Stufib
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The 2nd Generation Eurocode2 Key Updates in Concrete Structures

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a world of structures

Introduction

Key ambitions:

- ✓ **Increase ease-of-use**
  - Increased use of tables instead of text
  - Consistent models
  - Whenever possible formulations are expressed in terms which provide orders of magnitude (i.e. stresses instead of forces)
- ✓ **Reduce number of Nationally Determined Parameters (NDPs)**
- ✓ **Extend the scope of the Eurocode including:**
  - Evaluation of early-age and long-term cracking due to restraint (Annex D)
  - Assessment of Existing Structures (Annex I)
  - Strengthening of Existing Concrete Structures with CFRP (Annex J)
  - Steel Fibre Reinforced Concrete Structures (annex L)
  - Recycled aggregates concrete structures (Annex M)
  - Stainless reinforcing steel (Annex Q)
  - Embedded FRP reinforcement (Annex R)
  - Account for size effect throughout the text
- ✓ **Incorporate new knowledge**

Document structure:

- ✓ Document structure is the same for all Eurocodes

- ✓ Part 1-1: Eurocode 2 - Design of concrete structures - Part 1-1: General rules and rules for buildings, **bridges** and civil engineering structures -> no independent bridge part, to emphasize that design of concrete is the same no matter the type of structure it is used in:
  - ✓ Help keep models consistent
  - ✓ Avoid object-oriented design
  - ✓ Improve ease-of-use

There is an unparallelled amount of background information in which all significant changes are treated and explained:

CEN/TC 250/SC 2 "Eurocode 2: Design of concrete structures"	**CEN/TC 250/SC 2 N2087** Date: 31. March 2023	

Background document to FprEN 1992-1-1:2023-04 (Formal-Vote-Draft):

Eurocode 2 - Design of concrete structures - Part 1-1: General rules and rules for buildings, bridges and civil engineering structures

Please note: While wide distribution and use of the background documents is encouraged, any reproduction of an entire paper or parts of a paper, and/or models and figures shown in these papers should make reference to the authors of the papers as follows:

"Reproduced from [list of authors, title of paper given in Background Document, Background Document for FprEN 1992-1-1, CEN/TC 250/SC 2 N2087, pages xx to yy]"

Content list

EN 1992-1-1: 878 pages
EN 1992-1-2: 389 pages

Introduction

Also, there have been several diffusion initiatives. For example, this monographic number of Hormigón y Acero gives a nice overview of changes introduced with respect to EN 1992-1-1:2004. It is in English and free to download.

Vol. 74 Number 299-300 (2023): Hormigón y Acero

Key changes to EN 1992-1-1

Most relevant changes in EN 1992-1-1:

- ✓ Explicit statement on how to deal with restrained imposed strain in SLS and ULS
- ✓ Reference age for compressive strength of concrete
- ✓ Brittleness factor
- ✓ Exposure resistance classes – go towards a performance-based design
- ✓ New formulation for rotation capacity
- ✓ New formulation for shear resistance of members without shear reinforcement
- ✓ Modifications to cracking model to reduce scatter - improved sustainability
- ✓ Accounting for LL/TL in deflections explicitly
- ✓ Simplified model for deflection calculations based on linear elastic analysis
- ✓ New formulation for development length of reinforcement
- ✓ Formula for anchorage of headed bars
- ✓ Rational method to modify material partial factors

(4.2.1.3): Explicitly allows to neglect imposed deformation in ULS. This is a big step forward. The section on neglecting imposed strains if joints are less than a certain length apart has been deleted to avoid encouraging poor durability design.

4.2.1.3 Effects resulting from restrained, imposed deformations

(1) Effects resulting from restrained, imposed deformations should be quantified and considered when verifying serviceability limit states and fatigue.

NOTE Effects resulting from restrained, imposed deformations can be reduced, when necessary, using various methods such as varying the composition of the concrete mix (guidance is given in D.3) and adjusting the stiffness of integral structural restraints. The use of bearings and joints can also reduce these effects.

(2) The effects of restrained, imposed deformations may be neglected at ultimate limit states where it can be demonstrated or has been shown by experience with similar structures that:

a) there is sufficient deformation capacity to allow the respective movements to occur and fulfil the ultimate limit state; and

b) the structures behaviour is not sensitive to second order effects caused by large displacements.

(3) In all other cases, the effects of restrained imposed deformations should be considered.

NOTE For a detailed analysis, see Annex D.

Reference compressive strength can be defined at ages between 28 and 91 days -> meant to favour green concretes

(1) The compressive strength of concrete shall be denoted by concrete strength classes which relate to the characteristic (5 %) cylinder strength f_{ck} of the concrete in accordance with EN 206, determined at an age t_{ref}.

(2) The value for t_{ref}

a) should be taken as 28 days in general; or

b) may be taken between 28 and 91 days when specified for a project.

(3) The compressive and tensile strength characteristics necessary for design should be taken from Table 5.1.

Table 5.1 — Compressive and tensile strength of concrete [MPa]

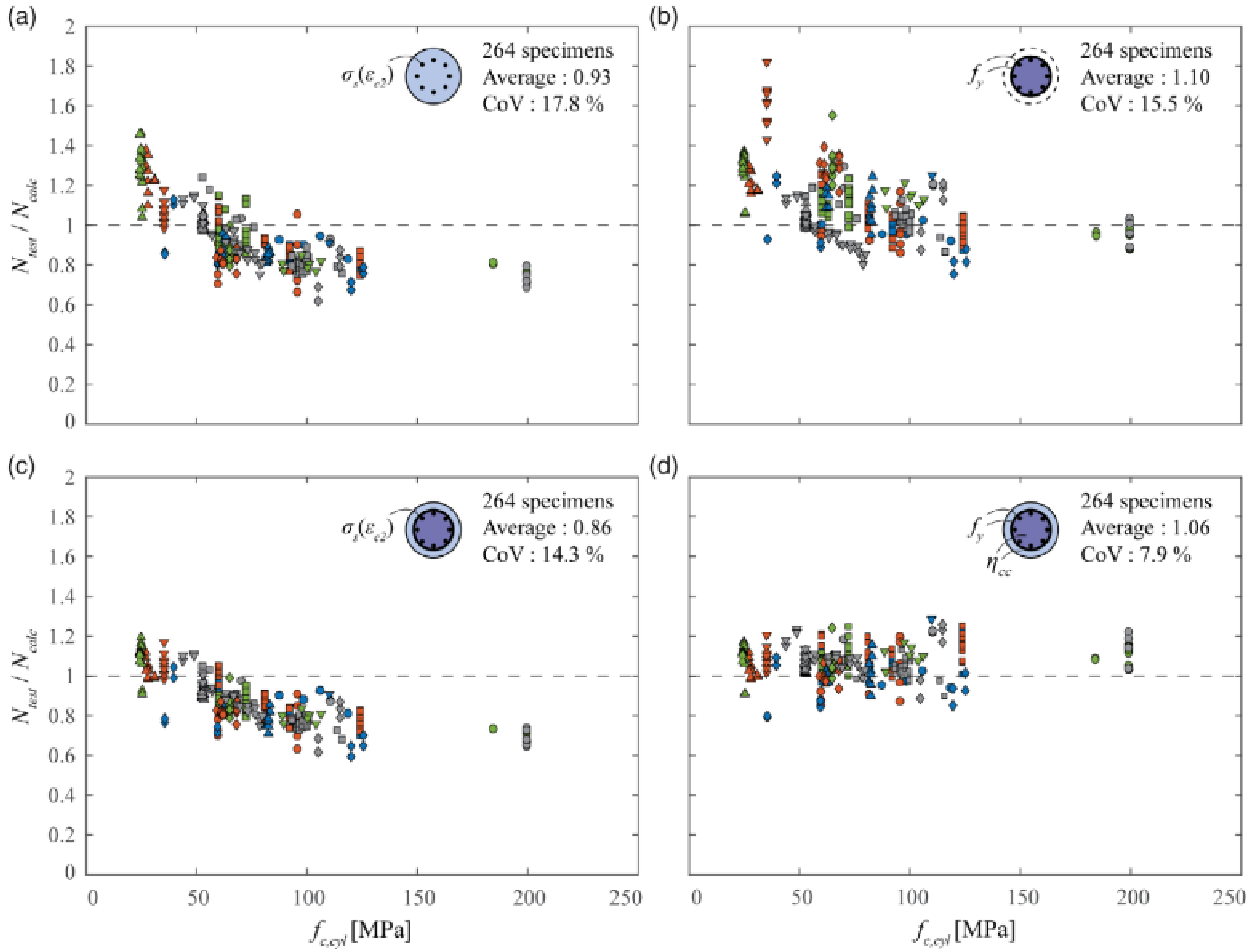
f	Strength classes for concrete [EN 206]															Governing formulae
	C12/15	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60	C55/67	C60/75	C70/85	C80/95	C90/105	C100/115	
f_{ck}	12	16	20	25	30	35	40	45	50	55	60	70	80	90	100	—
f_{cm}	20	24	28	33	38	43	48	53	58	63	68	78	88	98	108	$f_{cm} = f_{ck} + 8$ MPa
f_{ctm}	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,3	4,5	4,7	4,9	5,1	$f_{ctm} = 0{,}3 f_{ck}^{2/3}$ ($f_{ck} \leq 50$ MPa) $f_{ctm} = 1{,}1 f_{ck}^{1/3}$ ($f_{ck} > 50$ MPa)
$f_{ctk;0,05}$	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	2,9	3,0	3,2	3,3	3,5	3,6	$f_{ctk;0,05} = 0{,}7 f_{ctm}$ (5 %-fractile)
$f_{ctk;0,95}$	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,4	5,6	5,9	6,2	6,4	6,6	$f_{ctk;0,95} = 1{,}3 f_{ctm}$ (95 %-fractile)

Introduction of a brittleness factor in the definition of the design concrete strength (5.1.6):

$$f_{cd} = \eta_{cc} \cdot k_{tc} \frac{f_{ck}}{\gamma_C} \qquad \eta_{cc} = \left(\frac{f_{ck,ref}}{f_{ck}}\right)^{\frac{1}{3}} \leq 1{,}0$$

- ✓ Addresses overestimation of compressive strength in columns for high strength concrete
- ✓ Simplifies the definition of constitutive laws for concrete (ε_{c0}=0.20% and ε_{cu}=0.35% for all concrete strengths)

Introduction of a brittleness factor in the definition of the design concrete strength (5.1.6):



Durability: Exposure resistance classes: a performance-based approach

Exposure resistance classes for resistance against corrosion induced by CARBONATION:

According to the definition in table 6.3 the designation of ERCs for resistance against corrosion induced by carbonation (XRC) is derived from the carbonation depth in mm (characteristic value 90 % fractile) assumed to be obtained after 50 years under reference conditions (400 ppm CO2 in a constant 65 % RH environment and at 20°C). XRC has the dimension of a carbonation rate (mm / sqrt(years)).

