Design of concrete bridges (EN 1992-2)

Approved by CEN on 25 April 2005
Published on October 2005

Supersedes ENV 1992-2:1996

Prof. Ing. Giuseppe Mancini
Politecnico di Torino

Design of concrete bridges (EN 1992-2)

- EN 1992-2 contains principles and application rules for the design of bridges in addition to those stated in EN 1992-1-1

- Scope: basis for design of bridges in plain/reinforced/prestressed concrete made with normal/light weight aggregates

Section 3 ⇒ MATERIALS

- Recommended values for C_{min} and C_{max}

C_{min} → C30/37 (Durability)

C_{max} → C70/85 (Ductility)

- α_{cc} coefficient for long term effects and unfavourable effects resulting from the way the load is applied

Recommended value: 0.85 → high stress values during construction

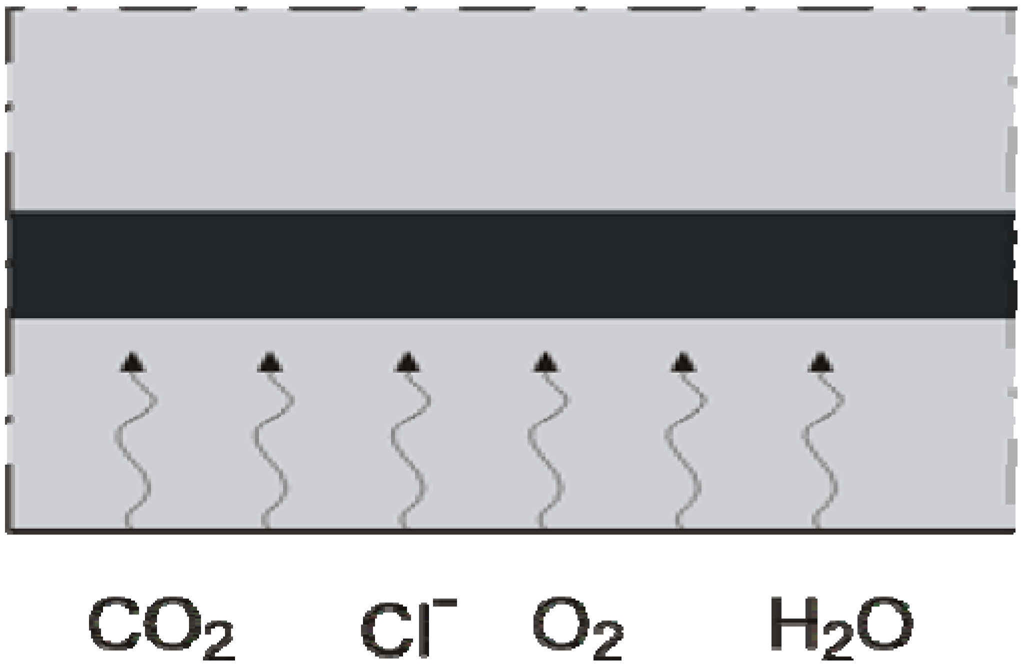
- Recommended classes for reinforcement:

“B” and “C”

(Ductility reduction with corrosion / Ductility for bending and shear mechanisms)

Section 4 ⇒ Durability and cover to reinforcement

Penetration of corrosion stimulating components in concrete



Deterioration of concrete

Corrosion of reinforcement by chloride penetration

Avoiding corrosion of steel in concrete

Design criteria

- Aggressivity of environment
- Specified service life

Design measures

- Sufficient cover thickness
- Sufficiently low permeability of concrete (in combination with cover thickness)
- Avoiding harmfull cracks parallel to reinforcing bars

Aggressivity of the environment

Main exposure classes:

- The exposure classes are defined in EN206-1. The main classes are:

- XO – no risk of corrosion or attack
- XC – risk of carbonation induced corrosion
- XD – risk of chloride-induced corrosion (other than sea water)
- XS – risk of chloride induced corrosion (sea water)
- XF – risk of freeze thaw attack
- XA – chemical attack

Aggressivity of the environment

Further specification of main exposure classes in subclasses (I)

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs

Procedure to determine $c_{min,dur}$

EC-2 leaves the choice of $c_{min,dur}$ to the countries, but gives the following recommendation:

The value $c_{min,dur}$ depends on the “structural class”, which has to be determined first. If the specified service life is 50 years, the structural class is defined as 4. The “structural class” can be modified in case of the following conditions:

- The service life is 100 years instead of 50 years
- The concrete strength is higher than necessary
- Slabs (position of reinforcement not affected by construction process)
- Special quality control measures apply

The finally applying service class can be calculated with Table 4.3N

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Table for determining final Structural Class

Structural Class							
Criterion	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class [1) 2)]	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

Final determination of $c_{min,dur}$ (1)

The value $c_{min,dur}$ is finally determined as a function of the structural class and the exposure class:

Table 4.4N: Values of minimum cover, $c_{min,dur}$, requirements with regard to durability for reinforcement steel in accordance with EN 10080.

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Special considerations

In case of stainless steel the minimum cover may be reduced. The value of the reduction is left to the decision of the countries (0 if no further specification).

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- XC3 class recommended for surface protected by waterproofing

- When de-icing salt is used

Exposed concrete surfaces within (6 m) of the carriage way and supports under expansion joints: directly affected by de-icing salt

Recommended classes for surfaces directly affectd by de-icing salt: XD3 – XF2 – XF4, with covers given in tables 4.4N and 4.5N for XD classes

Section 5 ⇒ Structural analysis

- Linear elastic analysis with limited redistributions

Limitation of δ due to uncertaintes on size effect and bending-shear interaction

$\delta \geq 0.85$ (recommended value)

- Plastic analysis

Restrictions due to uncertaintes on size effect and bending-shear interaction:

$$\frac{x_u}{d} \leq \begin{cases} 0.15 \text{ for concrete strength classes } \leq \text{ C50/60} \\ 0.10 \text{ for concrete strength classes } \geq \text{ C55/67} \end{cases}$$

- Rotation capacity

Restrictions due to uncertaintes on size effect and bending-shear interaction:

in plastic hinges

$$\frac{x_u}{d} \leq \begin{cases} 0.30 \text{ for concrete strength classes } \leq \text{ C50/60} \\ 0.23 \text{ for concrete strength classes } \geq \text{ C55/67} \end{cases}$$

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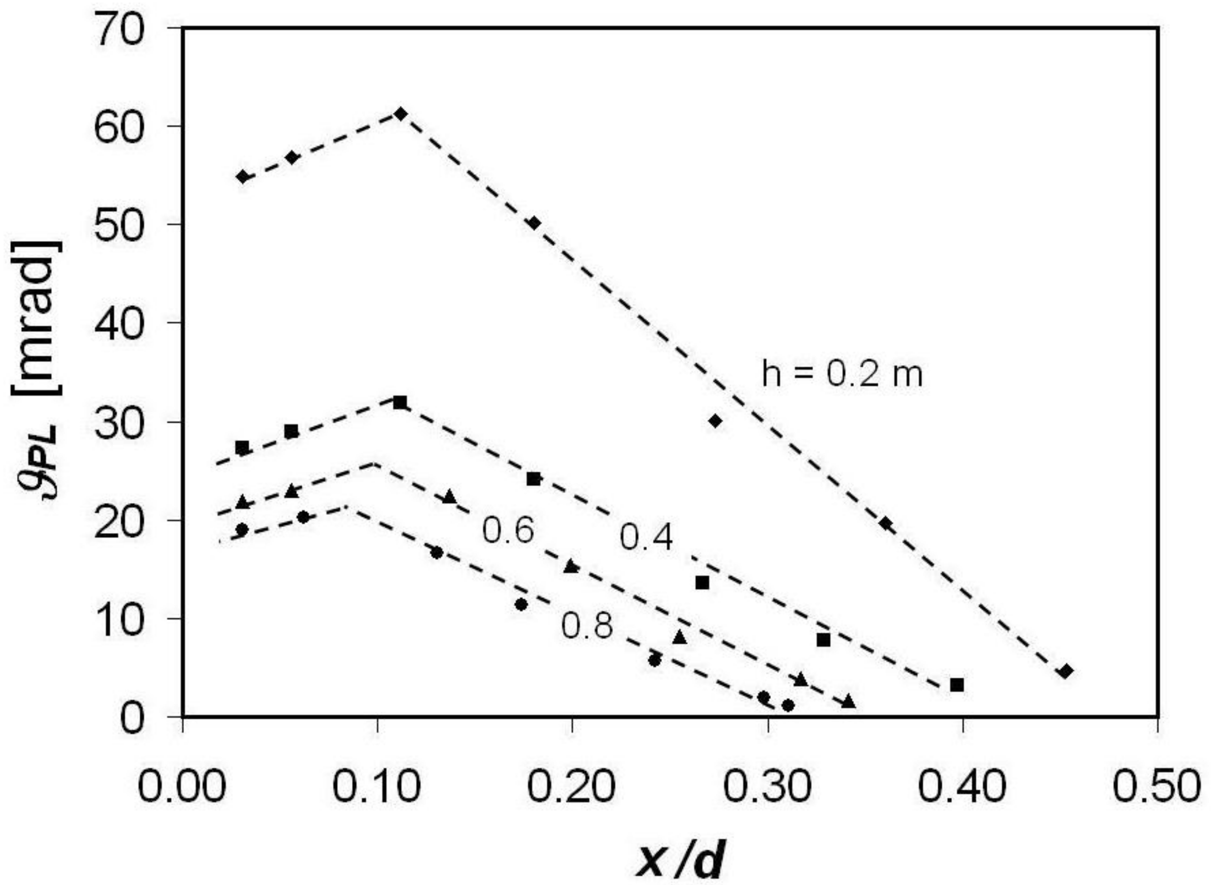
Numerical rotation capacity



- Nonlinear analysis ⇒ Safety format

Reinforcing steel

Mean values

$1.1\ f_{yk}$ $1.1\ k\ f_{yk}$

Prestressing steel

Mean values

$1.1\ f_{pk}$

Concrete

Sargin modified mean values

$\gamma_{cf}\, f_{ck}$

$\gamma_{cf} = 1.1\ \gamma_s / \gamma_c$

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- Design format

  - Incremental analysis from SLS, so to reach $\gamma_G G_k + \gamma_Q Q$ in the same step

  - Continuation of incremental procedure up to the peak strength of the structure, in corrispondance of ultimate load q_{ud}

  - Evaluation of structural strength by use of a global safety factor γ_0

$$R\left(\frac{q_{ud}}{\gamma_0}\right)$$

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- Verification of one of the following inequalities

$$\gamma_{Rd} E\left(\gamma_G G + \gamma_Q Q\right) \leq R\left(\frac{q_{ud}}{\gamma_O}\right)$$

$$E\left(\gamma_G G + \gamma_Q Q\right) \leq R\left(\frac{q_{ud}}{\gamma_{Rd} \cdot \gamma_O}\right)$$

(i.e.) $R\left(\frac{q_{ud}}{\gamma_{O'}}\right)$

$$\gamma_{Rd} \gamma_{Sd} E\left(\gamma_g G + \gamma_q Q\right) \leq R\left(\frac{q_{ud}}{\gamma_O}\right)$$

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With

γ_{Rd} = 1.06 partial factor for model uncertainties (resistence side)

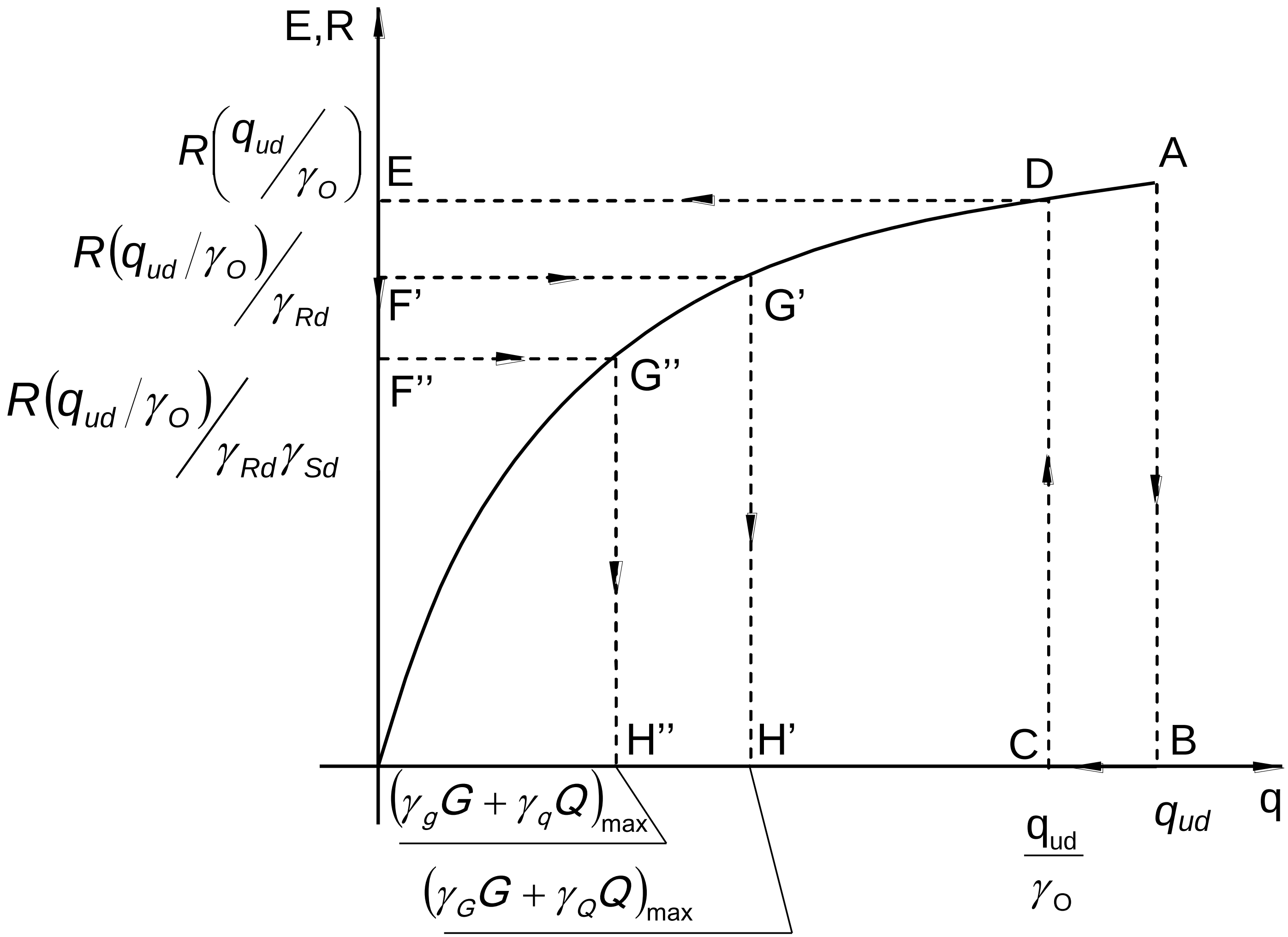
γ_{Sd} = 1.15 partial factor for model uncertainties (actions side)

