EN 1992-2

EUROCODE 2 – Design of concrete structures
Concrete bridges: design and detailing rules

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Supersedes ENV 1992-2:1996

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- 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

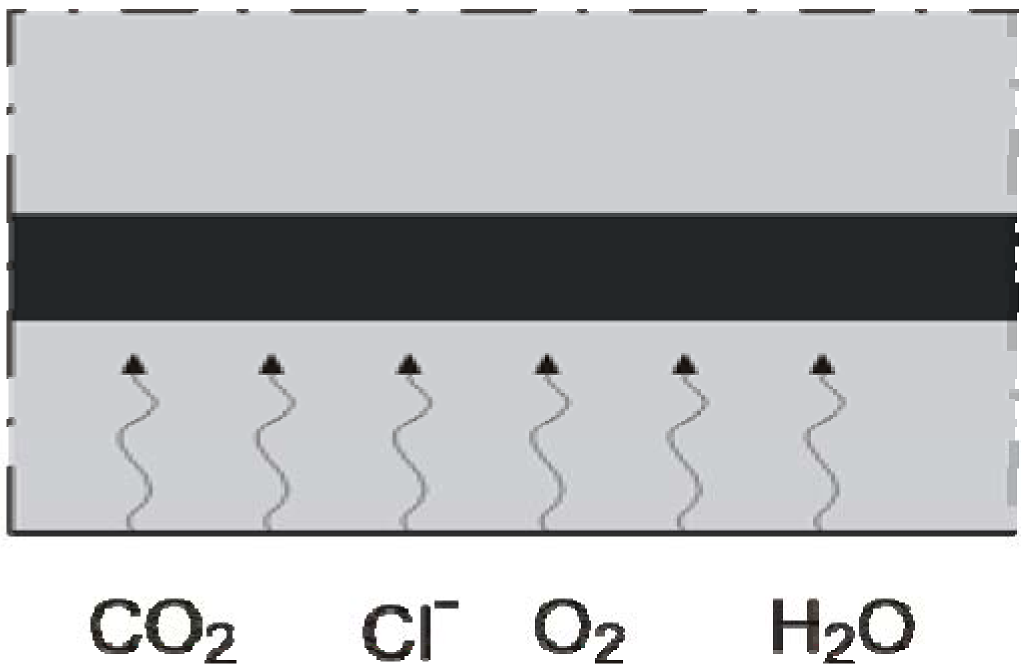
- Recommended classes for reinforcement:

  "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

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).

- 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:

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

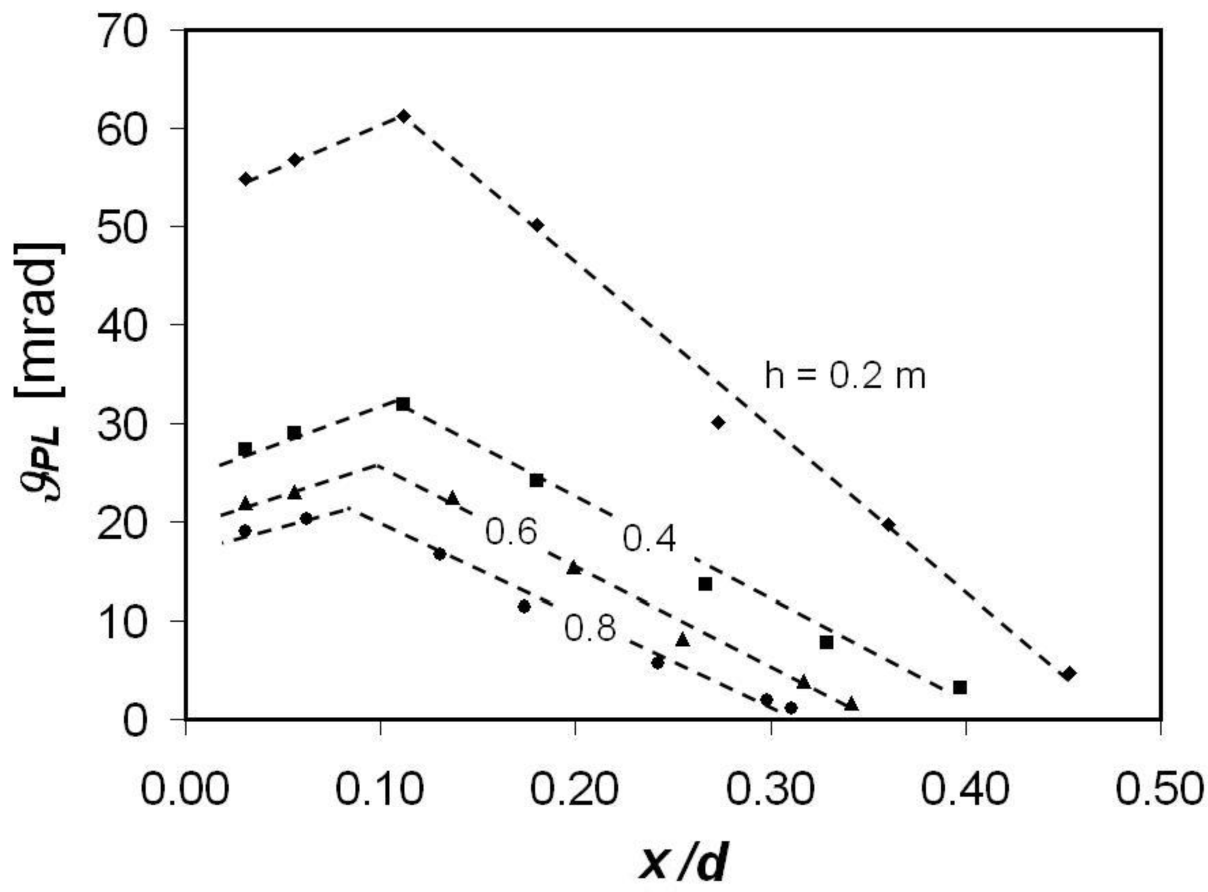
- Rotation capacity

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

in plastic hinges

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

Numerical rotation capacity



- Nonlinear analysis $\Rightarrow$ 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

