THE FUTURE OF STEEL CONSTRUCTION

Technical improvements for steel bridge design in the second generation EurocodeS

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Eurocodes – the second generation

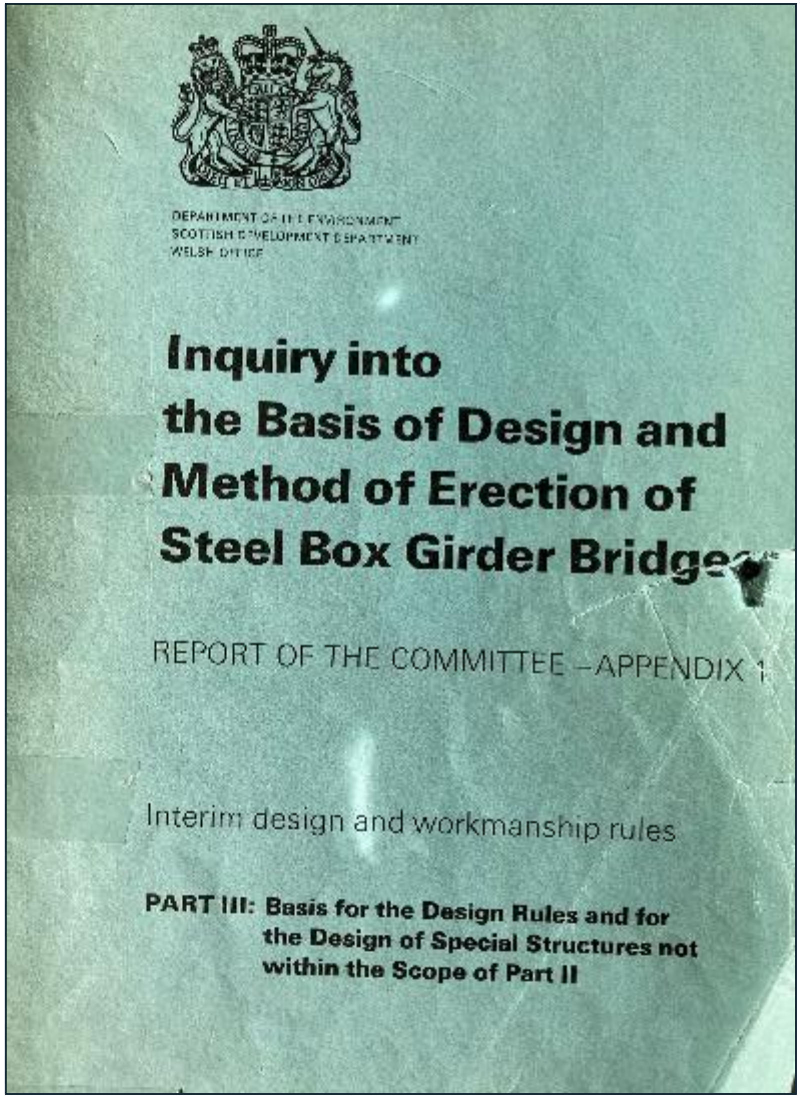
THE FUTURE OF STEEL CONSTRUCTION

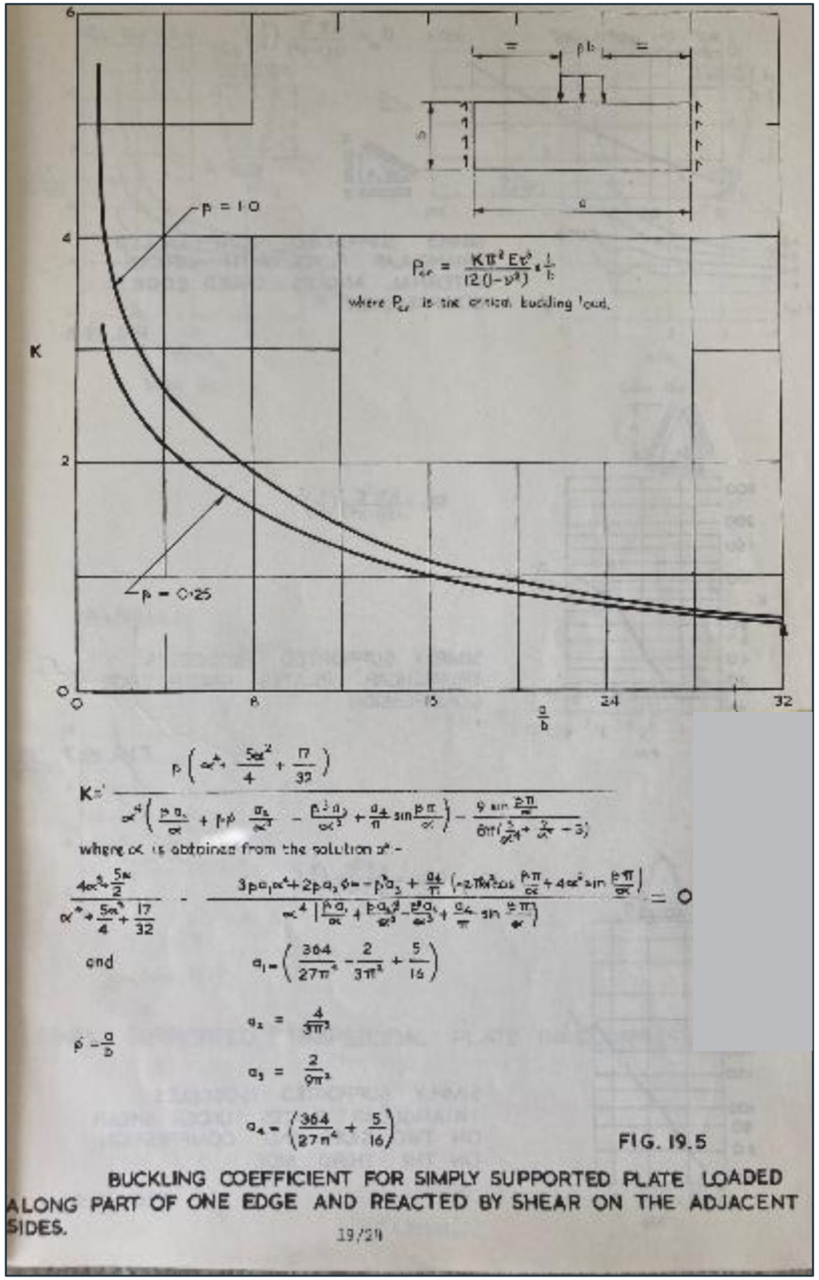
- Second generation Eurocodes and timescales
- Main technical changes in EN 1993-1-5 and EN 1993-2 for steel bridge design
- Other changes in EN 1993-1-1, EN 1993-1-9, EN 1993-1-10 and EN 1993-1-11 relating to bridges

Introduction - Role of Codes of Practice

Most Codes of Practice have got longer over time, but not more complicated in all cases

- IDWR was produced following the collapse of West Gate and Cleddau bridges to deal with complex buckling issues
- Critical stresses were introduced as basis for buckling calculations
- Rules simplified in BS 5400:Part 3 as designers struggled with IDWR
- Eurocodes returned to critical stresses but with simpler formulae or use of FE
- Second Generation Eurocodes have provided more material to cover more topics





Eurocodes – the second generation

Evolved Eurocodes

EN 1990	Eurocode 0	Basis of structural and geotechnical design
EN 1991	Eurocode 1	Actions on structures
EN 1992	Eurocode 2	Design of concrete structures
EN 1993	Eurocode 3	Design of steel structures
EN 1994	Eurocode 4	Design of composite steel and concrete structures
EN 1995	Eurocode 5	Design of timber structures
EN 1996	Eurocode 6	Design of masonry structures
EN 1997	Eurocode 7	Geotechnical design
EN 1998	Eurocode 8	Design of structures for earthquake resistance
EN 1999	Eurocode 9	Design of aluminium alloy structures
EN 19100	Eurocode 10	Design of Glass Structures

New Eurocodes

- EN 19100 Design of glass structures – Eurocode 10

New CEN Technical Specifications to become Eurocodes

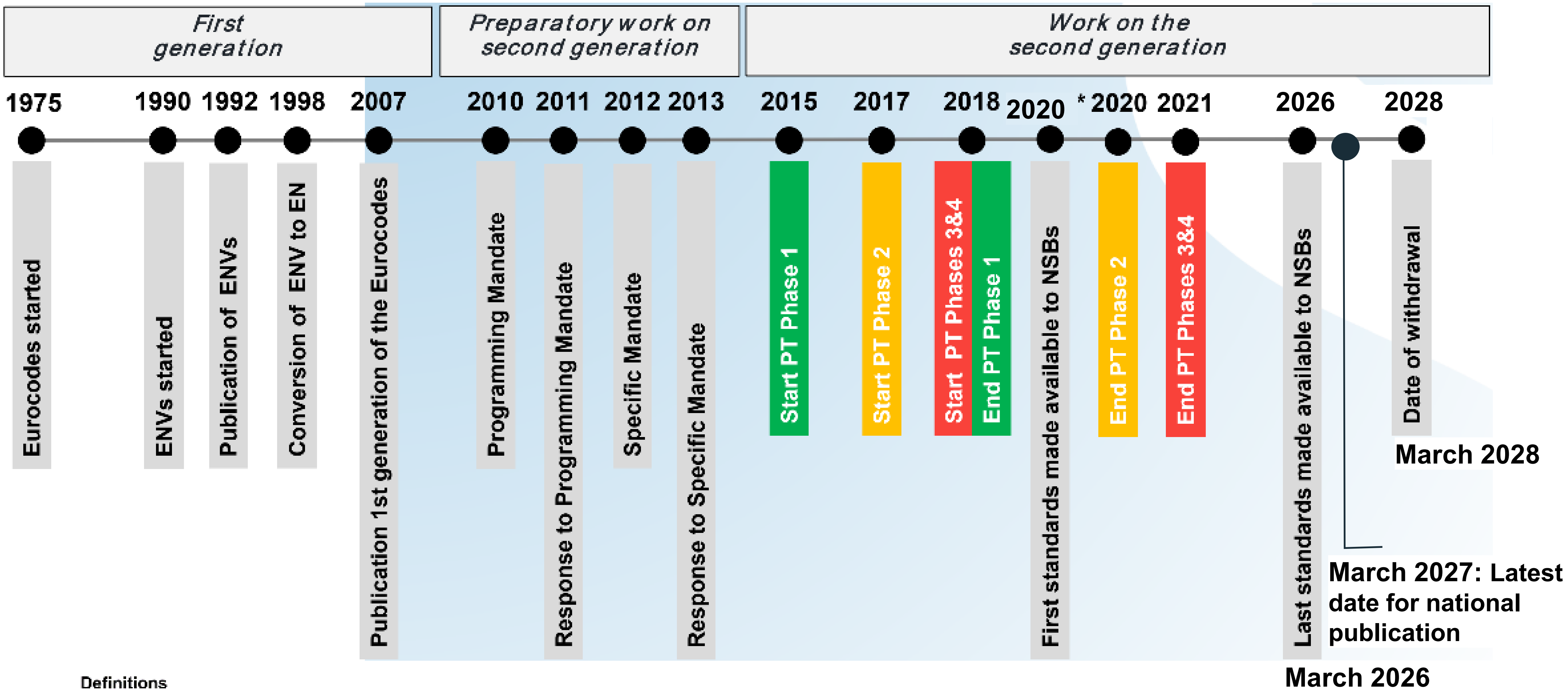
- CEN/TS 19101 Design of fibre-polymer composite structures – Eurocode 11
- CEN/TS 19102 Design of membrane structures – Eurocode 12
- CEN/TS 17440:2020 Assessment of existing structures (currently prEN 1990-2 with prEN 1990-1 New structures)

Plus other new CEN Technical Specifications

[Second generation of the Eurocodes: what is new? | Eurocodes: Building the future (europa.eu)]

[https://eurocodes.jrc.ec.europa.eu/2nd-generation/eurocodes-evolution-explained-video-series]

Eurocodes – the second generation



Definitions

'Date of availability (DAV) : Date when the definitive text in the official language versions of an approved CEN/CENELEC publication is distributed by the Central Secretariat •

Date of publication (DoP) : Latest date by which an EN has to be implemented at national level by publication of an identical national standard or by endorsement •

Date of withdrawal (DoW) : Latest date by which national standards conflicting with an EN have to withdraw.'

Eurocodes – the second generation

- New Eurocodes will aim to improve ease of use, including:
  - Address specific needs of diverse users
  - Improve clarity and provide missing material
  - Reduce inconsistencies and mistakes
  - Reduce length
  - Reduce NDPs
  - Update rules where more reliable material exists
  - Encourage innovation
  - Adopt ISO standards where possible
  - Consider climate change and robustness
- Systematic review comments and queued amendments addressed
- This has led to a moderate amount of amended and new material in EN 1993 and a lot of change in EN 1992
- Existing clause numbers will be incremented by two e.g. current *Section 3 – Materials*, will be *Section 5 – Materials*

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Examples from EN 1993-1-5

Moderate change

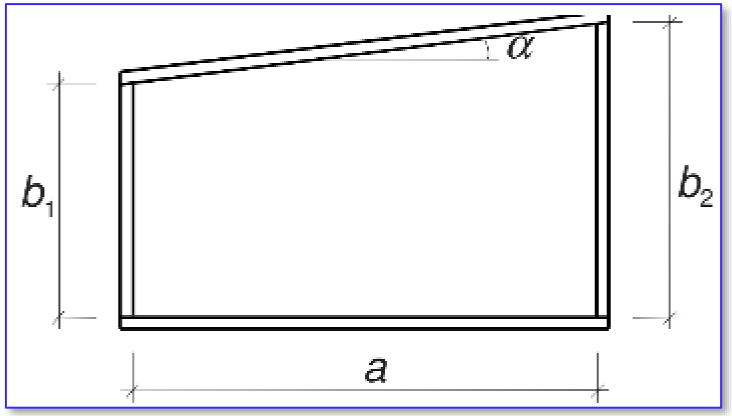
Eurocode 3: Design of steel structures – Part 1-5: Plated structural elements

Einführendes Element — Haupt-Element — Ergänzendes Element

Élément introductif — Élément central — Élément complémentaire

Eurocode is now published by BSI

Examples from EN 1993-1-5
Improved clarity on coverage of curved panels



- Clause is currently ambiguous about direction of application of curvature; “*a*” is usually “panel length”
- Caused arguments on design of at least one arch project

NOTE 4: Single plate elements may be considered as flat where the curvature radius r satisfies:

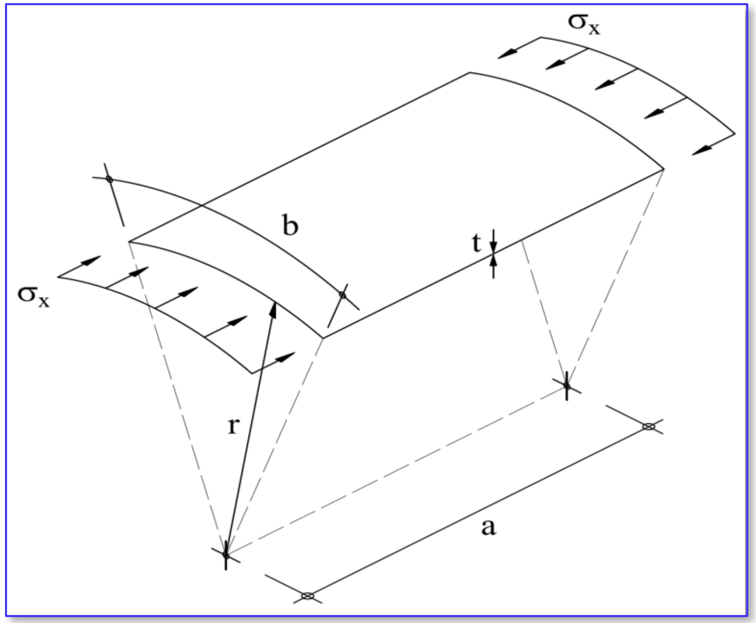
$$r \geq \frac{a^2}{t} \tag{1.1}$$

Current rule

where a is the panel width

t is the plate thickness

Examples from EN 1993-1-5
Improved clarity on coverage of curved panels



- Now resolved and aligned with new rules for curved panels in EN 1993-2 i.e. a plate is “flat” if curvature *Z* = $b^2/rt \leq 1.0$
- Curvature in the “*a*” direction **not** addressed

(3) Single plate elements may be considered as flat where the curvature radius r in the direction perpendicular to the compression satisfies:

$$r \geq \frac{b^2}{t} \tag{1.1}$$

New rule

where b is the panel width

t is the plate thickness

- For guidance on arches, see PD 6695-2 and *Walton Bridge – a new arch bridge over the River Thames, UK,* ICE Bridge Engineering

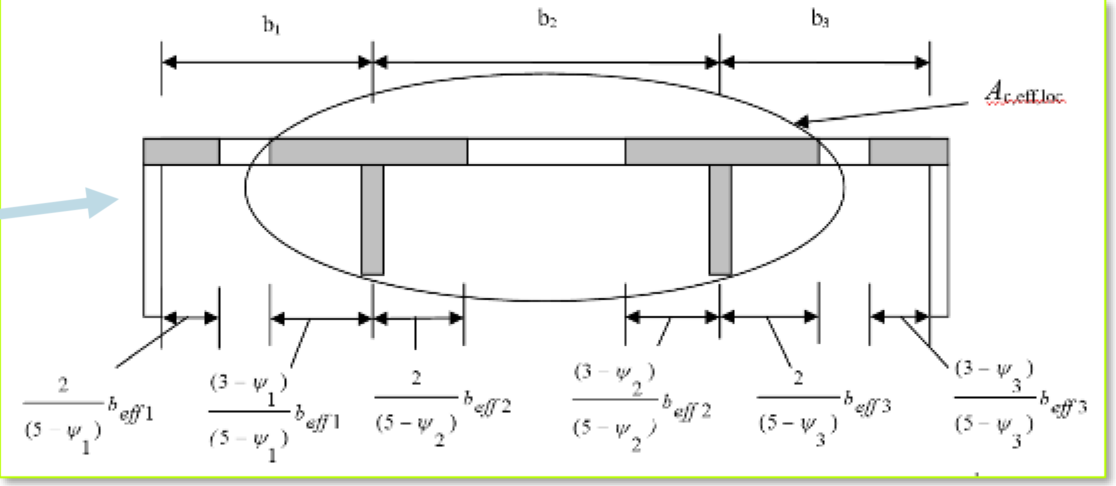
Example changes in EN 1993-1-5 Class 4 Sections

(2) The reduction factor ρ may be taken as

– For internal compression elements:

$$\rho = 1{,}0 \qquad \text{for } \bar{\lambda}_p \leq 0{,}5 + \sqrt{0{,}085 - 0{,}055\,\psi}$$

$$\rho = \frac{\bar{\lambda}_p - 0{,}055\,(3+\psi)}{(\bar{\lambda}_p)^2} \qquad \text{for } \bar{\lambda}_p > 0{,}5 + \sqrt{0{,}085 - 0{,}055\,\psi} \qquad (6.2)$$

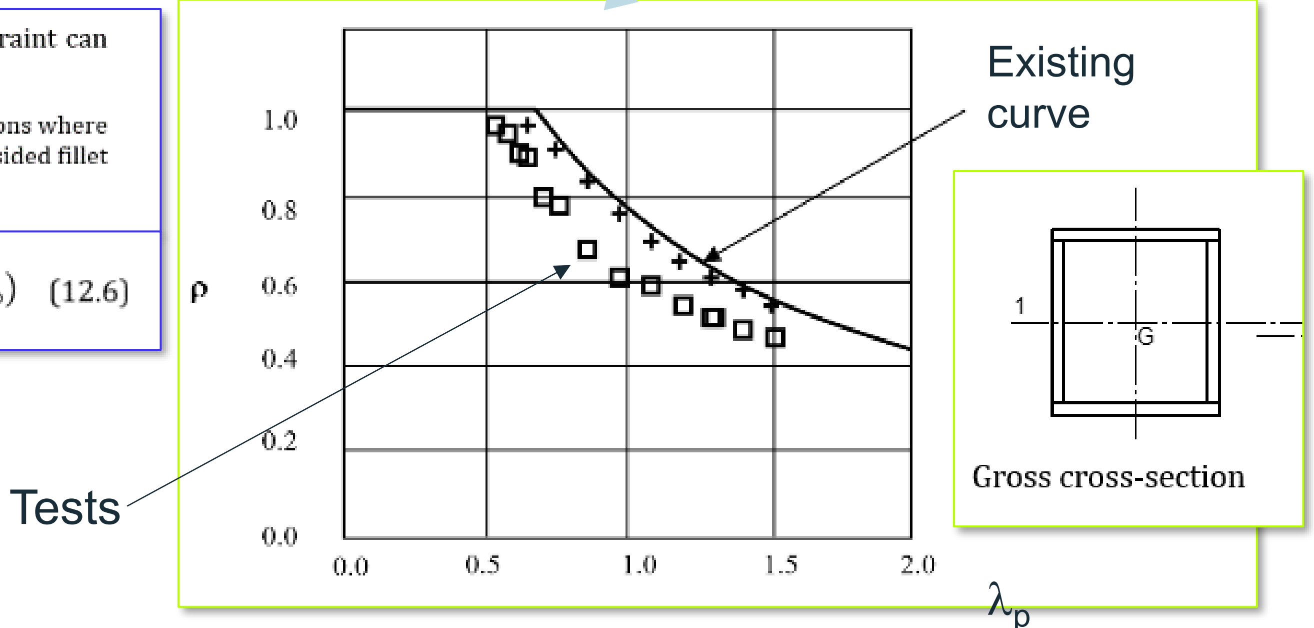


- Parametric study on stub box columns identified that current reduction factors for single plates or plates with no membrane restraint are not safe-sided so addition proposed to cover these cases:

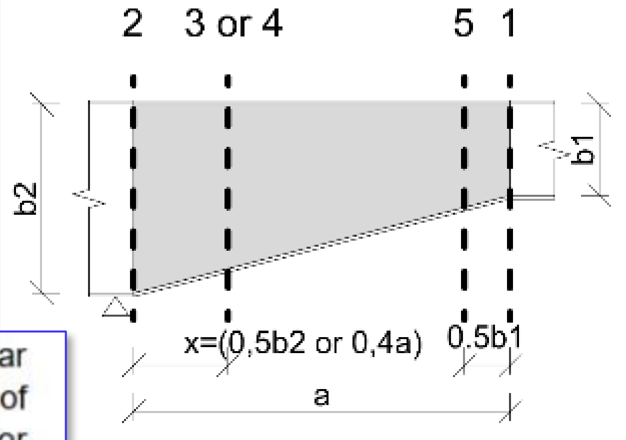
(10) For unstiffened internal compression elements where no membrane or clamping restraint can develop, Formula (6.2) should be substituted by Formula (12.6) with $\alpha_p = 0{,}34$ and $\bar{\lambda}_{p0} = 0{,}70$.

NOTE: This is typically the case of unstiffened square hollow sections under pure compression or situations where the plate into consideration is not connected to adjacent plates with a full penetration weld or with two-sided fillet welds. The National Annex can specify additional situations identified as non-restrained.

$$\rho = \frac{1}{\varphi_p + \sqrt{\varphi_p^2 - \bar{\lambda}_p}} \qquad \text{for } \bar{\lambda}_p > \bar{\lambda}_{p0} \quad \text{with} \quad \varphi_p = \frac{1}{2}\left(1 + \alpha_p\left(\bar{\lambda}_p - \bar{\lambda}_{p0}\right) + \bar{\lambda}_p\right) \qquad (12.6)$$



Examples from EN 1993-1-5
Non-rectangular panels



$$x = \begin{cases} \text{If } \dfrac{M_{0.5b2}}{M_{f,0.5b2}} \geq \dfrac{M_{0.4a}}{M_{f,0.4a}} & x=0{,}5b2 \\ \text{If } \dfrac{M_{0.5b2}}{M_{f,0.5b2}} < \dfrac{M_{0.4a}}{M_{f,0.4a}} & x=0{,}4a \end{cases}$$

If Φ does not exceed 10°, the panel may be assessed assuming it to be rectangular based on larger b_1 and b_2 of the panel, whereby the b_1 smaller and b_2 larger width of panel indicated. If Φ is higher than 10° and less than 17,5°, the corresponding rules for non-rectangular panel should be applied.