γ_0 = 1.20 structural safety factor

If $\gamma_{Rd} = 1.00$ then $\gamma_{0'} = 1.27$ is the structural safety factor

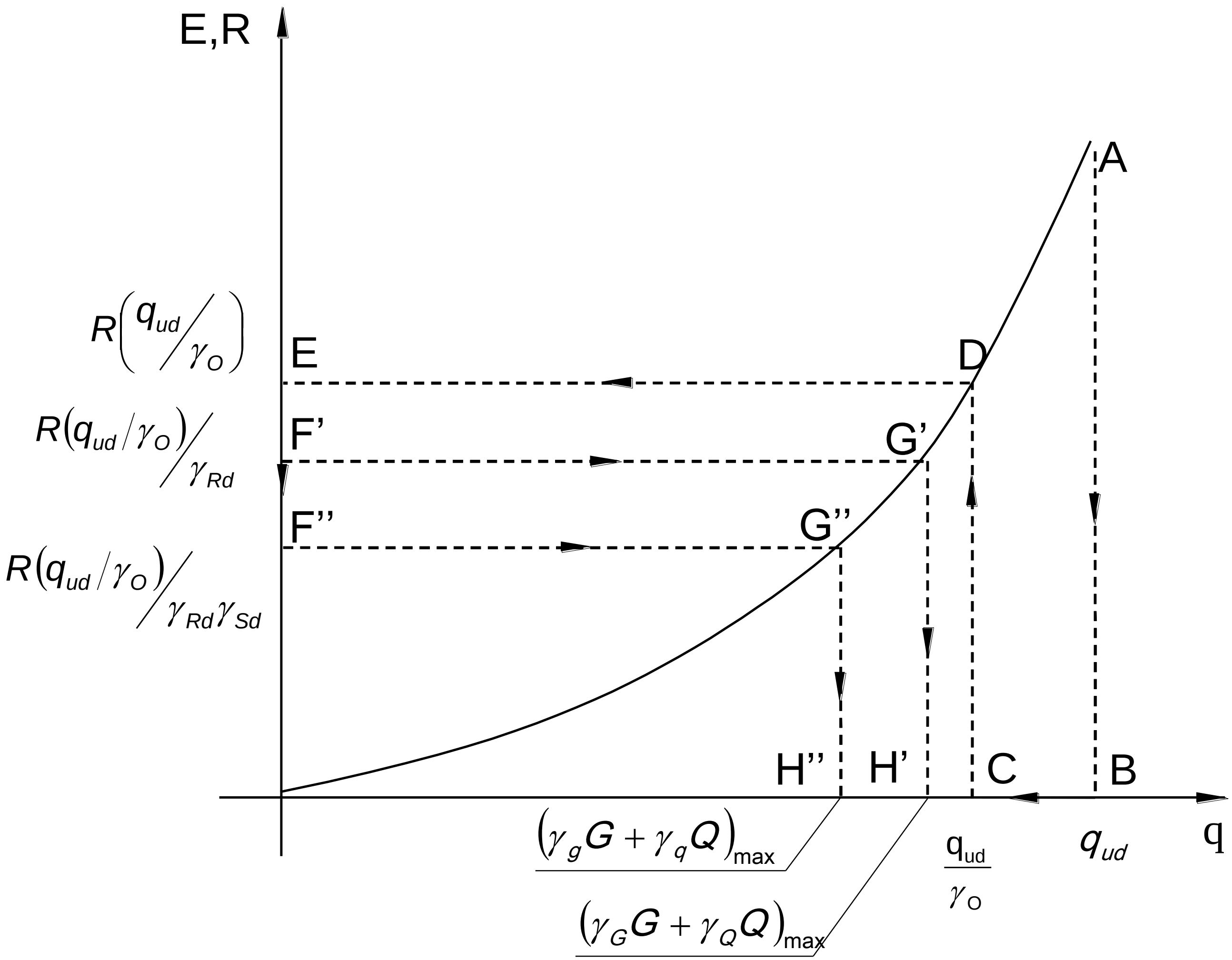
- Safety format

Application for scalar combination of internal actions and underproportional structural behaviour



- Safety format

Application for scalar combination of internal actions and overproportional structural behaviour



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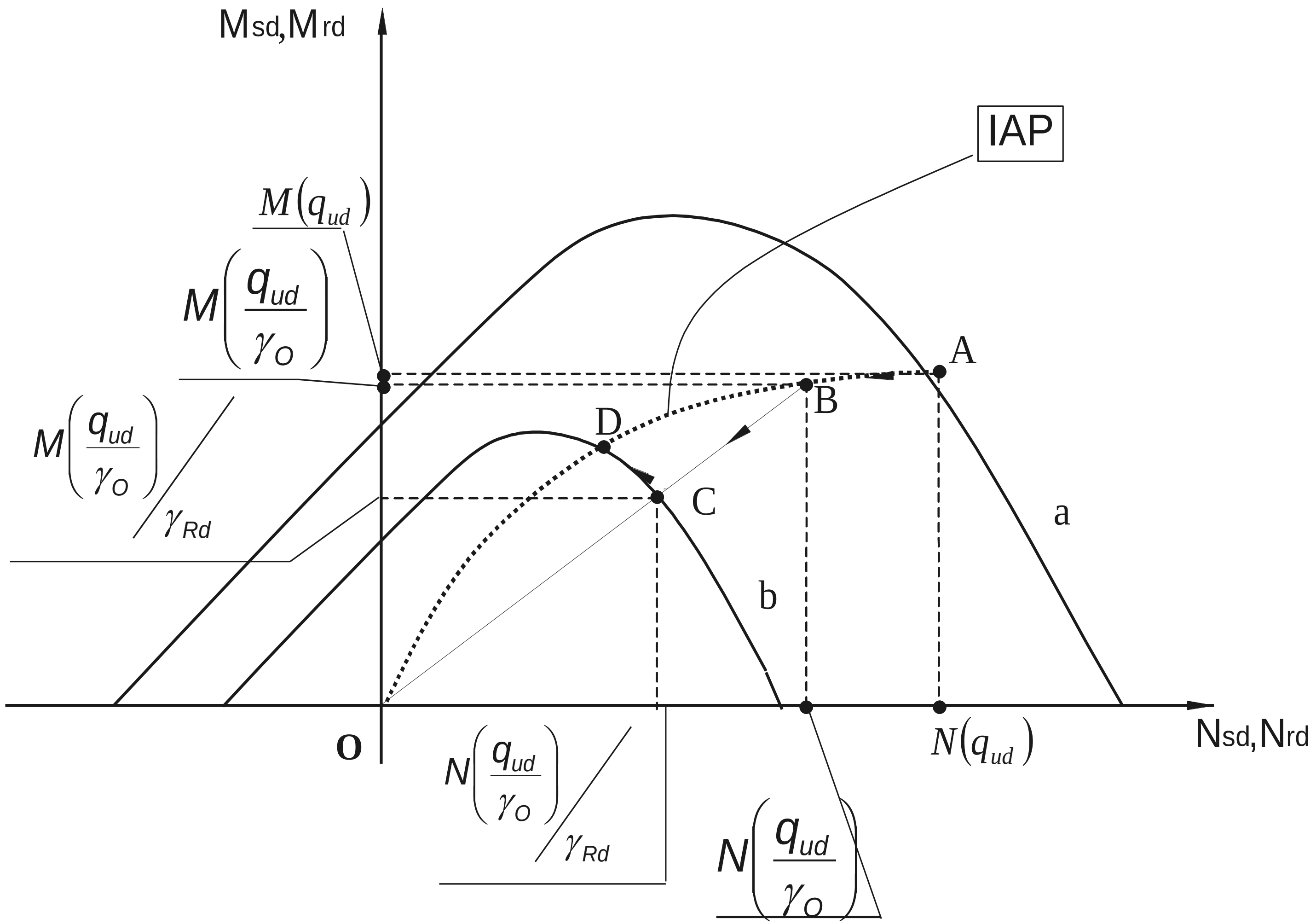
- Safety format

Application for vectorial combination of internal actions and underproportional structural behaviour



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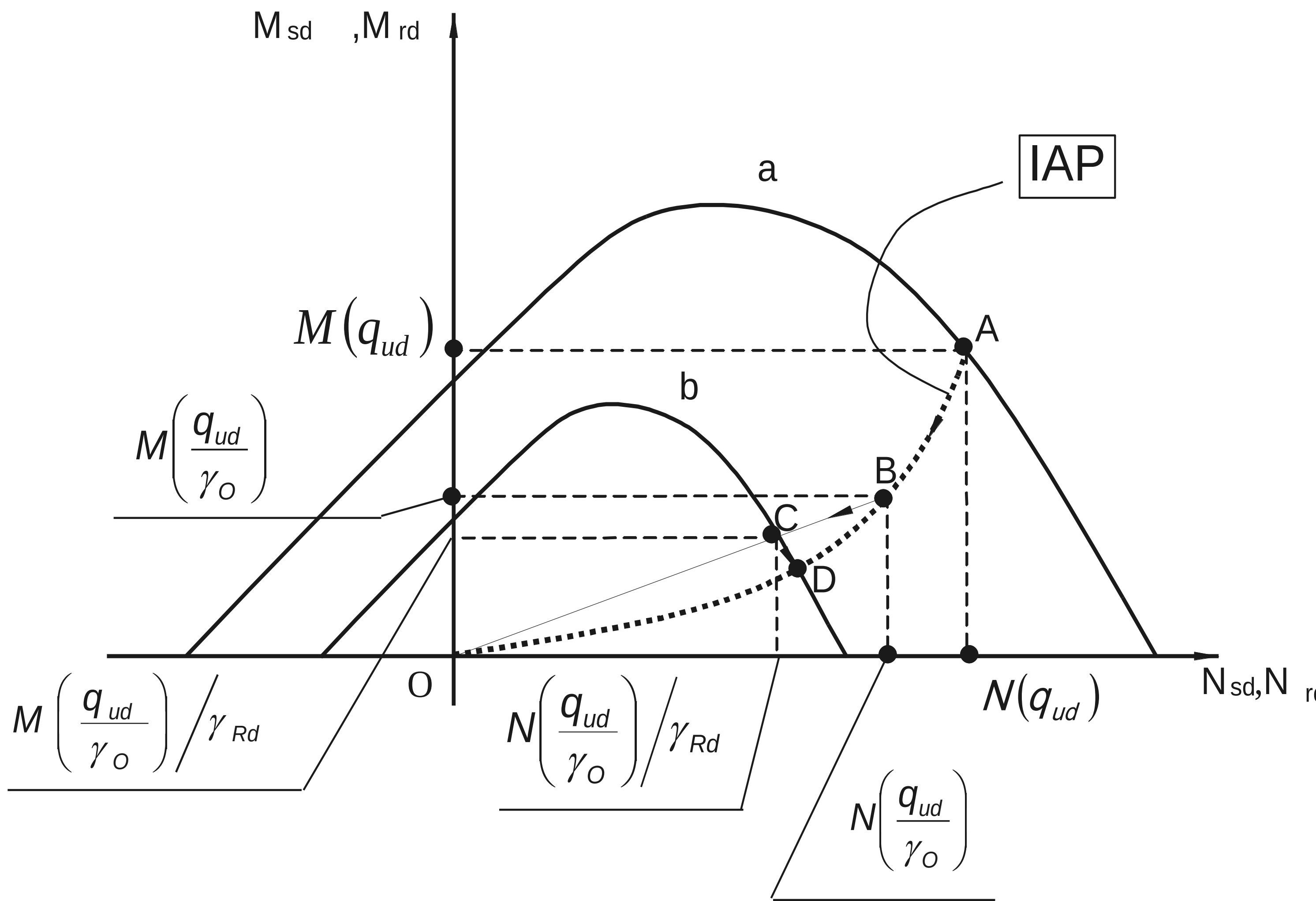
- Safety format

Application for vectorial combination of internal actions and overproportional structural behaviour