Durability: Exposure resistance classes: a performance-based approach

Exposure resistance classes for resistance against corrosion induced by CHLORIDE INGRESS:

The designation of XRDS classes for resistance against corrosion induced by chloride ingress is derived from the depth of chlorides (XRDS) penetration [mm] (characteristic value 90 % fractile), corresponding to a reference chlorides concentration (0,6 % by mass of binder cement + type II additions), assumed to be obtained after 50 years on a concrete exposed to one-sided penetration of reference seawater (30 g/l NaCl) at 20 °C. In the following, this depth is noted d_{Cl}. XRDS has the dimension of a diffusion coefficient [10^{-13} m²/s].

$$C(x,t) = C_i + (C_s - C_i)\left(erfc\left\{\frac{x}{\sqrt{D_{app}(t)t}}\right\}\right)$$

$$0.6 = 0.05 + (3.55 - 0.05)\left(erfc\left\{\frac{d_{lc}}{\sqrt{D_{app}(t)t}}\right\}\right)$$

$$erfc\left\{\frac{d_{lc}}{\sqrt{D_{app}(t_{50})t_{50}}}\right\} = \frac{0.05}{3.55} \rightarrow \frac{d_{lc}}{\sqrt{D_{app}(t_{50})t_{50}}} = 1$$

Durability: Exposure resistance classes: a performance-based approach

(1) The minimum concrete covers $c_{min,dur}$ dependent on design service life, exposure class and exposure resistance class (ERC) are given in Table 6.3 (NDP) and Table 6.4 (NDP).

Table 6.3 (NDP) — Minimum concrete cover $c_{min,dur}$ for carbon reinforcing steel — Carbonation

ERC	Exposure class (carbonation)							
	XC1		XC2		XC3		XC4	
	Design service life (years)							
	50	100	50	100	50	100	50	100
XRC 0,5	10	10	10	10	10	10	10	10
XRC 1	10	10	10	10	10	15	10	15
XRC 2	10	15	10	15	15	25	15	25
XRC 3	10	15	15	20	20	30	20	30
XRC 4	10	20	15	25	25	35	25	40
XRC 5	15	25	20	30	25	45	30	45
XRC 6	15	25	25	35	35	55	40	55
XRC 7	15	30	25	40	40	60	45	60

Durability: Exposure resistance classes: a performance-based approach

(1) The minimum concrete covers $c_{min,dur}$ dependent on design service life, exposure class and exposure resistance class (ERC) are given in Table 6.3 (NDP) and Table 6.4 (NDP).

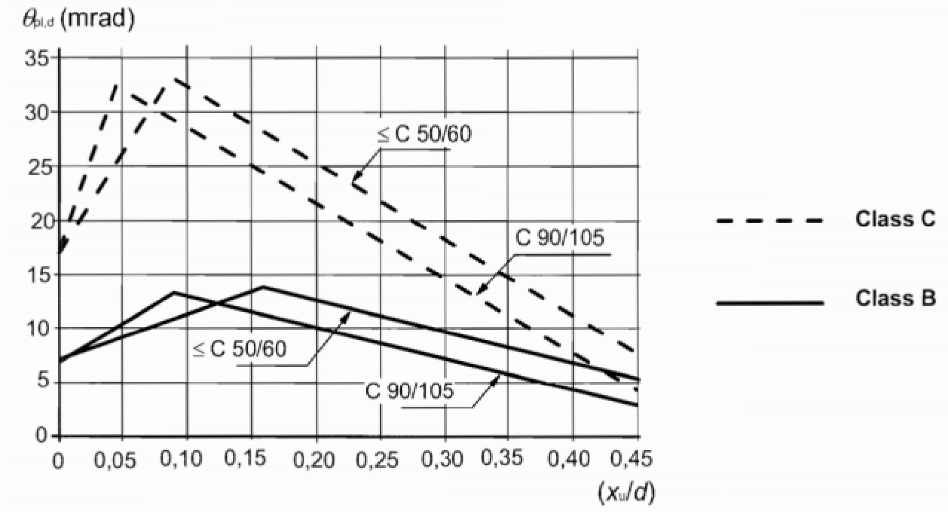
Table 6.4 (NDP) — Minimum concrete cover $c_{min,dur}$ for carbon reinforcing steel — Chlorides

ERC	Exposure class (chlorides)											
	XS1		XS2		XS3		XD1		XD2		XD3	
	Design service life (years)						Design service life (years)					
	50	100	50	100	50	100	50	100	50	100	50	100
XRDS 0,5	20	20	20	30	30	40	20	20	20	30	30	40
XRDS 1	20	25	25	35	35	45	20	25	25	35	35	45
XRDS 1,5	25	30	30	40	40	50	25	30	30	40	40	50
XRDS 2	25	30	35	45	45	55	25	30	35	45	45	55
XRDS 3	30	35	40	50	55	65	30	35	40	50	55	65
XRDS 4	30	40	50	60	60	80	30	40	50	60	60	80
XRDS 5	35	45	60	70	70	—	35	45	60	70	70	—
XRDS 6	40	50	65	80	—	—	40	50	65	80	—	—
XRDS 8	45	55	75	—	—	—	45	55	75	—	—	—
XRDS 10	50	65	80	—	—	—	50	65	80	—	—	—

Rotation capacity

Need for change:

- ✓ With the current proposal for factor η_{cc}, the ultimate strain of concrete is not a function of the concrete strength anymore, so there is no compelling need for the curves in Figure 5.6N of EN1992-1-1:2004 to be dependent on the concrete strength.
- ✓ The curves of Figure 5.6N do not seem to be totally correct, since for high values of the x_u/d ratio, the rotation capacity should be independent of the steel class since failure occurs by crushing of concrete.
- ✓ It is not fully transparent how the curves of Figure 6.6N have been derived. There is very limited background information which does not provide information on the how, for instance, tension stiffening effects were considered what the refined curves look like or how the simplified bi-linear law shown of Figure 5.6N was derived.
- ✓ There is no information available regarding the model uncertainty factor that was applied to produce this graph and no information regarding how it fits with experimental data.
- ✓ There is no information regarding the limits of validity of this simplification.
- ✓ EN 1992-1-1:2004 does not account for the **size effect** which has been shown by tests to be a major variable to predict rotation capacity (see, for instance, [C07-8]).



Rotation capacity

The rotation capacity may be derived from Formula (7.18).

$$\theta_{\mathrm{Rd}} = \frac{1{,}3d}{\gamma_\theta}\left(\left(\frac{1}{r}\right)_{\mathrm{u,m}} - TS_{\mathrm{My}}\frac{\varepsilon_{\mathrm{yd}}}{d-x}\right) \tag{7.18}$$

where

$$\left(\frac{1}{r}\right)_{\mathrm{u,m}} = TS_{\mathrm{Mu}} \cdot \min\left\{\frac{\varepsilon_{\mathrm{ud}}}{(d-x_{\mathrm{u}})};\frac{\varepsilon_{\mathrm{cu,d,}\rho_{\mathrm{w}}}}{x_{\mathrm{u}}}\right\} \tag{7.19}$$

$$\varepsilon_{\mathrm{cu,d,}\rho_{\mathrm{w}}} = 0{,}002 + \frac{1{,}35}{d} + 3\rho_w \leq 0{,}015 \tag{7.20}$$

$$TS_{\mathrm{Mu}} = 1 - \frac{1}{2\alpha}\cdot\left(1-\frac{\varepsilon_{\mathrm{yd}}}{\varepsilon_{\mathrm{ud,ef}}}\right) \text{ when } \alpha \geq 1 \tag{7.21}$$

$$TS_{\mathrm{Mu}} = \frac{\alpha}{2} + \frac{\varepsilon_{\mathrm{yd}}}{\varepsilon_{\mathrm{ud,ef}}}\left(1-\frac{\alpha}{2}+\left(\frac{f_{\mathrm{s,ef}}}{f_{\mathrm{yd}}}-1\right)\cdot\left(2-\frac{1}{\alpha}-\alpha\right)\right) \text{ when } \alpha < 1 \tag{7.22}$$

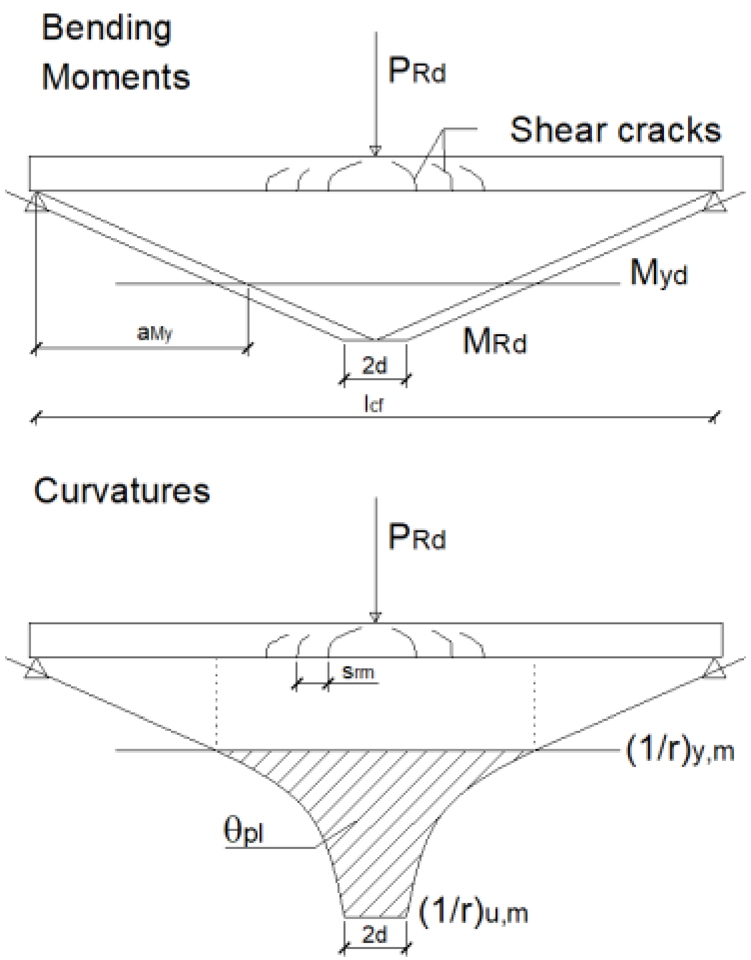
$$TS_{\mathrm{My}} = 1 - 0{,}6\left[\frac{M_{\mathrm{cr}}}{M_{\mathrm{y}}}\right] \geq 0{,}4 \tag{7.23}$$

$$\alpha = \frac{\left(f_{\mathrm{s,ef}} - f_{\mathrm{yd}}\right)}{0{,}6 f_{\mathrm{cm}}^{\frac{2}{3}}}\cdot\frac{\phi}{s_{\mathrm{r,m,cal}}} \tag{7.24}$$