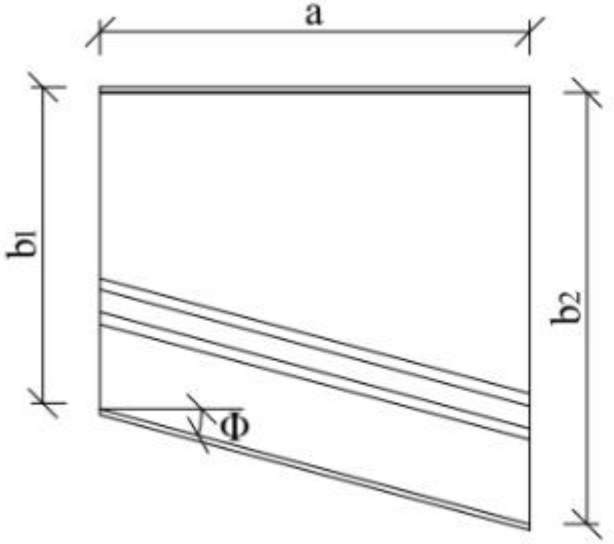
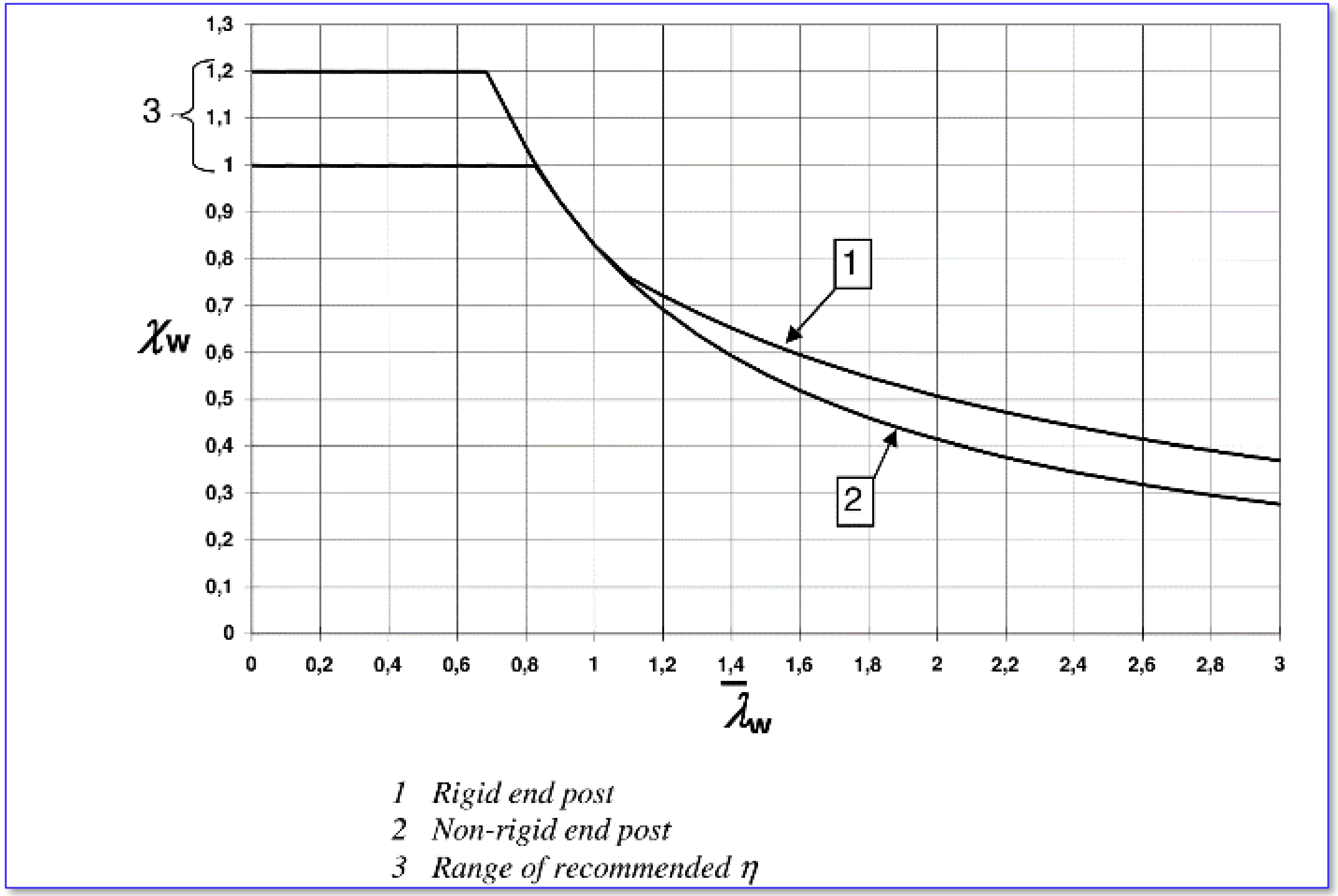


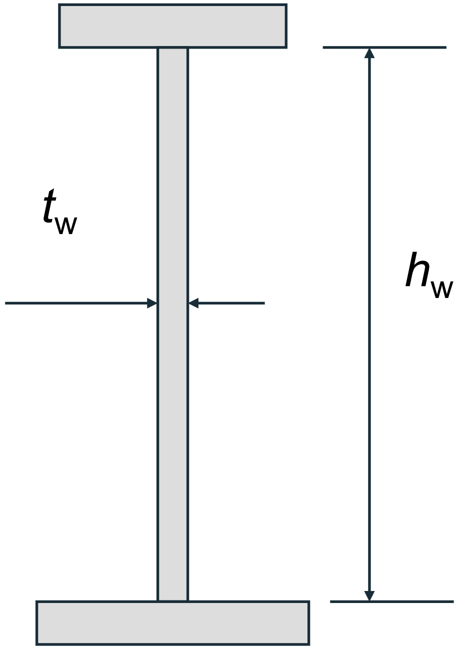
Fig. 3.1: Definition of angle Φ

- Rules for non-rectangular panels clarified and extended from current limit of $\Phi < 10^o$ to $\Phi < 17.5^o$
- Requires checks to be done using the stresses and resistances at a number of discrete cross-sections
- A unique slenderness is calculated at each cross section 3,4 and 5 (because $\sigma_{crit,p}$ changes at each location)

Examples from EN 1993-1-5
Shear – clarified rigid end post case

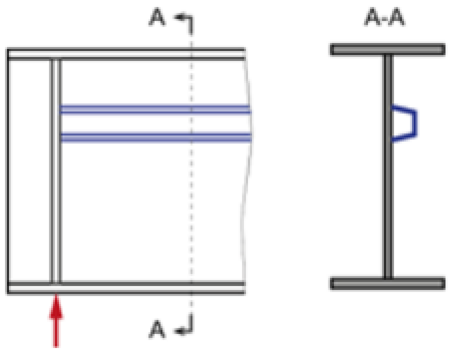


$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3}\gamma_{M1}}$$



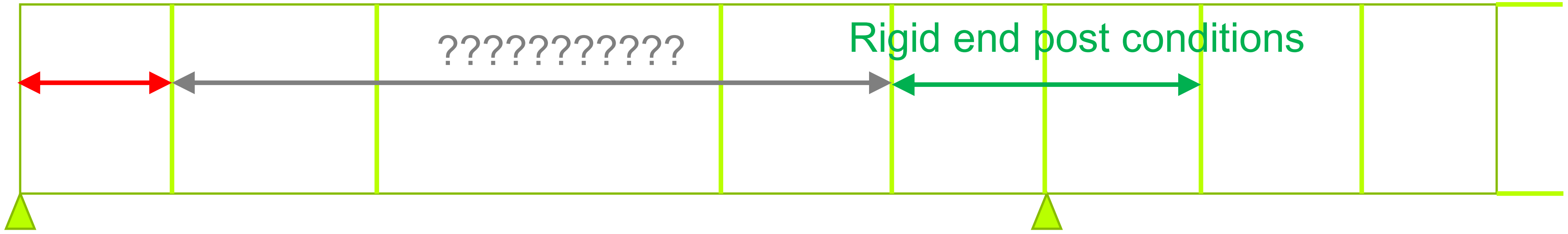
Examples from EN 1993-1-5
Shear – clarified rigid end post case

Rigid end post (two double sided stiffeners or now webs with **closed** longitudinal stiffeners)



Non-rigid end post

Non-rigid end post conditions



??????? = Unclear zone

Examples from EN 1993-1-5
Shear – clarified rigid end post case

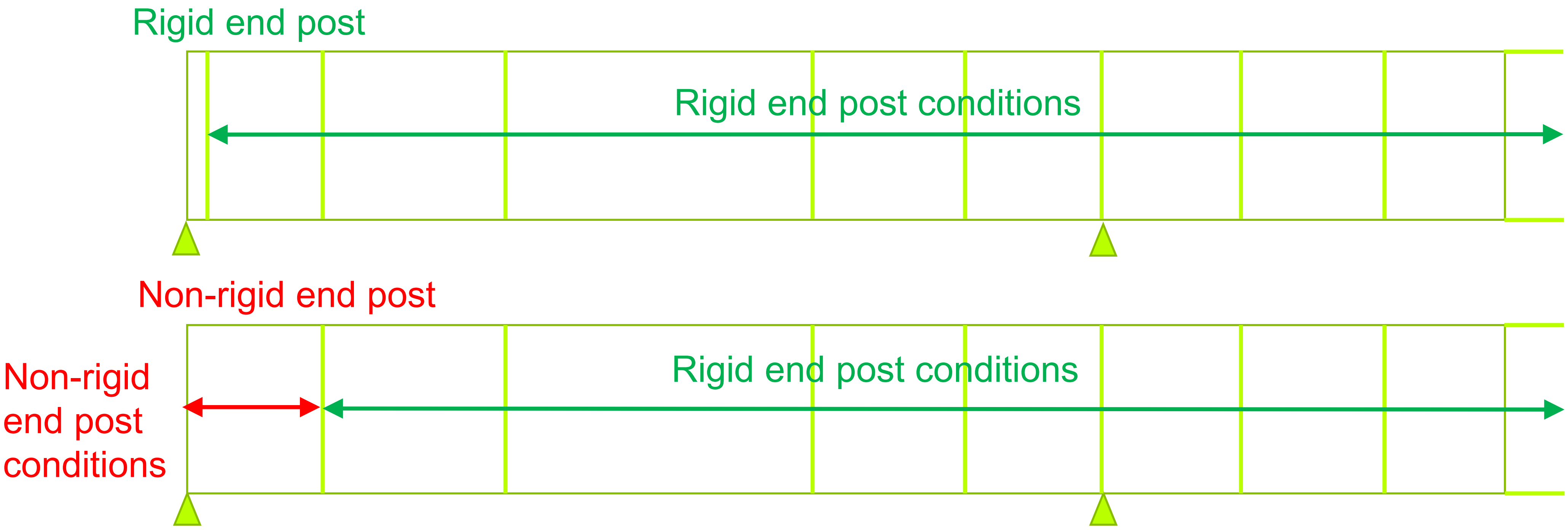


Table 7.1: Contribution from the web χ_w to shear buckling resistance

	End panels with non-rigid end post	All the other cases (intermediate panels and end panels with rigid end post)
$\bar{\lambda}_w < 0{,}83/\eta$	η	η
$0{,}83/\eta \le \bar{\lambda}_w < 1{,}08$	$0{,}83/\bar{\lambda}_w$	$0{,}83/\bar{\lambda}_w$
$\bar{\lambda}_w \ge 1{,}08$	$0{,}83/\bar{\lambda}_w$	$1{,}37/(0{,}7+\bar{\lambda}_w)$

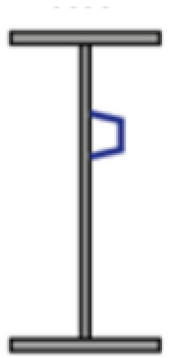
Examples from EN 1993-1-5
Shear – improved resistance with closed longitudinal stiffeners

- The second moment of area of a longitudinal stiffener currently needs to be reduced to 1/3 of its actual value when calculating the shear slenderness

- This reduction is now only required for **open** section longitudinal stiffeners as tests show closed stiffeners increase the shear strength compared to an equivalent open section
- τ_{cr} is now clarified to be determined with hinged boundary conditions for the web if got from FEM

$$\bar{\lambda}_w = 0{,}76\sqrt{\frac{f_{yw}}{\tau_{cr}}} = \frac{h_w}{37{,}4\ t_w\,\varepsilon\sqrt{k_\tau}}$$

Where $\tau_{cr} = k_\tau\,\sigma_E$



(1) For plates with rigid transverse stiffeners and without longitudinal stiffeners or with more than two longitudinal stiffeners, the shear buckling coefficient k_τ should be obtained as follows:

$$k_\tau = 5{,}34 + 4{,}00\,(h_w/a)^2 + k_{\tau s\ell} \quad \text{when } a/h_w \geq 1$$
$$k_\tau = 4{,}00 + 5{,}34\,(h_w/a)^2 + k_{\tau s\ell} \quad \text{when } a/h_w < 1 \qquad \text{(A.3)}$$

Where

$$k_{\tau s\ell} = 9\left(\frac{h_w}{a}\right)^2 \sqrt[4]{\left(\frac{\beta_{sl}\cdot I_{sl}}{t^3\,h_w}\right)^3} \quad \text{but not less than} \quad \frac{2{,}1}{t}\sqrt[3]{\frac{\beta_{sl}\cdot I_{sl}}{h_w}}$$

a is the distance between transverse stiffeners;

$I_{s\ell}$ is the second moment of area of the longitudinal stiffener about the z–z axis, (b). For webs with longitudinal stiffeners, not necessarily equally spaced, $I_{s\ell}$ is the sum of the moment of inertia of the individual stiffeners;

$\boxed{\beta_{sl}\ \text{is equal to 1,0 for open-section longitudinal stiffeners and is equal to 3,0 for closed-section longitudinal stiffeners.}}$

Examples from EN 1993-1-5
M-V Interaction improvements

- Extended numerical study was performed on the design of longitudinally stiffened and unstiffened plate girders subjected to M-V interaction. The results showed that the current M-V interaction is not always safe-sided but this is the case only for certain geometries with small flanges and large webs

7.1 Interaction between shear force, bending moment and axial force

(1) Provided that $\bar{\eta}_3$ (see below) does not exceed 0,5 , the design resistance to bending moment and axial force need not be reduced to allow for the shear force. If $\bar{\eta}_3$ is more than 0,5 the combined effects of bending and shear in the web of an I or box girder should satisfy:

$$\bar{\eta}_1 + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right)\left(2\bar{\eta}_3 - 1\right)^2 \leq 1{,}0 \qquad for\ \bar{\eta}_1 \geq \frac{M_{f,Rd}}{M_{pl,Rd}} \tag{7.1}$$

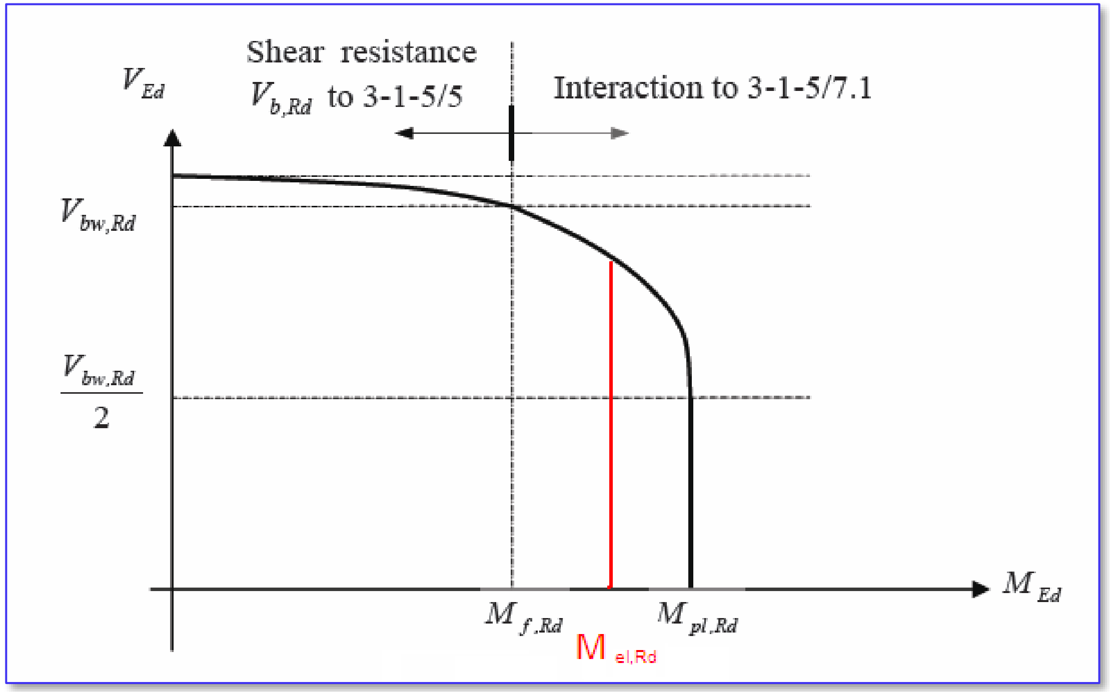
where $M_{f,Rd}$ is the design plastic moment of resistance of the section consisting of the effective area of the flanges;

$M_{pl,Rd}$ is the design plastic resistance of the cross section consisting of the effective area of the flanges and the fully effective web irrespective of its section class.

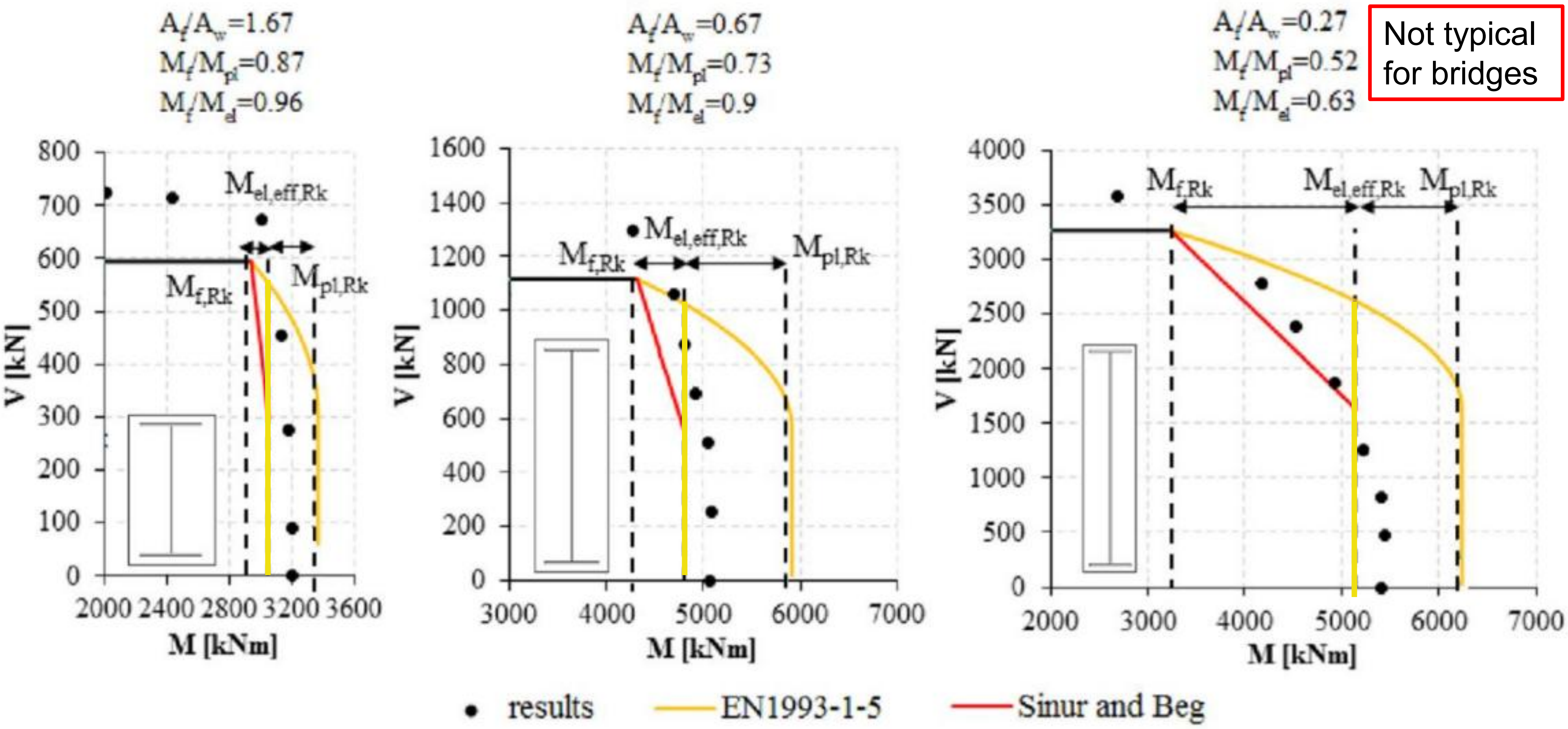
$$\bar{\eta}_1 = \frac{M_{Ed}}{M_{pl,Rd}}$$

$$\bar{\eta}_3 = \frac{V_{Ed}}{V_{bw,Rd}}$$ AC for $V_{bw,Rd}$ see expression (5.2). AC

Current rule



Examples from EN 1993-1-5
M-V Interaction improvements



Examples from EN 1993-1-5
M-V Interaction improvements

THE FUTURE OF STEEL CONSTRUCTION

9.1 Interaction between shear force, bending moment and axial force

(1) Provided that $\bar{\eta}_3$ (see below) does not exceed 0,5, the design resistance to bending moment and axial force need not be reduced to allow for the shear force. If $\bar{\eta}_3$ is more than 0,5 the combined effects of bending and shear in the web of an I or box girder should satisfy:

$$\eta_1 + \left(1 - \frac{M_{f,Rd}}{M_{eff,Rd}}\right)\left(2\bar{\eta}_3 - 1\right)^{\mu} \leq 1,0 \quad \text{for } \eta_1 \geq \frac{M_{f,Rd}}{M_{eff,Rd}} \tag{9.1}$$

where

$M_{f,Rd}$ is the characteristic plastic moment of resistance of the cross-section consisting of the effective area of the flanges only;

$$M_{eff,Rd} = W_{eff}\, f_y / \gamma_{M0} \tag{9.2}$$

η_1 see 6.7(1)

New rule

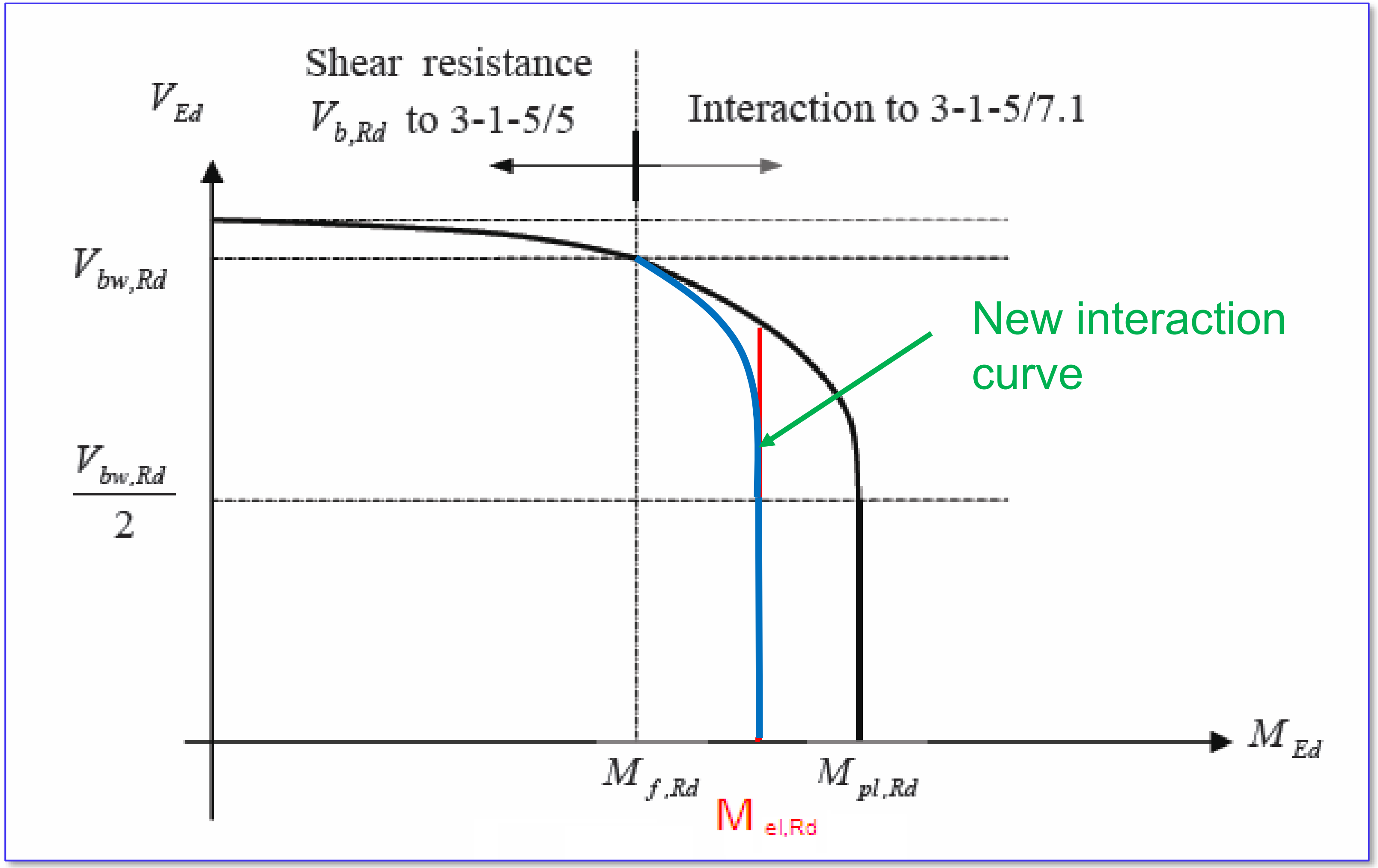
$$\bar{\eta}_3 = \frac{V_{Ed}}{V_{bw,Rd}} \tag{9.3}$$