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

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

- 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

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

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

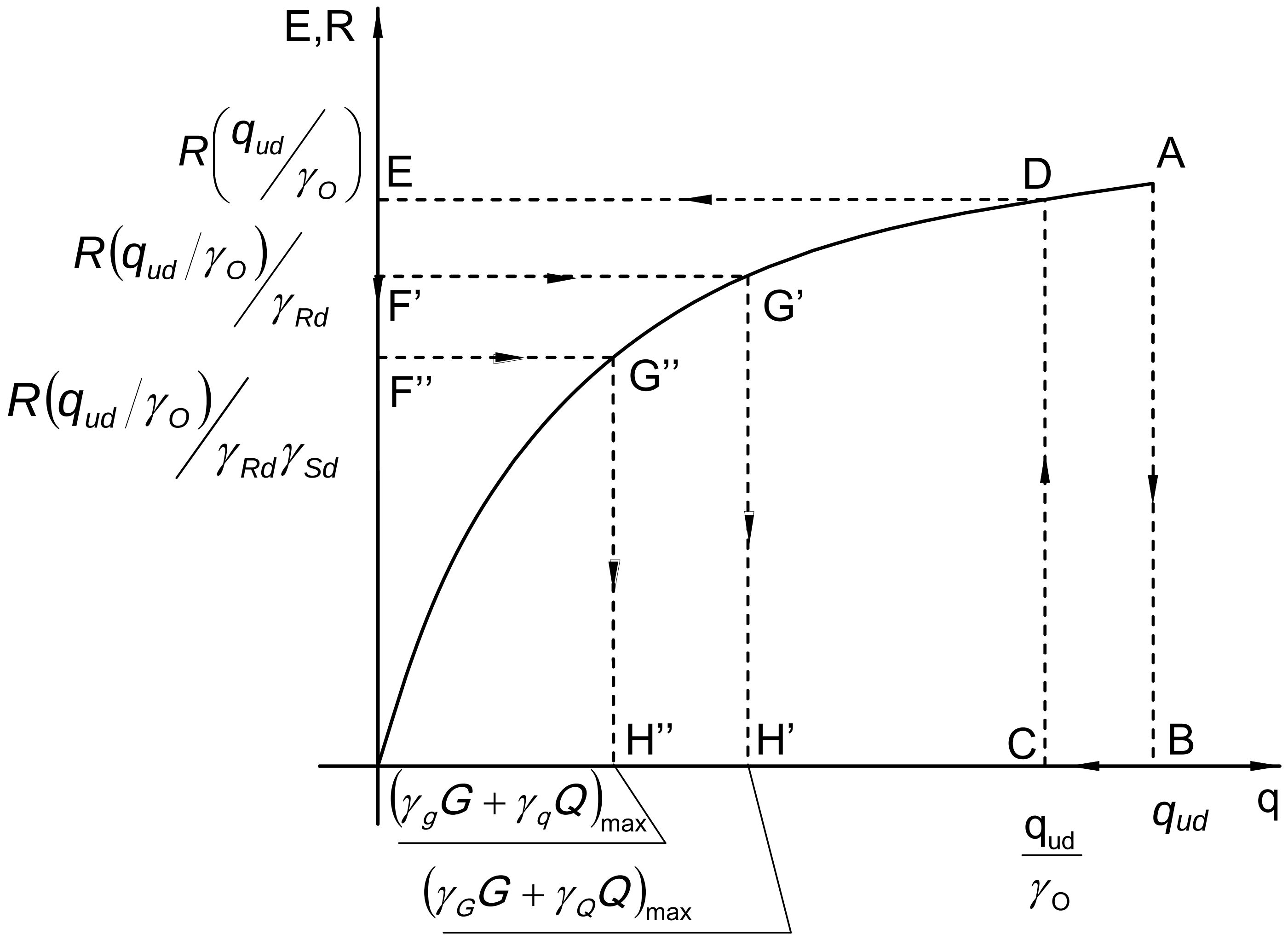
$$\gamma_{Rd} \gamma_{Sd} E\left(\gamma_g G + \gamma_q Q\right) \leq R\left(\frac{q_{ud}}{\gamma_O}\right)$$

With

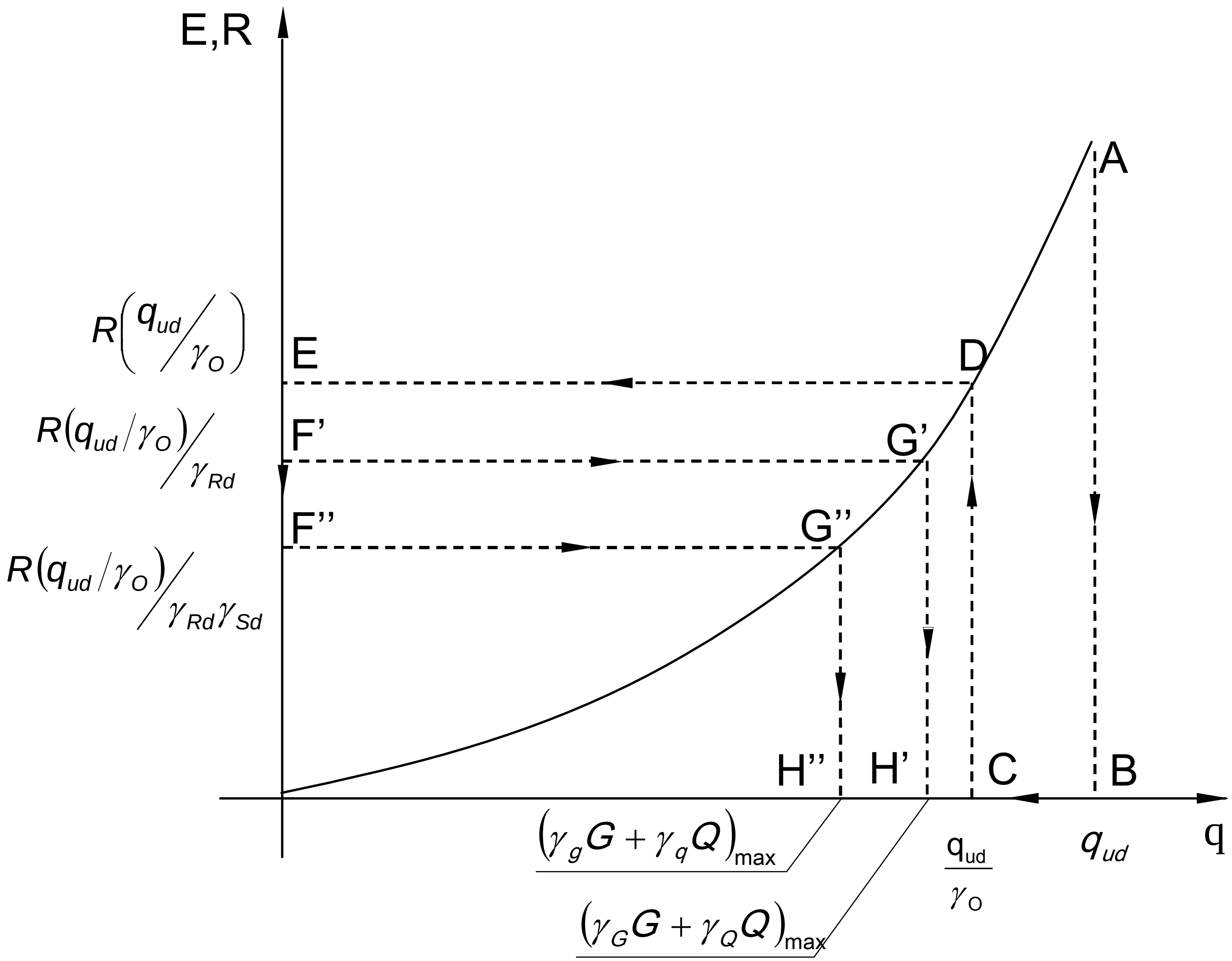
$$\begin{cases} \gamma_{Rd} = 1.06 & \text{partial factor for model uncertainties (resistence side)} \\ \gamma_{Sd} = 1.15 & \text{partial factor for model uncertainties (actions side)} \\ \gamma_0 = 1.20 & \text{structural safety factor} \end{cases}$$