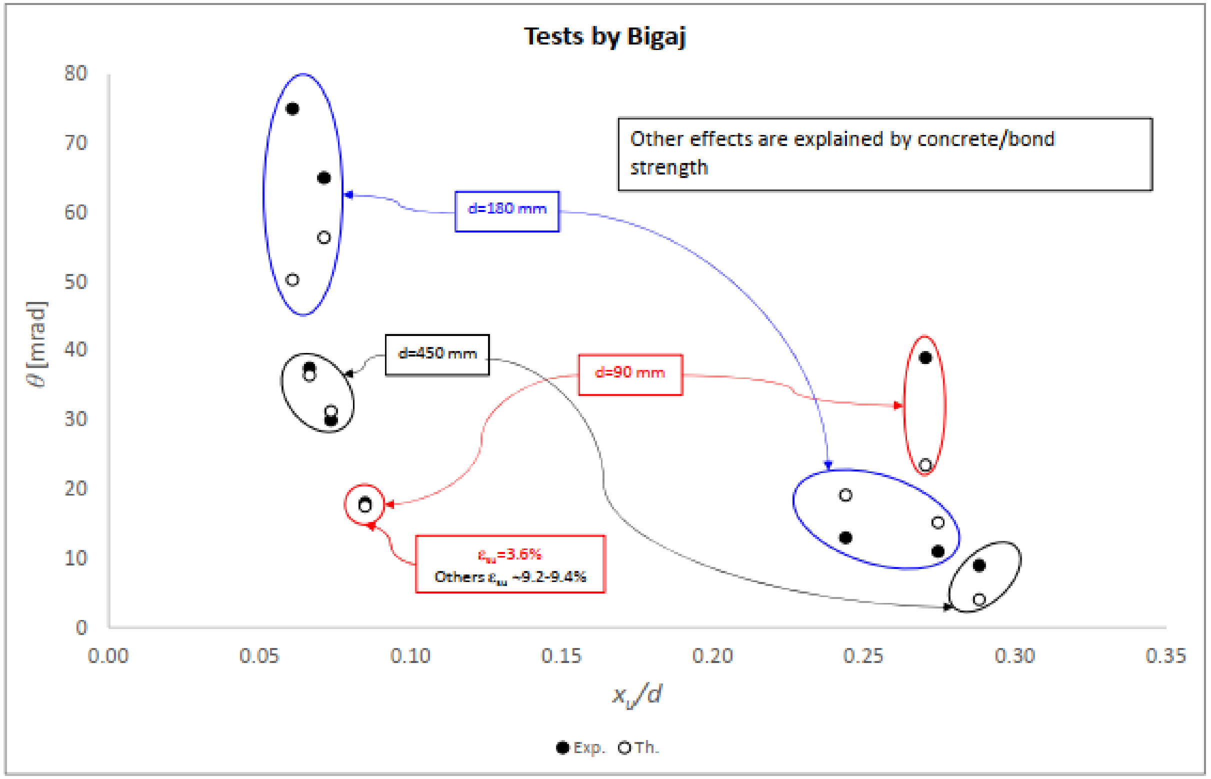
where

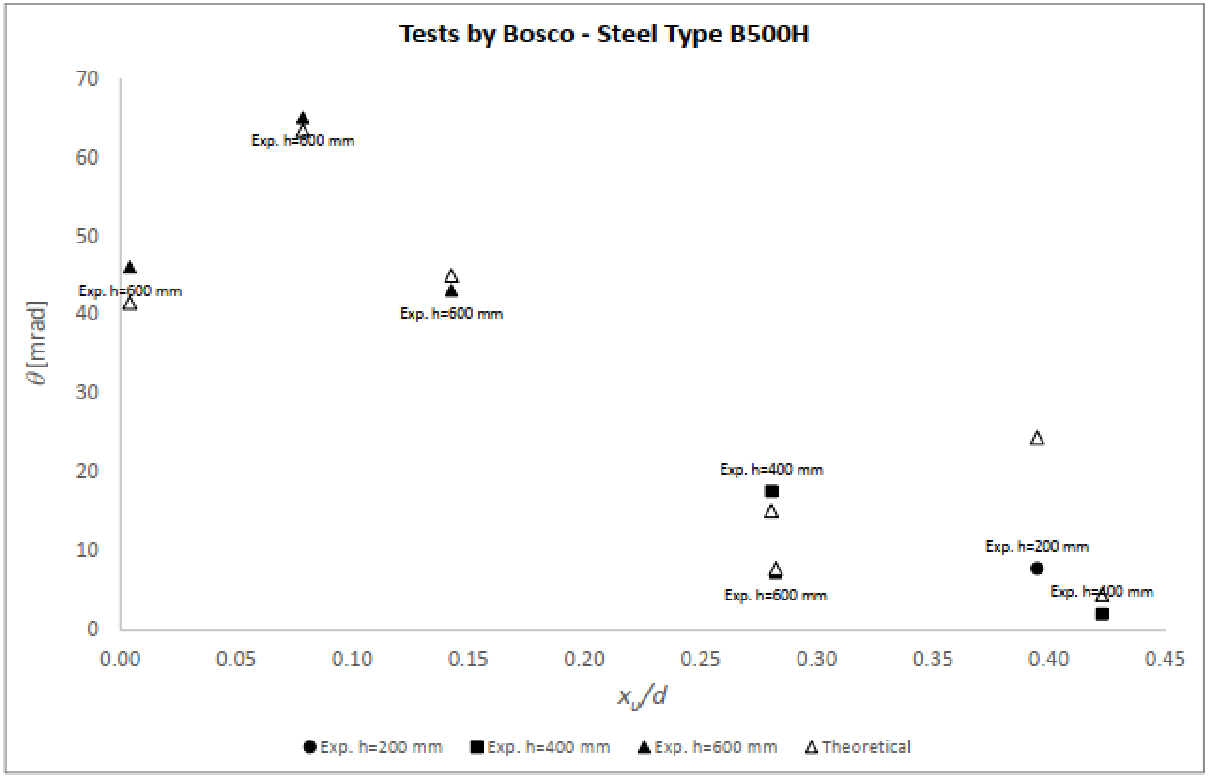
- $f_{\mathrm{s,ef}}$ is the tensile stress in the reinforcement when M_{Rd} is reached, assuming an inclined post-elastic stress-strain-relation, see Figure 5.2;
- $\varepsilon_{\mathrm{ud,ef}}$ is the strain in the reinforcement with a stress equal to $f_{\mathrm{s,ef}}$;
- M_{y} is the internal moment when the strain in the tension reinforcement equals $\varepsilon_{\mathrm{yd}}$;
- $\varepsilon_{\mathrm{yd}}$ $= f_{\mathrm{yd}} / E_{\mathrm{s}}$
- γ_θ is a partial factor for model uncertainty.

NOTE The value of $\gamma_\theta = 3{,}0$ applies unless the National Annex gives a different value.



Rotation capacity

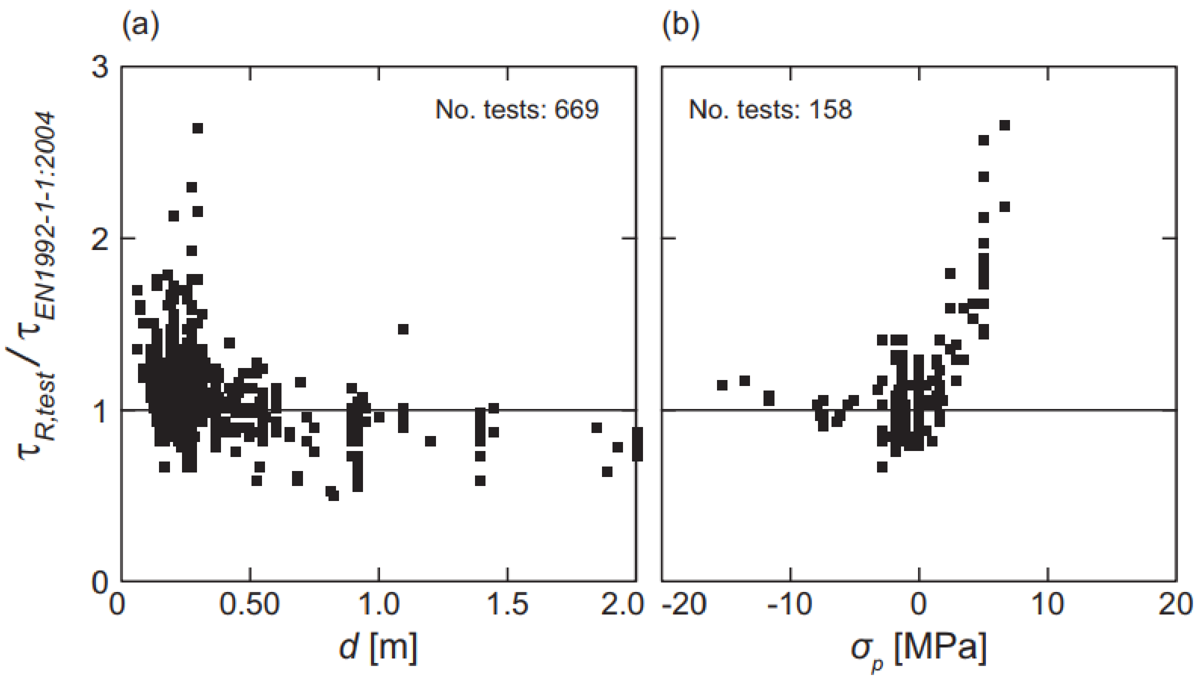




Shear in members without shear reinforcement:

New formulation derived from the Critical Shear Crack Theory (CSCT)

- Reasons for change:
  - The current formulation underestimates the influence of size effect
  - The current formulation is too conservative for members subjected to tensile axial forces
  - The effect of shear slenderness is not considered and could yield unsafe results for slender beams
  - Current formulation derived for point loading in simply supported beams which is not representative of practical cases