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For vectorial combination and $\gamma_{Rd} = \gamma_{Sd} = 1.00$ the safety check is satisfied if:

$$M_{ED} \leq M_{Rd}\left(\frac{q_{ud}}{\gamma_{0'}}\right)$$

and

$$N_{ED} \leq N_{Rd}\left(\frac{q_{ud}}{\gamma_{0'}}\right)$$

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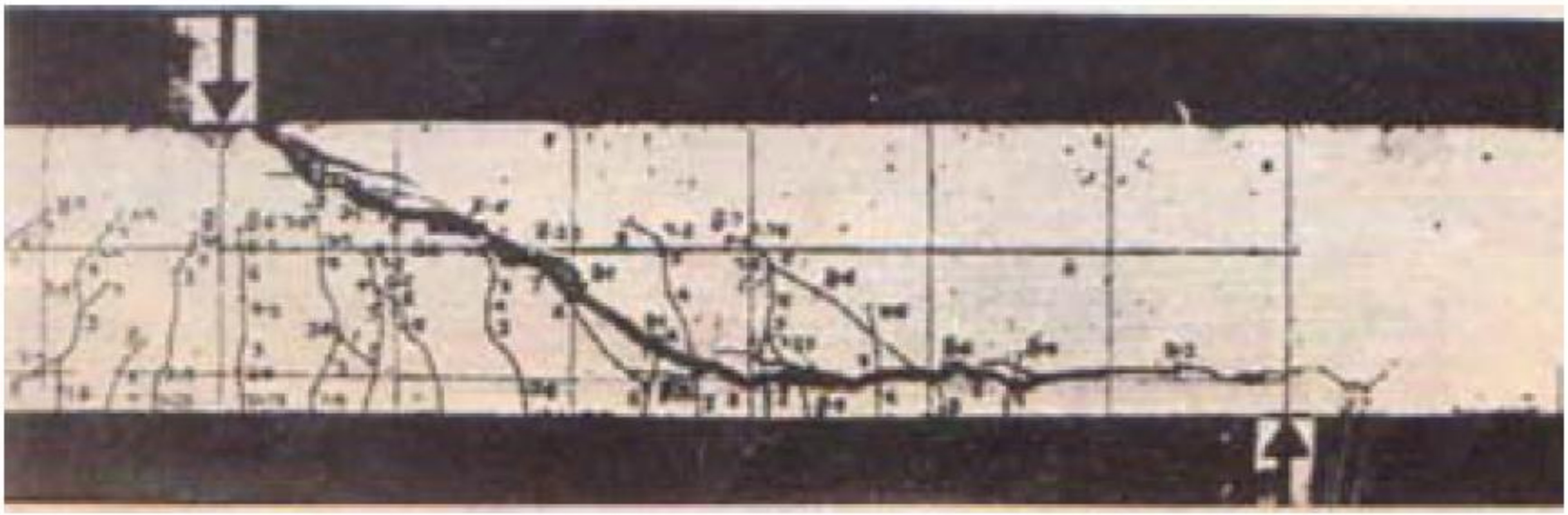
Concrete slabs without shear reinforcement



Shear resistance $V_{Rd,c}$ governed by shear flexure failure:
shear crack develops from flexural crack

Concrete slabs without shear reinforcement

Prestressed hollow core slab

Shear resistance $V_{Rd,c}$ governed by shear tension failure:
crack occurs in web in region uncracked in flexure

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Shear design value under which no shear reinforcement is necessary in elements unreinforced in shear (general limit)

$$V_{Rd,c} = C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} b_w d$$

$C_{Rd,c}$	coefficient derived from tests (recommended 0.12)
k	size factor = 1 + $\sqrt{(200/d)}$ with d in meter
ρ_l	longitudinal reinforcement ratio ($\leq$0,02)
f_{ck}	characteristic concrete compressive strength
b_w	smallest web width
d	effective height of cross section

Shear design value under which no shear reinforcement is necessary in elements unreinforced in shear (general limit)

Minimum value for $V_{Rd,c}$

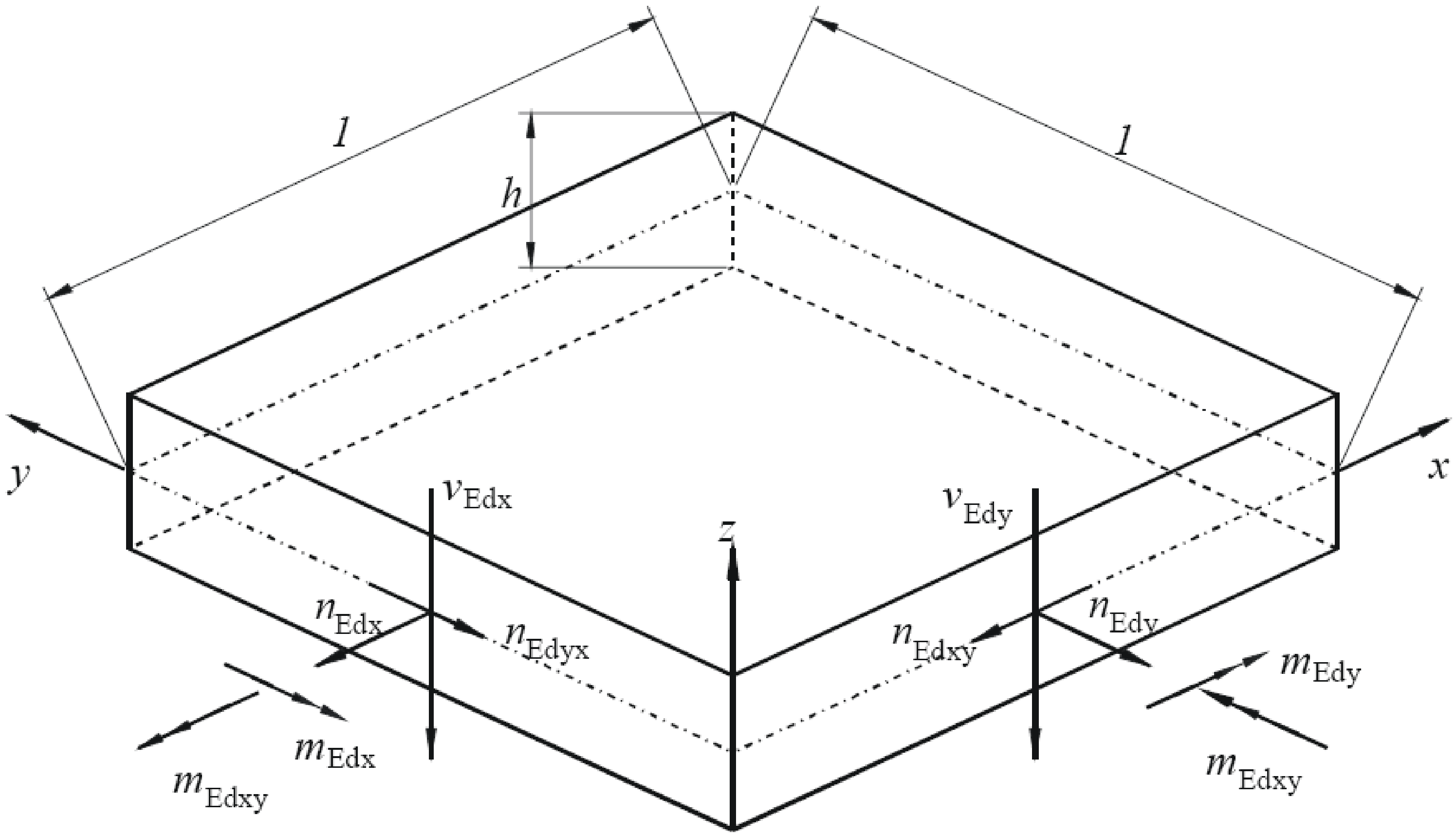
$$V_{Rd,c} = v_{min}\, b_w\, d$$

Values for v_{min} (N/mm^2)

	d=200	d=400	d=600	d=800
C20	0,44	0,35	0,25	0,29
C40	0,63	0,49	0,44	0,41
C60	0,77	0,61	0,54	0,50
C80	0,89	0,70	0,62	0,58

Annex LL ⇒ Concrete shell elements

A powerfull tool to design 2D elements



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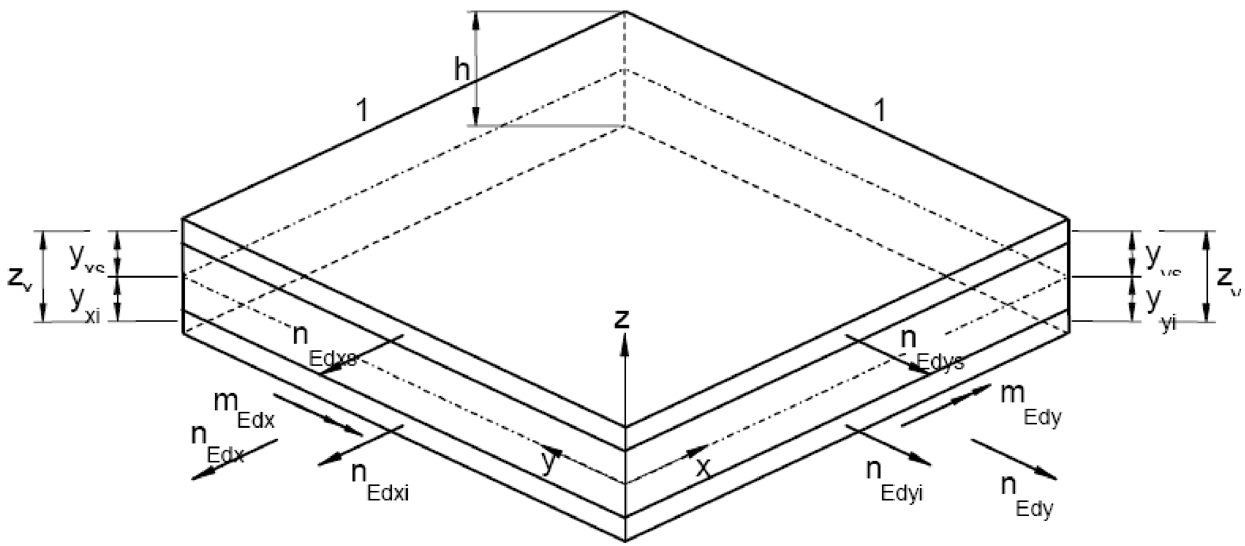
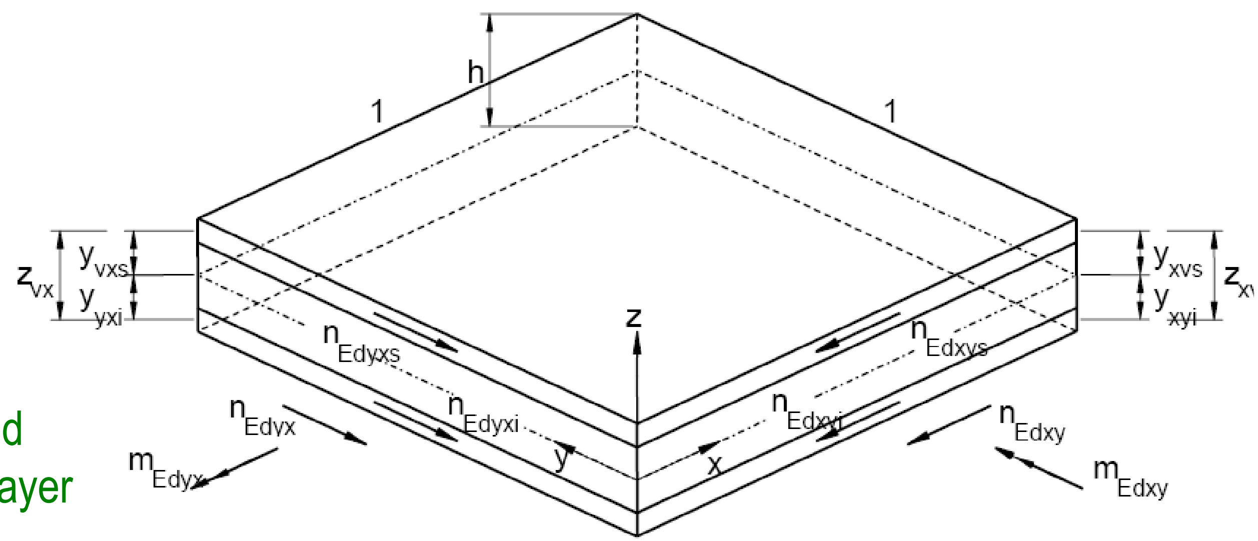


Axial actions and bending moments in the outer layer



Membrane shear actions and twisting moments in the outer layer

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- Out of plane shear forces v_{Edx} and v_{Edy} are applied to the inner layer with lever arm z_c, determined with reference to the centroid of the appropriate layers of reinforcement.