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*

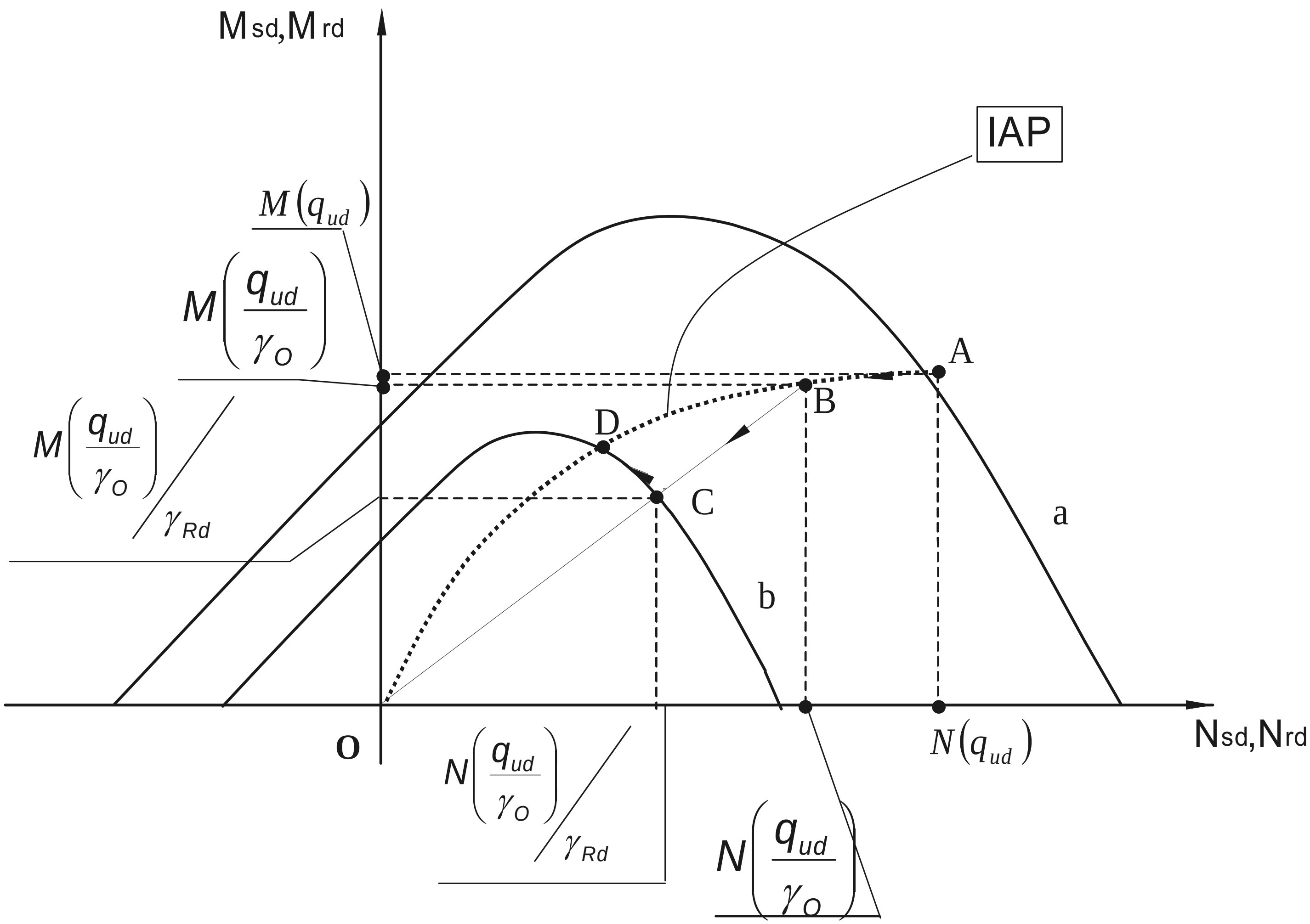


- Safety format *Application for scalar combination of internal actions and overproportional structural behaviour*



- Safety format

Application for vectorial combination of internal actions and underproportional structural behaviour