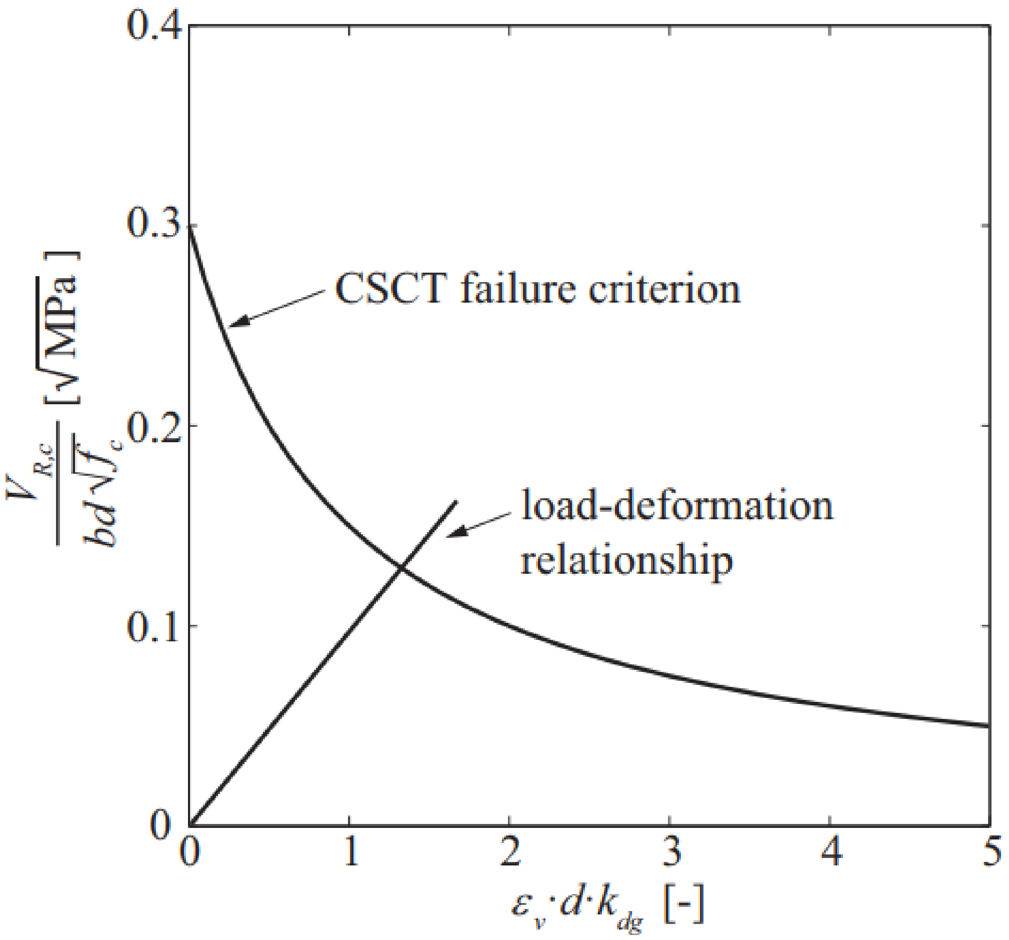
Shear in members without shear reinforcement:

New formulation derived from the Critical Shear Crack Theory (CSCT)

$$V_{R,c} = \frac{0.3}{1 + \varepsilon_v \cdot d \cdot k_{dg}} \cdot \sqrt{f_c} \cdot b_w \cdot d$$

$$\varepsilon_v = \frac{M_E}{z \cdot A_s \cdot E_s} = \frac{M_E}{z \cdot \rho_l \cdot b_w \cdot d \cdot E_s} = \frac{V_E \cdot a_{cs}}{z \cdot \rho_l \cdot b_w \cdot d \cdot E_s}$$

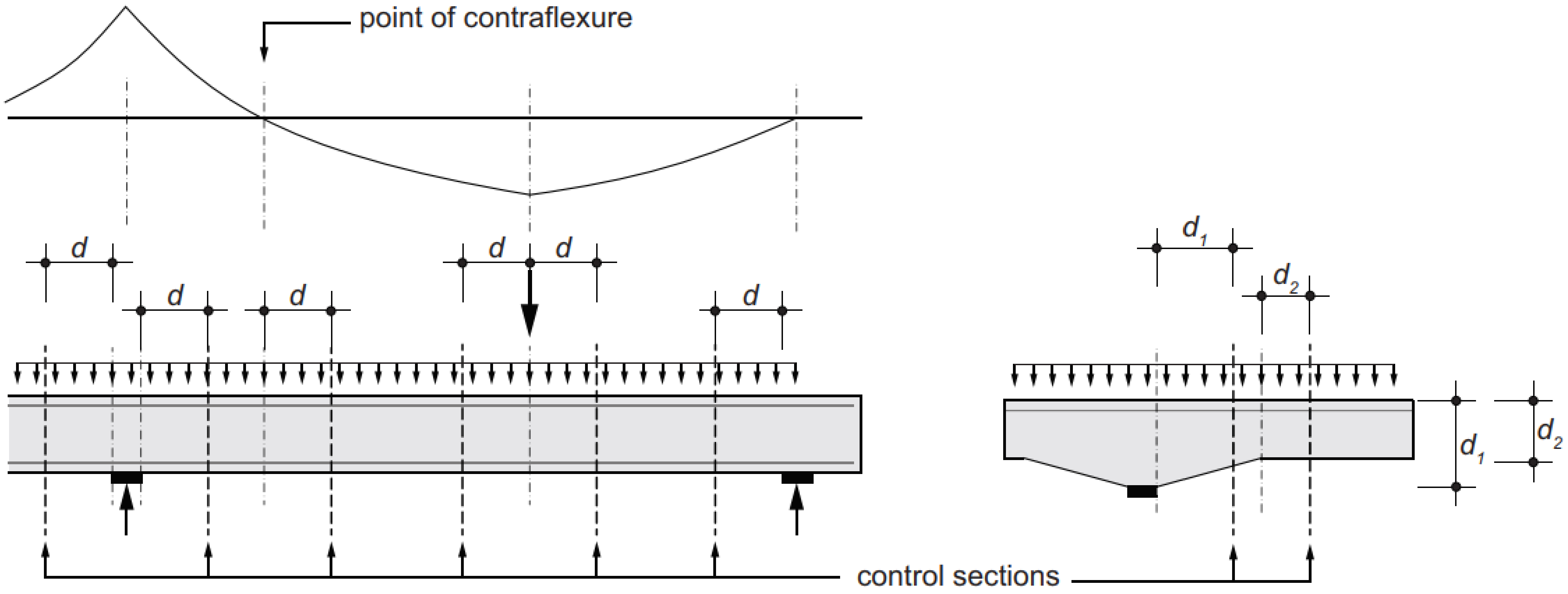
$a_{cs} = \left| M_E / V_E \right|$ is the effective shear span at the control section;



Need for iterations to determine failure point (intersection)

Shear in members without shear reinforcement:

Location of control sections



Shear in members without shear reinforcement:

Refinement of the mechanical of the CSCT, allows a closed-form solution

$$V_{R,c} = k \cdot \left(\frac{f_c \cdot d_{dg}}{\varepsilon_v \cdot d} \right)^{1/2} \cdot b_w \cdot d \leq V_{R,c0}$$

where:

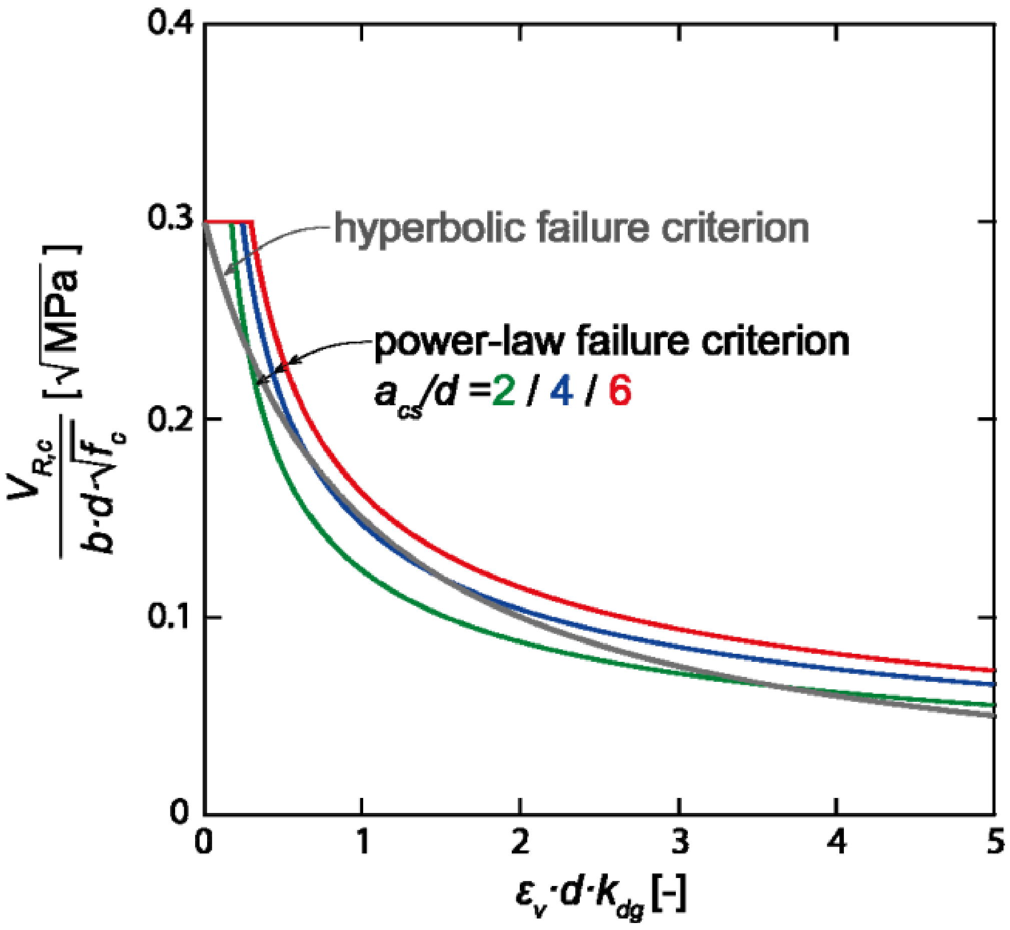
k depends on the location and shape (inclination) of the critical shear crack and, based on [C8.4,C8.5, C8.6], can be estimated to depend upon the ratio a_{cs}/d resulting into $k = 0.015 \cdot (a_{cs}/d)^{1/4}$;

d_{dg} = 16 mm+D_{lower} ≤ 40 mm refers to the average roughness dimension of the critical shear crack (see definition in 8.2.1(4));

$V_{R,c0}$ is the maximum shear resistance.