- For the design of the inner layer the principal shear v_{Edo} and its direction φ_o should be evaluated as follows:

$$v_{\mathrm{Edo}} = \sqrt{v_{\mathrm{Edx}}^{2} + v_{\mathrm{Edy}}^{2}}$$

$$\tan \varphi_o = \frac{v_{\mathrm{Edy}}}{v_{\mathrm{Edx}}}$$

- In the direction of principal shear the shell element behaves like a beam and the appropriate design rules should therefore be applied.

$$\rho_l = \rho_x \cos^2 \varphi_o + \rho_y \sin^2 \varphi_o$$

- When shear reinforcement is necessary, the longitudinal force resulting from the truss model $V_{Edo} \cdot \cot\theta$ gives rise to the following membrane forces in *x* and *y* directions:

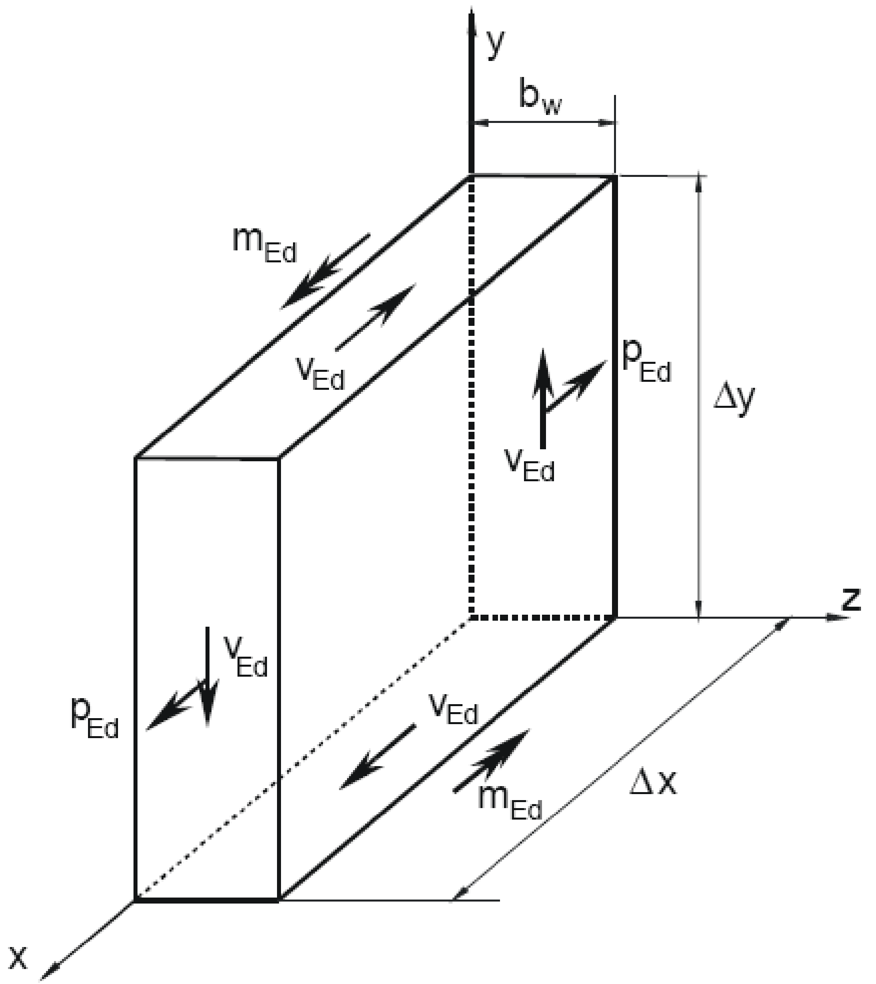
$$n_{\mathrm{Edyc}} = \frac{{v_{\mathrm{Edy}}}^2}{v_{\mathrm{Edo}}} \cot\theta \qquad n_{\mathrm{Edxc}} = \frac{{v_{\mathrm{Edx}}}^2}{v_{\mathrm{Edo}}} \cot\theta$$

$$n_{\mathrm{Edxyc}} = \frac{v_{\mathrm{Edx}}\, v_{\mathrm{Edy}}}{v_{\mathrm{Edo}}} \cot\theta \qquad n_{\mathrm{Edyxc}} = n_{\mathrm{Edxyc}} = \frac{v_{\mathrm{Edx}}\, v_{\mathrm{Edy}}}{v_{\mathrm{Edo}}} \cot\theta$$

- The outer layers should be designed as membrane elements, using the design rules of clause 6 (109) and Annex F.

Annex MM ⇒ Shear and transverse bending

Webs of box girder bridges



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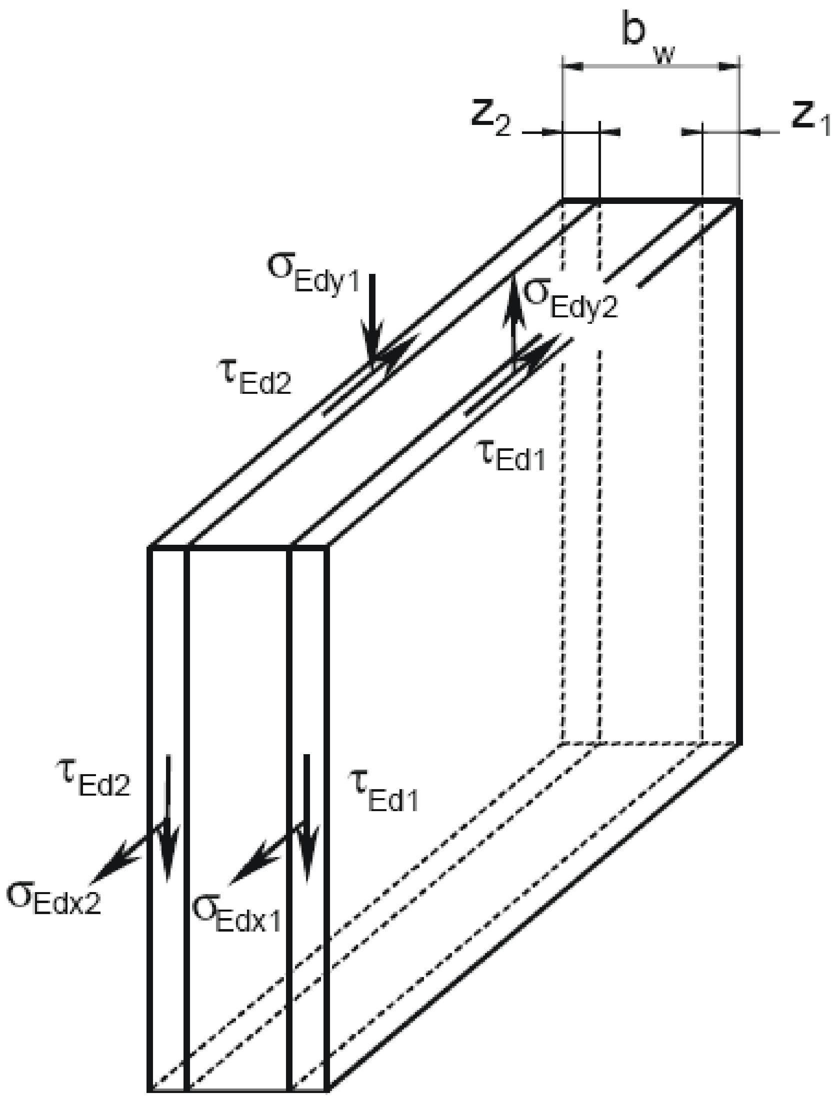

Modified sandwich model

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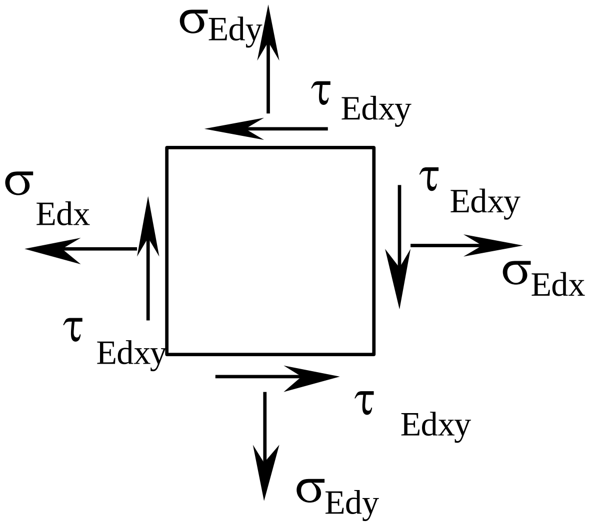
- Membrane elements



- Compressive stress field strength defined as a function of principal stresses
- If both principal stresses are comprensive

$$\sigma_{cd\,\max} = 0.85 f_{cd} \frac{1+3,80\alpha}{(1+\alpha)^2}$$

is the ratio between the two principal stresses ($\alpha \leq 1$)

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- Where a plastic analysis has been carried out with $\theta = \theta_{el}$ and at least one principal stress is in tension and no reinforcement yields

$$\sigma_{cd\,\max} = f_{cd}\left[0{,}85 - \frac{\sigma_s}{f_{yd}}\left(0{,}85 - \nu\right)\right]$$

σ_s is the maximum tensile stress value in the reinforcement

- Where a plastic analysis is carried out with yielding of any reinforcement

$$\sigma_{cd\,\max} = \nu\ f_{cd}\left(1 - 0{,}032\left|\theta - \theta_{el}\right|\right)$$

θ is the angle to the X axis of plastic compression field at ULS (principal compressive stress)

$\left|\theta - \theta_{el}\right| \le 15$ degrees

θ_{el} is the inclination to the X axis of principal compressive stress in the elastic analysis

Model by Carbone, Giordano, Mancini

Assumption: strength of concrete subjected to biaxial stresses is correlated to the angular deviation between angle ϑ_{el} which identifies the principal compressive stresses in incipient cracking and angle ϑ_u which identifies the inclination of compression stress field in concrete at ULS

⇓

With increasing $\Delta\vartheta$ concrete damage increases progressively and strength is reduced accordingly

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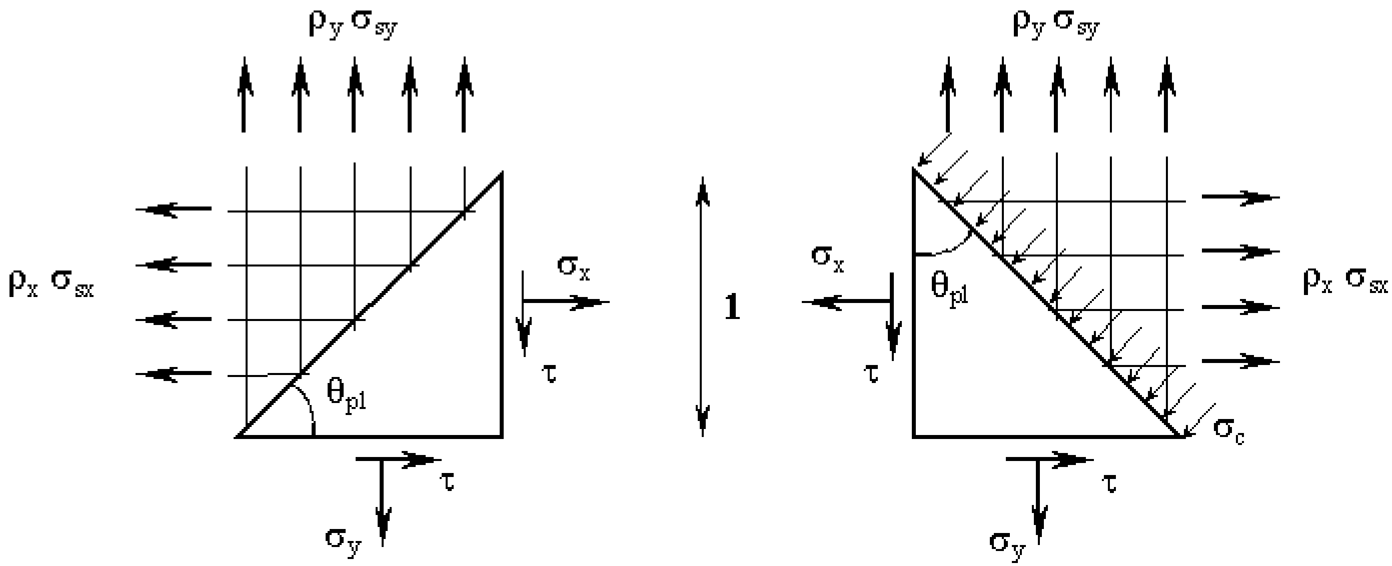
Plastic equilibrium condition



$$\sigma_x + \tau \cot \vartheta_{pl} - \sigma_{sx}\,\rho_x = 0$$

$$\tau + \sigma_x \cot \vartheta_{pl} - \sigma_{sy}\,\rho_y \cot \vartheta_{pl} = 0$$

$$\tau \tan \vartheta_{pl} - \sigma_x + \sigma_{sx}\,\rho_x - \sigma_c = 0$$

$$\tau - \sigma_y \tan \vartheta_{pl} + \sigma_{sy}\,\rho_y \tan \vartheta_{pl} - \sigma_c \tan \vartheta_{pl} = 0$$