For $V_{bw,Rd}$, see Formula (7.2)

$$\mu = \left(\frac{M_{f,Rd}}{M_{eff,Rd}} + 0,2\right)^{15} + 1 \tag{9.4}$$

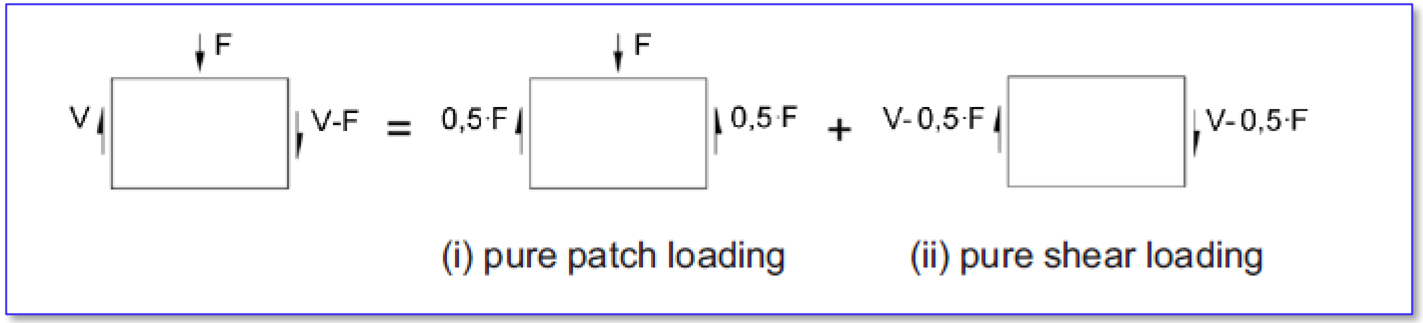
μ varies between 1.0 and 16

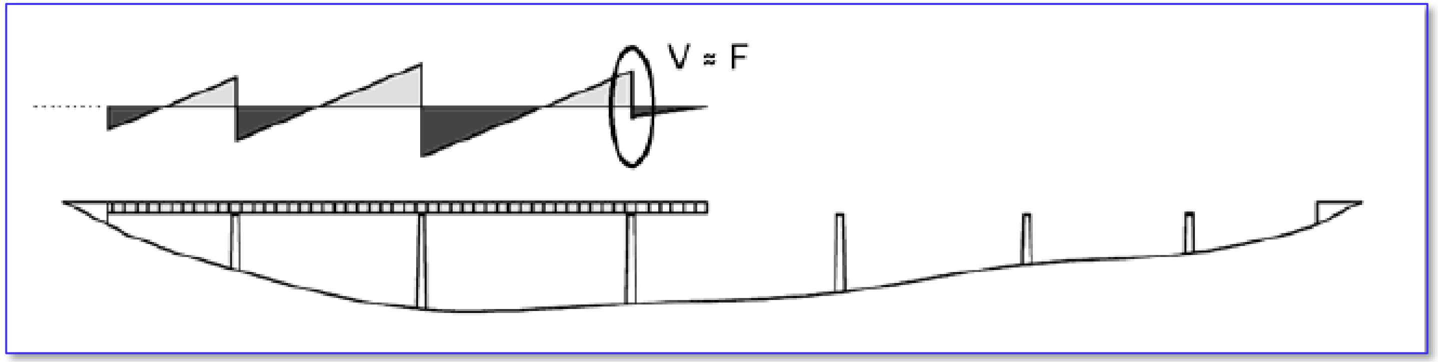
Examples from EN 1993-1-5
M-V Interaction improvements



Examples from EN 1993-1-5
Patch loading – addressing an omission

- F-M-V interaction is not covered in first generation EN 1993-1-5 (only F-M)
- Important for bridge launches for example
- Needs addition for safety





Examples from EN 1993-1-5
Patch loading – addressing an omission

- F-M-V interaction is not covered in first generation EN 1993-1-5 (only F-M)
- Important for bridge launches for example
- Needs addition for safety

9.3 Interaction between transverse force, bending moment and shear force

(1) This interaction should be verified if $\eta_2 > 0{,}1$ with η_2 defined in 8.6(1). If $\eta_2 \leq 0{,}1$, the verification is limited to a bending moment and shear force interaction according to 9.1.

Important because otherwise this is more conservative than M-V interaction at zero patch load

(2) If the girder is subjected to a concentrated transverse force acting on the compression flange in conjunction with bending moment and shear force, the resistance should be verified using 6.7, 7.5, 8.6 and the following interaction formula:

$$(\bar{\eta}_1)^{3,6} + \left[\bar{\eta}_3 \left(1 - \frac{F_{\mathrm{Ed}}}{2\,V_{\mathrm{Ed}}}\right)\right]^{1,6} + \eta_2 \leq 1{,}0 \qquad (9.6)$$

Where

$$\bar{\eta}_1 = \frac{M_{\mathrm{Ed}}}{M_{\mathrm{f,eff,Rd}}}$$

$M_{\mathrm{f,eff,Rd}}$ is the design plastic moment of resistance of the cross section consisting of the effective area of the flanges and the fully effective web irrespective of its section class.

$\bar{\eta}_3$ see 9.1(1)

$$(\bar{\eta}_1)^{3,6} + \left[\bar{\eta}_3 \left(1 - \frac{F_{\mathrm{Ed}}}{2\,V_{\mathrm{Ed}}}\right)\right]^{1,6} + \eta_2 \leq 1{,}0$$

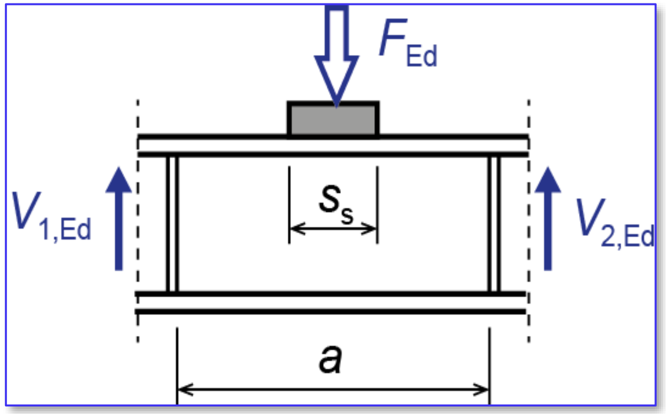
Bending utilisation

Shear utilisation

Patch load utilisation

Examples from EN 1993-1-5
Patch loading – improving the accuracy

- Current resistance overestimates patch loading resistance with hybrid girders (flanges with greater f_y) and underestimated for other girders



6.4 Reduction factor χ_F for effective length for resistance

(1) The reduction factor χ_F should be obtained from:

$$\chi_F = \frac{0{,}5}{\bar{\lambda}_F} \leq 1{,}0$$

where $\bar{\lambda}_F = \sqrt{\frac{\ell_y t_w f_{yw}}{F_{cr}}}$

$$\boxed{F_{cr} = 0{,}9\, k_F E \frac{t_w^3}{h}}$$

k_F from formula only

Current rule

8.4 Reduction factor χ_F

(1) The reduction factor χ_F should be obtained from:

$$\chi_F = \frac{1{,}0}{\varphi_F + \sqrt{\varphi_F^2 - \bar{\lambda}_F}} \leq 1{,}0$$

New rule

where:

$$\varphi_F = \frac{1}{2}\left(1 + \alpha_{F0}\left(\bar{\lambda}_F - \bar{\lambda}_{F0}\right) + \bar{\lambda}_F\right)$$

$$\bar{\lambda}_F = \sqrt{\frac{\ell_y t_w f_{yw}}{F_{cr}}}$$

$$\alpha_{F0} = 0{,}75$$

$$\bar{\lambda}_{F0} = 0{,}50$$

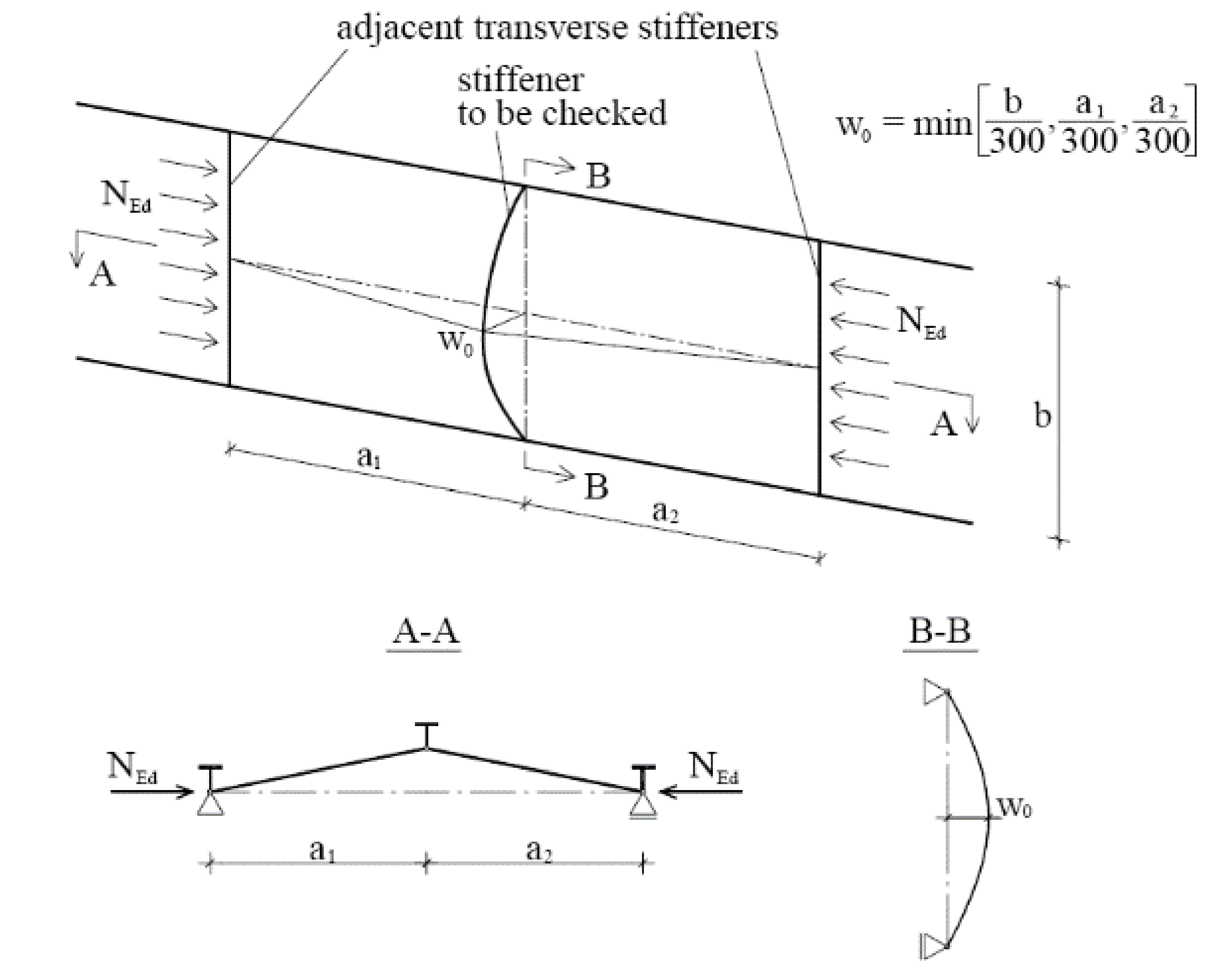
$$\boxed{F_{cr} = 0{,}9\, k_F E \frac{t_w^3}{h_w}}$$

k_F from formula or FEM (allows for beneficial torsional stiffness of closed stiffeners)

Examples from EN 1993-1-5
Transverse stiffener improvements

Existing rules:

- Current rule used simple mechanical model
- Transverse stiffener under consideration has initial bow
- Creates a kink in the web which pushes the stiffener out of plane
- Second order problem
- Critical forces for web panel based on shortest panel length, even though mode of buckling being prevented involves a_1 + a_2

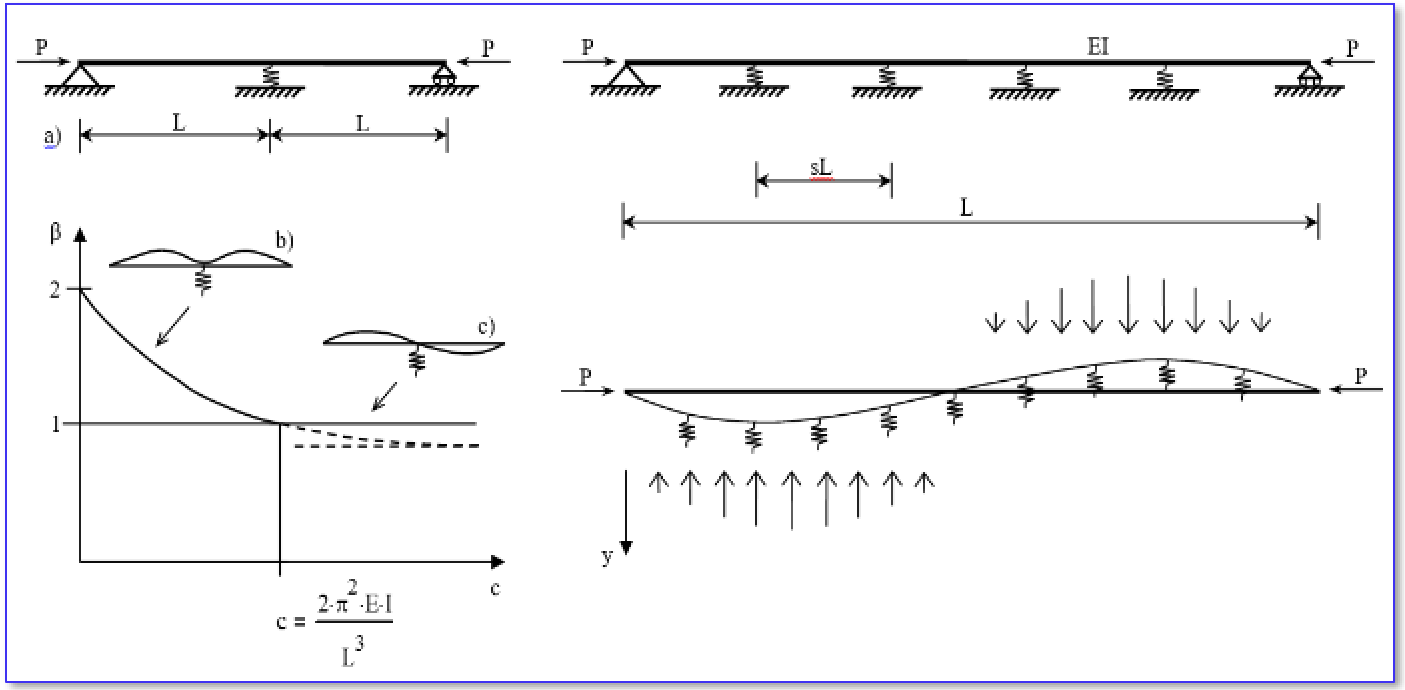


Examples from EN 1993-1-5
Transverse stiffener improvements

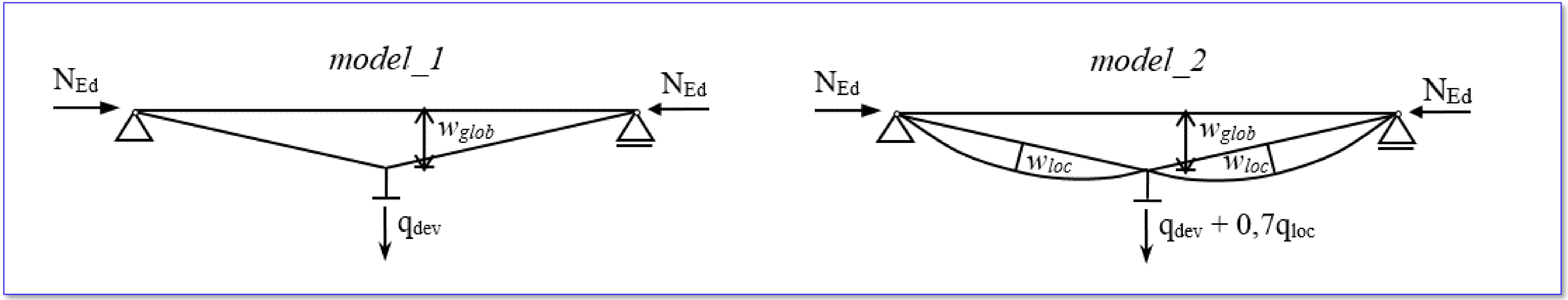
Existing rules:

Two problems identified:

(i) Stiffness of transverse stiffener is important or other modes may occur

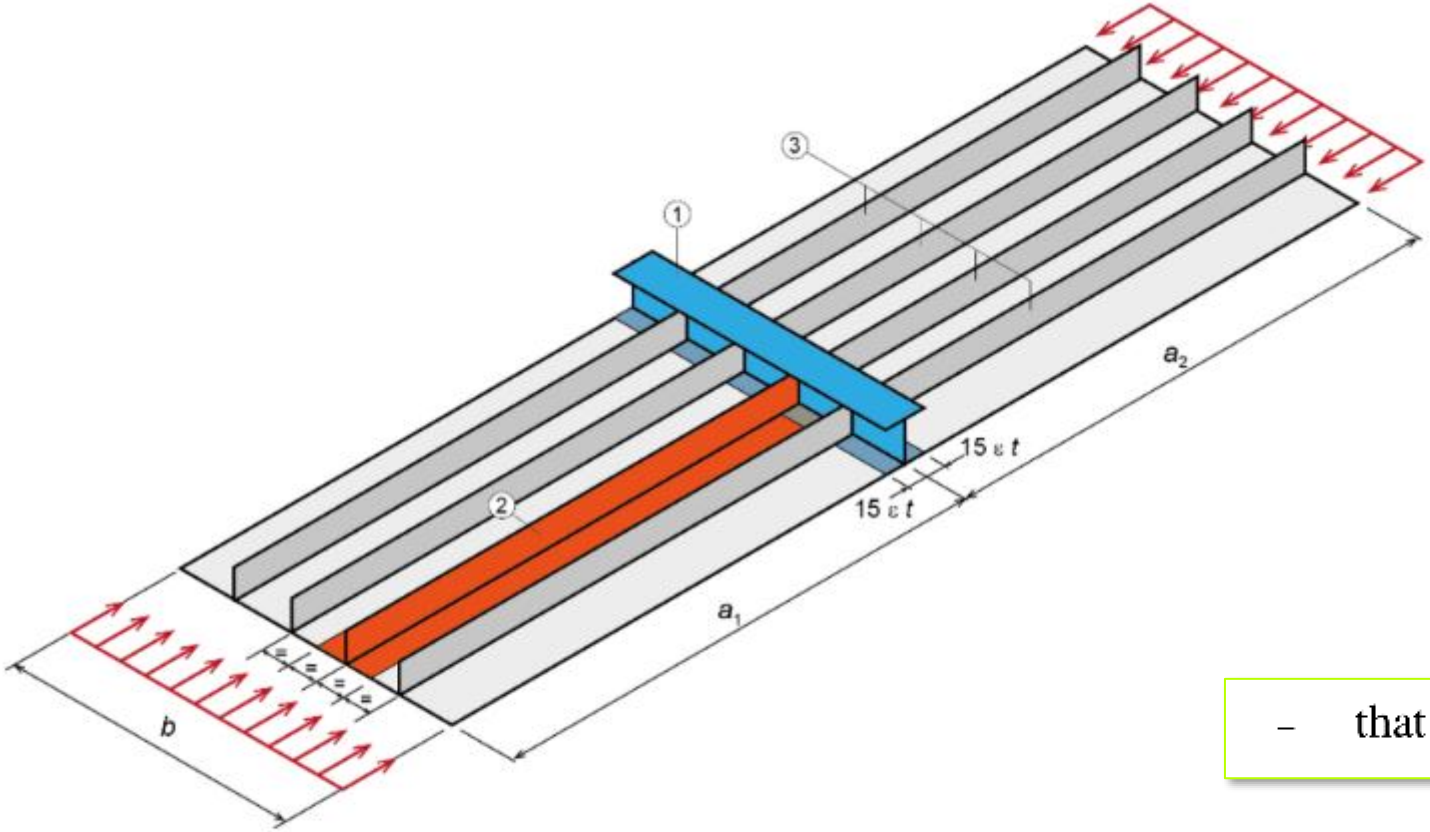


(ii) Local imperfection is important



Examples from EN 1993-1-5
Transverse stiffener improvements

New rules – maintains same mechanical model with different limits



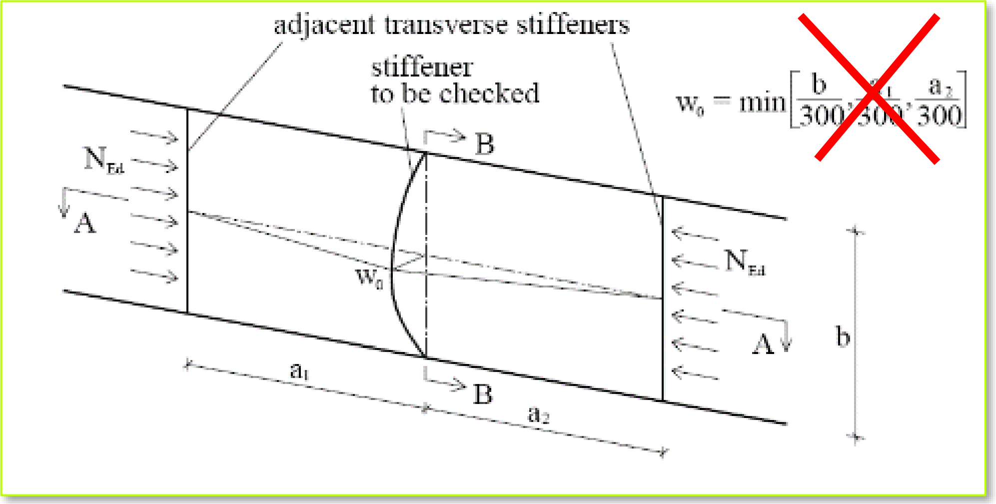
1 – Transverse stiffener: minimum second moment of area = I_{st}

2 – Longitudinal stiffener: Out-of-plane inertia I_b for the cross-section

3 – n number of the longitudinal stiffeners

Figure 11.3: Example of a box girder flange under pure compression

$b/200$, $a_1/200$, $a_2/200$



– that the additional deflection should not exceed ~~b/300~~.

$b/500$

Examples from EN 1993-1-5
Transverse stiffener improvements

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New rules – maintains same mechanical model with different limits

(2) The transverse stiffener should be treated as a simply supported member subject to lateral loading with an initial sinusoidal imperfection w_0 equal to $s/200$, where s is the smallest of a_1, a_2 or b, see Figure 11.2 , where a_1 and a_2 are the lengths of the panels adjacent to the transverse stiffener under consideration and b is the height between the centroids of the flanges or span of the transverse stiffener. Eccentricities should be accounted for.

(4) It should be verified that using a second order elastic method analysis all the following criteria are satisfied at the ultimate limit state:

- that the maximum stress in the stiffener should not exceed f_y/γ_{M1};
- that the additional deflection should not exceed b/500;
- that the second moment of area I_{st} of the transverse stiffener is not less than:

$$I_{st} = \frac{3.92 \cdot n \cdot I_b \cdot b^3}{\pi^2 \cdot a^3} \tag{11.1}$$

where: I_b out-of-plane inertia of the longitudinal stiffener with adjacent plate part,

n number of longitudinal stiffeners within the plate,

$a=min(a_1,a_2)$ minimum length of the investigated panels.