$M(q_{ud})$

$M\left(\frac{q_{ud}}{\gamma_O}\right)$

$M\left(\frac{q_{ud}}{\gamma_O}\right) / \gamma_{Rd}$

$N(q_{ud})$

$N\left(\frac{q_{ud}}{\gamma_O}\right) / \gamma_{Rd}$

$N\left(\frac{q_{ud}}{\gamma_O}\right)$

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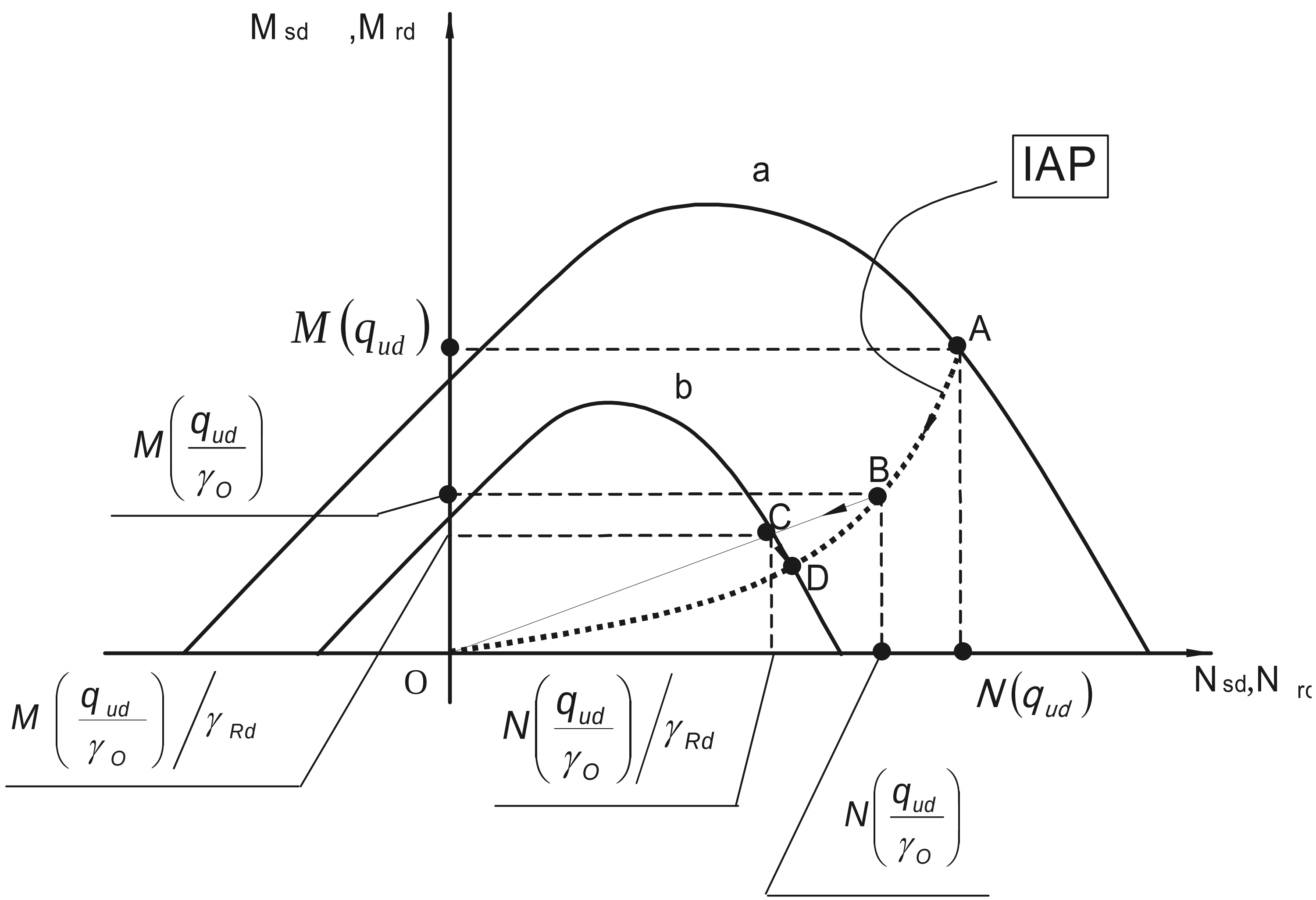
- Safety format

Application for vectorial combination of internal actions and overproportional structural behaviour



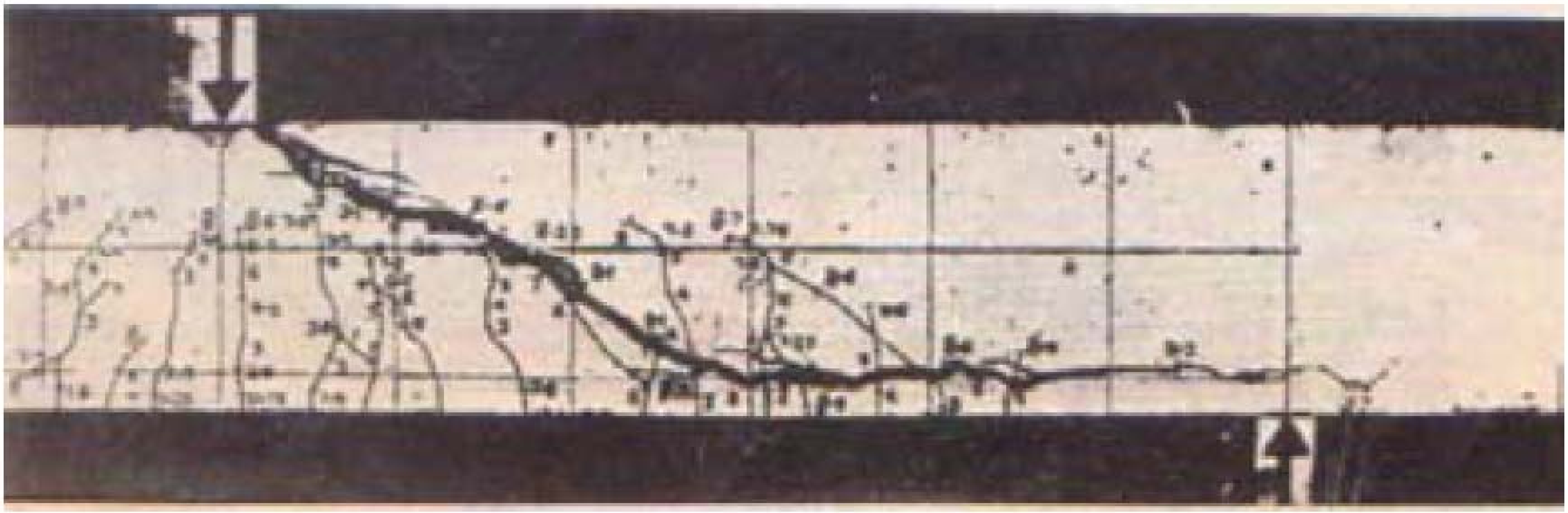
For vectorial combination and $\gamma_{Rd} = \gamma_{Sd} = 1.00$ the safety check is satisfied if:

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

and

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

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

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 + √(200/d) with d in meter
ρ_l	longitudinal reinforcement ratio (≤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 $V_{Rd,c}$

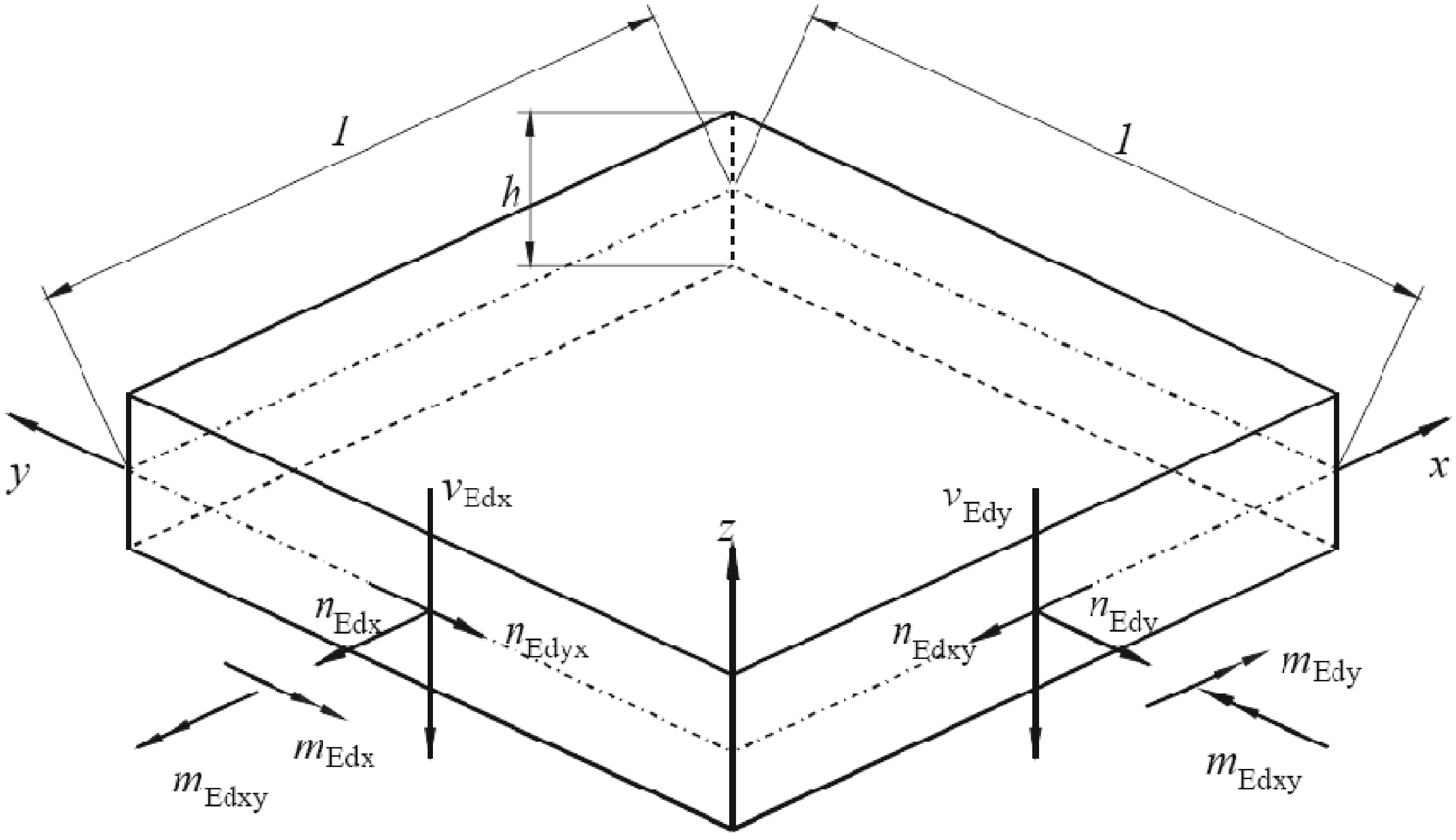
$$V_{Rd,c} = v_{min}\, b_w\, d$$

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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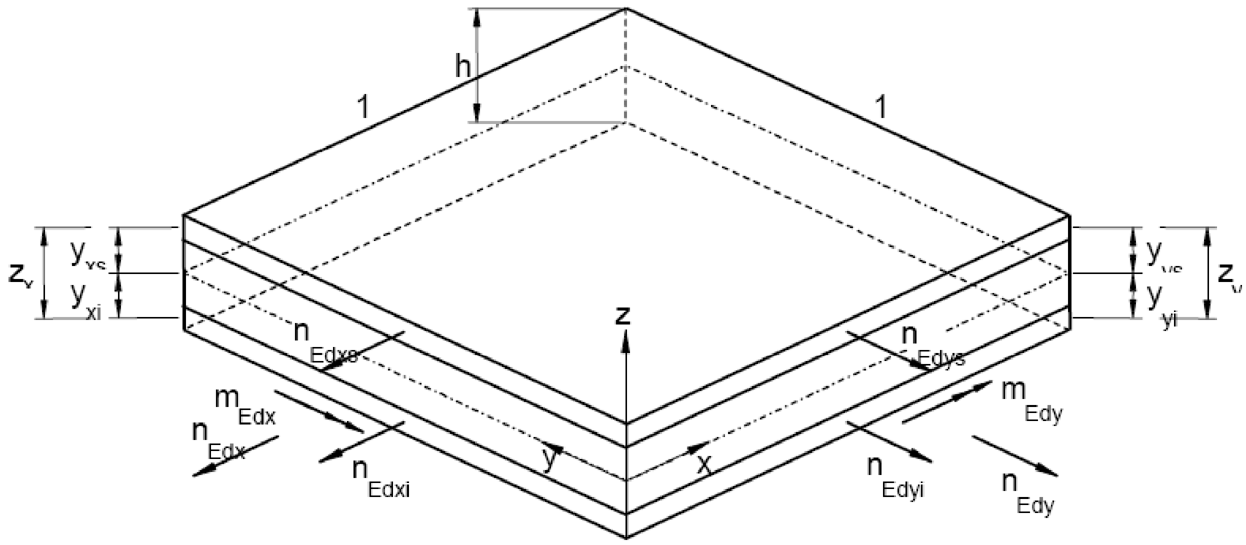
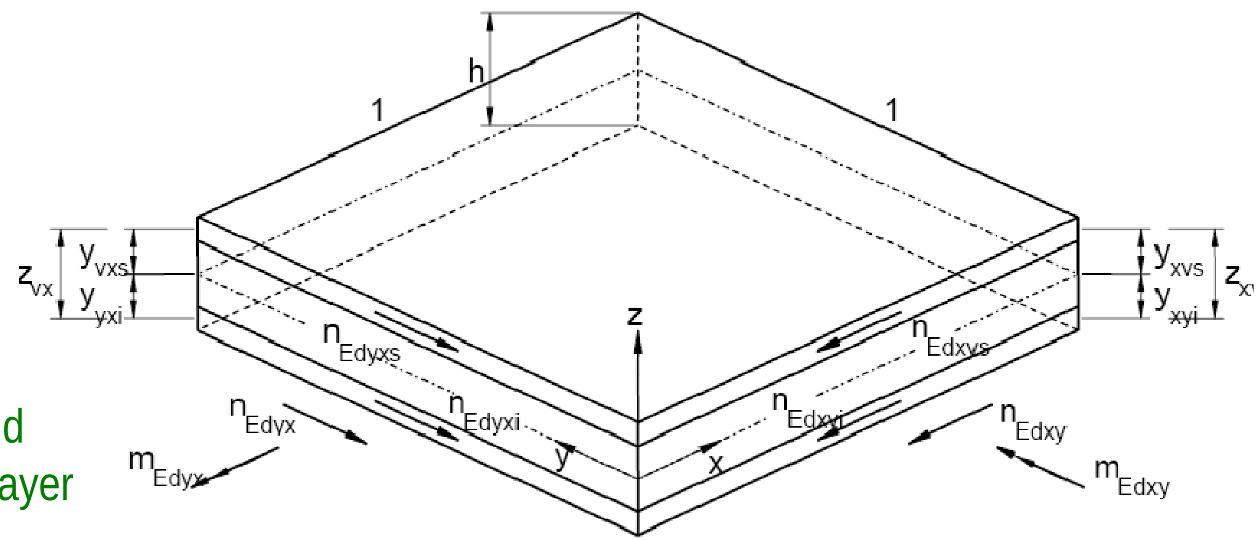


Axial actions and bending moments in the outer layer



Membrane shear actions and twisting moments in the outer layer

- 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.