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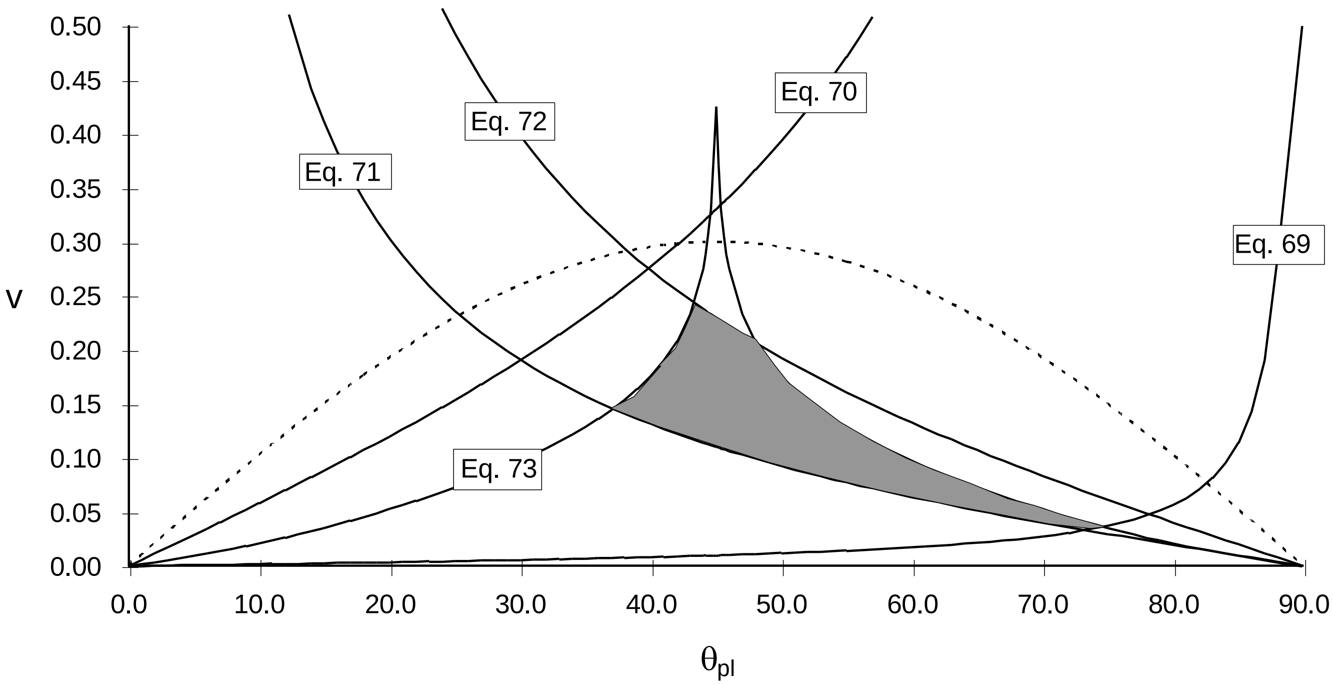
Graphical solution of inequalities system



$\omega_x = 0.16$

$\omega_y = 0.06$

$n_x = n_y = -0.17$

$\vartheta_{el} = 45°$

$$\frac{\tau}{|f_c'|}\left(\tan\vartheta_{pl} + \cot\vartheta_{pl}\right) - \left[0.55 - 0.12\ln\left|\vartheta_{pl} - \vartheta_{el}\right|\right] = 0$$

$$v \geq -(\omega_x + n_x)\tan\vartheta_{pl} \quad (69)$$

$$v \leq (\omega_x - n_x)\tan\vartheta_{pl} \quad (70)$$

$$v \geq (-\omega_y + n_y)\cot\vartheta_{pl} \quad (71)$$

$$v \leq (\omega_y - n_y)\cot\vartheta_{pl} \quad (72)$$

$$v \leq \nu\sin\vartheta_{pl}\cos\vartheta_{pl} \quad (73)$$

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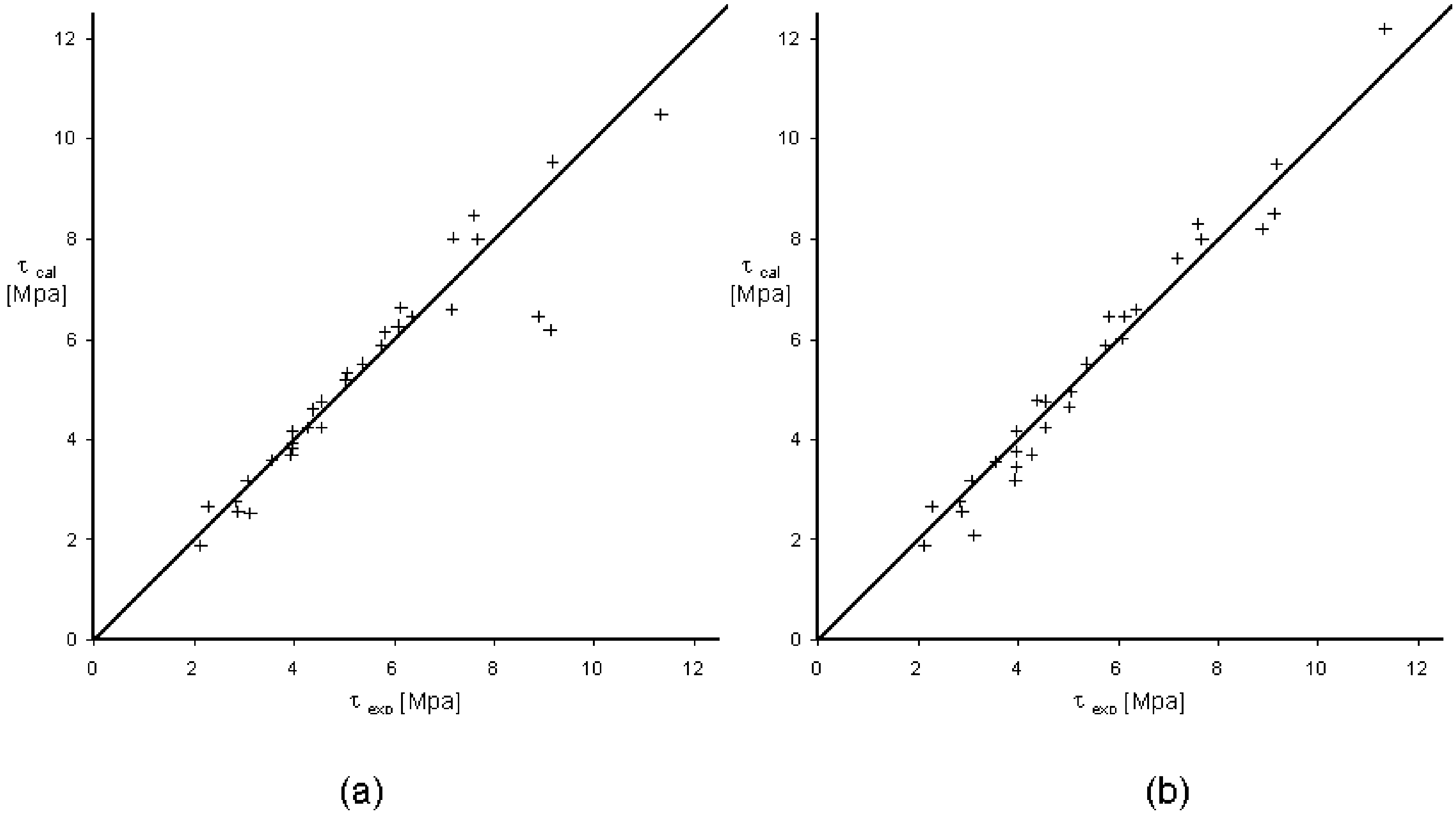

Experimental versus calculated panel strenght by Marti and Kaufmann (a) and by Carbone, Giordano and Mancini (b)

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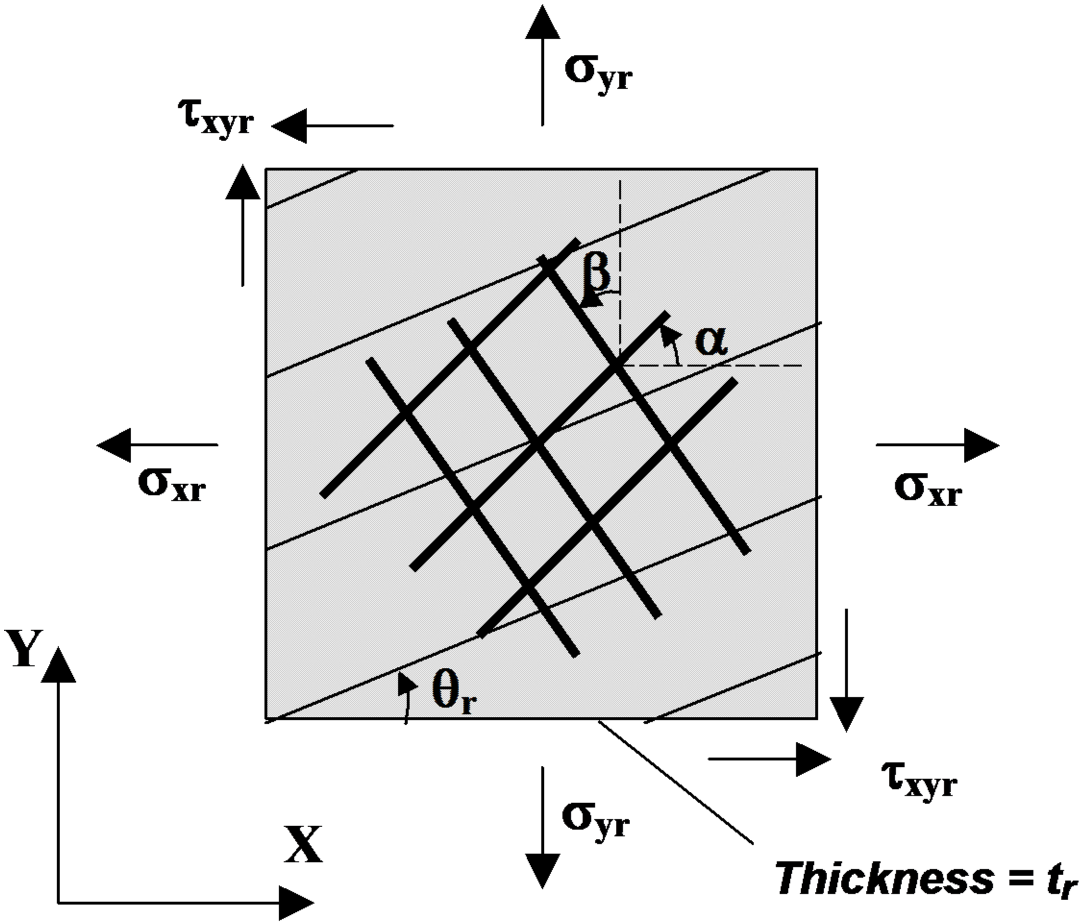
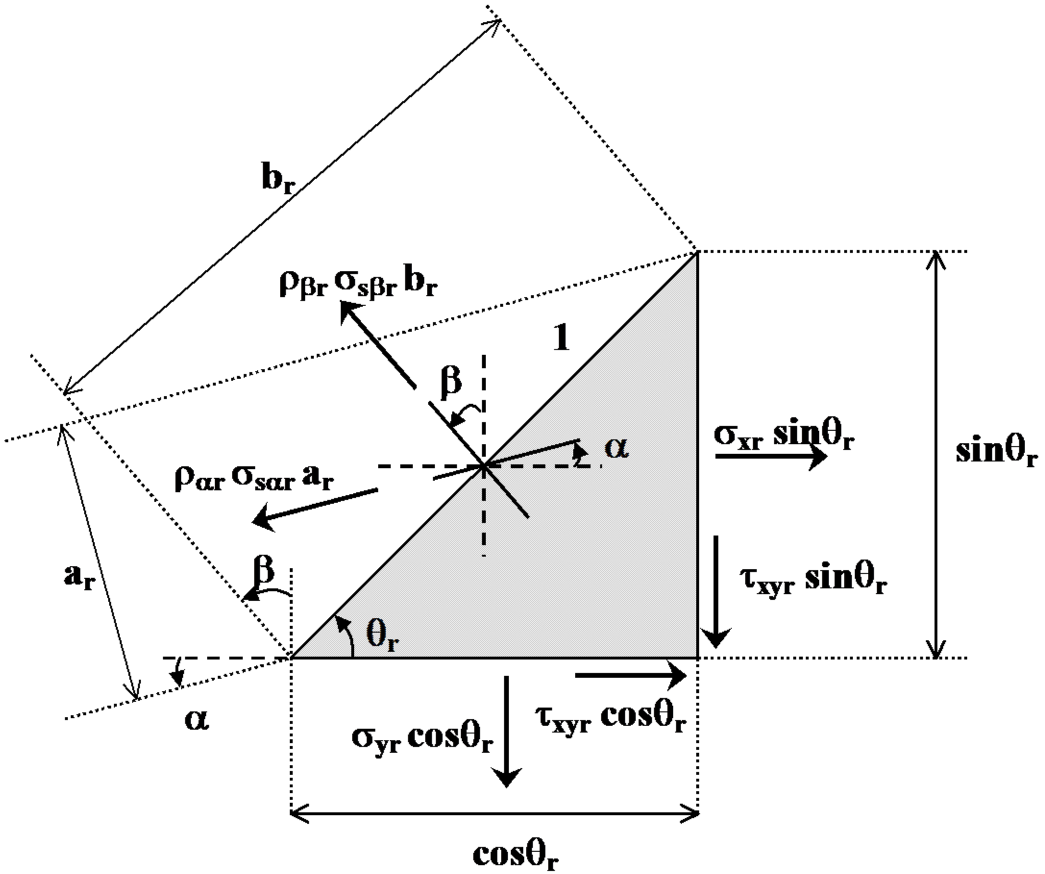
Skew reinforcement



Plates conventions



Equilibrium of the section parallel to the compression field

$$\rho_{\alpha r}\sigma_{s\alpha r} = \frac{\sigma_{xr}\sin\theta_r\cos\beta - \sigma_{yr}\cos\theta_r\sin\beta + \tau_{xyr}\cos(\theta_r + \beta)}{\sin(\theta_r - \alpha)\cos(\alpha - \beta)}$$

$$\rho_{\beta r}\sigma_{s\beta r} = \frac{\sigma_{xr}\sin\theta_r\sin\alpha + \sigma_{yr}\cos\theta_r\cos\alpha + \tau_{xyr}\sin(\theta_r + \alpha)}{\cos(\theta_r - \beta)\cos(\alpha - \beta)}$$