Examples from EN 1993-2

Moderate change

Eurocode 3 — Design of steel structures — Part 2: Steel Bridges

Eurocode 3 — Bemessung und konstruktion von Stahlbauten — Teil 2: Stahlbrücken

Eurocode 3 — Calcul des structures en acier — Partie 2 : Ponts métalliques

Eurocode is not yet published by BSI

Examples from EN 1993-2 Bracing design

- Current EN 1993-2 force for bracings is **too large** - uses imperfection of approximately $L_e/200$ for buckling of strut itself
- Revised version uses $L_w/500$ based on the actual geometrical bow (compatible with bow in EN 1993-1-1-), but also over half-wavelength not effective length
- BS5400 used L_w / 667

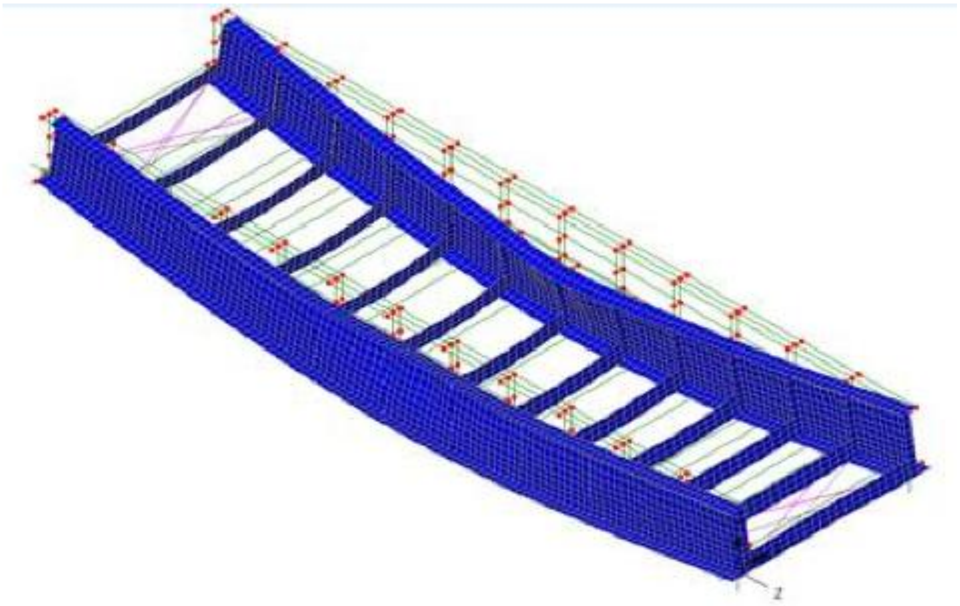
$$F_{Ed} = \frac{N_{Ed}}{100} \text{ if } l_k \leq 1{,}2l$$

$$F_{Ed} = \frac{l}{l_k}\frac{N_{Ed}}{80}\frac{1}{1-\frac{N_{Ed}}{N_{crit}}} \text{ if } l_k > 1{,}2l$$

where

$$l_k = \pi\sqrt{\frac{EI}{N_{crit}}}$$

ℓ is the distance between the springs.



(4) For chords in compression or the bottom flanges of continuous girders between rigid supports, the effect of initial imperfections and second order effects on a supporting spring may be taken into account by a first order calculation, by applying an additional lateral force F_{Ed} at the connection of the chord to the spring according to the Formulae (8.5):

$$F_{Ed} = \frac{N_{Ed}}{100} \qquad \text{if } L_{cr} \leq 1{,}2\, d_s$$

$$F_{Ed} = \frac{N_{Ed}}{N_{cr} - N_{Ed}} \times \frac{C_d\, L_w}{500} \qquad \text{if } L_{cr} > 1{,}2\, d_s \qquad (8.5)$$

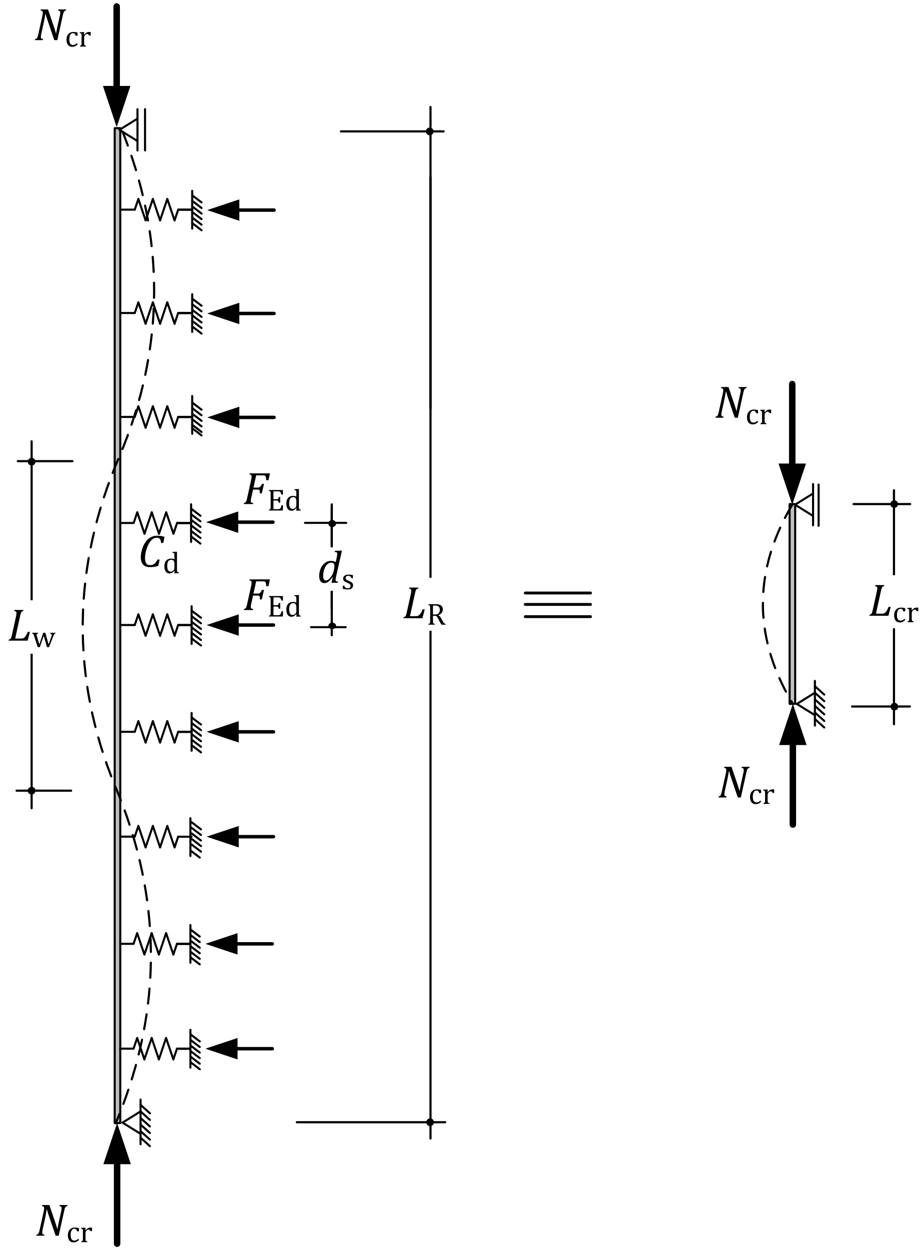
where

$$L_{cr} = \pi\sqrt{\frac{EI}{N_{cr}}}$$

L_w is the half wavelength of buckling determined by taking L/L_w as the next integer below L/L_{cr} but not less than unity;

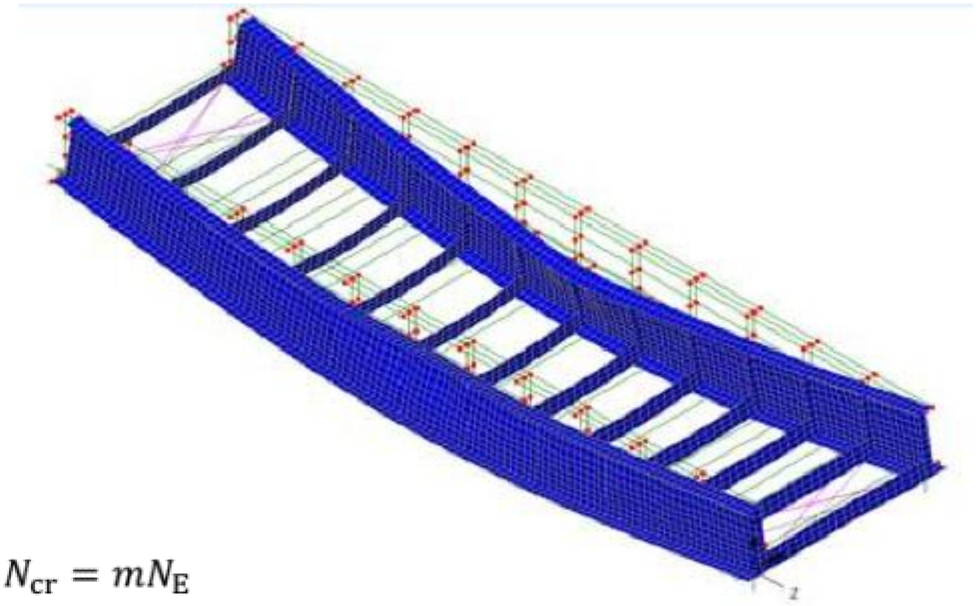
Examples from EN 1993-2 Bracing design

- Current EN 1993-2 force for bracings is **too large** - uses imperfection of approximately $L_e/200$ for buckling of strut itself
- Revised version uses $L_w/500$ based on the actual geometrical bow (compatible with bow in EN 1993-1-1-), but also over half-wavelength not effective length
- BS5400 used L_w / 667



Examples from EN 1993-2
Elastic buckling moment

- The calculation of critical flange force, based on buckling factor "*m*", in EN1993-2 clause 6.3.4.2 did not cover the possibility of there being no rigid restraints at beam supports
- Cautionary note added: If there are no rigid lateral supports at girder ends to define the length *L*, ……………… , the value of *m* should be modified in the last half wavelength of buckling to allow for the end restraint flexibility in accordance with elastic buckling theory
- A formula for *m* adjusted in this way is provided in section 9 of the BSI document **PD 6695-2:2008 - Recommendations for the design of bridges to BS EN 1993**



$$N_{cr} = mN_E$$

where

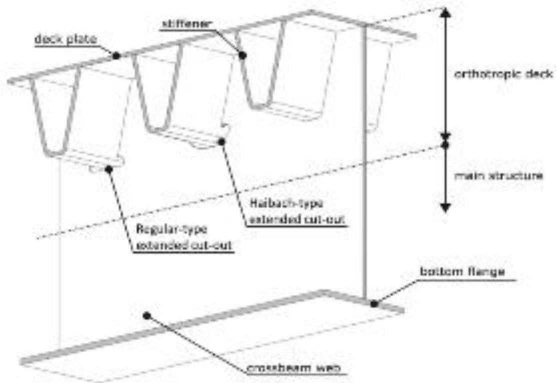
$$N_E = \pi^2 \frac{EI}{L^2}$$

$$m = \frac{\sqrt{\gamma}}{\left(\frac{\pi}{\sqrt{2}} + \frac{0,69}{X + 0,5}\right)^2};$$

with $X = \frac{C_e}{\sqrt{2}}\left(\frac{l^3}{C_d^3 EI}\right)^{0,25}$ and C_e is the stiffness of the end support determined in the same way as the stiffness of intermediate supports, C_d, other terms are as defined in BS EN 1993-2:2006, **6.3.4.2**.

Examples from EN 1993-2 Fatigue of orthotropic plates

- Fatigue calculations and classifications for orthotropic decks covered in - CEN/TC 250/TS 1993-1-901
- This provides a more detailed assessment approach than provided in EN 1993-2 Annex C based on hot spot stress method for toe cracking and nominal stress for weld cracking
- Different details for toe cracking have categories based on 100 MPa x f:



CEN/TC 250

Date: 20XX -XX

prTS 1993-1-901

Secretariat: XXX

CEN/TC 250/TS 1993-1-901 — Fatigue design of orthotropic bridge decks with the hot spot stress method

Einführendes Element — Haupt-Element — Ergänzendes Element

Élément introductif — Élément central — Élément complémentaire

$$\Delta\sigma_C = f \cdot \Delta\sigma_{C,Table}$$

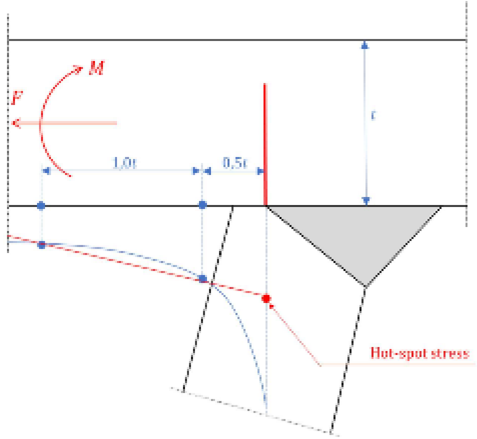


Figure 6.1: Hot-spot stress for single sided welds where a crack initiates from the root into a plate, example for linear extrapolation with "fine" mesh model.

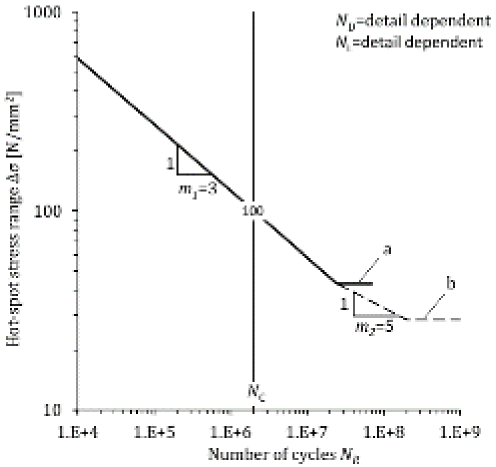
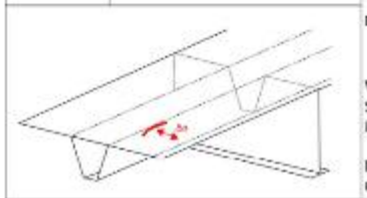


Figure 7.1: Characteristic fatigue resistance curves for single sided welds where a crack initiates from the root into a plate.

Detail	Description	f	$f \cdot \Delta\sigma_{C,Table}$	N_D	N_L	m_1	m_2	Detail category specific requirements
C1a	Stiffener-to-deck plate weld, weld toe crack in deck plate	1,25	125	5	100	3	5	$t_{dp} \geq 18$mm
		1,25	125	5	100	3	5	14mm < t_{dp} < 18mm
		1,40	140	5	100	3	5	10mm ≤ t_{dp} ≤ 14mm



Execution requirements

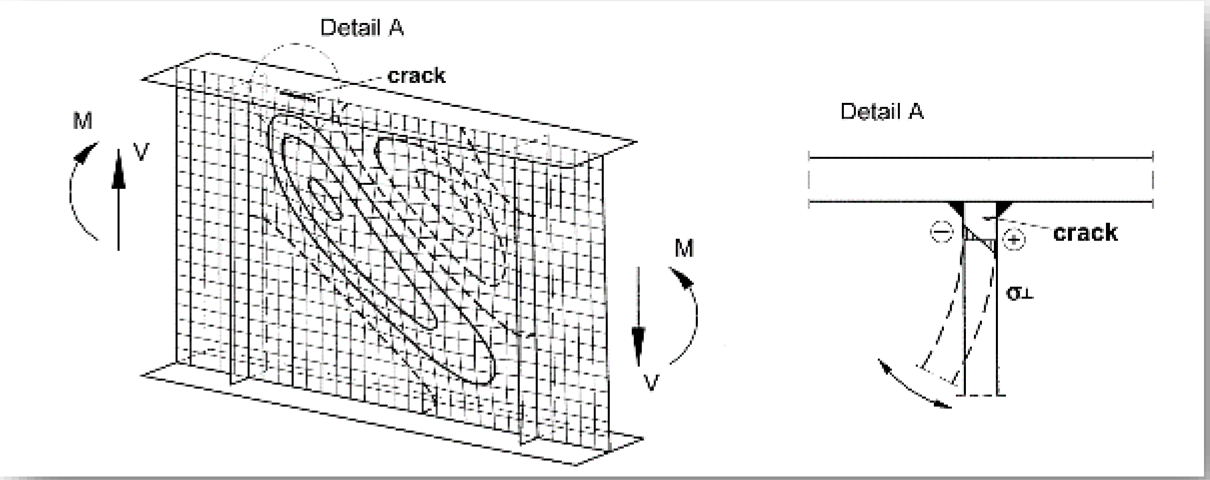
Gap (fit between plates)	h_1 = 0 mm for >90% of weld length
	h_1 < 0,5mm for <10% of weld length
Lack of penetration	h_2 ≤ 1,0 mm as average value (Note: relax)
	h_2 ≤ 1,5 mm as maximum value (Note: relax)
Extension of weld	h_3 = t_s +/- 1mm
	h_4 = t_s +/- 1mm

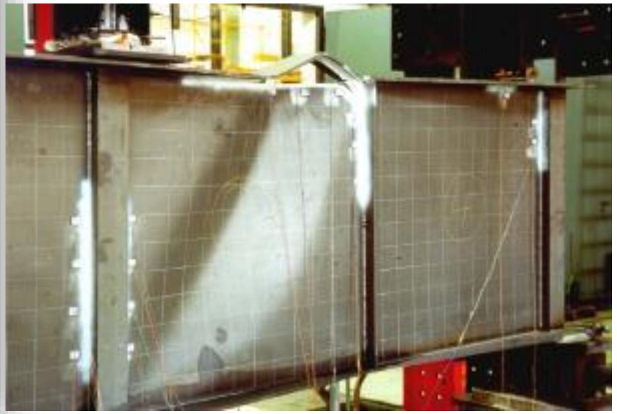
Weld geometry
Smooth transition of weld toes at deck plate and at stringer web.
Blow through of the weld at the inner side of the stringer is not allowed.

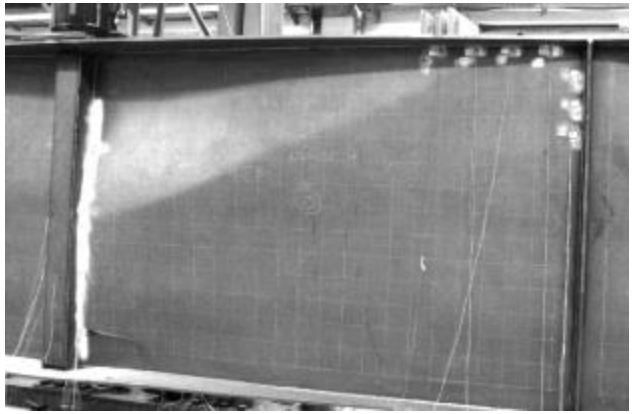
Preparation
Chamfer stringer web

Examples from EN 1993-2
Web breathing

- Slender webs contain initial out-of-plane geometric imperfections which grow under action of in-plane bending and shear stresses
- Stresses in slender webs of steel plate girders can exceed the linear elastic buckling stresses - then relatively large out-of-plane deformations of the web plate may result
- Out-of-plane cyclic deformation known as web breathing - can initiate fatigue cracks





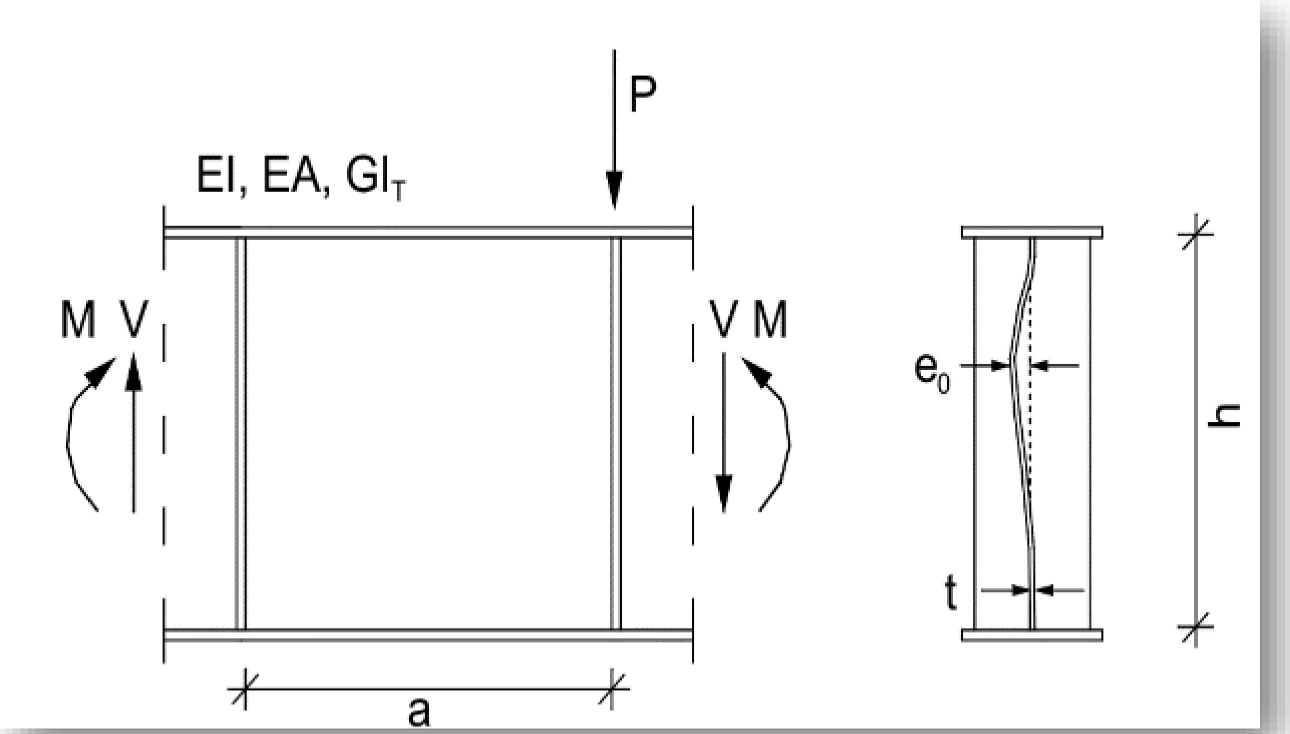


Examples from EN 1993-2
Web breathing

Introduction to web breathing

The magnitude of web breathing is complex non-linear calculation dependent on very many parameters, including:

- Panel length, *a*
- Panel height, *h*
- Web thickness, *t*
- Panel aspect ratio $\alpha = a/h$
- Web slenderness $\beta = h/t$
- Shape and magnitude of initial web geometric imperfection, e_0
- Magnitude of in-plane direct stresses, σ_0, and shear stresses, τ_0, produced by M and V
- Boundary stiffnesses: bending stiffness *EI,* normal stiffness *EA* and torsional rigidity *GI*T



Examples from EN 1993-2
Web breathing

- Large number of parametric studies conducted on typical highway and railway bridges; simple slenderness limitations were derived for longitudinally unstiffened web panels

7.4 Limitation of web breathing

(1) The slenderness of web plates should be limited to avoid excessive breathing that might result in fatigue at or adjacent to the web-to-flange connections.

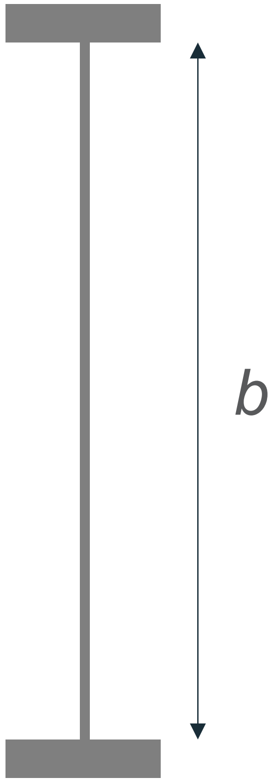
NOTE: The National Annex may define cases where web breathing checks are not necessary.

(2) Web breathing may be neglected for web panels without longitudinal stiffeners or for subpanels of stiffened webs, where the following criteria are met:

$$b/t \leq 30 + 4{,}0\,L \quad \leq 300 \qquad \text{for road bridges} \tag{7.5}$$

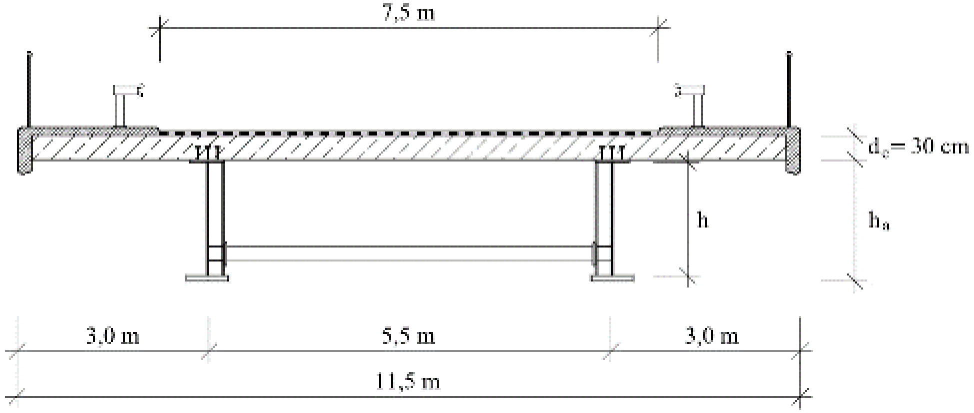
$$b/t \leq 55 + 3{,}3\,L \quad \leq 250 \qquad \text{for railway bridges} \tag{7.6}$$

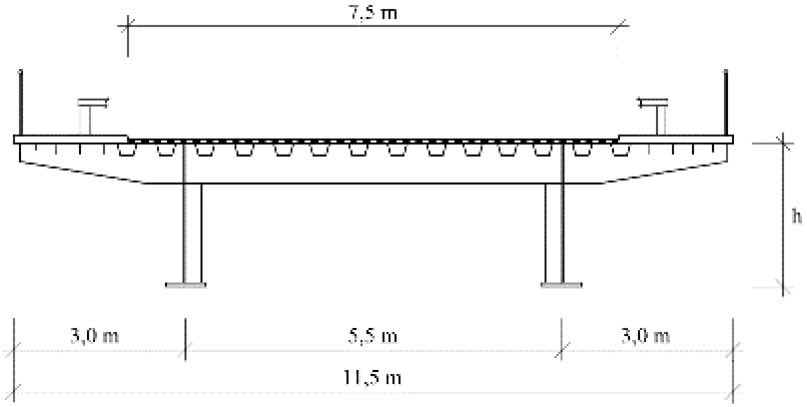
where L is the span length in m, but not less than 20 m.