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

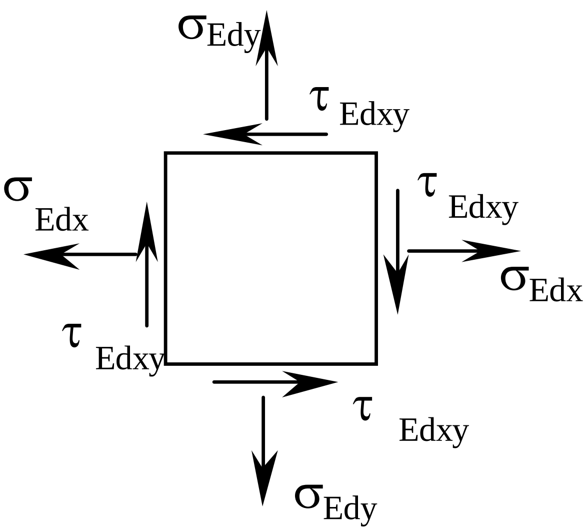
- When shear reinforcement is necessary, the longitudinal force resulting from the truss model $V_{Edo} \cdot \cot\theta$ gives rise to the following membrane forces in *x* and *y* directions:

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

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

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

- 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 ($\alpha \leq 1$)

- 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

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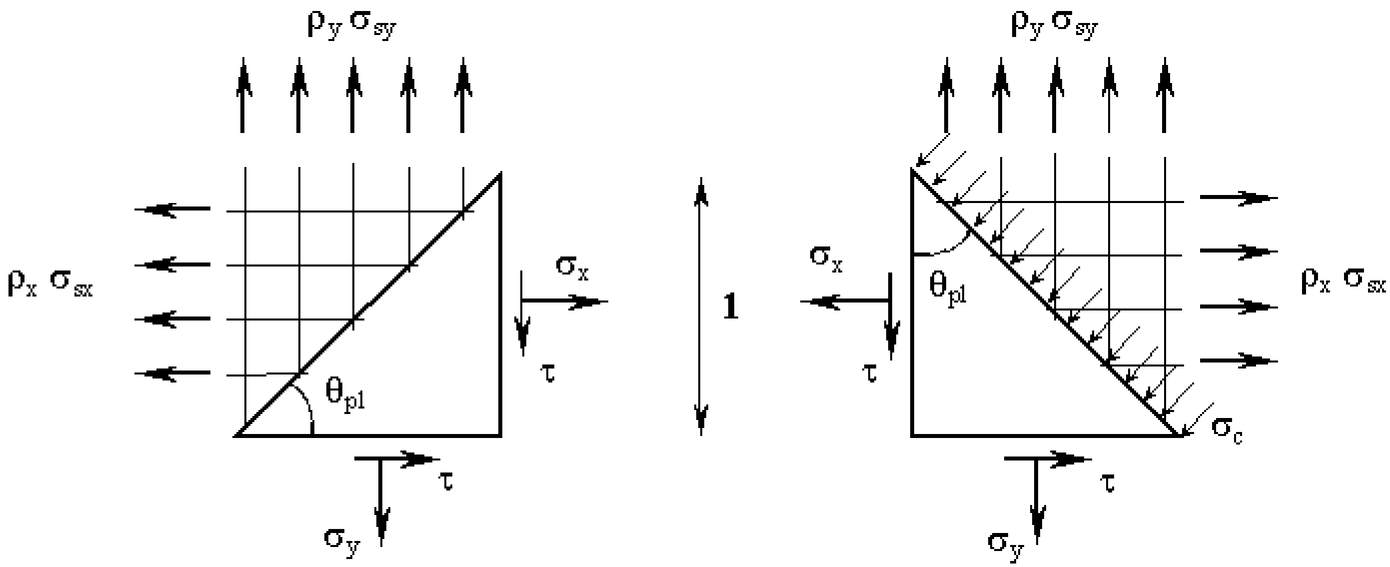
With increasing $\Delta\vartheta$ concrete damage increases progressively and strength is reduced accordingly