Shear in members without shear reinforcement:

Refinement of the mechanical of the CSCT, allows a closed-form solution



$$V_{R,c} = k \cdot \left(\frac{f_c \cdot d_{dg}}{\varepsilon_v \cdot d} \right)^{1/2} \cdot b_w \cdot d \leq V_{R,c0} \qquad k = 0.015 \cdot (a_{cs}/d)^{1/4}$$

$$+$$

$$\varepsilon_v = \frac{M_E}{z \cdot A_s \cdot E_s} = \frac{M_E}{z \cdot \rho_l \cdot b_w \cdot d \cdot E_s} = \frac{V_E \cdot a_{cs}}{z \cdot \rho_l \cdot b_w \cdot d \cdot E_s}$$

$$=$$

$$V_{R,c} = 0.015 \cdot \left(\frac{f_c \cdot d_{dg}}{V_{R,c} \cdot \sqrt{a_{cs} \cdot d}} \cdot \rho_l \cdot b_w \cdot E_s \cdot z \right)^{1/2} \cdot b_w \cdot d$$

Shear in members without shear reinforcement:

Refinement of the mechanical of the CSCT, allows a closed-form solution

$$V_{R,c} = 0.015 \cdot \left(\frac{f_c \cdot d_{dg}}{V_{R,c} \cdot \sqrt{a_{cs} \cdot d}} \cdot \rho_l \cdot b_w \cdot E_s \cdot z \right)^{1/2} \cdot b_w \cdot d$$

$$\downarrow$$

$a_{cs}=4d$

$$V_{R,c} = 0.015^{2/3} \cdot \left(\rho_l \cdot E_s \cdot f_c \cdot \frac{d_{dg}}{\sqrt{a_{cs} \cdot d}} \cdot \frac{z}{d} \right)^{1/3} \cdot b_w \cdot d \rightarrow \tau_{Rd,c} = \frac{V_{Rd,c}}{b_w \cdot z} \simeq \frac{V_{Rd,c}}{b_w \cdot 0.9 \cdot d} \simeq \frac{0.66}{\gamma_V} \cdot \left(100 \cdot \rho_l \cdot f_{ck} \frac{d_{dg}}{d} \right)^{1/3}$$

Key changes to EN 1992-1-1

Changes in verification of cracking:

Mean crack spacing:

$$s_m = k_c \cdot c + k_{\phi/\rho} \cdot k_{fl} \cdot k_b \cdot \frac{f_{ctm}}{\tau_{bms}} \cdot \frac{\phi_s}{\rho_{s,ef}} \rightarrow s_m = 1.5 \cdot c + \frac{k_{fl} \cdot k_b \cdot}{7.2} \frac{\phi_s}{\rho_{s,ef}}$$

accounts for distribution of stresses

accounts for bond conditions

Mean strain of steel with respect to concrete:

accounts for curvature

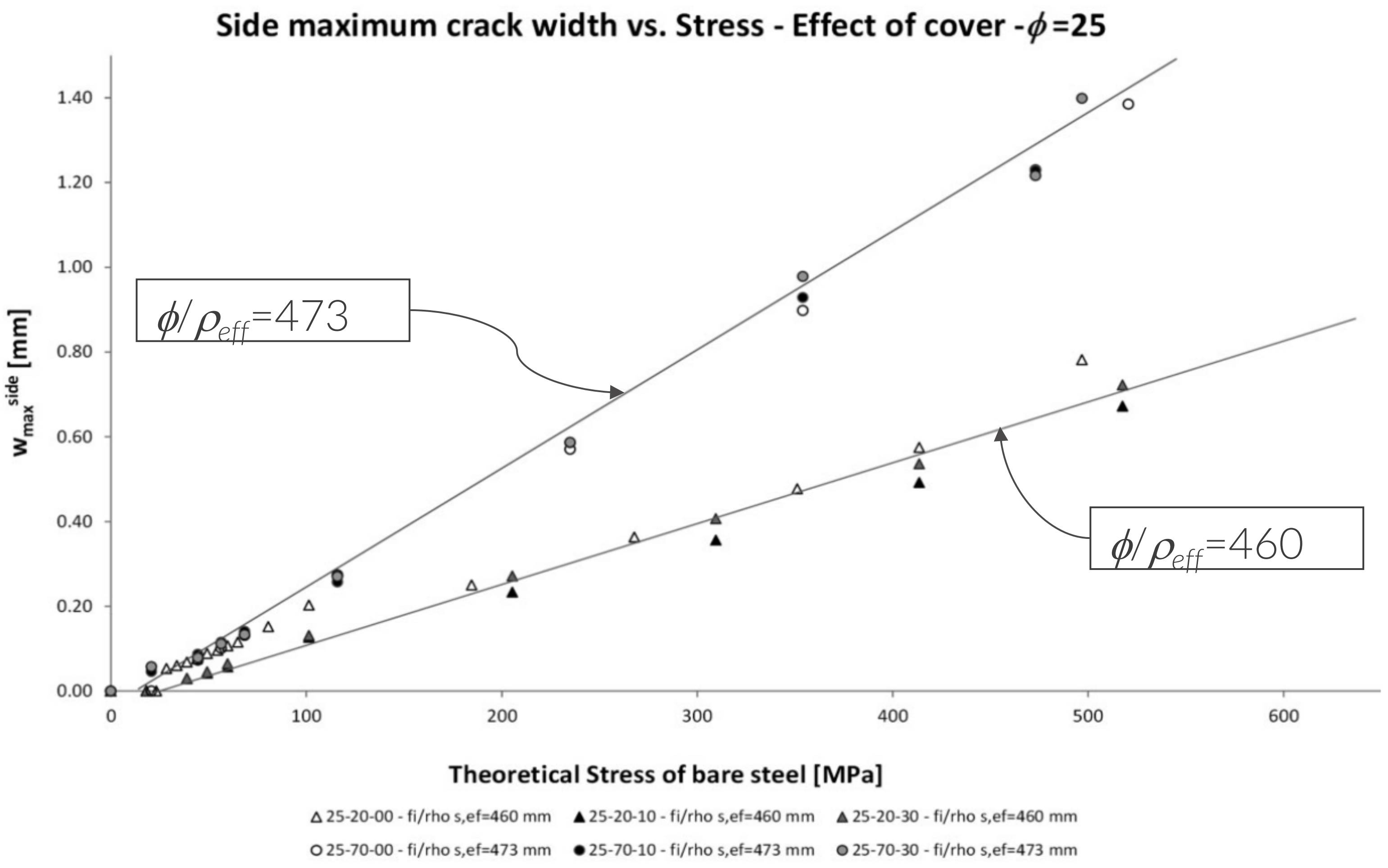
$$\varepsilon_{sm} - \varepsilon_{cm} = k_{1/r} \frac{\sigma_s - k_t \dfrac{f_{ctm}}{\rho_{ef}}\left(1 + \alpha_e \rho_{ef}\right)}{E_c} \geq 0.6 \frac{\sigma_s}{E_c}$$

Design crack width:

From mean to characteristic values

$$w_d = \beta_w \cdot s_m \left(\varepsilon_{sm} - \varepsilon_{cm}\right) \rightarrow 1.7 \cdot s_m \left(\varepsilon_{sm} - \varepsilon_{cm}\right)$$

The need for a cover term – experimental evidence is unquestionable:



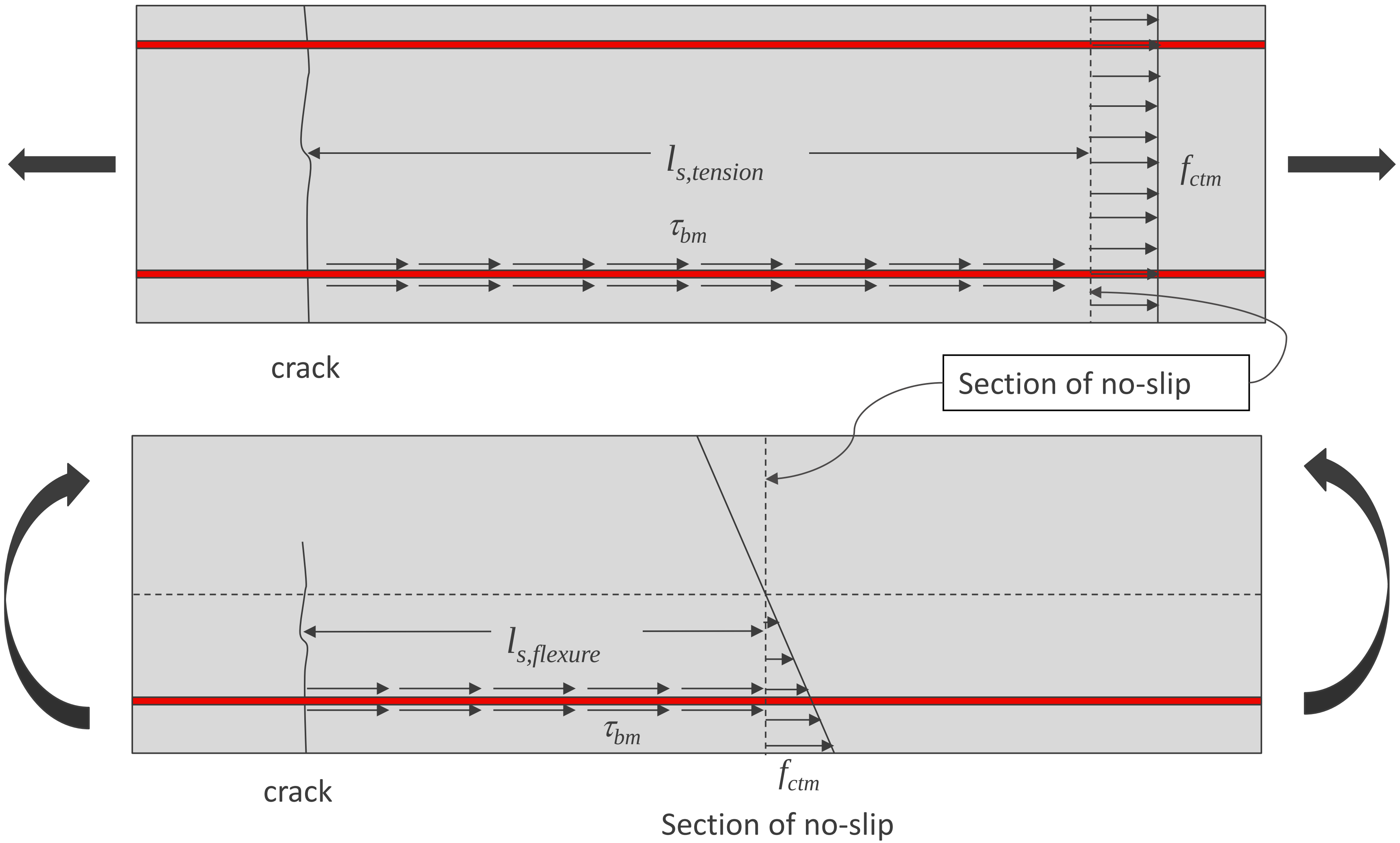
Bending vs tension:

Effect of stress distribution – Coefficient k_{fl}

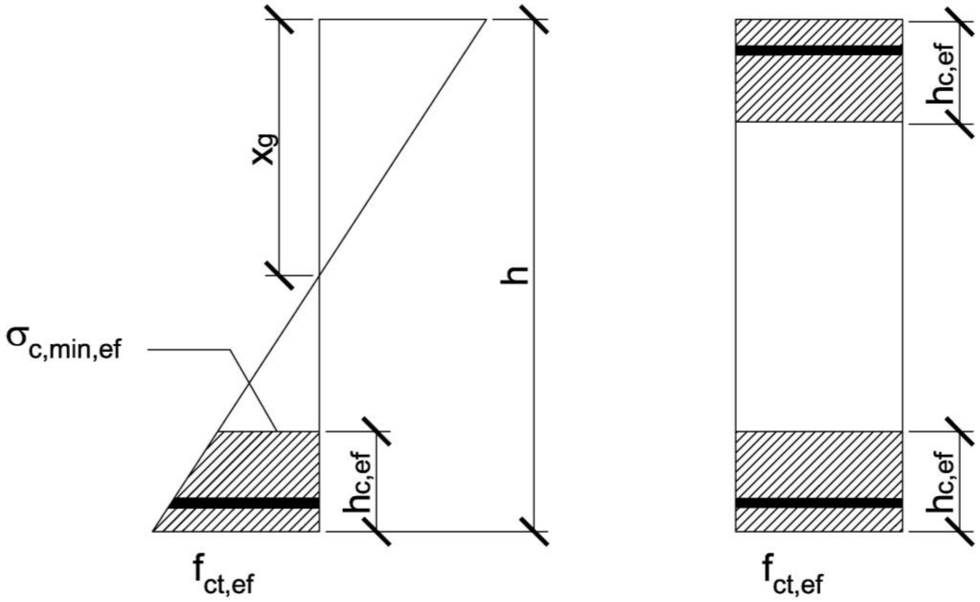
(h-x)/3 and coefficient k_{fl}

EN 1992-1-1:2004

$$s_{r,max}=k_3\cdot c+k_1\cdot k_2\cdot k_4\cdot \phi/\rho_{p,eff}$$



Bending vs tension: when k_{fl} is introduced, there is no need for the (h-x)/3 limit for the effective area -> New definition of $A_{c,eff}$



a) Bending b) Tension

$$\sigma_{mean} = \frac{1}{2}\left(f_{ct,ef} + \sigma_{c,min,ef}\right) = \frac{1}{2}\left(f_{ct,ef} + \frac{f_{ct,ef}}{h-x_g}\left(h - x_g - h_{c,ef}\right)\right) =$$

$$= f_{ct,ef} \underbrace{\frac{1}{2}\left(1 + \frac{\left(h - x_g - h_{c,ef}\right)}{h - x_g}\right)}_{k_{fl}}$$

Rectangular CS:

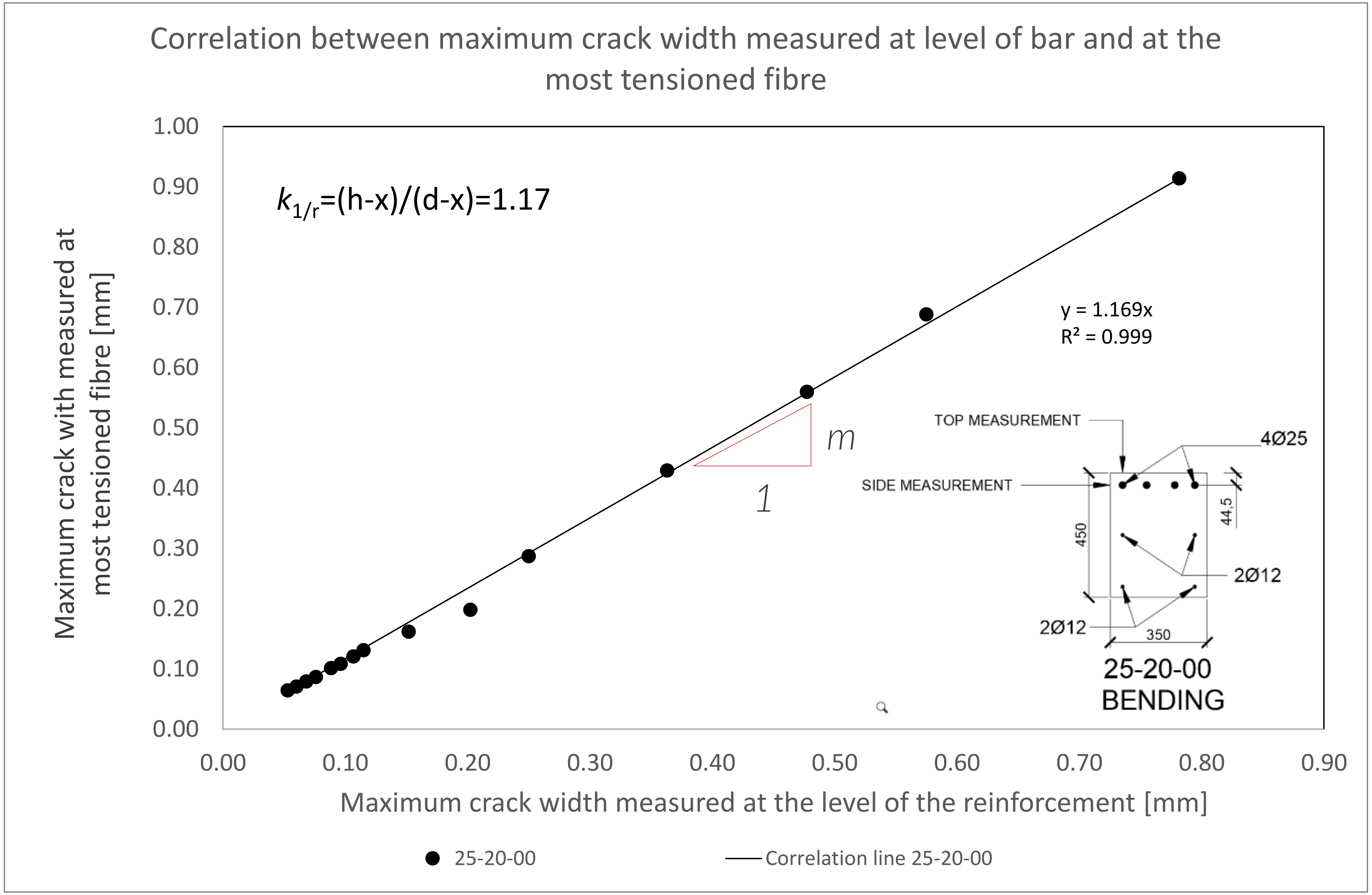
$$k_{fl} = \frac{h - h_{c,ef}}{h}$$

Bending vs tension:

(h-x)/3 and coefficient k_{fl}

				FprEN 1992-1-1:2022			MC 2010			EN 1992-1-1:2004		
BEAMS	**Measured mean crack spacing (mm)**			**Predicted mean crack spacing (mm)**								
	B	**T**	**Inc. (T-B)/B**	**B**	**T**	**Inc. (T-B)/B**	**B**	**T**	**Inc. (T-B)/B**	**B**	**T**	**Inc. (T-B)/B**
12-20-B/T-G	115	162	0.41	137	162	0.18	182	182	0.00	152	240	0.58
16-20-B/T-G	105	147	0.40	125	152	0.22	151	151	0.00	134	203	0.52
16-70-B/T-G	183	220	0.20	213	262	0.23	233	352	0.51	248	477	0.93
16-20-B/T-PL	109.8	170.5	0.55	150	187	0.25	151	151	0.00	134	203	0.52
16-70-B/T-PL	188	232	0.23	243	309	0.27	233	352	0.51	248	477	0.93
25-20-B/T-G	86.7	115	0.33	105	135	0.29	113	119	0.05	110	163	0.48
25-70-B/T-G	148.3	184	0.24	186	245	0.32	175	260	0.49	212	365	0.72
25-20-B/T-PL	96.5	171	0.77	124	164	0.32	113	119	0.05	110	163	0.48
Average	**129**	**175**	**0.39**	**160**	**202**	**0.26**	**169**	**211**	**0.20**	**168**	**287**	**0.64**

Effect of curvature:



Key changes to EN 1992-1-1

Better modelling → reduction in scatter → improved sustainability (more reinforcement only when actually necessary)

Statistics of calibration data set:

		MC 2010		EN 1992-1-1:2004		New proposal	
		Flexure	**Tension**	**Flexure**	**Tension**	**Flexure**	**Tension**
	nº of tests	37	36	37	36	37	36
	$\sqrt{(\Sigma\varepsilon^2/N)}$	**38.28**	**37.96**	**35.25**	**83.01**	**29.01**	**30.97**
Model/Measured	**min($s_{m,model}$/s**	0.46	0.51	0.60	0.68	0.53	0.67
	max($s_{m,model}$/	1.78	1.51	1.74	2.04	1.47	1.34
	μ =	1.04	1.05	1.08	1.43	1	1.01
	σ =	0.29	0.22	0.27	0.30	0.24	0.158
	COV =	**27.68%**	**21.18%**	**24.69%**	**21.25%**	**23.57%**	**15.62%**

Better modelling → reduction in scatter → improved sustainability (more reinforcement only when actually necessary)

Statistics of test data set (independent – post-tested)

		Test data set (flexure)		
		MC 2010	**EN 1992-1-1:2004**	**New proposal**
	nº of tests	144	144	144
	$\sqrt{(\Sigma\varepsilon^2/N)}$	**26.97**	**26.30**	**23.55**
Model/Measured	**min($s_{m,model}$/s**	0.54	0.51	0.53
	max($s_{m,model}$/	1.87	1.63	1.76
	μ =	1.05	0.99	1.04
	σ =	0.24	0.24	0.2
	COV =	**23.04%**	**24.56%**	**18.85%**

Account explicitly for influential, but previously implicit, variable – Live Load/Total Load + Slabs

Table 9.3 — Limiting span/effective depth ratios l/d for buildings

	Structural system	Required mechanical reinforcement ratio[a]								
		$\omega_r = 0{,}3$			$\omega_r = 0{,}2$			$\omega_r = 0{,}1$		
		LL/TL[b]			LL/TL[b]			LL/TL[b]		
		60 %	45 %	30 %	60 %	45 %	30 %	60 %	45 %	30 %
1	Simply supported beam, one-way spanning simply supported slab	15	14	12	17	15	13	22	19	17
2	End span of continuous beam or one-way spanning slab	20	18	16	22	20	17	29	25	22
3	Interior span of continuous beam or one-way spanning slab	23	21	18	26	23	20	33	29	26
4	Cantilever	7	7	6	8	7	6	10	9	8

NOTE This table assumes the quasi-permanent value of the live load with $\psi_2 = 0{,}3$ and that the deflection limit for long-term deflection is $l/250$, where l is the span of the beam or slab.

[a] $\omega_r = A_{s,req}/(b_w \cdot d) \cdot f_{yd}/f_{cd}$ is the required mechanical tension reinforcement ratio to resist the moment due to the design loads, at mid span for continuous or simply supported elements and at the support for cantilevers. Intermediate values may be interpolated. The limits are conservative for flanged sections.

[b] Characteristic values of: LL = characteristic value of live load (imposed or variable); TL = characteristic value of total load. Intermediate values may be interpolated.

