} = (k_s n \mu) F_{p,C}) / \gamma_{M3}$

k_s = 1 for bolts in normal holes (Table 3.6)

n is the number of friction surfaces

μ: slip factor (Table 3.7)

Class A: shot/ grit blasted, spray metallised with aluminium or zinc based coating certified to provide a slip factor of 0.5

Class B: shot/ grit blasted, painted with alkali-zinc silicate paint to produce a thickness of 50 – 80 μm

Class C: wire brushed or flame cleaned

Class D: untreated

3.10.2 Design for block tearing

(2) $V_{eff,1,Rd} = f_u A_{nt} / \gamma_{M2} + (1/\sqrt{3}) f_y A_{nv} / \gamma_{M0}$ Eq 3.9

(3) $V_{eff,2,Rd} = 0.5 f_u A_{nt} / \gamma_{M2} + (1/\sqrt{3}) f_y A_{nv} / \gamma_{M0}$ Eq 3.10

Section 4 Welded connections

4.5.2 Effective throat thickness

4.5.3.2 Directional method

4.5.3.3 Simplified method for the design resistance of fillet weld

(1) $F_{w,Ed} \leq F_{w,Rd}$

(2) $F_{w,Rd} = f_{vw,d} * a$ (throat thickness) = 0.7 * leg length

(3) $f_{vw,d} = f_u/(\sqrt{3} * \beta * \gamma_{M2})$ β: Table 4.1 (= 0.85 for S275)

5 Analysis, classification and modelling

Table 5.1 Joint modelling

6 Structural Joints connecting H or I sections

Table 6.1 Basic joint components

Table 6.2 Design resistance of a T-stub

Column Base plate

6.2.8.2, Figure 6.19

6.2.5 Equivalent T-stub in compression

(3) $F_{C,Rd} = f_{jd} * b_{eff} * l_{eff}$

(7) $f_{jd} = \beta_j F_{Rdu} / (b_{eff} \, l_{eff})$ $\beta_j = 2/3$

EN 1992-1-1 Clause 3.1.6 (1) $f_{cd} = \alpha_{cc} f_{ck} / \gamma_c ; \alpha_{cc} = 1, \gamma_c = 1.5$

NCCI SN037a $f_{jd} = \beta_j * \alpha * f_{cd}$ $(\alpha = 1.5)$

$$A_{c,0} = MAX\left(\frac{N_{j,Ed}}{f_{jd}}, \frac{1}{h_c b_{fc}} \left[\frac{N_{j,Ed}}{f_{jd}} \right]^2 \right)$$

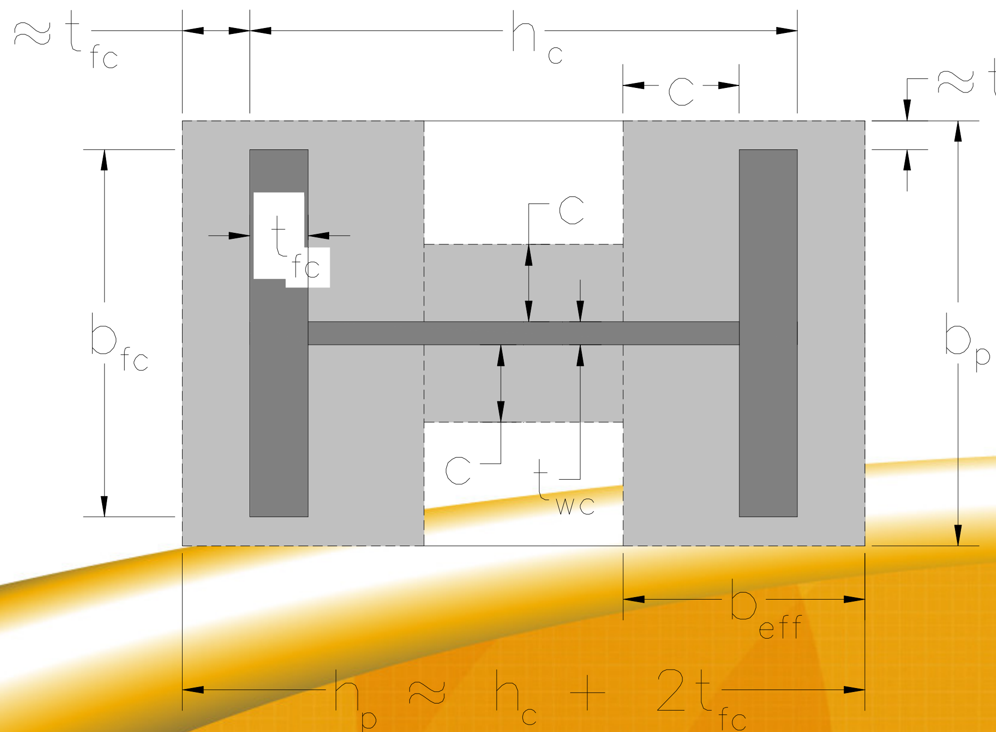
Large projection

Short projection (when $A_{c,0} < 0.95bh$) base plate

Select plate dimensions – determine c (Figure 6.4)