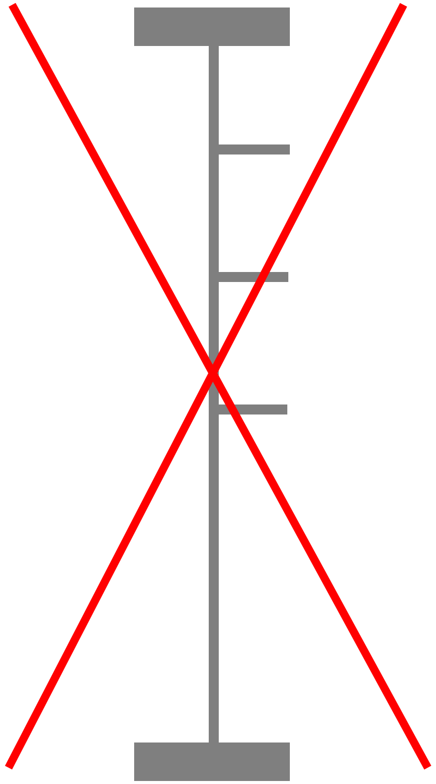


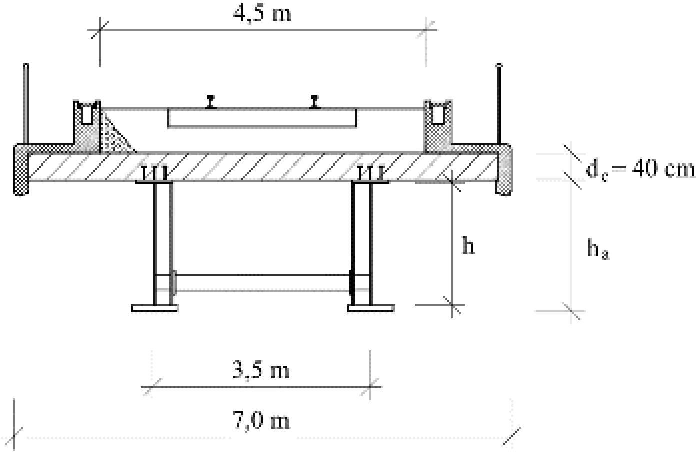
Examples from EN 1993-2
Web breathing

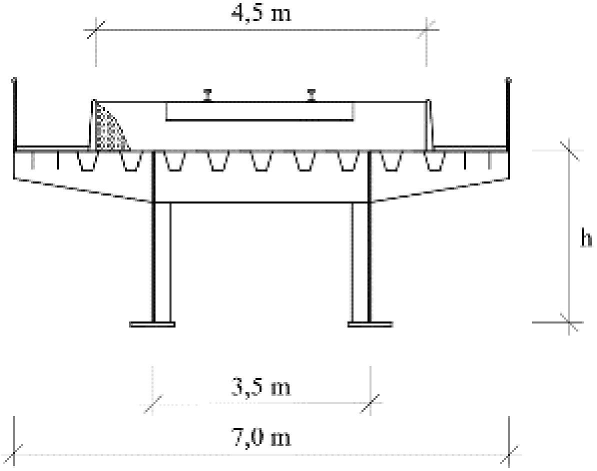
- Study looked at typical bridge geometries/spans and web slenderness h/t and evaluated both fatigue (due to in-plane and out-of-plane deformations) and ULS conditions
- But longitudinally stiffened webs were not considered – the sub-panels and overall stiffened web panel had to meet a more complex interaction check











Examples from EN 1993-2
Web breathing

- Webs that were non-compliant with simple rules (including those with longitudinal stiffeners) needed to be checked with a more complex approach that was very conservative:

(3) If the provision in (2) is not satisfied web breathing should be checked as follows:

$$\sqrt{\left(\frac{\sigma_{x,Ed,ser}}{k_\sigma\, \sigma_E}\right)^2 + \left(\frac{1{,}1\, \tau_{x,Ed,ser}}{k_\tau\, \sigma_E}\right)^2} \leq 1{,}1 \qquad (7.7)$$

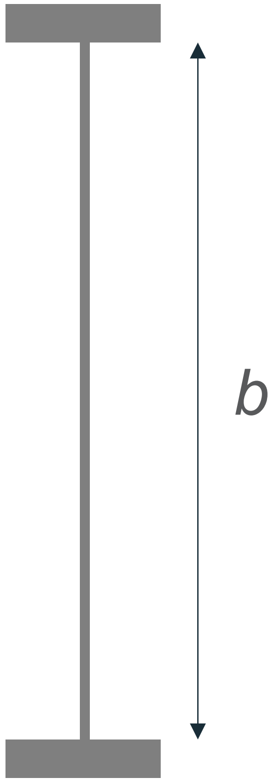
where $\sigma_{x,Ed,ser}$, $\tau_{Ed,ser}$ are the stresses for the frequent load combination. If the stresses are not uniform along the length of the panel, see section 4.6(3) of EN 1993-1-5;

k_σ, k_τ are the linear elastic buckling coefficients assuming hinged edges of the panel, see EN 1993-1-5;

AC1 $\sigma_E = 190000\left(\frac{t}{b_p}\right)^2 \quad [N/mm^2]$ AC1

b_p is the smaller of a and b.

NOTE: For stresses varying along the panel see EN 1993-1-5, 4.6(3).



Examples from EN 1993-2
Web breathing

9.4 Limitation of web breathing

(1) The slenderness of web plates should be limited to avoid excessive breathing that might result in fatigue at or adjacent to the web-to-flange connections.

NOTE The National Annex can define cases where web breathing checks are not necessary.

(2) Web breathing may be neglected for unstiffened panels where the condition (9.5) or (9.6) is met:

$$b_p/t \leq 30 + 4{,}0\,L \leq 300 \quad \text{for road bridges} \tag{9.5}$$

$$b_p/t \leq 55 + 3{,}3\,L \leq 250 \quad \text{for railway bridges} \tag{9.6}$$

where

b_p is the width of the panel

t is the thickness of the panel

L is the span length in metres, but not less than 20 m.

(3) Web breathing may be neglected for longitudinally stiffened web panels with a relative stiffness $\gamma^*_{s\ell,i}$ according to EN 1993-1-5, 6.5.1(5) greater than 25, where the condition (9.5) or (9.6) is met for each subpanel, assuming b as the width of the subpanel.

- More recent study has now considered longitudinally stiffened girders
- No need now to consider conservative alternative method to consider overall stiffened panel if sub-panels also meet (9.5)/(9.6) and stiffeners meet minimum stiffness requirement

(3) If the provision in (2) is not satisfied web breathing should be checked as follows:

$$\sqrt{\left(\frac{\sigma_{x,Ed,ser}}{k_\sigma\,\sigma_E}\right)^2 + \left(\frac{1{,}1\,\tau_{x,Ed,ser}}{k_\tau\,\sigma_E}\right)^2} \leq 1{,}1 \tag{7.7}$$

where $\sigma_{x,Ed,ser}$, $\tau_{Ed,ser}$ are the stresses for the frequent load combination. If the stresses are not uniform along the length of the panel, see section 4.6(3) of EN 1993-1-5;

k_σ, k_τ are the linear elastic buckling coefficients assuming hinged edges of the panel, see EN 1993-1-5;

AC1 $\sigma_E = 190000\left(\frac{t}{b_p}\right)^2$ $[N/mm^2]$ AC1

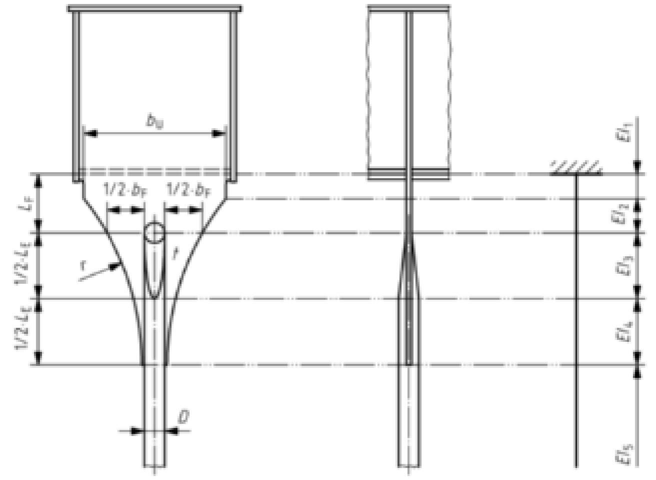
b_p is the smaller of a and b.

NOTE: For stresses varying along the panel see EN 1993-1-5, 4.6(3).

Examples from EN 1993-2
New Annex A for Design of hangers for tied-arch bridges

Covers:

- Deemed to satisfy geometry for which no fatigue check needed
- Fatigue from cable vortex shedding, wind-rain, galloping and traffic addressed
- Design rules for:
  - Welded round bar hangers
  - Flat steel plates
  - Ropes
- But rules very prescriptive in terms of adherence to geometry
- Were originally normative and now informative rules



Key:

Diameter of hanger: $D = 2\sqrt{\frac{N_{max}}{\pi\ \sigma}}$

Thickness of gusset plate: $t = 0,2\ D$

Width at cope hole level: $b_F = \frac{N_{max}}{\sigma_{sat}\ t}$

Embedded length: $L_E = \frac{N_{max}}{2\ r\ t}$

Maximum width of plate: $b_U = 1,5\ (b_F + D)$

Outside radius: $r = 1,9\left(\frac{L_E^2}{b_F} + 0,25\ b_F\right)$

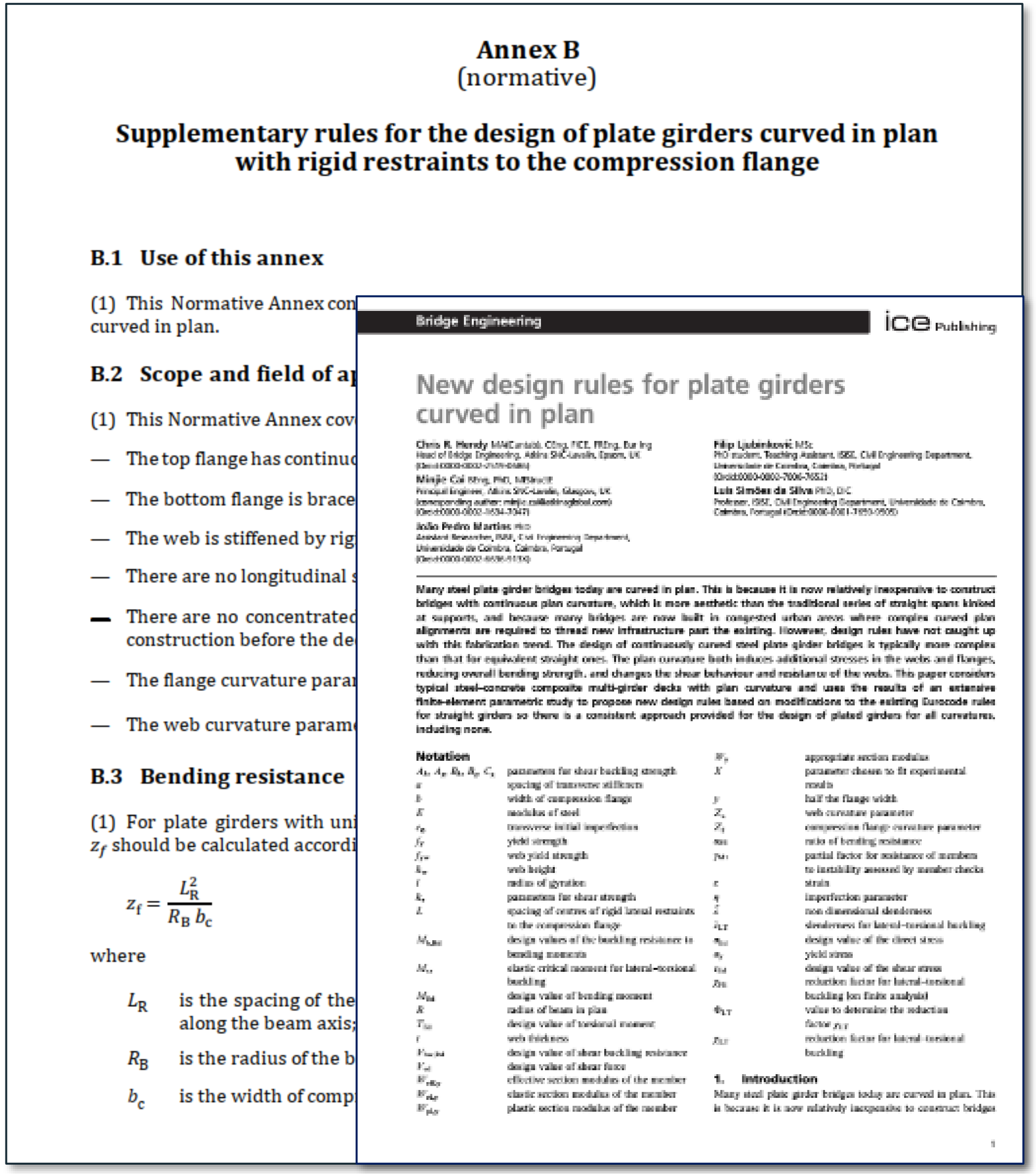
Clear height of gusset plate: $L_F = 0,45\ L_E$

where

N_{max} is the maximum axial force acting on the hanger at the ultimate limit state due to the persistent design situation according to 3.2 of EN 1990;

Examples from EN 1993-2
New Annex B for girders curved in plan

- Relatively inexpensive to construct today, more aesthetic than straight, in demand in congested urban area
- Absence of any codified guidance in the Eurocodes
- M_{Rd} and V_{Rd} affected by curvature and “T_{Ed}” introduced
- New rules developed through European RFCS project
- Published in ICE Bridge Engineering journal



Annex B
(normative)

Supplementary rules for the design of plate girders curved in plan with rigid restraints to the compression flange

B.1 Use of this annex

(1) This Normative Annex con…
curved in plan.

B.2 Scope and field of a…

(1) This Normative Annex cov…

— The top flange has continu…

— The bottom flange is brace…

— The web is stiffened by rig…

— There are no longitudinal s…

— There are no concentrated… construction before the de…

— The flange curvature para…

— The web curvature param…

B.3 Bending resistance

(1) For plate girders with uni… z_f should be calculated accordi…

$$z_f = \frac{L_R^2}{R_B\, b_c}$$

where

L_R is the spacing of the… along the beam axis;

R_B is the radius of the b…

b_c is the width of comp…

Bridge Engineering

ice Publishing

New design rules for plate girders curved in plan

Chris R. Hendy

Minjie Cai

João Pedro Martins

Filip Ljubinković

Luís Simões da Silva

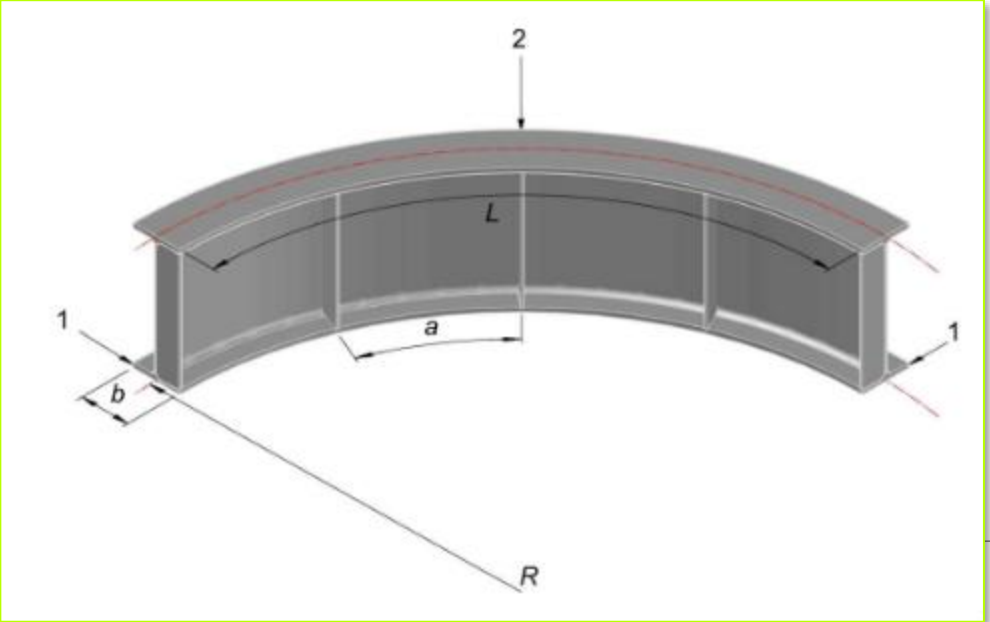
Many steel plate girder bridges today are curved in plan. This is because it is now relatively inexpensive to construct bridges with continuous plan curvature, which is more aesthetic than the traditional series of straight spans kinked at supports, and because many bridges are now built in congested urban areas where complex curved plan alignments are required to thread new infrastructure past the existing. However, design rules have not caught up with this fabrication trend. The design of continuously curved steel plate girder bridges is typically more complex than that for equivalent straight ones. The plan curvature both induces additional stresses in the webs and flanges, reducing overall bending strength, and changes the shear behaviour and resistance of the webs. This paper considers typical steel–concrete composite multi-girder decks with plan curvature and uses the results of an extensive finite-element parametric study to propose new design rules based on modifications to the existing Eurocode rules for straight girders so there is a consistent approach provided for the design of plated girders for all curvatures, including none.

Notation

1. Introduction

Many steel plate girder bridges today are curved in plan. This is because it is now relatively inexpensive to construct bridges

Examples from EN 1993-2
New Annex B for girders curved in plan



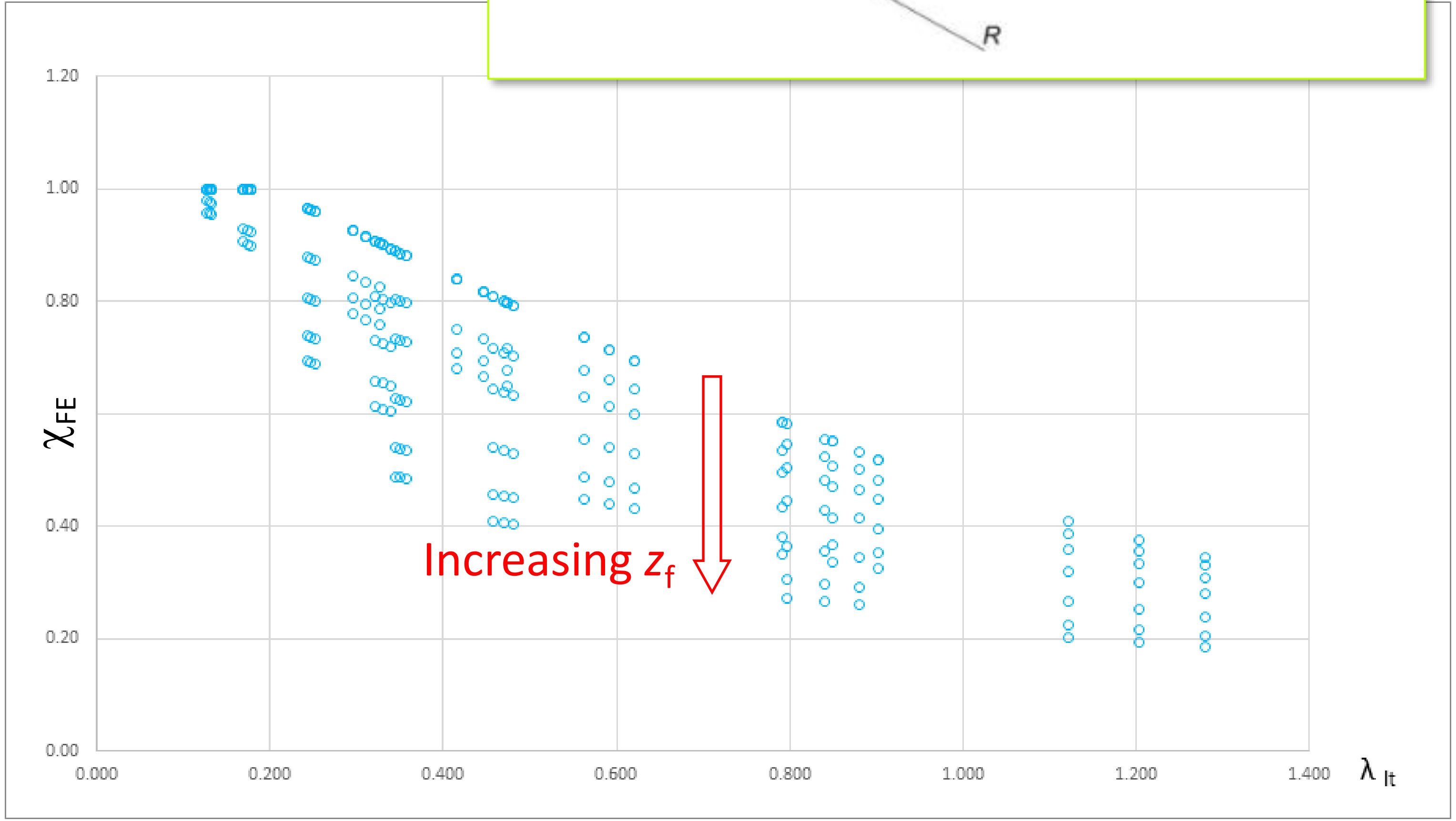
Bending

- Curvature factor: Z_f

$$Z_f = \frac{L^2}{R \cdot B_f}$$

L	Lateral support spacing;
R	Radius of girder in plan;
B_f	Flange width (*b* in figure);
a	Transverse stiffener spacing;
t_w	Web thickness.

- Results χ_{FE} show effect of curvature at a given slenderness



Examples from EN 1993-2
New Annex B for girders curved in plan

B.3 Bending resistance

(1) For plate girders with uniform curvature in plan, the compression flange curvature parameter z_f should be calculated defined according to Formula (B.1):

$$z_f = \frac{L^2}{R\,b} \qquad \text{(B.1)}$$

where:

L is the spacing of the centres of rigid lateral restraints to the compression flange; see Figure B.1;
R is the radius of the beam web axis in plan; see Figure B.1;
b is the width of compression flange.

(2) Where $z_f \leq 0{,}2$, the beam may be treated as straight for the purposes of determining lateral torsional buckling resistance.

(3) Where $0{,}2 < z_f \leq 9{,}0$, the design buckling resistance should be determined in accordance with EN 1993-1-1, 8.3.2, but with Φ_{LT} calculated according to Formula (B.2):

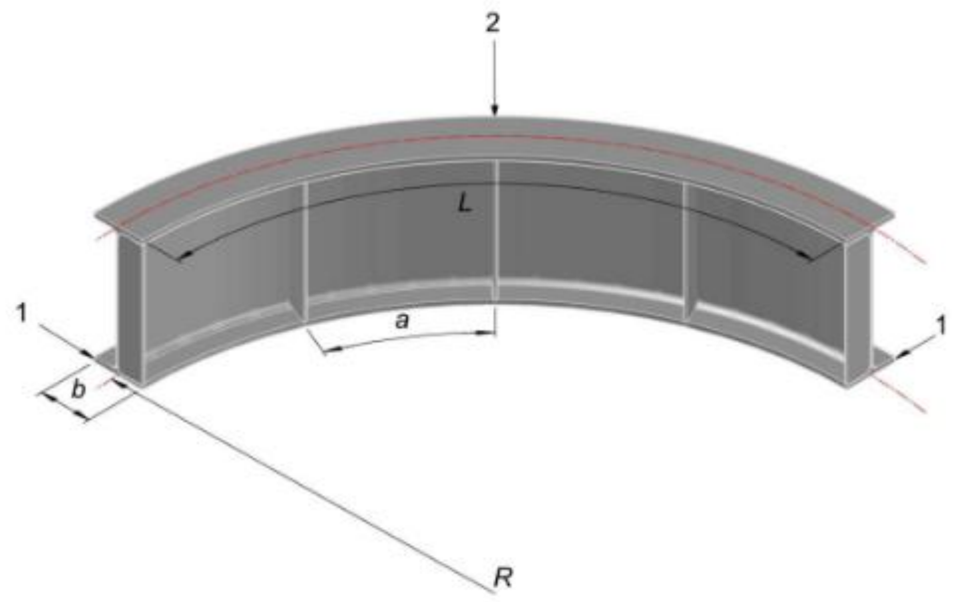
$$\Phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0{,}2\right) + C_{zf}\,z_f + \bar{\lambda}_{LT}^2\right] \qquad \text{(B.2)}$$

where C_{zf} has the following values:

$C_{zf} = 0{,}2$ for $z_f \leq 4$

$C_{zf} = 0{,}25$ for $4 < z_f \leq 9$

(4) When determining the relative slenderness $\bar{\lambda}_{LT}$ in accordance with 8.3.2 of EN 1993-1-1, the elastic critical moment, M_{cr}, should be calculated for either the curved girder or an equivalent straight girder with the same restraints and restraint spacings.