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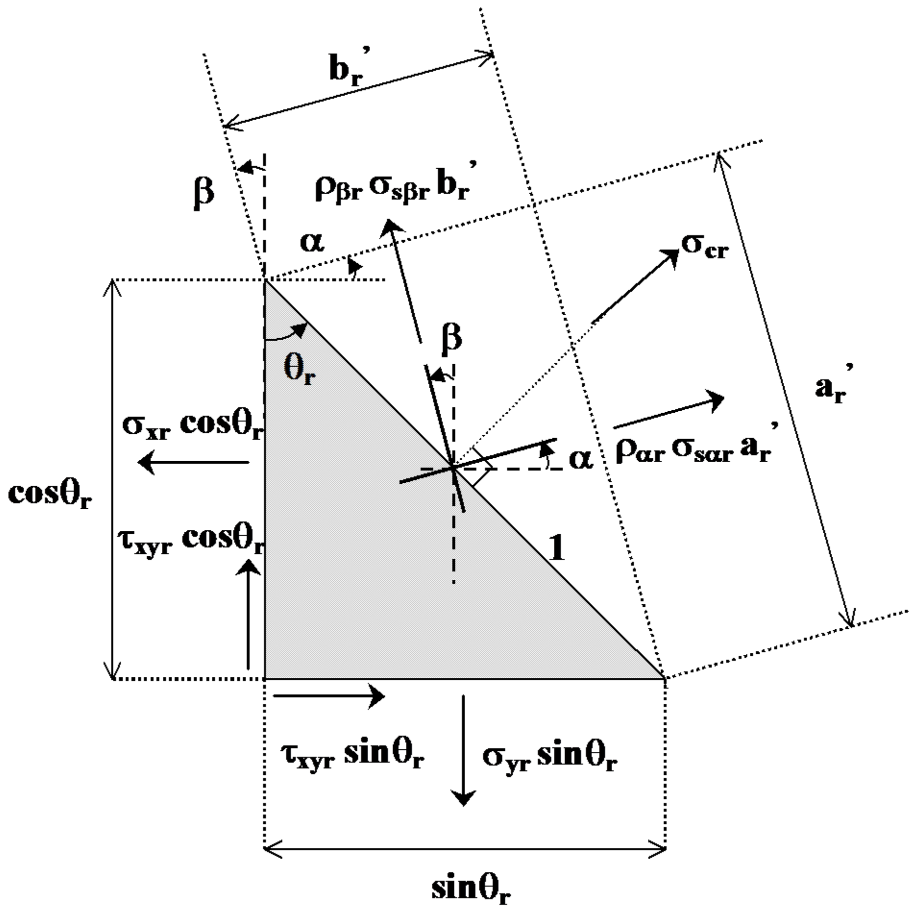

Equilibrium of the section orthogonal to the compression field

$$-\sigma_{xr}\cos\theta_r + \tau_{xyr}\sin\theta_r + \rho_{\alpha r}\sigma_{s\alpha r}a'_r\cos\alpha - \rho_{\beta r}\sigma_{s\beta r}b'_r\sin\beta + \sigma_{cr}\cos\theta_r = 0$$

$$-\sigma_{yr}\sin\theta_r + \tau_{xyr}\cos\theta_r + \rho_{\alpha r}\sigma_{s\alpha r}a'_r\sin\alpha - \rho_{\beta r}\sigma_{s\beta r}b'_r\cos\beta + \sigma_{cr}\sin\theta_r = 0$$

Use of genetic algorithms (Genecop III) for the optimization of reinforcement and concrete verification

Objective: minimization of global reinforcement

Stability: find correct results also if the starting point is very far from the actual solution

Section 7 ⇒ Serviceability limit state (SLS)

- Compressive stresses limited to $k_1 f_{ck}$ with exposure classes XD, XF, XS (Microcracking)

$k_1 = 0.6$ (reccommended value)

$k_1 = 0.66$ in confined concrete (reccommended value)

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- Crack control

Exposure Class	Reinforced members and prestressed members with unbonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
X0, XC1	0,3[1]	0,2
XC2, XC3, XC4	0,3	0,2[2]
XD1, XD2, XD3 XS1, XS2, XS3		Decompression

Note 1: For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed.

Note 2: For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.

Decompression requires that concrete is in compression within a distance of 100 mm (reccommended value) from bondend tendons

For skew cracks where a more refined model is not available, the following expression for the may be used:

$$s_{rm} = \left(\frac{\cos\theta}{s_{rm,x}} + \frac{\sin\theta}{s_{rm,y}} \right)^{-1}$$

where $s_{rm,x}$ and $s_{rm,y}$ are the mean spacing between the cracks in two ideal ties arranged in the x and y directions. The mean opening of cracks can than evaluated as:

$$w_m = s_{rm}(\varepsilon_\perp - \varepsilon_{c,\perp})$$

where $\varepsilon_\perp$ and $\varepsilon_{c,\perp}$ represent the total mean strain and the mean concrete strain, evaluated in the direction orthogonal to the crack

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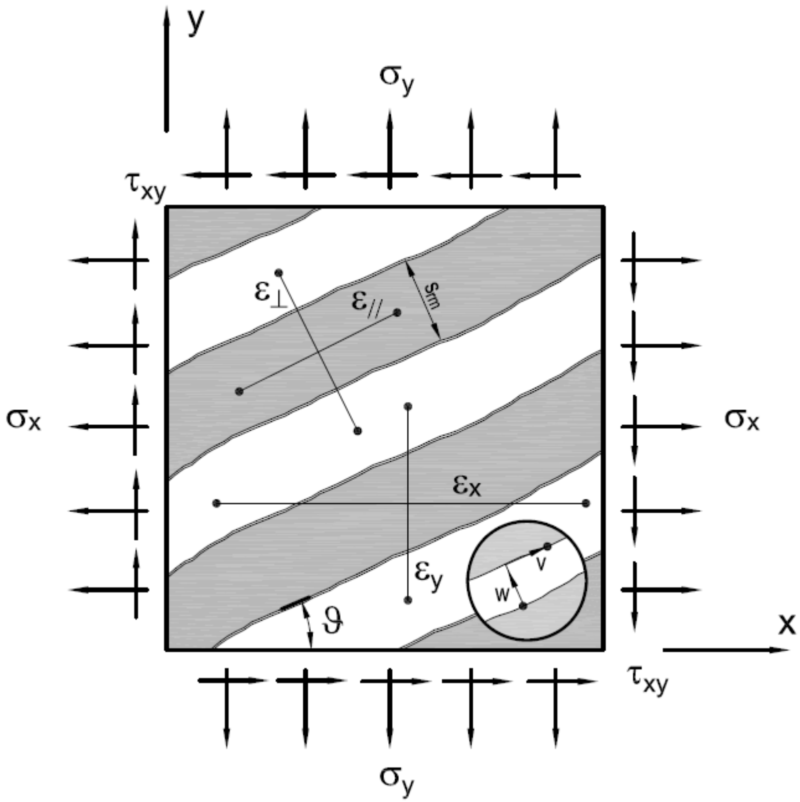

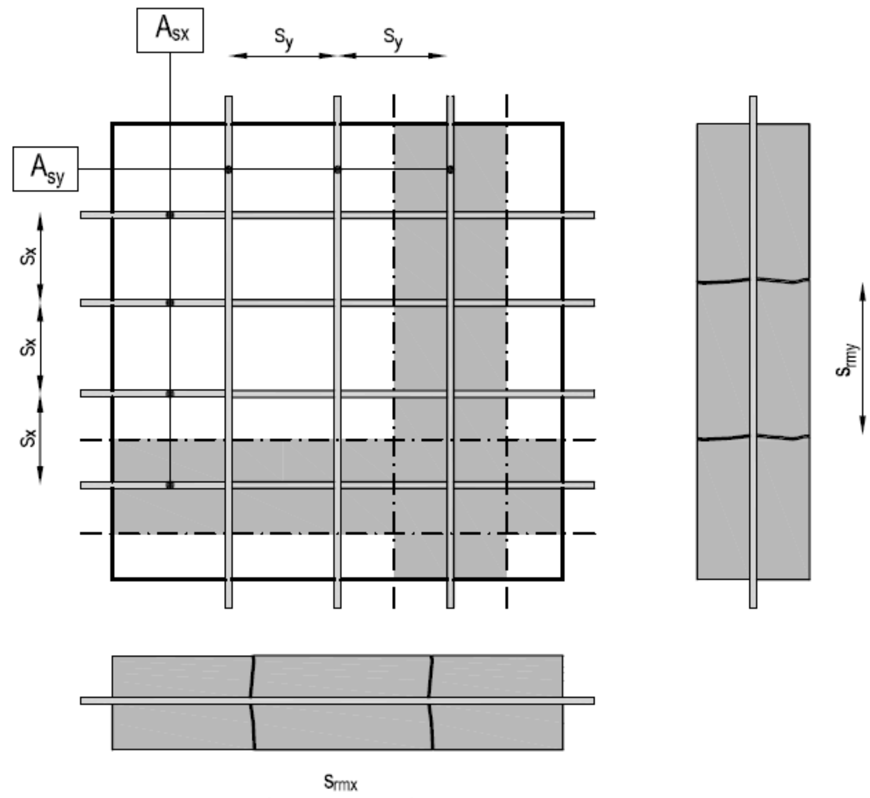

Expressing the compatibility of displacement along the crack, the total strain and the corresponding stresses in reinforcement in x and y directions may be evaluated, as a function of the displacements components w and v, respectively orthogonal and parallel to the crack direction.

Moreover, by the effect of w and v, tangential and orthogonal forces along the crack take place, that can be evaluated by the use of a proper model able to describe the interlock effect.

Finally, by imposition of equilibrium conditions between internal actions and forces along the crack, a nonlinear system of two equations in the unknowns w and v may be derived, from which those variables can be evaluated.

Annex B ⇒ Creep and shrinkage strain

- HPC, class R cement, strength ≥ 50/60 MPa with or without silica fume
- Thick members → kinetic of basic creep and drying creep is different
- Distiction between
  - ***Autogenous shrinkage:*** *related to process of hydration*
  - ***Drying shrinkage:*** *related to humidity exchanges*
- Specific formulae for SFC (content > 5% of cement by weight)

- Autogenous shrinkage

- For t < 28 days $f_{ctm}(t) / f_{ck}$ is the main variable

$$\frac{f_{cm}(t)}{f_{ck}} < 0.1 \qquad \varepsilon_{ca}(t, f_{ck}) = 0$$

$$\frac{f_{cm}(t)}{f_{ck}} \geq 0.1 \qquad \varepsilon_{ca}(t, f_{ck}) = (f_{ck} - 20)\left(2.2\frac{f_{cm}(t)}{f_{ck}} - 0.2\right)10^{-6}$$

- For t ≥ 28 days

$$\varepsilon_{ca}(t,f_{ck}) = (f_{ck} - 20)\left[2.8 - 1.1\exp(-t/96)\right]10^{-6}$$

97% of total autogenous shrinkage occurs within 3 mounths

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- Drying shrinkage (RH ≤ 80%)

$$\varepsilon_{cd}(t, t_s, f_{ck}, h_0, RH) = \frac{\mathrm{K}(f_{ck})\left[72\exp(-0.046 f_{ck}) + 75 - RH\right](t - t_s)10^{-6}}{(t - t_s) + \beta_{cd}\, h_0^{\,2}}$$

with: $K(f_{ck}) = 18$ if $f_{ck} \leq 55$ MPa

$K(f_{ck}) = 30 - 0.21 f_{ck}$ if $f_{ck} > 55$ MPa

$$\beta_{cd} = \begin{cases} 0.007 & \text{for silica-fume concrete} \\ 0.021 & \text{for nonsilica-fume concrete} \end{cases}$$

- Creep

$$\varepsilon_{cc}(t,t_0) = \frac{\sigma(t_0)}{E_{c28}} \left[\Phi_b(t,t_0) + \Phi_d(t,t_0) \right]$$

Basic creep

Drying creep

- Basic creep

$$\Phi_b\left(t,t_0,f_{ck},f_{cm}(t_0)\right)=\phi_{b0}\,\frac{\sqrt{t-t_0}}{\left[\sqrt{t-t_0}+\beta_{bc}\right]}$$

with:

$$\phi_{b0}=\begin{cases}\dfrac{3.6}{f_{cm}(t_0)^{0,37}} & \text{for silica-fume concrete}\\[2ex] 1.4 & \text{for non silica-fume concrete}\end{cases}$$

$$\beta_{bc}=\begin{cases}0.37\exp\left(2.8\dfrac{f_{cm}(t_0)}{f_{ck}}\right) & \text{for silica-fume concrete}\\[2ex] 0.4\exp\left(3.1\dfrac{f_{cm}(t_0)}{f_{ck}}\right) & \text{for non silica-fume concrete}\end{cases}$$

- Drying creep

$$\Phi_d(t,t_s,t_0,f_{ck},RH,h_0) = \phi_{d0}\left[\varepsilon_{cd}(t,t_s) - \varepsilon_{cd}(t_0,t_s)\right]$$

with: $$\phi_{d0} = \begin{cases} 1000 & \text{for silica- fume concrete} \\ 3200 & \text{for non silica- fume concrete} \end{cases}$$

- Experimental identification procedure

At least 6 months

- Long term delayed strain estimation

Formulae

Experimental determination

- Safety factor for long term extrapolation γ_{lt}

t (age of concrete for estimating the delayed strains)	γ_{lt}
t < 1 year	1
t = 5 years	1,07
t = 10 years	1,1
t = 50 years	1,17
t = 100 years	1,20
t = 300 years	1,25

Annex KK ⇒ Structural effects of time dependent behaviour of concrete

Assumptions

- *Creep and shrinkage indipendent of each other*
- *Average values for creep and shrinkage within the section*
- *Validity of principle of superposition (Mc-Henry)*

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Type of analysis	Comment and typical application
General and incremental step-by-step method	These are general methods and are applicable to all structures. Particularly useful for verification at intermediate stages of construction in structures in which properties vary along the length (e.g.) cantilever construction.
Methods based on the theorems of linear viscoelasticity	Applicable to homogeneous structures with rigid restraints.
The ageing coefficient method	This mehod will be useful when only the long -term distribution of forces and stresses are required. Applicable to bridges with composite sections (precast beams and in-situ concrete slabs).
Simplified ageing coefficient method	Applicable to structures that undergo changes in support conditions (e.g.) span-to- span or free cantilever construction.