(4) For rectangular 2-way flat slabs supported on columns, with slenderness ratio l_{max}/d, the limiting span/effective depth ratios of Table 9.3 may be multiplied by the following factor:

$$\left(\frac{1}{1+\left(\frac{l_{min}}{l_{max}}\right)^4}\right)^{1/4} \tag{9.21}$$

where

l_{min} is the minimum span of the slab;

l_{max} is the maximum span of the slab.

(5) For rectangular 2-way flat slabs, simply supported on walls or supported on stiff beams on all four sides, with slenderness ratio l_{min}/d, the limiting span/effective depth ratios of Table 9.3 may be multiplied by the following factor:

$$\left(\frac{1}{1-0{,}65\frac{l_{min}}{l_{max}}}\right)^{1/4} \tag{9.22}$$

Simplified calculation of deflections:

- ✓ Deflection:

Elastic deflection due to loads

Elastic deflection due to shringage

$$\delta = k_I \left[\delta_{LOADS} + k_S \delta_{\varepsilon cs} \right]$$

Accounts for cracking (I_{cr}/I_g) and TS

Accounts for cracking (S_e/S_g) and TS

If the section cracks:

$$\frac{I_{cr}}{I_g} = 2.7\left(\alpha_{e,ef}\,\rho\right)^{0.6}\left(\frac{d}{h}\right)^3$$

$$\zeta = \left[1 - 0.5\left(\frac{M_{cr}}{M_k}\right)^2\right]$$

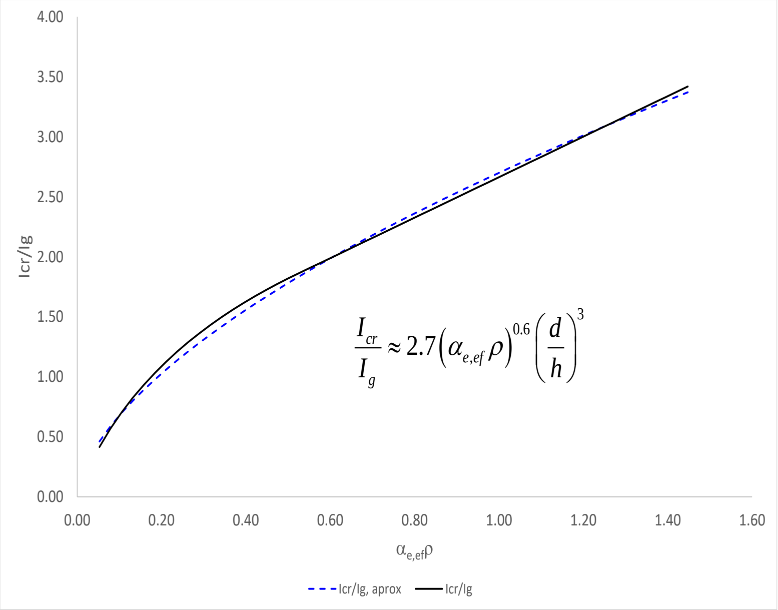
$$k_I = \zeta\frac{I_g}{I_{cr}} + (1-\zeta)$$

$$k_S = 455\rho^2 - 35\rho + 1,6$$

If the section does not crack:

$$k_I = 1,00$$

$$k_s = 1,00$$



Development length:

$$l_{\mathrm{bd}} = k_{\mathrm{lb}} \cdot k_{\mathrm{cp}} \cdot \phi \cdot \left(\frac{\sigma_{\mathrm{sd}}}{435}\right)^{n_\sigma} \cdot \left(\frac{25}{f_{\mathrm{ck}}}\right)^{\frac{1}{2}} \cdot \left(\frac{\phi}{20}\right)^{\frac{1}{3}} \cdot \left(\frac{1{,}5\phi}{c_{\mathrm{d}}}\right)^{\frac{1}{2}} \geq 10\phi \quad (11.3)$$

where

Ratios in Formula (11.3) shall be limited to $(\phi/20\ \mathrm{mm}) \geq 0{,}6$ and $(25/f_{\mathrm{ck}}) \geq 0{,}3$:

c_{d} — $c_{\mathrm{d}} = \min\{0{,}5c_{\mathrm{s}};\ c_{\mathrm{x}};\ c_{\mathrm{y}};\ 3{,}75\phi\}$, see Figure 11.3.

k_{cp} — coefficient accounting for casting effects on bond conditions:

- $k_{\mathrm{cp}} = 1{,}0$ for bars with good bond conditions according to (4);
- $k_{\mathrm{cp}} = 1{,}2$ for poor bond conditions and for all bars used in slipform construction unless it is shown that the vertical bars cannot move during casting;
- $k_{\mathrm{cp}} = 1{,}4$ for all bars executed under bentonite or similar slurries unless data is available for the specific slurry to be used;

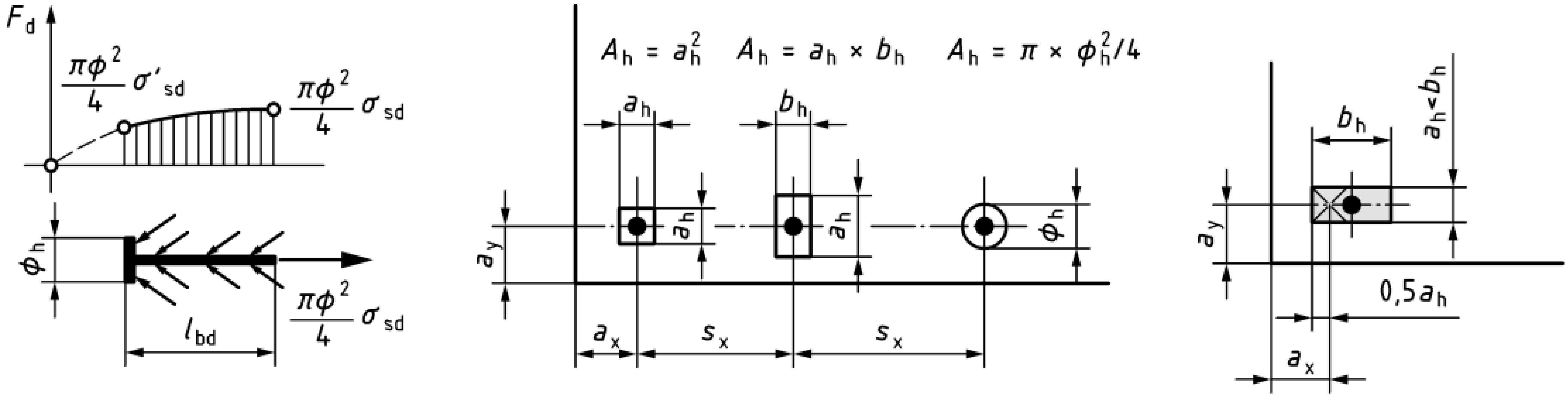
NOTE For anchorages, the following values for k_{lb} and n_σ apply unless the National Annex gives different values:

$k_{\mathrm{lb}} = 50$ for persistent and transient design situations with $n_\sigma = 3/2$; and

$k_{\mathrm{lb}} = 35$ for accidental design situations with $n_\sigma = 3/2$.

Anchorage of headed bars – formulation based on a physical model:

$$\sigma'_{sd} = k_{h,A} \cdot f_{cd} + \nu_{part} \frac{\sqrt{f_{ck}}}{\gamma_C} \cdot \frac{a_d}{\phi} \cdot \left(\frac{\phi_h}{\phi}\right)^{\frac{5}{6}} \cdot \left(\frac{d_{dg}}{\phi}\right)^{\frac{1}{3}} \leq k_{h,A} \cdot \nu_{part} \cdot f_{cd}$$



Adjustment of partial factors for materials (Annex A):

(3) The adjusted partial factors may be calculated as:

$$\gamma_{M} = \frac{\exp(\alpha_{R} \cdot \beta_{tgt} \cdot V_{RM})}{\mu_{RM}} \quad (A.1)$$

where

index M is S for reinforcement, C for concrete in compression and V for shear;

α_{R} is the sensitivity factor for resistance according to Table A.4 (NDP);

β_{tgt} is the target value for the 50-year reliability index according to Table A.4 (NDP);

V_{RM} is the coefficient of variation of the resistance which may be calculated from

$$V_{RS} = \sqrt{V_{fy}^{2} + V_{d}^{2} + V_{\theta s}^{2}} \quad (A.2)$$

$$V_{RC} = \sqrt{V_{fc}^{2} + V_{\eta is}^{2} + V_{Ac}^{2} + V_{\theta c}^{2}} \quad (A.3)$$

$$V_{RV} = \sqrt{\left(\frac{V_{fc}}{3}\right)^{2} + \left(\frac{V_{\eta is}}{3}\right)^{2} + V_{d}^{2} + V_{\theta v}^{2} + V_{res,v}^{2}} \quad (A.4)$$

where the coefficients of variation of each uncertainty are defined in Table A.3 or updated (for conditions, see Table A.1 (NDP) and Table A.2 (NDP));

μ_{RM} is the bias factor of the resistance and may be calculated from:

$$\mu_{RS} = \frac{f_{ym}}{f_{yk}} \mu_{d} \cdot \mu_{\theta S} \quad (A.5)$$

$$\mu_{RC} = \frac{f_{cm}}{f_{ck}} \mu_{\eta is} \cdot \mu_{Ac} \cdot \mu_{\theta C} \quad (A.6)$$

$$\mu_{RV} = \left(\frac{f_{cm}}{f_{ck}} \cdot \mu_{\eta is}\right)^{1/3} \cdot \mu_{d} \cdot \mu_{\theta V} \quad (A.7)$$

where the bias factors of each uncertainty are defined in Table (A.3) or updated (for conditions, see Tables A.1 (NDP) and A.2 (NDP)).

Table A.4 (NDP) — Sensitivity factors for resistance α_{R} and target values for the 50-year reliability index β_{tgt}

Design situations/Limit states	Sensitivity factors for resistance α_{R}	target value for the 50-year reliability index β_{tgt}
Persistent or transient design situation	0,8	3,8
Fatigue design situation	0,8	3,8
Accidental design situation	0,8	2,0
NOTE These values refer to CC2. For others Consequence Classes, refer to EN 1990.		