Examples from EN 1993-2
New Annex B for girders curved in plan

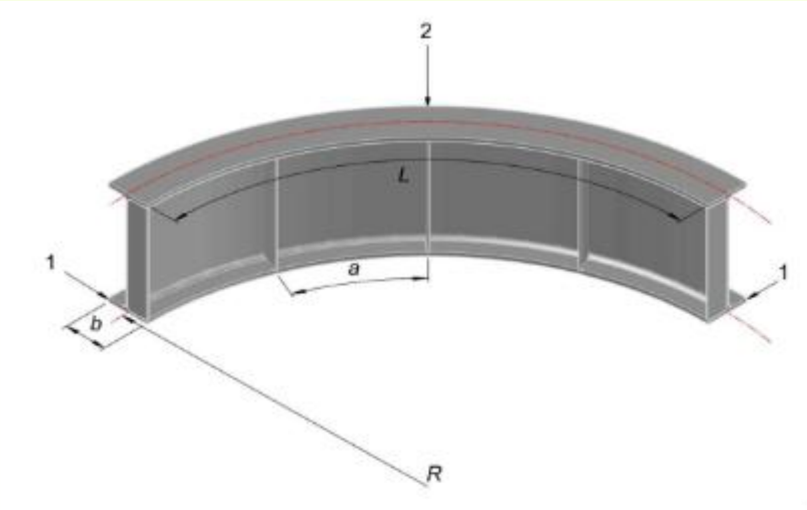
Shear

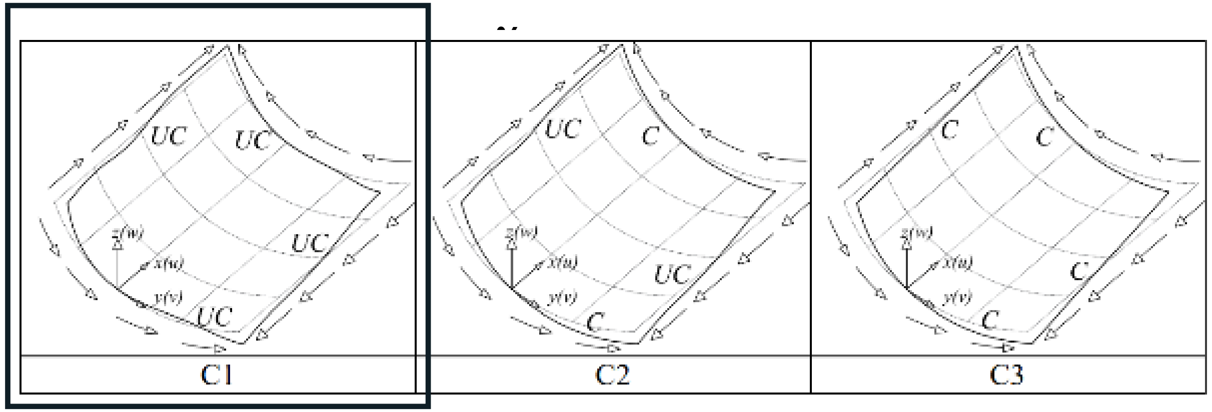
- Curvature factor: Z_a

$$Z_a = \frac{a^2}{R \cdot t}$$

R	Radius of girder in plan;
a	Transverse stiffener spacing;
t	Web thickness.

- Study looked at different boundary conditions but case without membrane restraint used in new rules





Typical "rigid end post" case for webs of girders curved in plan is straight (vertical) edges forced to remain straight – so C1 conservative

BC: All simply supported

C1 – Edges free to wave

C2 – Curved edges forced to remain straight

C3 – All edges forced to remain straight

Examples from EN 1993-2
New Annex B for girders curved in plan

Shear

- Curvature factor: Z_a

$$Z_a = \frac{a^2}{R \cdot t}$$

If $Z_a \leq 1.0$ then treat as flat

R	Radius of girder in plan;
a	Transverse stiffener spacing;
t	Web thickness.

- Study looked at different boundary conditions but case without membrane restraint used in new rules

Add new paragraphs to clause 7.1 of EN 1993-1-5 as follows:

(4) For beams with curvature in plan, the web curvature parameter Z_a shall be defined as follows:

$$Z_a = \frac{a^2}{Rt}$$

where:

R is the radius of the beam in plan

(5) Where $Z_a \leq 1.0$, the design shear buckling resistance may be determined in accordance with 5.1(1).

(6) Where $Z_a > 1.0$, the design shear buckling resistance may be taken as:

$$V_{b,Rd} = V_{bw,Rd} = \chi_w h_w t \frac{f_{yw}}{\sqrt{3}\gamma_{M1}}$$

where χ_w is determined as follows:

$\chi_w = 1.0$	for	$\bar{\lambda}_w \leq \bar{\lambda}_{w,0}$
$\chi_w = A_\chi \bar{\lambda}_w^{2} + B_\chi \bar{\lambda}_w + C_\chi$	for	$\bar{\lambda}_{w,0} < \bar{\lambda}_w < 1.1$
$\chi_w = \dfrac{A_\chi}{B_\chi + \bar{\lambda}_w}$	for	$1.1 \leq \bar{\lambda}_w \leq 3.0$

where $\bar{\lambda}_{w,0} = \begin{cases} 0.4, & Z_a \leq 30 \\ 0.3, & Z_a > 30 \end{cases}$

and A_χ, B_χ and C_χ should be taken from Table (i).

Table (i): A_χ, B_χ and C_χ parameters

$\bar{\lambda}_{w,0} < \bar{\lambda}_w < 1.1$	$A_\chi = -\frac{Z_a}{57.7} - 0.48$	$B_\chi = \frac{Z_a}{86} + 0.25$	$C_\chi = -\frac{Z_a}{367} + 1.0$
$1.1 \leq \bar{\lambda}_w \leq 3.0$	$A_\chi = \frac{335.5 - Z_a}{380}$	$B_\chi = \frac{3.7 + Z_a}{47.5}$	

(7) The slenderness $\bar{\lambda}_w = \frac{a/t}{37.4\varepsilon\sqrt{k_{\tau,Z_a}}}$ with $k_{\tau,Z_a} = A_k + B_k \left(\frac{1}{\alpha}\right)^2$ and A_k and B_k taken from Table (ii) where $\alpha = \frac{h_w}{a}$.

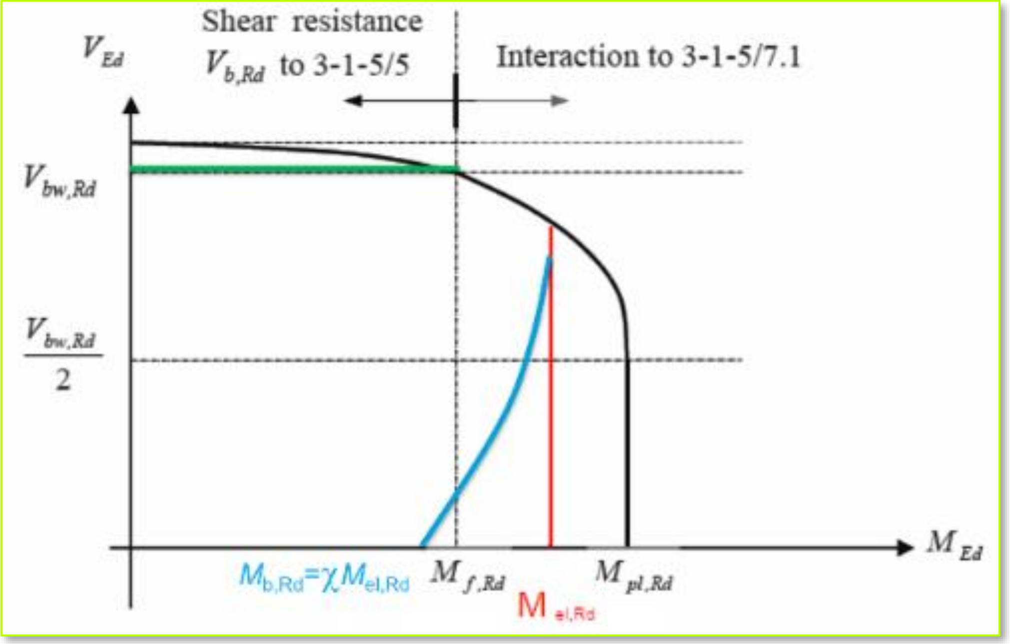
Table (ii): A_k and B_k parameters

$\alpha \leq 1$	$A_k = 0.214Z_a + 2.88$	$B_k = 5.343 - \frac{Z_a}{175.6}$
$\alpha > 1$	$A_k = 0.096Z_a + 5.15$	$B_k = 0.135Z_a + 3.18$

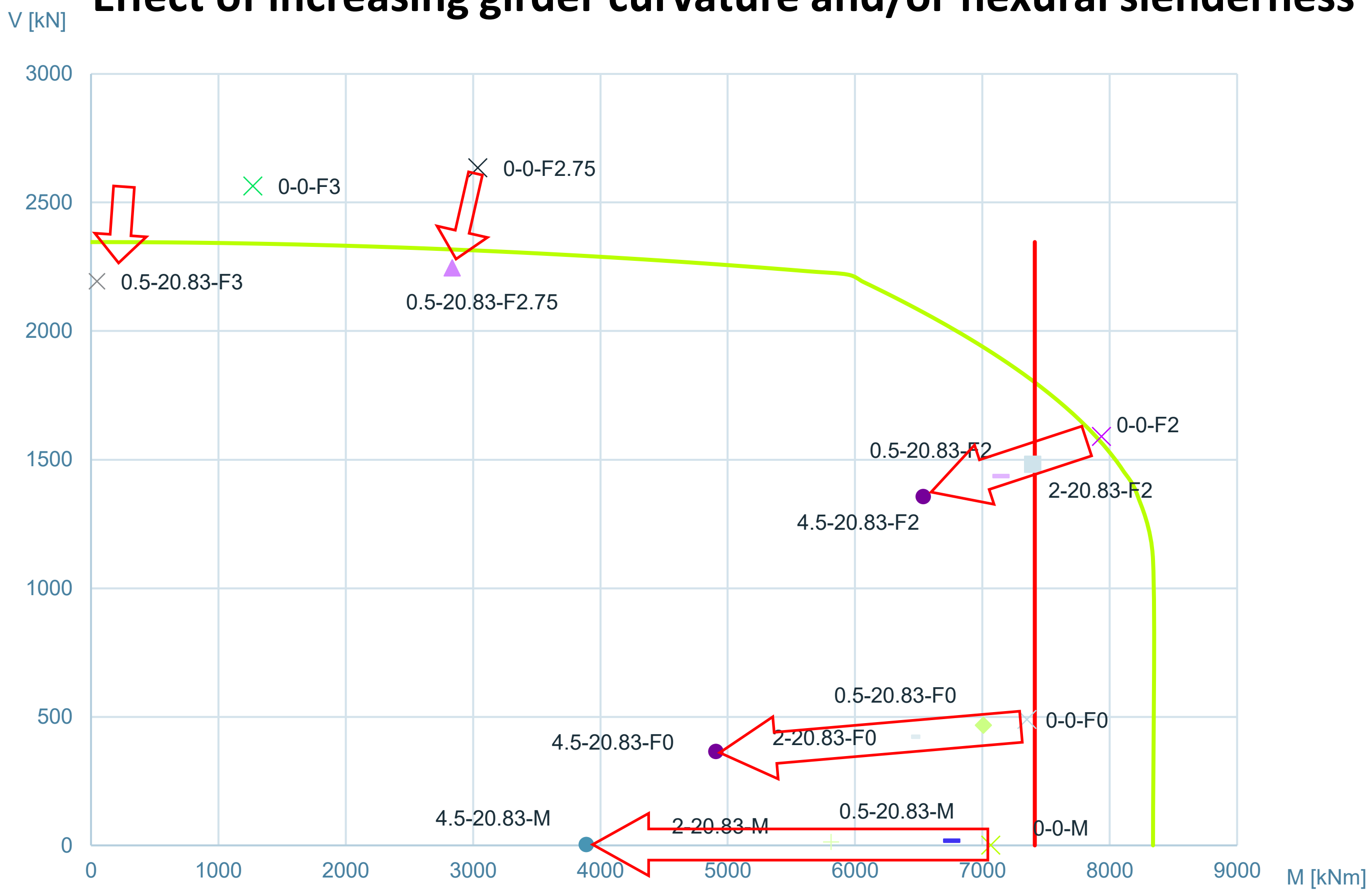
Examples from EN 1993-2
New Annex B for girders curved in plan

Shear – moment interaction

- Uses interaction for straight girders with resistance for curved girders



Effect of increasing girder curvature and/or flexural slenderness



Examples from EN 1993-2
New Annex B for girders curved in plan

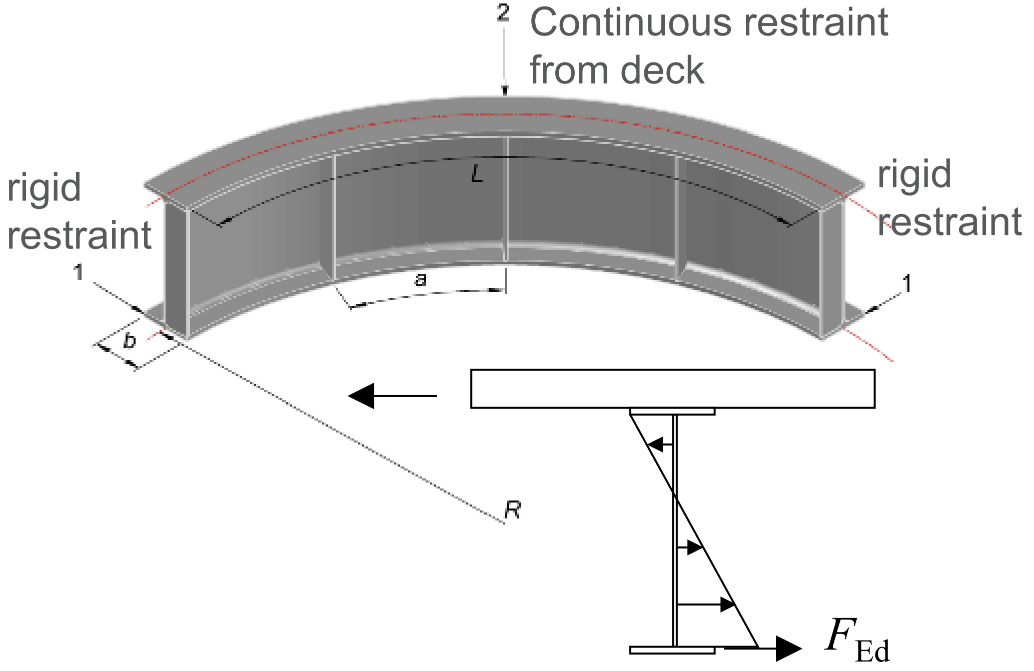
- Updated to include design force for **rigid** restraints to curved in plan compression flanges
- Restraints should be designed for the forces developed resisting the lateral buckling deflections of the flange, considering both initial intended curvature and geometrical imperfections
- The force, F_{Ed}, to apply to each discrete rigid compression flange restraint, with spacing L, may conservatively be calculated by introducing the additional compression flange bow from intended plan radius, R, into EN 1993-2 clause 6.3.4.2(5) as follows:

$$F_{\mathrm{Ed}} = N_{\mathrm{Ed}}\left(\frac{1}{100}+\frac{L}{R}\right)$$

Compression flange force (EN1993-2)

due to fabrication imperfection - corresponds to bow of *L*/800 over length *L*

due to intended radius



Examples from EN 1993-2
Amended Annex D – buckling effective lengths removed

Covers:

- Only imperfection for arches now covered
- All buckling critical forces moved to CEN Technical Report

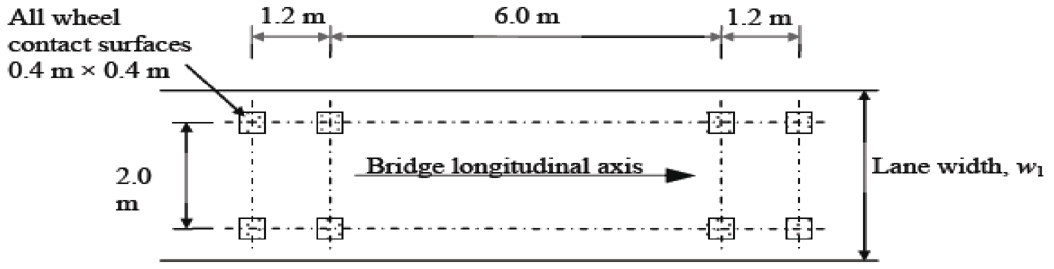
Content

CEN/TR 1993-1-103

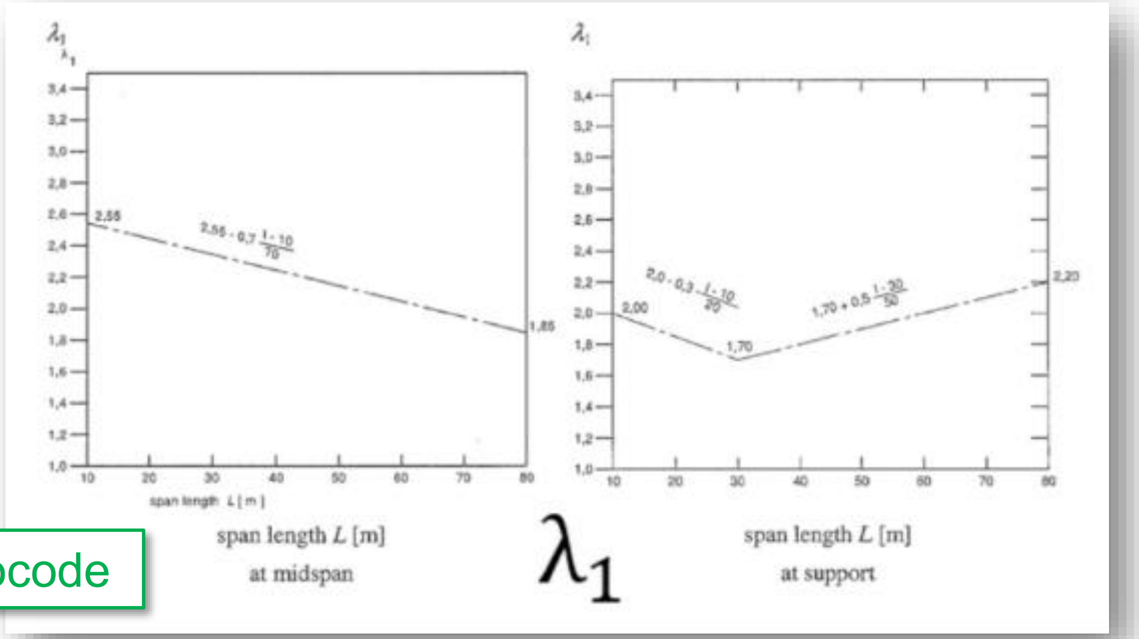
Eurocode 3 – Design of Steel Structures –
Part 1-103: Elastic Critical Buckling of Members

Examples from EN 1993-2
New Annex F for fatigue damage equivalent factors for road bridges

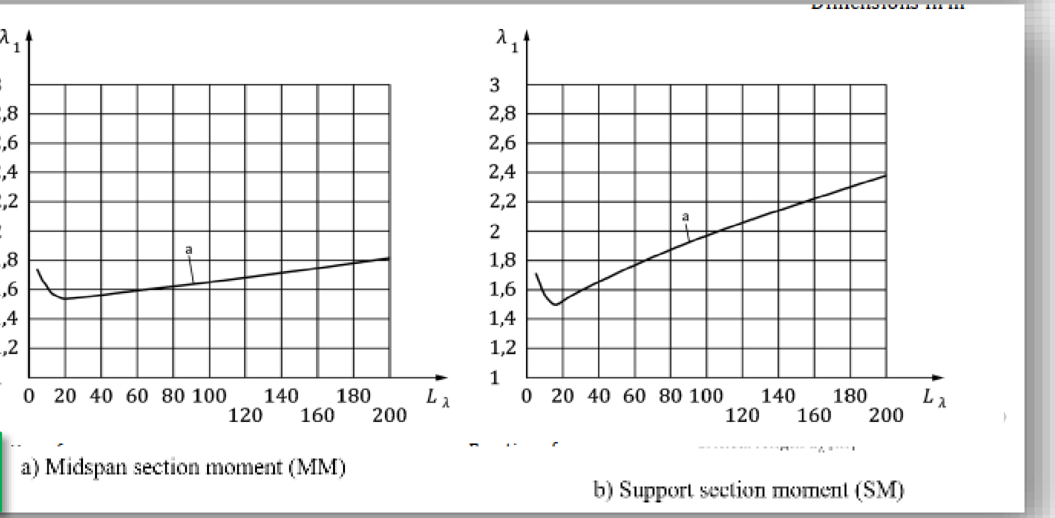
- Current lambda factors for damage equivalent stress determination have no obtainable background – BGN152
- Recent studies show they are not accurate (don't account for influence of axles at short span) and are not defined above 80 m
- Mostly safe-sided, but not always, and sometimes follow wrong trend!



Current Eurocode



New Eurocode



Eurocodes – the second generation

THE FUTURE OF STEEL CONSTRUCTION

Examples from EN 1993-1-11

Moderate change

Eurocode is not yet published by BSI

Contents Page

Examples from EN 1993-1-11 Miscellaneous changes

Lots of minor changes:

- Impact factor to consider for sudden loss of cable – taken as 2.0 unless other value justified e.g. time history analysis (was 1.5)
- Improvements in grouping of cable types

Group	Tension element	Tension component
A	rod	tension rod system made of structural steel or stainless steel
		tension rod system made of concrete reinforcement steel
		tension rod system made of prestressing steel
B	rope made of circular wire	spiral strand rope with terminations
	rope made of circular wire and stranded wire	stranded rope (type IWRC or WSC) with terminations
	rope made of circular and Z-shaped wires	full-locked coil rope with terminations
C	bundle made of circular prestressing wires	parallel wire system (PWS)
	bundle made of 7-wire prestressing strands	parallel strand system (PSS)
D	**Suspension bridge cables (new Annex C**	**Parallel wire system**

(5) The analysis may be performed by applying opposing forces, including dynamic amplification, at both terminations of the lost tension component(s) or, in case of linear system behaviour, by superposition of design values, including the dynamic amplification, as per the following formulae:

$$\Delta E_{d} = k\left(E_{d2} - E_{d1}\right) \tag{4.1}$$

$$E_{d} = E_{d1} + \Delta E_{d} \tag{4.2}$$

where

ΔE_{d} represents the dynamic effect of an accidental loss of the tension component(s);

E_{d1} represents the design value of the effect of the action with all tension component(s) intact;

E_{d2} represents the design value of the effect of the action with the relevant tension component(s) removed;

NOTE 1 The value of k is 2,0 unless the National Annex gives a different value.