Plastic equilibrium condition



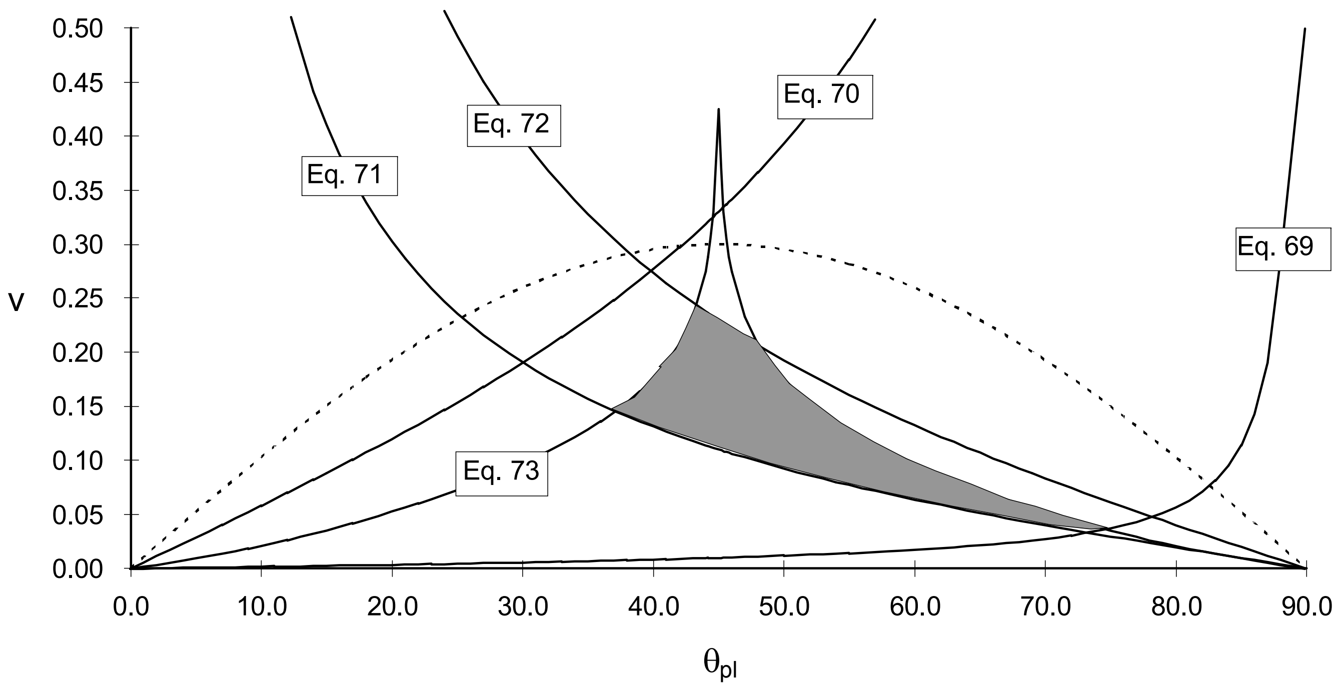
$$\sigma_x + \tau \cot \vartheta_{pl} - \sigma_{sx} \rho_x = 0$$

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

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

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

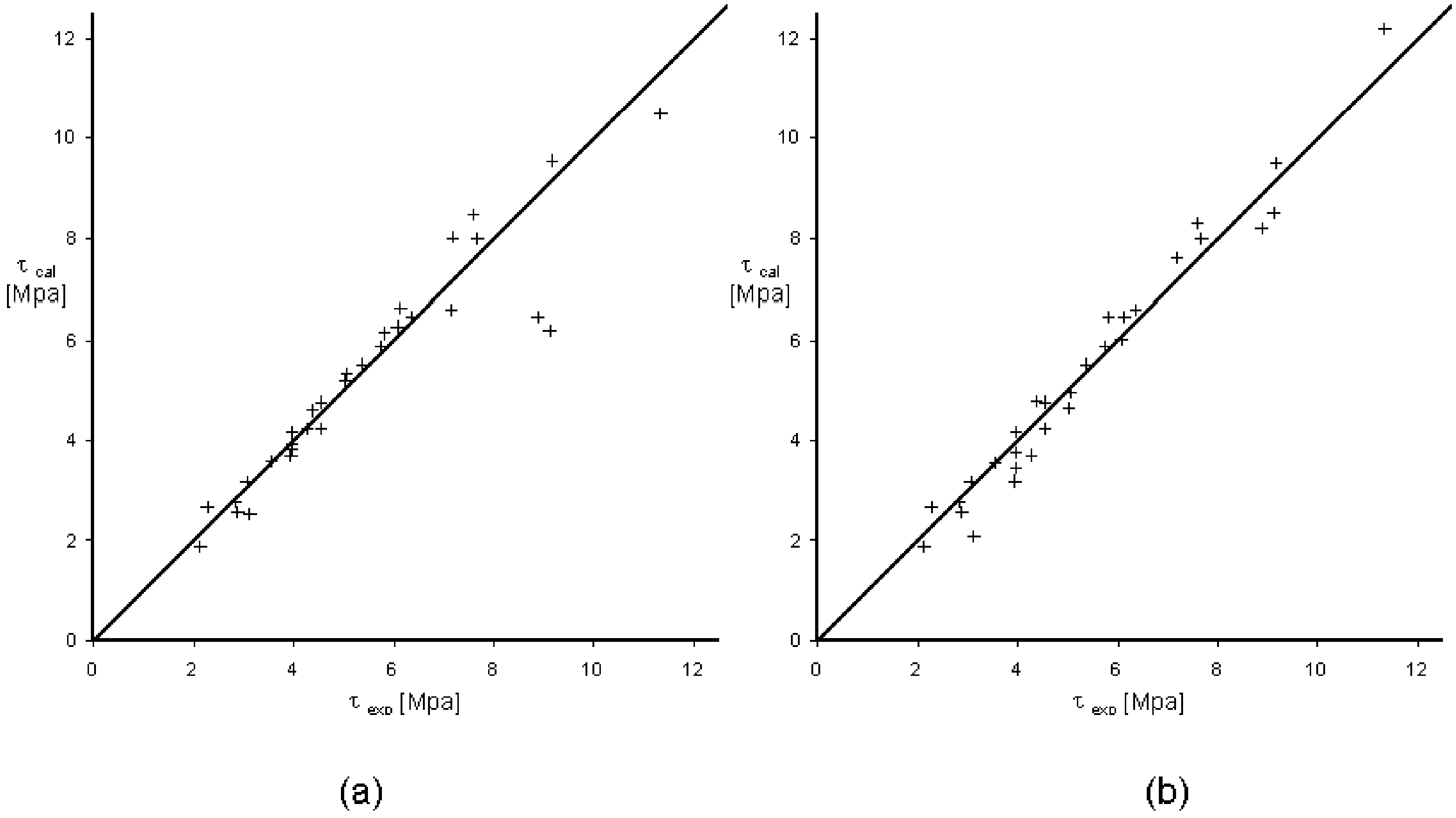
Graphical solution of inequalities system