- General method

$$\varepsilon_c(t) = \frac{\sigma_0}{E_c(t_0)} + \varphi(t,t_0)\frac{\sigma_0}{E_c(28)} + \sum_{i=1}^{n}\left(\frac{1}{E_c\left(t_i\right)} + \frac{\varphi(t,t_i)}{E_c(28)}\right)\Delta\sigma\left(t_i\right) + \varepsilon_{cs}\left(t,t_s\right)$$

A step by step analysis is required

- Incremental method

- At the time t of application of $\boldsymbol{\sigma}$ the creep strain $\boldsymbol{\varepsilon}_{cc}(t)$, the potential creep strain $\boldsymbol{\varepsilon}_{\propto cc}(t)$ and the creep rate are derived from the whole load history

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- The potential creep strain at time t is:

$$\frac{d\varepsilon_{\infty cc}(t)}{dt} = \frac{d\sigma}{dt}\frac{\varphi(\infty,t)}{E_{c28}}$$

- t $\Rightarrow$ t_e

under constant stress from te the same $\varepsilon_{cc}(t)$ and $\varepsilon_{\infty cc}(t)$ are obtained

$$\varepsilon_{\infty cc}\left(t\right)\cdot\beta_c\left(t,t_e\right)=\varepsilon_{cc}\left(t\right)$$

- Creep rate at time t may be evaluated using the creep curve for t_e

$$\frac{d\varepsilon_{cc}(t)}{dt} = \varepsilon_{\infty cc}\left(t\right)\frac{\partial\beta_c\left(t,t_e\right)}{\partial t}$$

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- For unloading procedures

$$|\boldsymbol{\varepsilon}_{cc}(t)| \;>\; |\boldsymbol{\varepsilon}_{\infty cc}(t)|$$

and t_e accounts for the sign change

$$\varepsilon_{ccMax}(t)-\varepsilon_{cc}(t)=\left(\varepsilon_{ccMax}(t)-\varepsilon_{\infty cc}(t)\right)\cdot\beta_c\left(t,t_e\right)$$

$$\frac{d\left(\varepsilon_{ccMax}(t)-\varepsilon_{cc}(t)\right)}{dt}=\left(\varepsilon_{ccMax}(t)-\varepsilon_{\infty cc}(t)\right)\cdot\frac{\partial\beta_c\left(t,t_e\right)}{\partial t}$$

where $\boldsymbol{\varepsilon}_{ccMax}(t)$ is the last extreme creep strain reached before t

- Application of theorems of linear viscoelasticity

- $J(t,t_0)$ an $R(t,t_0)$ fully characterize the dependent properties of concrete
- Structures homogeneous, elastic, with rigid restraints
- Direct actions effect

$$S(t) = S_{el}\left(t\right)$$

$$D(t) = E_C \int_0^t J\left(t,\tau\right) dD_{el}\left(\tau\right)$$

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- Indirect action effect

$$D(t) = D_{el}(t)$$

$$S(t) = \frac{1}{E_C}\int_0^t R(t,\tau)\, dS_{el}(\tau)$$

- Structure subjected to imposed constant loads whose initial statical scheme (1) is modified into the final scheme (2) by introduction of additional restraints at time $t_1 \geq t_0$

$$S_2(t) = S_{el,1} + \xi(t,t_0,t_1)\Delta S_{el,1}$$

$$\xi(t,t_0,t_1) = \int_{t_1}^{t} R(t,\tau)\, dJ(\tau,t_0)$$

$$\xi(t,t_0,t_0^+) = 1 - \frac{R(t,t_0)}{E_C(t_0)}$$

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- When additional restraints are introduced at different times $t_i \geq t_0$, the stress variation by effect of restrain j introduced at t_j is indipendent of the history of restraints added at $t_i < t_j$

$$S_{j+1} = S_{el,1} + \sum_{i=1}^{j} \xi\left(t, t_0, t_i\right) \Delta S_{el,i}$$

- Ageing coefficient method

Integration in a single step and correction by means of χ ($\chi \cong 0.8$)

$$\int_{\tau=t_0}^{t} \left[\frac{E_c(28)}{E_c(\tau)} + \varphi_{28}\left(t,\tau\right) \right] d\sigma\left(\tau\right) = \left[\frac{E_c(28)}{E_c(t_0)} + \chi\left(t,t_0\right)\varphi_{28}\left(t,t_0\right) \right] \Delta\sigma_{t_0 \to t}$$

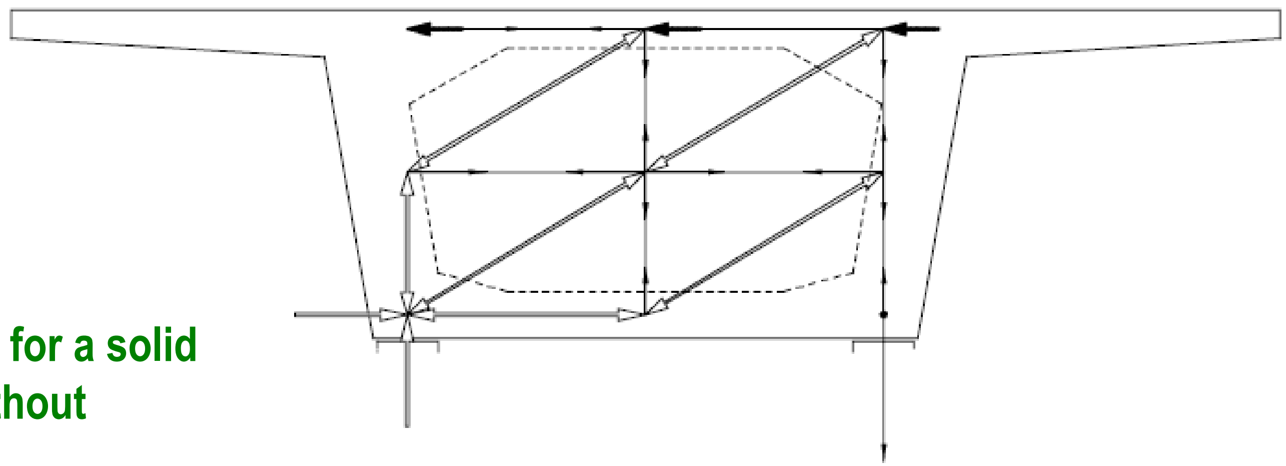
- Simplified formulae

$$S_{\infty} = S_0 + \left(S_1 - S_0\right) \frac{\varphi(\infty, t_0) - \varphi(t_1, t_0)}{1 + \chi \varphi\left(\infty, t_1\right)} \frac{E_c(t_1)}{E_c(t_0)}$$

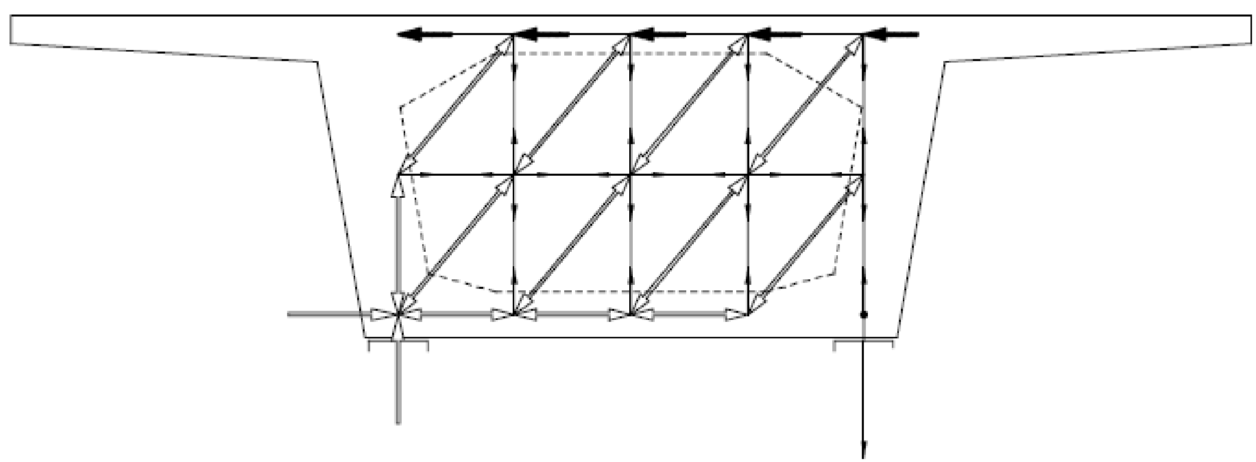
where: S_0 and S_1 refer respectively to construction and final statical scheme

t_1 is the age at the restraints variation

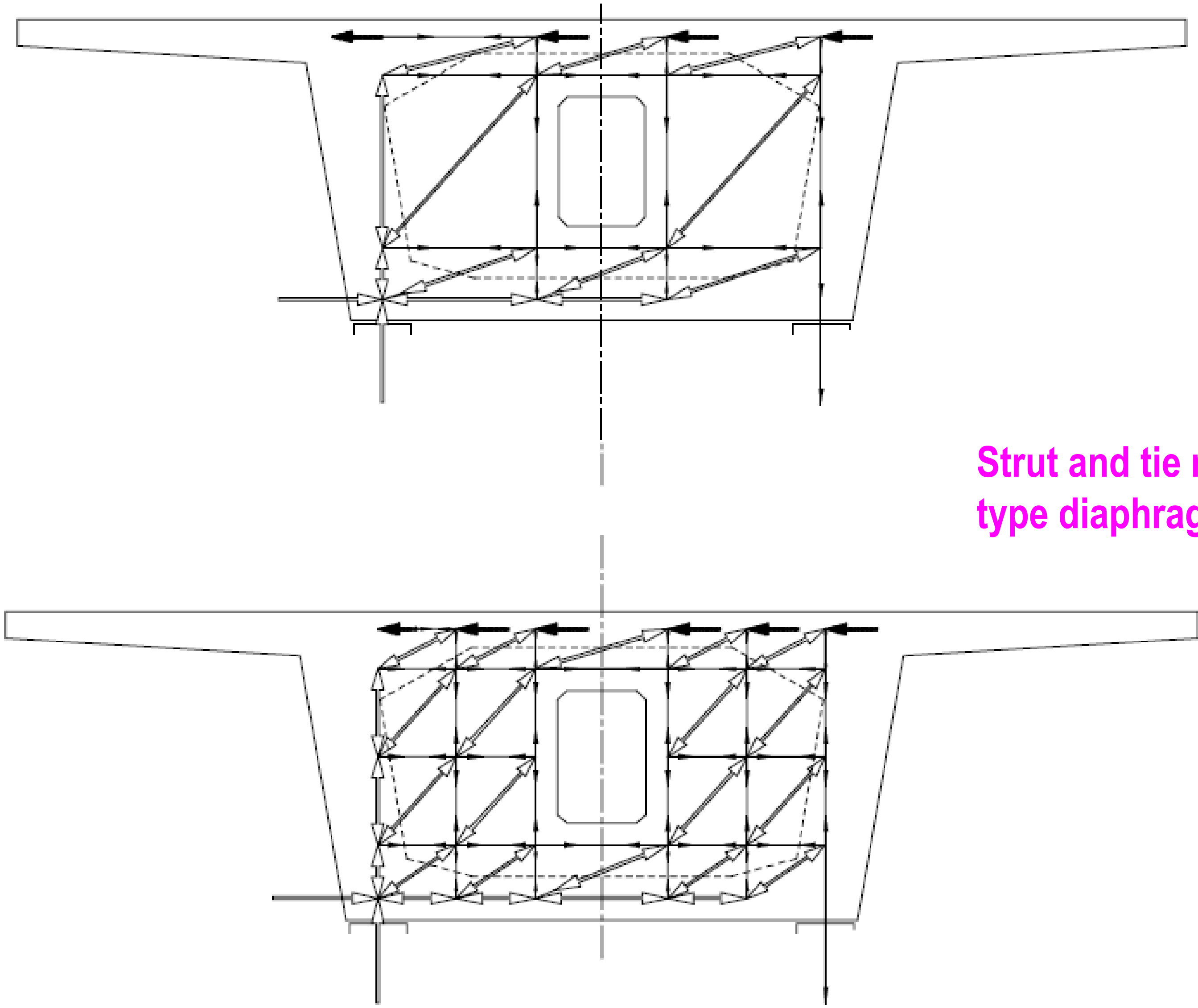
Annex OO ⇒ Typical bridge discontinuity regions