Min. outstand: t_f

6.2.5(4) $$t_p = c\sqrt{\frac{3f_{jd}\gamma_{M0}}{f_y}}$$



EN 1993-1-5 Plated structural elements

Section 6 Resistance to transverse forces (web bearing and buckling)

Figure 6.1 k_F

6.2 Design resistance $F_{Rd} = (f_{yw} L_{eff} t_w) / \gamma_{M1}$

$$L_{eff} = \chi_F l_y$$

6.3 Length of stiff bearing

6.4 Reduction factor $\chi_F = \dfrac{0.5}{\bar{\lambda}_F} \leq 1.0$

$$\bar{\lambda}_F = \sqrt{\frac{l_y t_w f_{yw}}{F_{cr}}} \qquad F_{cr} = 0.9 k_F E \frac{t_w^{\,3}}{h_w}$$

6.5 Effective loaded length

$m_1 = f_{yf}b_f/f_{yw}t_w$

$m_2 = 0.02\ (h_w/t_f)^2$ if $\bar{\lambda}_F > 0.5,$

$\quad 0$ if $\bar{\lambda}_F \leq 0.5$

l_y is the minimum of:

$$l_e + t_f\sqrt{\frac{m_1}{2} + \left(\frac{l_e}{t_f}\right)^2 + m_2}\,, l_e + t_f\sqrt{m_1 + m_2}$$

$$l_e = \frac{k_F E t_w^{\ 2}}{2 f_{yw} h_w} but \leq s_s + c$$

6.6 Verification

$$\eta_2 = \frac{F_{Ed}}{(f_{yw} L_{eff} t_w)/\gamma_{M1}} \leq 1$$

Effective lengths

Truss members: EN 1993-1-1:

Annex BB.1.1: $L_{cr} = L$ (for chord members)

Annex BB.1.2 (angles as web members):

$$\bar{\lambda}_{eff,v} = 0.35 + 0.7\bar{\lambda}_v \quad \bar{\lambda}_{eff,y} = 0.50 + 0.7\bar{\lambda}_y \quad \bar{\lambda}_{eff,z} = 0.35 + 0.7\bar{\lambda}_z$$

Members in compression – ends held in position (BS5950 – in absence of EC3 guidance)

Both ends effectively restrained in direction	0.7L
Partially restrained in direction at both ends	0.85L
Restrained in direction at one end	0.85L
Not restrained in direction at either end	1.0L

Members in bending – compression flange laterally restrained, nominal torsional restraint against rotation about longitudinal axis at supports (NCCI SN009a)

Support conditions	K
Both flanges fully restrained against rotation on plan	0.7
Compression flange fully restrained against rotation on plan	0.75
Both flanges partially restrained against rotation on plan	0.8
Compression flange partially restrained against rotation on plan	0.85
Both flanges free to rotate on plan	1.0

NCCI SN005a:

Simple construction – assume loads applied at 100 mm from face of column (flange or web). Resulting moment can be distributed between upper and lower levels in accordance with stiffness

End Plate NCCI: SN013, SN014

Use full depth end plate if $V_{Ed} > 0.75\ V_{c,Rd}$

Min. number of bolts = $V_{Ed}/75$

Plate dimensions

Beam depth	t_p (mm)	b_p (mm)	Cross-centres (mm)
< 500 mm	8, 10	150	90
> 500 mm	10	200	140

Weld size (S275):

Beam web (mm)	9	12	15
Throat (mm)	4	5.5	7
Leg length (mm)	6	8	10

Bolts in shear: Shear resistance multiplied by 0.8 to account for presence of some tension in bolts

End plate in shear (gross section): Shear resistance divided by 1.27 to allow for bending moment in plate

Weld design: Shear resistance divided by 1.27 to allow for bending moment in plate

End plate in shear (block shear): may need to consider $V_{eff,1,Rd}$ (3.10.2 (2)) and $V_{eff,2,Rd}$ (3.10.2 (3))

End plate in bending – when bolt cross-centre distance is large, bending in plate reduces shear resistance

Beam web in shear to be checked for depth = plate depth

Weld design $a \geq 0.39t_{w,b1}$

Ductility requirements: $$t_p \leq \frac{d}{2.8}\sqrt{\frac{f_{ub}}{f_{y,p}}}$$

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