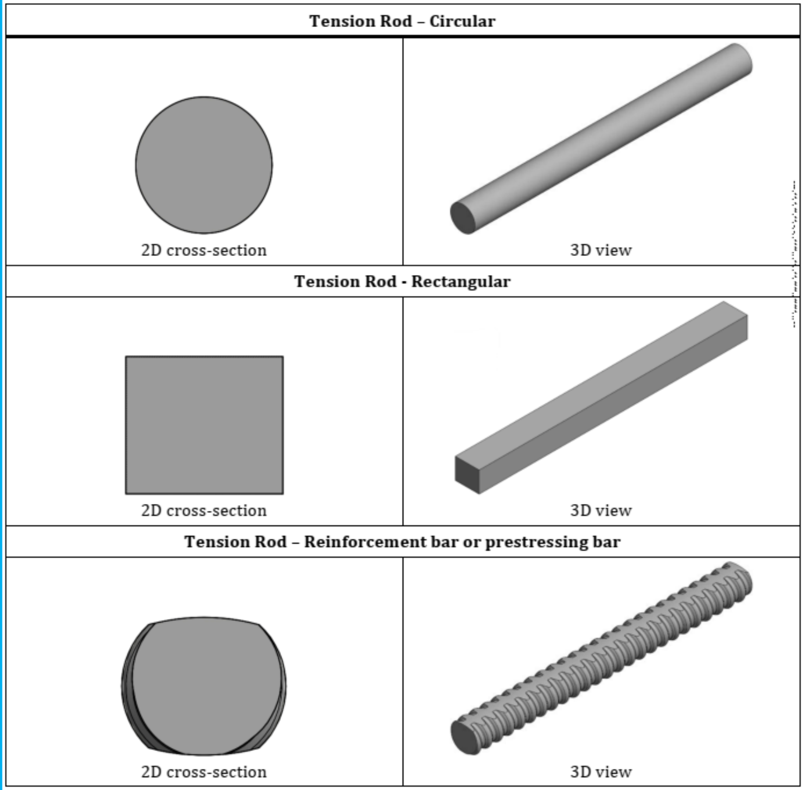


Figure B.2 — Group A — Tension elements

Examples from EN 1993-1-11
Miscellaneous changes

Lots of minor changes:

- Impact factor to consider for sudden loss of cable – taken as 2.0 unless other value justified e.g. time history analysis (was 1.5)
- Improvements in grouping of cable types

Group	Tension element	Tension component
A	rod	tension rod system made of structural steel or stainless steel
		tension rod system made of concrete reinforcement steel
		tension rod system made of prestressing steel
B	rope made of circular wire	spiral strand rope with terminations
	rope made of circular wire and stranded wire	stranded rope (type IWRC or WSC) with terminations
	rope made of circular and Z-shaped wires	full-locked coil rope with terminations
C	bundle made of circular prestressing wires	parallel wire system (PWS)
	bundle made of 7-wire prestressing strands	parallel strand system (PSS)
D	**Suspension bridge cables (new Annex C**	**Parallel wire system**

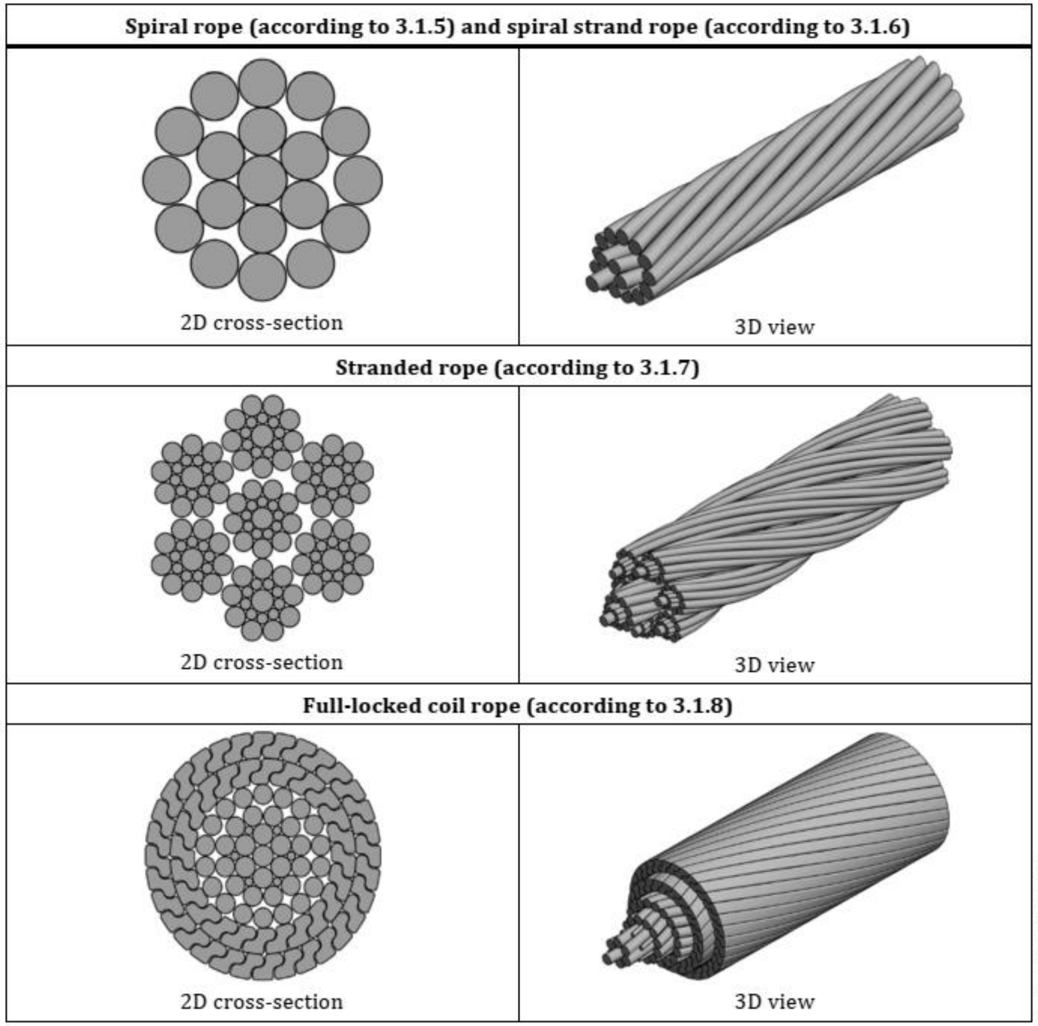


Figure B.5 — Group B — Tension Elements

Examples from EN 1993-1-11
Miscellaneous changes

Lots of minor changes:

- Impact factor to consider for sudden loss of cable – taken as 2.0 unless other value justified e.g. time history analysis (was 1.5)
- Improvements in grouping of cable types

Group	Tension element	Tension component
A	rod	tension rod system made of structural steel or stainless steel
		tension rod system made of concrete reinforcement steel
		tension rod system made of prestressing steel
B	rope made of circular wire	spiral strand rope with terminations
	rope made of circular wire and stranded wire	stranded rope (type IWRC or WSC) with terminations
	rope made of circular and Z-shaped wires	full-locked coil rope with terminations
C	bundle made of circular prestressing wires	parallel wire system (PWS)
	bundle made of 7-wire prestressing strands	parallel strand system (PSS)
D	**Suspension bridge cables (new Annex C**	**Parallel wire system**

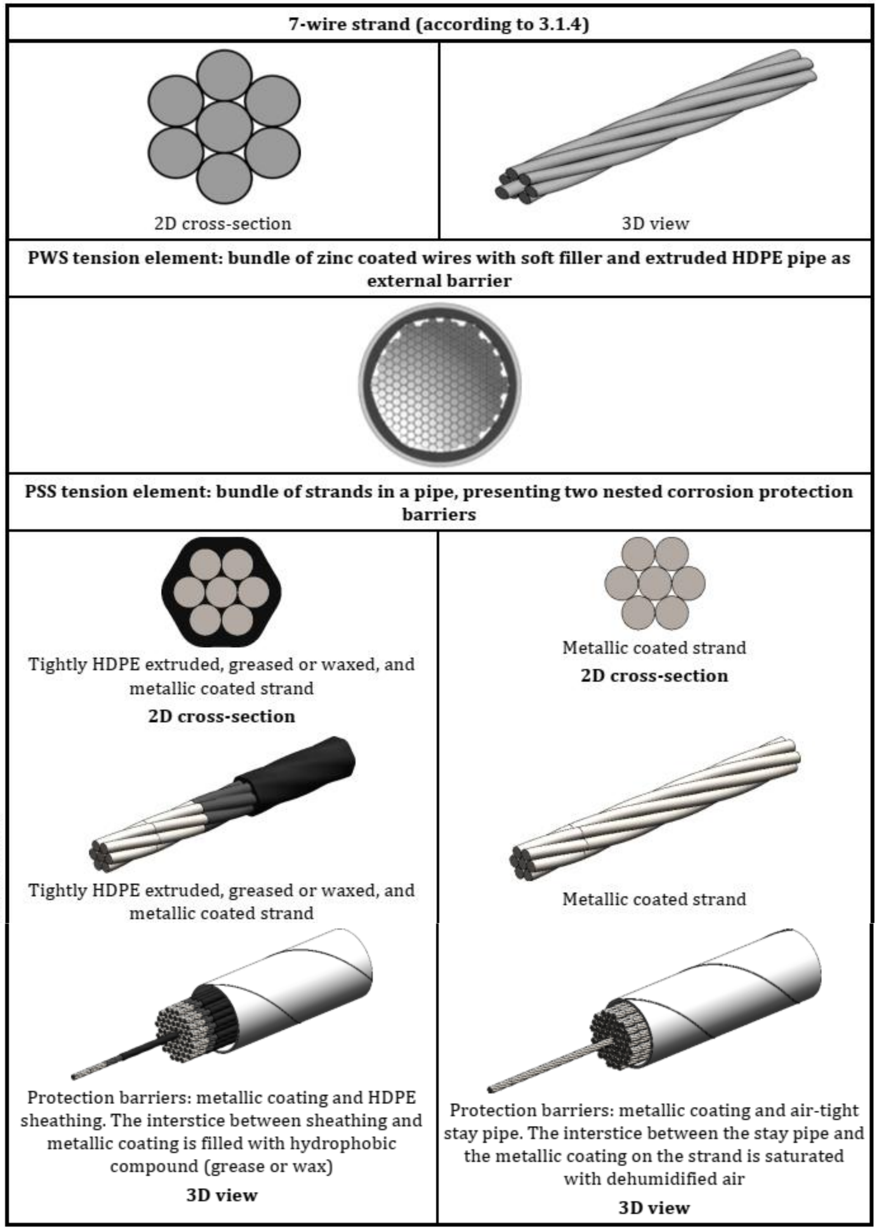


Figure B.8 — Group C — Tension Elements

Examples from EN 1993-1-11 Miscellaneous changes

Lots of minor changes:

- Impact factor to consider for sudden loss of cable – taken as 2.0 unless other value justified e.g. time history analysis (was 1.5)
- Improvements in grouping of cable types

Group	Tension element	Tension component
A	rod	tension rod system made of structural steel or stainless steel
		tension rod system made of concrete reinforcement steel
		tension rod system made of prestressing steel
B	rope made of circular wire	spiral strand rope with terminations
	rope made of circular wire and stranded wire	stranded rope (type IWRC or WSC) with terminations
	rope made of circular and Z-shaped wires	full-locked coil rope with terminations
C	bundle made of circular prestressing wires	parallel wire system (PWS)
	bundle made of 7-wire prestressing strands	parallel strand system (PSS)
D	**Suspension bridge cables (new Annex C**	**Parallel wire system**

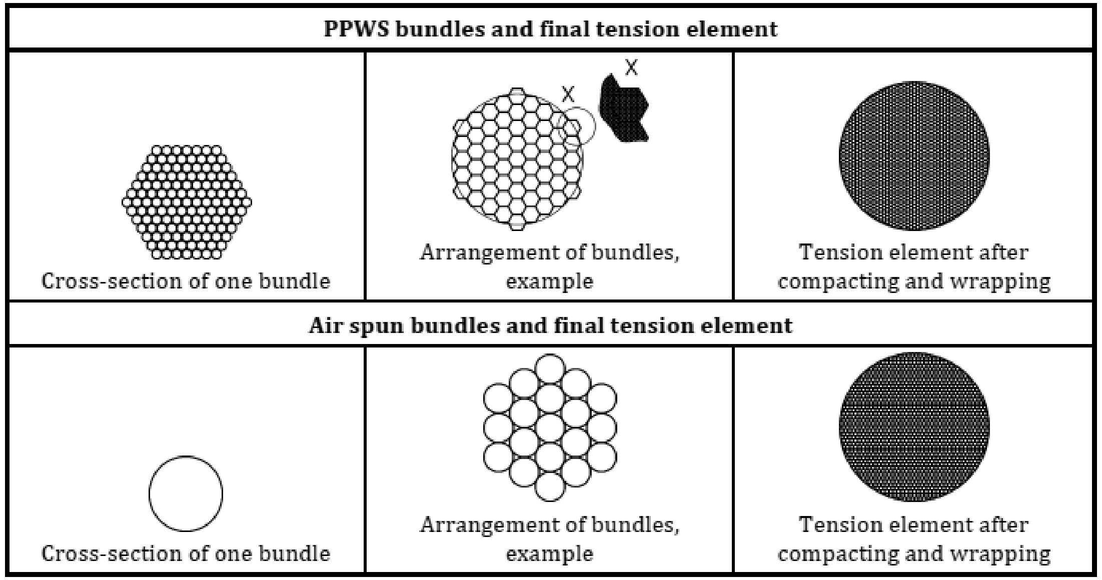


Figure B.11 — Group D — Tension Elements

Table C.1 — Group D tension components

Group	Tension element	Tension component
D	hexagonal bundle made of parallel circular high strength steel wires	prefabricated parallel wire bundle (strand) with terminations (PPWS)
	circular bundle made by arial spinning of parallel circular high strength steel wires	air spun parallel wire bundle with terminations (cable shoes)

Examples from EN 1993-1-11 Miscellaneous changes

- New Annex covering suspension bridge cables
- Covers:
  - Cable strength evaluation

(1) For the tension component the design value of the tension resistance, F_{Rd}, should be taken as follows:

$$F_{\mathrm{Rd}} = \frac{k_{\mathrm{e}} F_{\mathrm{uk}}}{\gamma_{\mathrm{Mt,D}}} \qquad \text{(C.1)}$$

where

F_{uk} is the calculated minimum breaking force $F_{\mathrm{c,min}}$ based upon the nominal wire size and wire tensile strength;

k_{e} is the loss factor for the termination, if relevant, ($k_e \leq 1,0$). For metal filled sockets, $k_{\mathrm{e}} = 1,0$;

$\gamma_{\mathrm{Mt,D}}$ is the partial factor for group D tension element.

NOTE The partial factor $\gamma_{\mathrm{Mt,D}}$ is equal to 1,8 unless the National Annex gives a different value. This value of the partial factor considers the bending stresses over saddles designed according to C.4.3.1.

  - Design of saddles
  - Design of cable clamps

Annex C
(normative)

Additional rules for Group D parallel wire tension components for suspended structures

C.1 Use of this annex

(1) This Normative Annex contains additional rules for Group D parallel wire tension components for suspended structures, as defined in Table C.1.

Table C.1 — Group D tension components

Group	Tension element	Tension component
D	hexagonal bundle made of parallel circular high strength steel wires	prefabricated parallel wire bundle (strand) with terminations (PPWS)
	circular bundle made by arial spinning of parallel circular high strength steel wires	air spun parallel wire bundle with terminations (cable shoes)

NOTE 1 Examples of tension components for Group D are illustrated in Annex B.

NOTE 2 For Group D the final tension element will usually consist of several bundles that are compacted with use of hydraulic jacks into a circular shape and are then wrapped with a wire of mild steel.

C.2 Scope and field of application

(1) This Normative Annex covers additional rules for Group D, parallel wire tension components, when used e.g. as main cables for suspension bridges. These rules may also be applied for parallel wire tension components in other types of suspended structures.

(2) This Normative Annex contains additional rules for the capacity of the parallel wire tension component as well as requirements related to the tension element at saddles and at clamps.

C.3 Materials

C.3.1 Nominal strength grade of steel wires

(1) The nominal tensile strength, f_{uk}, should be used for the design of steel wires.

NOTE A maximum value for f_{uk} is given in Table C.2 unless the National Annex gives a different value.

Table C.2 (NDP) — Nominal tensile strength grade of steel wires and references

Non-alloyed steel wire	EN 10264, Part 1 to 3
Recommended maximum values of f_{uk}	1 770 N/mm²

Examples from EN 1993-1-11
Fatigue of tension elements

Biggest changes are:

- Different fatigue curves
- SLS limit under ***characteristic*** combination removed; now SLS stress level $\sigma_{\mathrm{sup,Ed}}$ under ***frequent*** combination explicitly included in consideration of fatigue

$$\frac{\sigma_{\mathrm{sup,Ed}}}{\boxed{\sigma_{\mathrm{sup,Cd}}}} \leq 1$$

Figure 10.1 — Axial fatigue resistance curve for tension elements

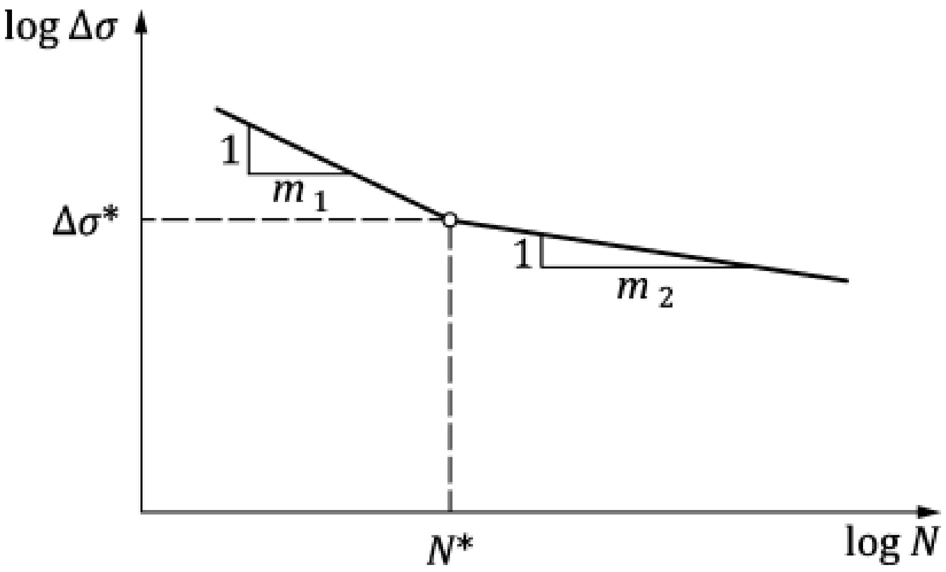


Table 10.1 (NDP) — Detail categories $\Delta\sigma_c$ and axial fatigue resistance curve parameters

Group	Tension element	$\sigma_{sup,Cd}/\sigma_{u,k}$	N^*	m_1	m_2	$\Delta\sigma^*$ N/mm²	$\Delta\sigma_C$ N/mm²
A	tension rod made of structural steel or stainless steel	values according to EN 1993-1-9					
A	tension rod made of concrete reinforcement steel with weld/ cut or rolled thread	≤ 0,55	1×10^7	3	5	35	60
A	tension rod made of prestressing steel with rolled thread	≤ 0,65	2×10^6	5	5	70	70
B	spiral strand rope with metal or resin socketing	≤ 0,45	5×10^6	4	6	115	145
B	stranded rope	There are no validated values – Qualification testing according to Clause 11 is required					
B	full-locked coil rope with metal or resin socketing	≤ 0,45	5×10^6	4	6	115	145
C	parallel wires / bundle of parallel wires	≤ 0,45	2×10^6	4	6	160	160
C	parallel wires / bundle of parallel wires	> 0,45 ≤ 0,55	2×10^6	4	6	112	112
C	bundle of parallel strands	≤ 0,45	2×10^6	4	6	160	160
C	bundle of parallel strands	> 0,45 ≤ 0,55	2×10^6	4	6	112	112

$\sigma_{u,k} = f_t$ for group A tension elements made of concrete reinforcement steel
$\sigma_{u,k} = f_{pk}$ for group A tension elements made of prestressing steel
$\sigma_{u,k} = F_{uk}/A_m$ for group B tension elements
$\sigma_{u,k} = f_{uk}$ for group C tension elements

Examples from EN 1993-1-1

Limited change

Eurocode 3 — Design of steel structures — Part 1-1: General rules and rules for buildings

Eurocode 3 — Bemessung und Konstruktion von Stahlbauten — Teil 1-1: Allgemeine Bemessungsregeln und Regeln für den Hochbau

Eurocode 3 — Calcul des structures en acier — Partie 1-1 : Règles générales et règles pour les bâtiments

Eurocode is now published by BSI

Examples from EN 1993-1-1 - materials

- Relatively few reductions to NDPs
- Steel grades to S700 introduced (previously up to S460)
- EN 1993-1-12 to cover up to S960
- EN 1993-1-10 introduces relaxations for fracture toughness considerations of steel structures without significant fatigue crack growth (see BGN 158 - Fracture mechanics methods to select steel sub-grades)

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 (all parts), EN 10210 (all parts), and EN 10219 (all parts)

Steel grade[a]	Nominal thickness of the element t mm			
	$t \leq 40$ mm		40 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
S235	235	360	215	360
S275	275	390	245	370
S355	355	490	325	470
S420	420	510	390	490
S460	460	540	410	510
S500	500	580	450	580
S550	550	600	500	600
S600	600	650	550	650
S620	620	700	560	660
S650	650	700	-	-
S690	690	770	630	710
S700	700	750	-	-

[a] Principal symbols in EN 10027-1.

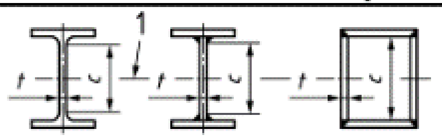
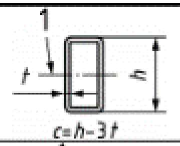
Examples from EN 1993-1-1 – Section classification

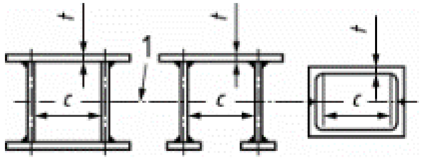
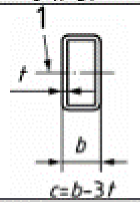
- Section classification limits for internal plates made more onerous generally, and Class 3-4 boundary made compatible with EN 1993-1-5

(XX) = current values in EN 1993-1-1

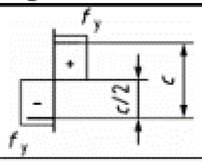
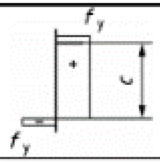
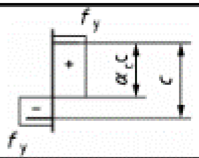
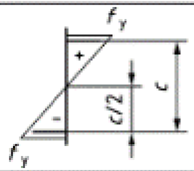
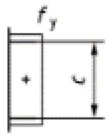
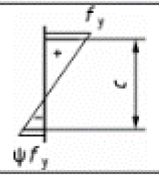
Table 7.3 — Maximum width-to-thickness ratios for compression parts (sheet 1 of 3)

Internal compression parts

Key
1 Axis of bending

Stress distribution in parts (compression positive)			
Class 1	$c/t \le 72\,\varepsilon$	$c/t \le 28\,\varepsilon$ (33ε)	when $\alpha_c > 0{,}5$: $c/t \le \frac{126\,\varepsilon}{5{,}5\,\alpha_c - 1}$ when $\alpha_c \le 0{,}5$: $c/t \le \frac{36\,\varepsilon}{\alpha_c}$
Class 2	$c/t \le 83\,\varepsilon$	$c/t \le 34\,\varepsilon$ (38ε)	when $\alpha_c > 0{,}5$: $c/t \le \frac{188\,\varepsilon}{6{,}53\,\alpha_c - 1}$ when $\alpha_c \le 0{,}5$: $c/t \le \frac{41{,}5\,\varepsilon}{\alpha_c}$
Stress distribution in parts (compression positive)			
Class 3	$c/t \le 121\,\varepsilon$ (124ε)	$c/t \le 38\,\varepsilon$ (42ε)	when $\psi > -1$: $c/t \le \frac{38\,\varepsilon}{0{,}608 + 0{,}343\,\psi + 0{,}049\,\psi^2}$ when $\psi \le -1$[a]: $\frac{c}{t} \le 60{,}5\,\varepsilon\,(1-\psi)$

For I or H sections with equal flanges, under axial force and bending moment about the main axis parallel to the flanges, the parameter α_c that defines the position of the plastic neutral axis may be calculated as follows:

If $N_{ed} \ge c\,t_w f_y$ $\quad \alpha_c = 1{,}0$

If $N_{ed} \le -c\,t_w f_y$ $\quad \alpha_c = 0$

In other cases: $\quad \alpha_c = 0{,}5\left(1 + \frac{N_{Ed}}{c\,t_w\,f_y}\right)$

Where N_{Ed} is the design axial force taken as positive for compression and negative for tension.

[a] $\psi \le -1$ and a compression stress of $\sigma_{com,Ed} = f_y$ applies where the tensile strain $\varepsilon_t > f_y/E$

Examples from EN 1993-1-1 – Class 3 behaviour

- New Annex B covering partial plastification of Class 3 doubly symmetric members
- Offsets significance of changing the class boundaries

Annex B
(normative)

Design of semi-compact sections

B.1 Scope and field of application

(1) This Annex provides additional rules for the design of semi-compact (Class 3) doubly symmetric I- or H-sections, rectangular hollow sections, doubly symmetric box sections, circular hollow sections and elliptical hollow sections against mono- and bi-axial bending and axial force.

B.2 Elasto-plastic section modulus

(1) The elasto-plastic section modulus W_{ep} for doubly symmetric cross-sections should be determined from an interpolation between the plastic section modulus and the elastic section modulus about one principal axis of a cross-section as follows:

$$W_{ep,y} = W_{pl,y} - \left(W_{pl,y} - W_{el,y}\right)\beta_{ep,y} \tag{B.1}$$

$$W_{ep,z} = W_{pl,z} - \left(W_{pl,z} - W_{el,z}\right)\beta_{ep,z} \tag{B.2}$$

where the values of $\beta_{ep,y}$ and $\beta_{ep,z}$ depend on the material parameter ε and the width-to-thickness ratios as defined in Table 7.3. They should be taken as:

— For I- or H- sections, rolled or welded:

$$\beta_{ep,y} = \mathrm{Max}\left(\frac{\frac{c}{t_f} - 10\varepsilon}{4\varepsilon}; \frac{\frac{c}{t_w} - 83\varepsilon}{38\varepsilon}; 0\right) \quad \text{But } \beta_{ep,y} \leq 1{,}0 \tag{B.3}$$

$$\beta_{ep,z} = \mathrm{Max}\left(\frac{\frac{c}{t_f} - 10\varepsilon}{6\varepsilon}; 0\right) \quad \text{But } \beta_{ep,z} \leq 1{,}0 \tag{B.4}$$

— For rectangular hollow sections or doubly symmetric welded box sections:

$$\beta_{ep,y} = \mathrm{Max}\left(\frac{\frac{c}{t_f} - 34\varepsilon}{4\varepsilon}; \frac{\frac{c}{t_w} - 83\varepsilon}{38\varepsilon}; 0\right) \quad \text{But } \beta_{ep,y} \leq 1{,}0 \tag{B.5}$$

$$\beta_{ep,z} = \mathrm{Max}\left(\frac{\frac{c}{t_w} - 34\varepsilon}{4\varepsilon}; 0\right) \quad \text{But } \beta_{ep,z} \leq 1{,}0 \tag{B.6}$$

NOTE $t_f = t_w = t$ for rectangular hollow sections.