Strut and tie model for a solid type diaphragm without manhole

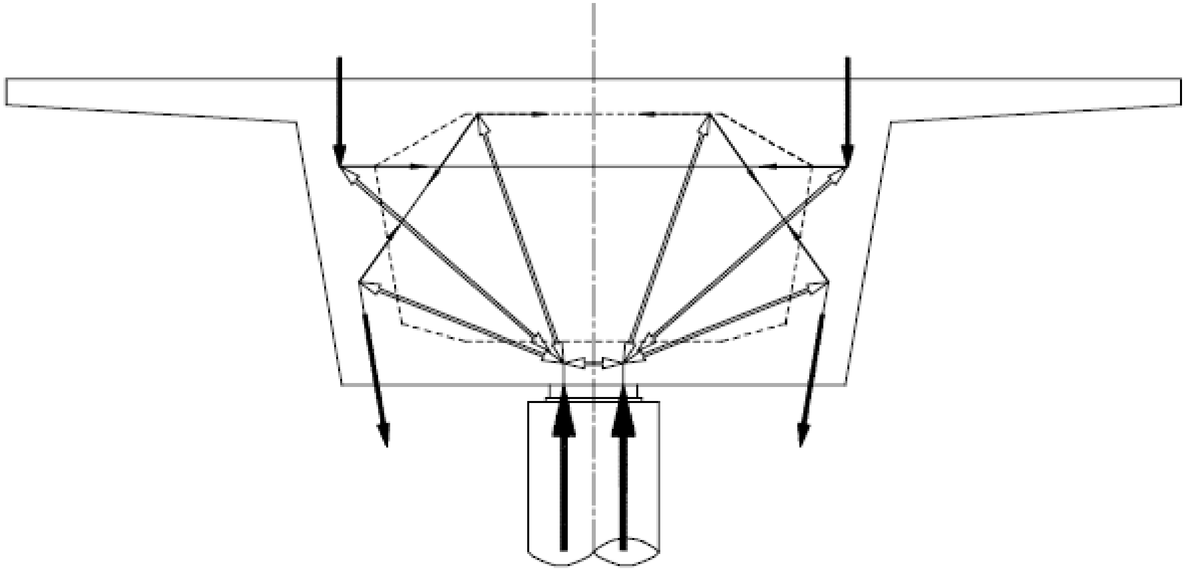


Design of concrete bridges
(EN 1992-2)



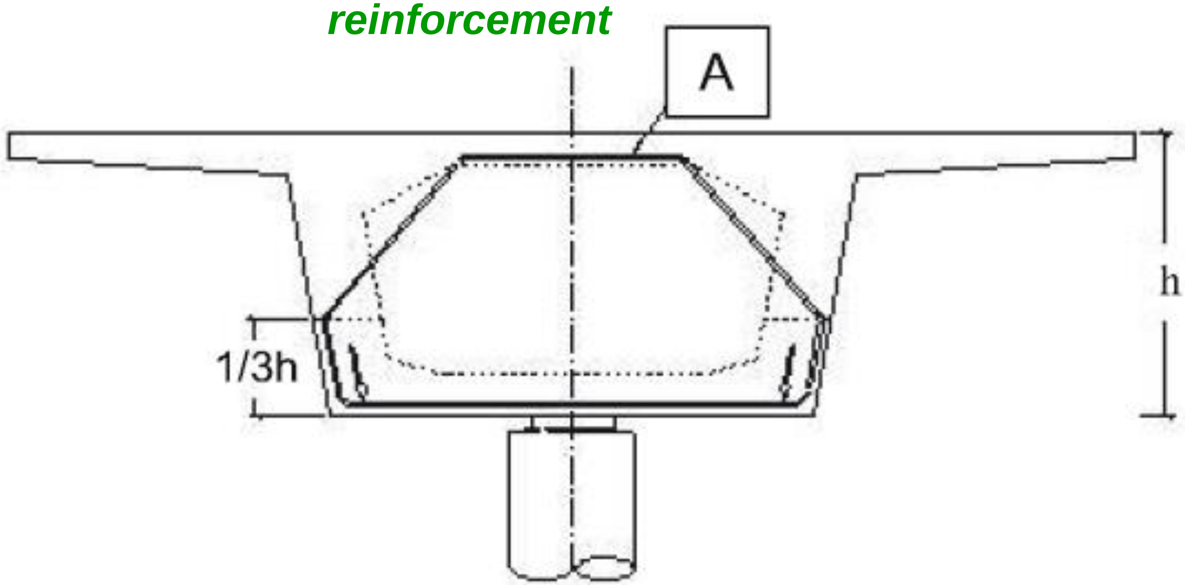
Strut and tie model for a solid type diaphragm with manhole

Design of concrete bridges
(EN 1992-2)



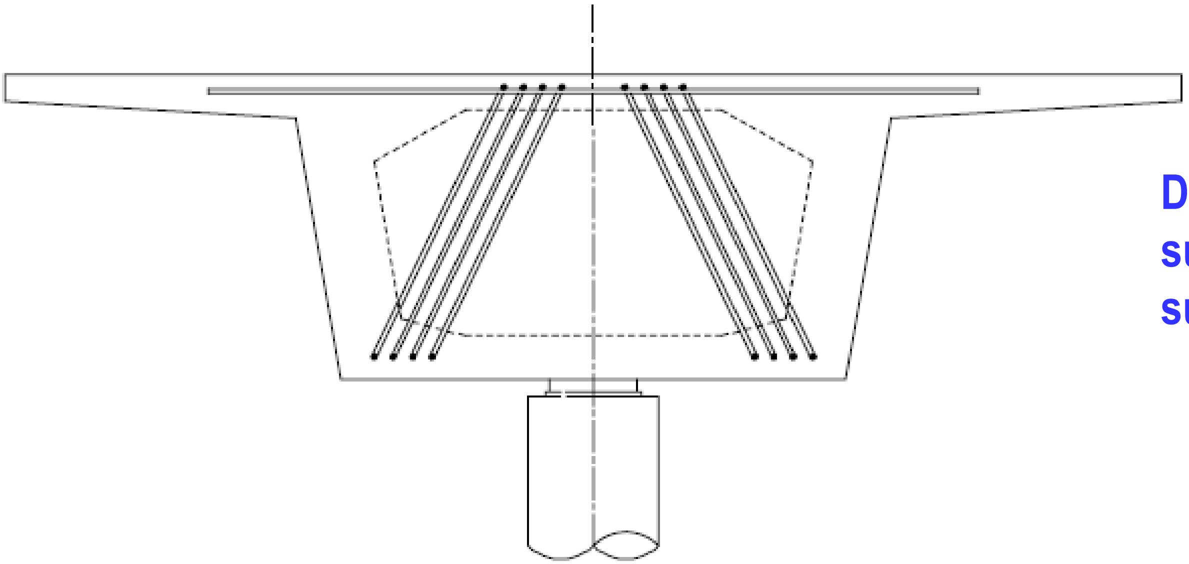
Diaphragms with indirect support. Strut and tie model

reinforcement



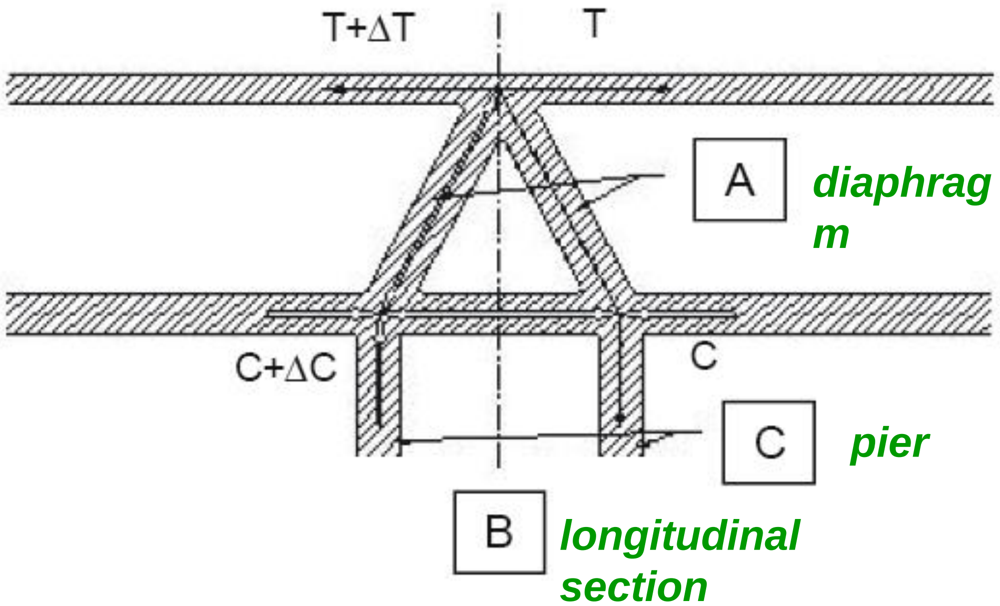
Diaphragms with indirect support. Anchorage of the suspension reinforcement

Design of concrete bridges
(EN 1992-2)



Diaphragms with indirect support. Links as suspension reinforcement

Diaphragm in monolithic joint with double diaphragm: Equivalent system of struts and ties.



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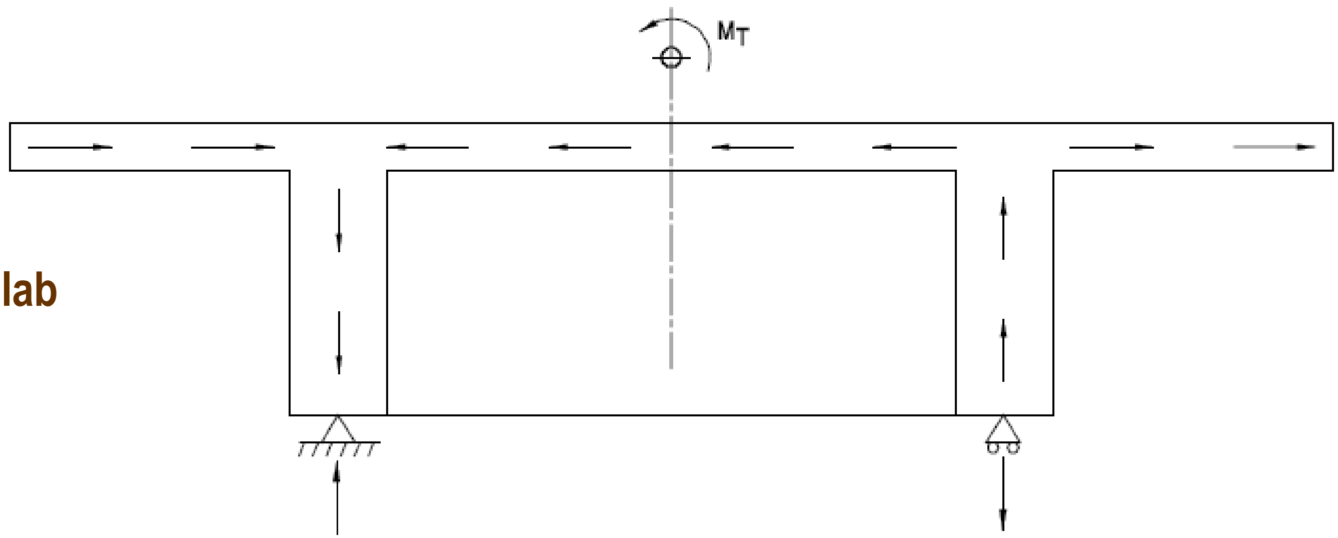
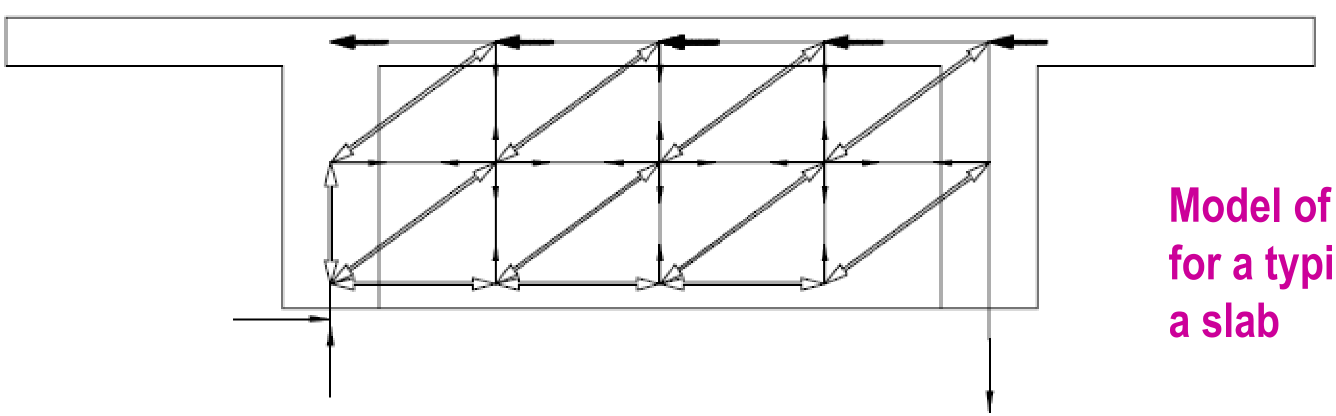


Torsion in the deck slab and reactions in the supports



Model of struts and ties for a typical diaphragm of a slab

EN 1992-2 $\Rightarrow$ **A new design code to help in conceiving more and more enhanced concrete bridges**

Thank you for the kind attention