Adjustment of partial factors for materials (Annex A):

Table A.3 — Statistical data assumed for the calculation of partial factor defined in Table 4.3 (NDP)

	Coefficient of variation	Bias factor[a]
Partial factor for reinforcement γ_S		
Yield strength f_y	$V_{fy} = 0{,}045$	$f_{ym}/f_{yk} = \exp(1{,}645 V_{fy})$
Effective depth d	$V_d = 0{,}050$[b]	$\mu_d = 0{,}95$[b]
Model uncertainty	$V_{\theta s} = 0{,}045$[c]	$\mu_{\theta s} = 1{,}09$[c]
Coefficient of variation and bias factor of resistance for reinforcement	$V_{RS} = 0{,}081$[i]	$\mu_{RS} = 1{,}115$[i]
Partial factor for concrete γ_C		
Compressive strength f_c (control specimen)	$V_{fc} = 0{,}100$	$f_{cm}/f_{ck} = \exp(1{,}645 V_{fc})$[d]
Insitu factor $\eta_{is} = f_{c,ais}/f_c$[e]	$V_{\eta is} = 0{,}120$	$\mu_{\eta is} = 0{,}95$
Concrete area A_c	$V_{Ac} = 0{,}040$	$\mu_{Ac} = 1{,}00$
Model uncertainty	$V_{\theta c} = 0{,}070$[f]	$\mu_{\theta c} = 1{,}02$[f]
Coefficient of variation and bias factor of resistance for concrete	$V_{RC} = 0{,}176$[i]	$\mu_{RC} = 1{,}142$[i]
Partial factor for shear and punching γ_V (see 8.2.1, 8.2.2, 8.4, I.8.3.1, I.8.5)		
Compressive strength f_c (control specimen)	$V_{fc} = 0{,}100$	$f_{cm}/f_{ck} = \exp(1{,}645 V_{fc})$[d]
Insitu factor $\eta_{is} = f_{c,ais}/f_c$[e]	$V_{\eta is} = 0{,}120$	$\mu_{\eta is} = 0{,}95$
Effective depth d	$V_d = 0{,}050$[b]	$\mu_d = 0{,}95$[b]
Model uncertainty	$V_{\theta v} = 0{,}107$[g]	$\mu_{\theta v} = 1{,}10$[g]
Residual uncertainties	$V_{res,v} = 0{,}046$[h]	–
Coefficient of variation and bias factor of resistance for shear and punching (members without shear reinforcement)	$V_{RV} = 0{,}137$[i]	$\mu_{RV} = 1{,}085$[i]

Key changes to EN 1992-1-2

Most relevant changes:

- ✓ How to deal with stainless steel
- ✓ Residual resistance of concrete after a fire
- ✓ Deletion of Isotherm 500 method and replacement by a new, more analytical and less conservative method
- ✓ Explicit rules for spalling

Key changes to EN 1992-1-2

Properties of stainless steel reinforcement at elevated temperatures :

(6) For the properties of stainless steel reinforcement at elevated temperatures, EN 1993 1 2 may be applied for simplified design method (Clause 7) and advanced design method (Clause 8). Tabulated design methods are not applicable with stainless steel.

Cooling phase 5.3.1.1 (5):

(5) For thermal actions in accordance with EN 1991-1-2:—[1], 5.3 (Physically based models), when considering the cooling phase, the strength of concrete heated to a maximum temperature $\theta_{c,max}$ and having cooled down to 20 °C may be taken according to Formula (5.15):

$$f_{c,\theta,20\,°C} = \varphi \, f_{ck} \tag{5.15}$$

where for:

— $f_{ck} < 70$ MPa

$$\varphi = f_{c,\theta max}/f_{ck} \quad \text{for } 20\,°C \leq \theta_{max} < 100\,°C \tag{5.16}$$

$$\varphi = (-0{,}0005 \times \theta_{max} + 1{,}05)\,(f_{c,\theta max}/f_{ck}) \quad \text{for } 100\,°C \leq \theta_{max} < 300\,°C \tag{5.17}$$

$$\varphi = 0{,}9\,(f_{c,\theta max}/f_{ck}) \quad \text{for } \theta_{max} \geq 300\,°C \tag{5.18}$$

— $f_{ck} \geq 70$ MPa

$$\varphi = f_{c,\theta max}/f_{ck} \quad \text{for } 20\,°C \leq \theta_{max} < 1\,200\,°C \tag{5.19}$$

The reduction factor $(f_{c,\theta max}/f_{ck})$, which corresponds to the coefficient $(f_{c,\theta}/f_{ck})$ at the maximum temperature $\theta_{c,max}$, should be taken according to Table 5.1.

Key changes to EN 1992-1-2

Analysis levels:

- ✓ Tabulated methods (Section 6)
  - ✓ Method A for columns
  - ✓ Method B for columns
- ✓ Simplified method (section 7)
  - ✓ Simplified verification
  - ✓ Refined verification
- ✓ Refined methods

Analysis levels:

Example of tabulated method:

Limits of minimum width and cover to centre of bar as a function of required fire resistance, and load level

Table 6.1 — Minimum column dimensions and axis distances for columns with rectangular or circular section exposed to fire on four sides

Standard fire resistance	Minimum dimensions (mm) Column width b_{min}/axis distance a of the main reinforcement		
	$\mu_{fi} = 0{,}2$	$\mu_{fi} = 0{,}5$	$\mu_{fi} = 0{,}7$
1	**2**	**3**	**4**
R 30	200/25	200/25	200/32 300/27
R 60	200/25	200/36 300/31	250/46 350/40
R 90	200/31 300/25	300/45 400/38	350/53 450/40[a]
R 120	250/40 350/35	350/45[a] 450/40[a]	350/57[a] 450/51[a]
R 180	350/45[a]	350/63[a]	450/70[a]
R 240	350/61[a]	450/75[a]	–

Simplified method

1. Determination of temperatures

$$\theta_1(x,t) = 345 \cdot \lg_{10}\left(\frac{7(t-\Delta t)}{60}+1\right) \cdot \exp\left(-x\sqrt{\frac{k}{t}}\right)$$

The temperature is used to determine the temperature of the reinforcement and the temperature at the centroid of the cross section or at other point for more refined calculations to determine the capacity of the section

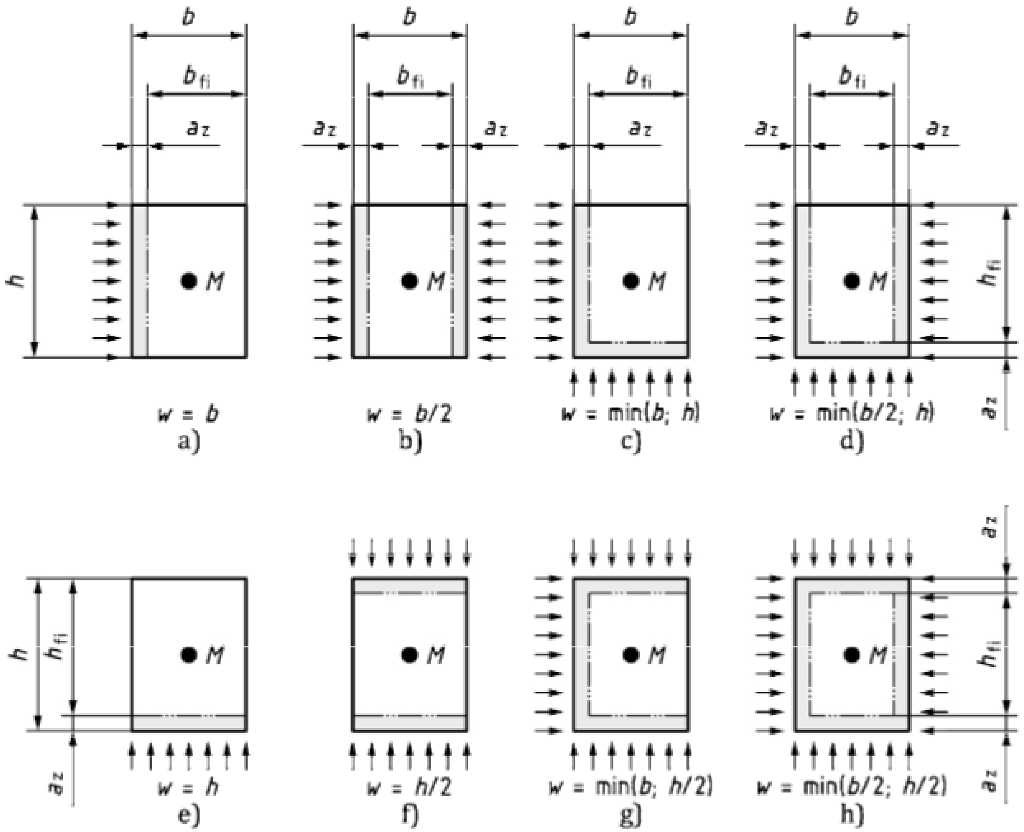
2. Structural analysis

- ✓ Calculation of the reduced cross-section (determination of the parameter a_z). For simple cases it is given directly. For more refined calculations, in the case of bending, the a_z parameter is determined by dividing the section into parallel zones of equal width, while in the case of bending and axial loading, the cross-section of the member should be discretized into a grid of small elemental zones each characterized by area A_{cij}.
- ✓ Verification of the structural behaviour:

  I) Simplified verification, based on the resistance without fire and the reinforcement temperature

  II) Refined verification, accounts for both the reinforcement temperature and concrete temperature at centroid of effective section

Simplified method: Deduction of rim area



$$a_z = \begin{cases} 0{,}011 \cdot \sqrt{1 + \dfrac{t-27}{27} \cdot \sqrt{\dfrac{w}{0{,}0125}}} & \text{for } 0{,}075 \le w < 0{,}20 \\ 0{,}011 \cdot \sqrt{1 + 4\dfrac{t-27}{27}} & \text{for } w \ge 0{,}20 \end{cases} \qquad (7.15)$$

where

t [min] duration of the standard fire

w [m] is a cross-sectional dimension used to obtain the reduced cross-section depending on the fire exposure defined as (see Figure 7.5):

— the member dimension perpendicular to the surface exposed to fire on one side or two non-opposite sides;

— half the member dimension perpendicular to the surface exposed to fire for members exposed to fire on at least two opposite sides; and

Specific rules for spalling

Table 10.1 — Overview of the rules for spalling

Verification for spalling	
R15	Verification of spalling may be omitted except Clause 10(2)
— structures in a water saturated environment — insulating permanent formwork which prevents concrete from drying	Specific assessment of spalling should be undertaken or polypropylene fibres should be specified See Clause 10(7), (8), (9) or (10)
f_{ck} < 70 MPa and silica fume content < 6 % by weight of cement	Verification of spalling may be omitted except Clause 10(3) and (5)
f_{ck} < 70 MPa and silica fume content ≥ 6 % by weight of cement or f_{ck} ≥ 70 MPa	Specific assessment of spalling should be undertaken or polypropylene fibres should be specified See Clause 10(7), (8), (9) or (10)

CONCLUDING REMARKS

EN 1992:2023 is a significant advance with respect to EN 1992:2004

The focus has been on:

- ✓ Improving ease-of-use
- ✓ Implementing consistent mechanical models
- ✓ Reducing Nationally determined parameters
- ✓ Updating the code to the State-of-the-Art
- ✓ Widening the scope to incorporate new materials and building techniques

It has been an enormous effort taking over 10 years of work

It is also the first time that background information is available to help others understand the reasons for all the major changes that were made

I think, as Europeans, we should be proud of what has been achieved: the revision and updating of a consistent set of standard which allows us to design all types of structures with increased convergence.