Examples from EN 1993-1-1 – references to buckling guidance

CEN Technical Report CEN/TR 1993-1-103

- Much detail on elastic buckling solutions transferred and added to this “Technical Report”

8.3.2.2 Slenderness for lateral torsional buckling curves

(1) The relative slenderness for lateral torsional buckling $\bar{\lambda}_{LT}$ should be taken as:

$$\bar{\lambda}_{LT} = \sqrt{\frac{M_{Rk}}{M_{cr}}} \tag{8.80}$$

where

M_{cr} is the elastic critical moment for lateral torsional buckling.

(2) M_{cr} should be based on gross cross-sectional properties and considers the loading conditions, the actual moment distribution and the lateral restraints.

NOTE Formulae for the elastic critical moment can be found in the technical report CEN/TR 1993-1-103.

CEN/TR 1993-1-103

Eurocode 3 – Design of Steel Structures –
Part 1-103: Elastic Critical Buckling of Members

Content

Examples from EN 1993-1-1 – selection of EXC

- Annex A – Selection of Execution Class: differentiation now made between static, seismic and fatigue structures for each Consequence Class
- This is however an NDP and UK NA will refer back to PD6705 for selection of EXC and QSC for fatigue
- Note DC is ductility class and not related to VC (verification case)

Consequence Class (CC)	Type of loading				
	Static, quasi-static	Seismic DC1[a]	Seismic DC2[a]	Seismic DC3[a]	Fatigue[b]
CC3	EXC3[c]	EXC3[c]	EXC3[c]	EXC3[c]	EXC3[c]
CC2	EXC2	EXC2	EXC2	EXC3[d]	EXC3
CC1	EXC1	EXC2[e]	EXC2	EXC2	EXC2

[a] Seismic ductility classes (DC's) are defined in EN 1998-1-1.

[b] See EN 1993-1-9.

[c] EXC4 may be considered for special cases, including those typically covered by CC4 of prEN 1990:2021.

[d] Only the primary seismic resisting system falls in EXC3; the gravity load resisting system may fall in EXC2.

[e] If the seismic action index is not greater than 2,5m/s2 (low seismic action class, see EN 1998-1-1), the execution class of structures in DC1 may be EXC1.

NOTE Structures in consequence class CC0 are not covered by this Annex, see prEN 1990:2021, 4.3.

Problematic as EN 1993-1-10 assumes no fatigue in brittle fracture table for EXC2; being discussed/resolved at present

THE FUTURE OF STEEL CONSTRUCTION

Examples from EN 1993-1-10

Limited change

Eurocode is not yet published by BSI

Examples from EN 1993-1-10 – selection of steel sub-grades

- Steel plates/sections need adequate toughness to avoid brittle fracture
- Toughness reduces with reducing temperature and increasing steel thickness
- The toughness is usually quantified through Charpy value
- Higher subgrades have greater toughness but longer order times
- The table of Charpy energy for steels in EN 1993-1-10:2005 Table 2.1 is not fully up to date so see [Steel material properties – SteelConstruction.info]

Specified minimum impact energy for carbon steel sub-grades

Standard	Subgrade	Impact strength	Test temperature
BS EN 10025-2[1] BS EN 10210-1[3]	JR	27J	20°C
	J0	27J	0°C
	J2	27J	-20°C
	K2	40J	-20°C
BS EN 10025-3[8]	N	40J	-20°c
	NL	27J	-50°c
BS EN 10025-4[9]	M	40J	-20°c
	ML	27J	-50°c
BS EN 10025-5[10]	J0	27J	0°C
	J2	27J	-20°C
	K2	40J	-20°C
	J4	27J	-40°C
	J5	27J	-50°C
BS EN 10025-6[11]	Q	30J	-20°c
	QL	30J	-40°c
	QL1	30J	-60°c

Examples from EN 1993-1-10 – selection of steel sub-grades

Current EN 1993-1-10

- The current EN 1993-1-10:2005 - single table of limiting thickness
- Initial flaw assumed to increase in size with fatigue
- Damage assumed = one quarter (500,000 cycles) of full fatigue damage obtained from nominal stress $\Delta\sigma_C$ according to EN 1993-1-9 (on basis of regular inspections during service life)

Table 2.1: Maximum permissible values of element thickness t in mm

Steel grade	Sub-grade	Charpy energy CVN at T [°C]	Charpy energy CVN J_{min}	Reference temperature T_{Ed} [°C] $\sigma_{Ed} = 0{,}75\,f_y(t)$ 10	0	-10	-20	-30	-40	-50	$\sigma_{Ed} = 0{,}50\,f_y(t)$ 10	0	-10	-20	-30	-40	-50	$\sigma_{Ed} = 0{,}25\,f_y(t)$ 10	0	-10	-20	-30	-40	-50
S235	JR	20	27	60	50	40	35	30	25	20	90	75	65	55	45	40	35	135	115	100	85	75	65	60
	J0	0	27	90	75	60	50	40	35	30	125	105	90	75	65	55	45	175	155	135	115	100	85	75
	J2	-20	27	125	105	90	75	60	50	40	170	145	125	105	90	75	65	200	200	175	155	135	115	100
S275	JR	20	27	55	45	35	30	25	20	15	80	70	55	50	40	35	30	125	110	95	80	70	60	55
	J0	0	27	75	65	55	45	35	30	25	115	95	80	70	55	50	40	165	145	125	110	95	80	70
	J2	-20	27	110	95	75	65	55	45	35	155	130	115	95	80	70	55	200	190	165	145	125	110	95
	M,N	-20	40	135	110	95	75	65	55	45	180	155	130	115	95	80	70	200	200	190	165	145	125	110
	ML,NL	-50	27	185	160	135	110	95	75	65	200	200	180	155	130	115	95	230	200	200	200	190	165	145
S355	JR	20	27	40	35	25	20	15	15	10	65	55	45	40	30	25	25	110	95	80	70	60	55	45
	J0	0	27	60	50	40	35	25	20	15	95	80	65	55	45	40	30	150	130	110	95	80	70	60
	J2	-20	27	90	75	60	50	40	35	25	135	110	95	80	65	55	45	200	175	150	130	110	95	80
	K2,M,N	-20	40	110	90	75	60	50	40	35	155	135	110	95	80	65	55	200	200	175	150	130	110	95
	ML,NL	-50	27	155	130	110	90	75	60	50	200	180	155	135	110	95	80	210	200	200	200	175	150	130
S420	M,N	-20	40	95	80	65	55	45	35	30	140	120	100	85	70	60	50	200	185	160	140	120	100	85
	ML,NL	-50	27	135	115	95	80	65	55	45	190	165	140	120	100	85	70	200	200	200	185	160	140	120
S460	Q	-20	30	70	60	50	40	30	25	20	110	95	75	65	55	45	35	175	155	130	115	95	80	70
	M,N	-20	40	90	70	60	50	40	30	25	130	110	95	75	65	55	45	200	175	155	130	115	95	80
	QL	-40	30	105	90	70	60	50	40	30	155	130	110	95	75	65	55	200	200	175	155	130	115	95
	ML,NL	-50	27	125	105	90	70	60	50	40	180	155	130	110	95	75	65	200	200	200	175	155	130	115
	QL1	-60	30	150	125	105	90	70	60	50	200	180	155	130	110	95	75	215	200	200	200	175	155	130
S690	Q	0	40	40	30	25	20	15	10	10	65	55	45	35	30	20	20	120	100	85	75	60	50	45
	Q	-20	30	50	40	30	25	20	15	10	80	65	55	45	35	30	20	140	120	100	85	75	60	50
	QL	-20	40	60	50	40	30	25	20	15	95	80	65	55	45	35	30	165	140	120	100	85	75	60
	QL	-40	30	75	60	50	40	30	25	20	115	95	80	65	55	45	35	190	165	140	120	100	85	75
	QL1	-40	40	90	75	60	50	40	30	25	135	115	95	80	65	55	45	200	190	165	140	120	100	85
	QL1	-60	30	110	90	75	60	50	40	30	160	135	115	95	80	65	55	200	200	190	165	140	120	100

Examples from EN 1993-1-10 – selection of steel sub-grades

Current EN 1993-1-10

- Paris law $\Delta a/\Delta n = C.\Delta K^m$ used to get crack growth with $\Delta K = YM_k \Delta\sigma\,(\pi a_i)^{0.5}$

a = crack length

a_i = initial crack length

Y, M_k constants for crack shape and detail type respectively

C, m constants from material tests

- Damage tolerance – leads to minimum number of periods between inspections
- Safe life – no in-service inspections needed for fatigue

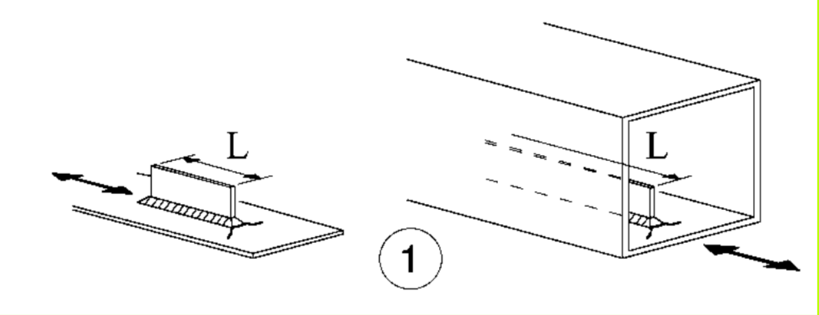
Table 2.1: Maximum permissible values of element thickness t in mm

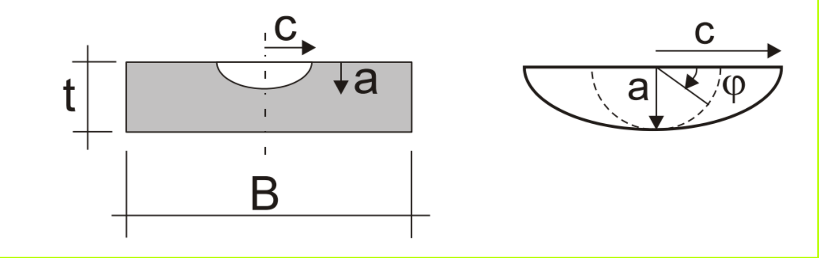
Steel grade	Sub-grade	Charpy energy CVN		Reference temperature T_{Ed} [°C]																				
		at T [°C]	J_{min}	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50	10	0	-10	-20	-30	-40	-50
				$\sigma_{Ed} = 0,75\ f_y(t)$							$\sigma_{Ed} = 0,50\ f_y(t)$							$\sigma_{Ed} = 0,25\ f_y(t)$						
S235	JR	20	27	60	50	40	35	30	25	20	90	75	65	55	45	40	35	135	115	100	85	75	65	60
	J0	0	27	90	75	60	50	40	35	30	125	105	90	75	65	55	45	175	155	135	115	100	85	75
	J2	-20	27	125	105	90	75	60	50	40	170	145	125	105	90	75	65	200	200	175	155	135	115	100
S275	JR	20	27	55	45	35	30	25	20	15	80	70	55	50	40	35	30	125	110	95	80	70	60	55
	J0	0	27	75	65	55	45	35	30	25	115	95	80	70	55	50	40	165	145	125	110	95	80	70
	J2	-20	27	110	95	75	65	55	45	35	155	130	115	95	80	70	55	200	190	165	145	125	110	95
	M,N	-20	40	135	110	95	75	65	55	45	180	155	130	115	95	80	70	200	200	190	165	145	125	110
	ML,NL	-50	27	185	160	135	110	95	75	65	200	200	180	155	130	115	95	230	200	200	200	190	165	145
S355	JR	20	27	40	35	25	20	15	15	10	65	55	45	40	30	25	25	110	95	80	70	60	55	45
	J0	0	27	60	50	40	35	25	20	15	95	80	65	55	45	40	30	150	130	110	95	80	70	60
	J2	-20	27	90	75	60	50	40	35	25	135	110	95	80	65	55	45	200	175	150	130	110	95	80
	K2,M,N	-20	40	110	90	75	60	50	40	35	155	135	110	95	80	65	55	200	200	175	150	130	110	95
	ML,NL	-50	27	155	130	110	90	75	60	50	200	180	155	135	110	95	80	210	200	200	200	175	150	130
S420	M,N	-20	40	95	80	65	55	45	35	30	140	120	100	85	70	60	50	200	185	160	140	120	100	85
	ML,NL	-50	27	135	115	95	80	65	55	45	190	165	140	120	100	85	70	200	200	200	185	160	140	120
S460	Q	-20	30	70	60	50	40	30	25	20	110	95	75	65	55	45	35	175	155	130	115	95	80	70
	M,N	-20	40	90	70	60	50	40	30	25	130	110	95	75	65	55	45	200	175	155	130	115	95	80
	QL	-40	30	105	90	70	60	50	40	30	155	130	110	95	75	65	55	200	200	175	155	130	115	95
	ML,NL	-50	27	125	105	90	70	60	50	40	180	155	130	110	95	75	65	200	200	200	175	155	130	115
	QL1	-60	30	150	125	105	90	70	60	50	200	180	155	130	110	95	75	215	200	200	200	175	155	130
S690	Q	0	40	40	30	25	20	15	10	10	65	55	45	35	30	20	20	120	100	85	75	60	50	45
	Q	-20	30	50	40	30	25	20	15	10	80	65	55	45	35	30	20	140	120	100	85	75	60	50
	QL	-20	40	60	50	40	30	25	20	15	95	80	65	55	45	35	30	165	140	120	100	85	75	60
	QL	-40	30	75	60	50	40	30	25	20	115	95	80	65	55	45	35	190	165	140	120	100	85	75
	QL1	-40	40	90	75	60	50	40	30	25	135	115	95	80	65	55	45	200	190	165	140	120	100	85
	QL1	-60	30	110	90	75	60	50	40	30	160	135	115	95	80	65	55	200	200	190	165	140	120	100

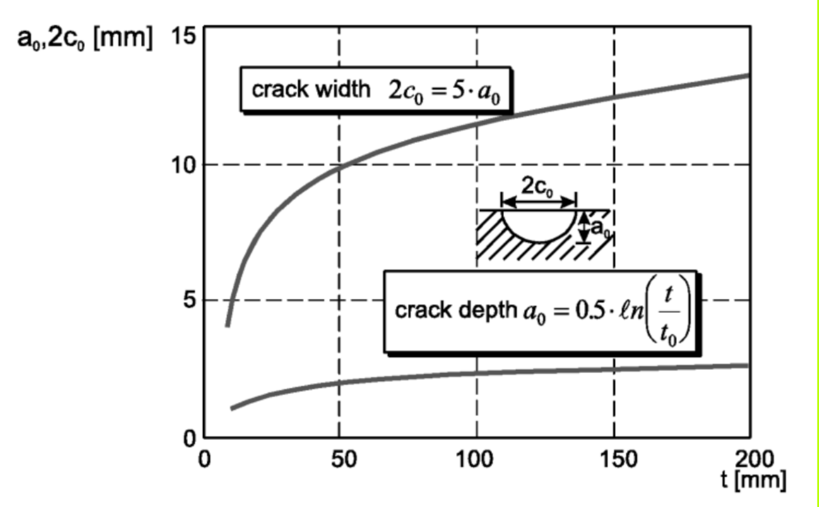
Examples from EN 1993-1-10 – selection of steel sub-grades

Current EN 1993-1-10

- Rules developed for steel bridge constructions
- Reference detail - plate subjected to tension with a welded non-load-carrying longitudinal attachment. (EN 1993-1-9 Table 8.4 Detail Category 56)
- All other detail categories covered safely by this
- A semi-elliptical surface crack is assumed at the hot-spot at the weld toe of the attachment
- The ratio of the semi axis crack depth *a* to crack length *c* is defined as *a*/*c* = 0.4
- Initial values from fabrication depend on plate thickness and are based on EUR23510
- UK NA gives additional requirements







Examples from EN 1993-1-10 – selection of steel sub-grades

Second generation EN 1993-1-10

- Table 4.2 covers fatigue structures as per first generation but Table 4.3 covers structures **not** subject to fatigue loading.
- Values in Table 4.3 are derived with same methodology as Table 4.2 but with the number of cycles for the calculation of the crack growth decreased from 500,000 to 20,000 cycles since (nominal fatigue loading)

Table 4.2: Maximum permissible values of element thickness t in mm for Execution Class EXC3 and EXC4

Steel grade	Quality	KV		Reference Temperature T_{Ed} [°C]																										
		T [°C]	J_{min}	10	0	-10	-20	-30	-40	-50	-80	-120	10	0	-10	-20	-30	-40	-50	-80	-120	10	0	-10	-20	-30	-40	-50	-80	-120
				$\sigma_{Ed} = 0.75 \cdot f_y(t)$									$\sigma_{Ed} = 0.5 \cdot f_y(t)$									$\sigma_{Ed} = 0.25 \cdot f_y(t)$								
S235	JR	20	27	60	50	40	35	30	25	20	10	5	90	75	65	55	45	40	35	20	15	135	115	100	85	75	65	60	40	30
	J0	0	27	90	75	60	50	40	35	30	15	10	125	105	90	75	65	55	45	30	15	175	155	135	115	100	85	75	50	35
	J2	-20	27	125	105	90	75	60	50	40	25	10	170	145	125	105	90	75	65	40	20	200	200	175	155	135	115	100	65	40
S275	JR	20	27	55	45	35	30	25	20	15	10	5	80	70	55	50	40	35	30	20	10	125	110	95	80	70	60	55	40	25
	J0	0	27	75	65	55	45	35	30	25	15	5	115	95	80	70	55	50	40	25	15	165	145	125	110	95	80	70	45	30
	J2	-20	27	110	95	75	65	55	45	35	20	10	155	130	115	95	80	70	55	35	20	200	190	165	145	125	110	95	60	40
	K2,M,N	-20	40	135	110	95	75	65	55	45	25	10	180	155	130	115	95	80	70	40	20	200	200	190	165	145	125	110	70	40
	ML,NL	-50	27	185	160	135	110	95	75	65	35	15	200	200	180	155	130	115	95	55	30	200	200	200	200	190	165	145	95	55
S355	JR	20	27	40	35	25	20	15	15	10	5	5	65	55	45	40	30	25	25	15	10	110	95	80	70	60	55	45	30	20
	J0	0	27	60	50	40	35	25	20	15	10	5	95	80	65	55	45	40	30	20	10	150	130	110	95	80	70	60	40	25
	J2	-20	27	90	75	60	50	40	35	25	15	5	135	110	95	80	65	55	45	25	15	200	175	150	130	110	95	80	55	30
	J4	-40	27	130	110	90	75	60	50	40	20	10	180	155	135	110	95	80	65	40	20	200	200	195	170	150	130	110	70	40
	K2,M,N	-20	40	110	90	75	60	50	40	35	20	5	155	135	110	95	80	65	55	30	15	200	200	175	150	130	110	95	60	35
	J5,ML,NL	-50	27	155	130	110	90	75	60	50	25	10	200	180	155	135	110	95	80	45	25	210	200	200	200	175	150	130	80	45
S420	JR	20	27	35	30	20	20	15	10	10	5	-	60	50	40	35	25	20	20	10	5	100	85	75	65	55	45	40	30	20
	J0	0	27	55	45	35	30	20	20	15	5	-	85	70	60	50	40	35	25	15	10	140	120	100	85	75	65	55	35	20
	J2	-20	27	80	65	55	45	35	30	20	10	5	120	100	85	70	60	50	40	20	10	185	160	140	120	100	85	75	45	30
	J4	-40	27	115	95	80	65	55	45	35	20	5	165	140	120	100	85	70	60	35	15	200	200	185	160	140	120	100	65	35
	K2,M,N	-20	40	95	80	65	55	45	35	30	15	5	140	120	100	85	70	60	50	25	15	200	185	160	140	120	100	85	55	30
	J5,ML,NL	-50	27	135	115	95	80	65	55	45	20	10	190	165	140	120	100	85	70	40	20	200	200	200	185	160	140	120	75	40

Examples from EN 1993-1-10 – selection of steel sub-grades

Second generation EN 1993-1-10

- Table 4.2 covers fatigue structures as per first generation but Table 4.3 covers structures **not** subject to fatigue loading.
- Values in Table 4.3 are derived with same methodology as Table 4.2 but with the number of cycles for the calculation of the crack growth decreased from 500,000 to 20,000 cycles since (nominal fatigue loading)

Table 4.3: Maximum permissible values of element thickness t in mm for EXC1 and EXC2

Steel grade	Quality	KV		Reference Temperature T_{Ed} [°C]																										
		T [°C]	J_{min}	10	0	-10	-20	-30	-40	-50	-80	-120	10	0	-10	-20	-30	-40	-50	-80	-120	10	0	-10	-20	-30	-40	-50	-80	-120
				$\sigma_{Ed} = 0.75 \cdot fy(t)$									$\sigma_{Ed} = 0.5 \cdot fy(t)$									$\sigma_{Ed} = 0.25 \cdot fy(t)$								
S235	JR	20	27	250	250	170	120	90	65	50	25	15	250	250	250	250	190	145	110	55	30	250	250	250	250	250	250	250	135	75
	J0	0	27	250	250	250	250	170	120	90	40	15	250	250	250	250	250	250	190	85	40	250	250	250	250	250	250	250	200	100
	J2	-20	27	250	250	250	250	250	250	170	65	25	250	250	250	250	250	250	250	145	55	250	250	250	250	250	250	250	250	135
S275	JR	20	27	250	185	130	95	70	50	40	20	10	250	250	250	205	150	115	90	45	25	250	250	250	250	250	250	215	120	65
	J0	0	27	250	250	250	185	130	95	70	30	15	250	250	250	250	250	205	150	70	30	250	250	250	250	250	250	250	175	85
	J2	-20	27	250	250	250	250	250	185	130	50	20	250	250	250	250	250	250	250	115	45	250	250	250	250	250	250	250	250	120
	K2,M,N	-20	40	250	250	250	250	250	250	185	70	25	250	250	250	250	250	250	250	150	55	250	250	250	250	250	250	250	250	140
	ML,NL	-50	27	250	250	250	250	250	250	250	130	40	250	250	250	250	250	250	250	250	90	250	250	250	250	250	250	250	250	215
S355	JR	20	27	165	115	85	60	45	35	25	10	5	250	250	190	140	105	80	60	30	15	250	250	250	250	250	210	165	90	50
	J0	0	27	250	240	165	115	85	60	45	20	5	250	250	250	250	190	140	105	45	20	250	250	250	250	250	250	250	135	65
	J2	-20	27	250	250	250	240	165	115	85	35	10	250	250	250	250	250	250	190	80	30	250	250	250	250	250	250	250	210	90
	J4	-40	27	250	250	250	250	250	240	165	60	20	250	250	250	250	250	250	250	140	45	250	250	250	250	250	250	250	250	135
	K2,M,N	-20	40	250	250	250	250	240	165	115	45	15	250	250	250	250	250	250	250	105	35	250	250	250	250	250	250	250	250	110
	J5,ML,NL	-50	27	250	250	250	250	250	250	240	85	25	250	250	250	250	250	250	250	190	60	250	250	250	250	250	250	250	250	165
S420	JR	20	27	120	85	60	45	30	25	20	10	5	250	200	145	105	80	60	45	25	10	250	250	250	250	225	175	140	75	40
	J0	0	27	250	175	120	85	60	45	30	15	5	250	250	250	200	145	105	80	35	15	250	250	250	250	250	250	225	110	55
	J2	-20	27	250	250	250	175	120	85	60	25	10	250	250	250	250	250	200	145	60	25	250	250	250	250	250	250	250	175	75
	J4	-40	27	250	250	250	250	250	175	120	45	15	250	250	250	250	250	250	250	105	35	250	250	250	250	250	250	250	250	110
	M,N	-20	40	250	250	250	250	175	120	85	30	10	250	250	250	250	250	250	200	80	30	250	250	250	250	250	250	250	225	90
	J5,ML,NL	-50	27	250	250	250	250	250	250	175	60	20	250	250	250	250	250	250	250	145	45	250	250	250	250	250	250	250	250	140

Technical improvements - Summary

The updates are balancing:

Making improvements:

- Improving reliability
- Increasing scope to cover missing material designers need
- Standardising further across CEN countries
- Making rules more understandable and consistent

with:

Minimising disruption upon introduction:

- Rework of design guides and software
- Re-education of designers
- Situations where recently designed structures are no longer “adequate”