$\omega_x = 0.16$
$\omega_y = 0.06$
$n_x = n_y = -0.17$
$\vartheta_{el} = 45°$

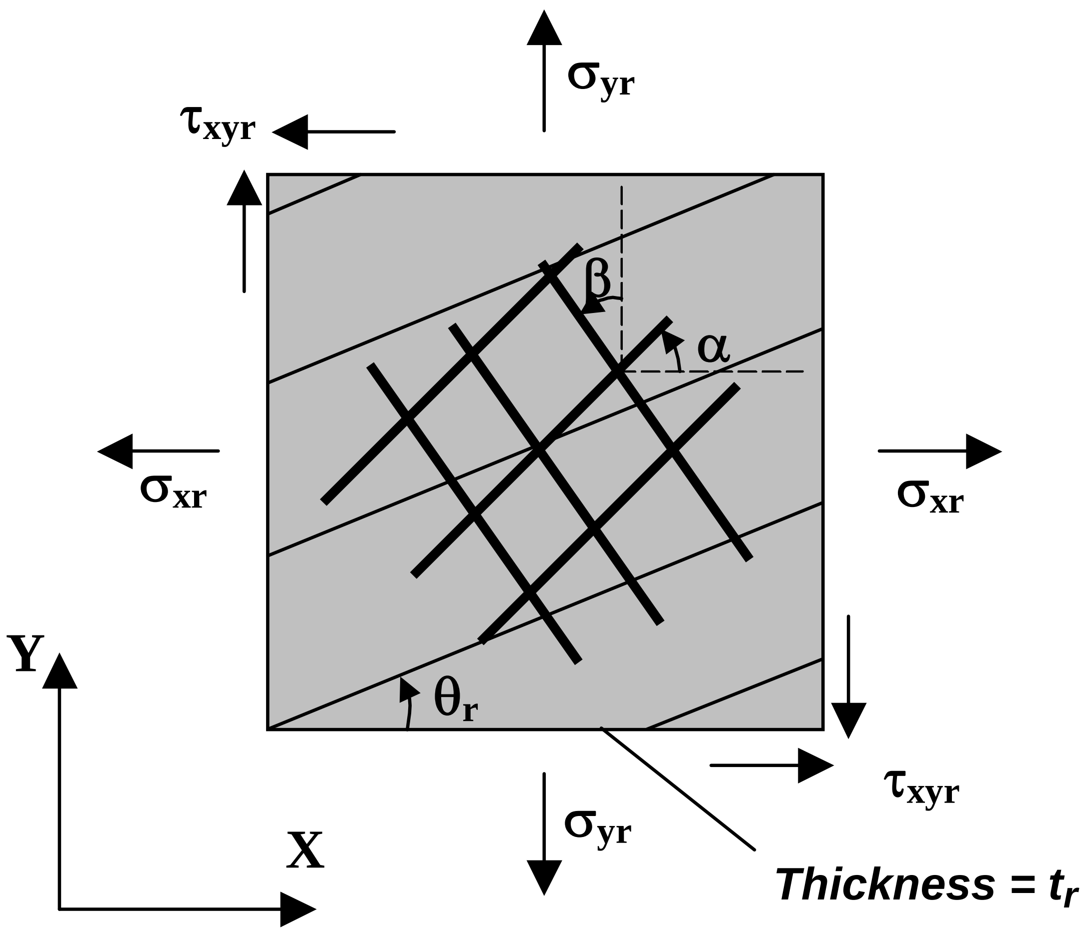
$$\frac{\tau}{|f_c'|}(\tan\vartheta_{pl} + \cot\vartheta_{pl}) - \left[0.55 - 0.12\ln|\vartheta_{pl} - \vartheta_{el}|\right] = 0$$

$$v \geq -(\omega_x + n_x)\tan\vartheta_{pl} \quad (69)$$
$$v \leq (\omega_x - n_x)\tan\vartheta_{pl} \quad (70)$$
$$v \geq (-\omega_y + n_y)\cot\vartheta_{pl} \quad (71)$$
$$v \leq (\omega_y - n_y)\cot\vartheta_{pl} \quad (72)$$
$$v \leq \nu\sin\vartheta_{pl}\cos\vartheta_{pl} \quad (73)$$



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

Skew reinforcement



Plates conventions

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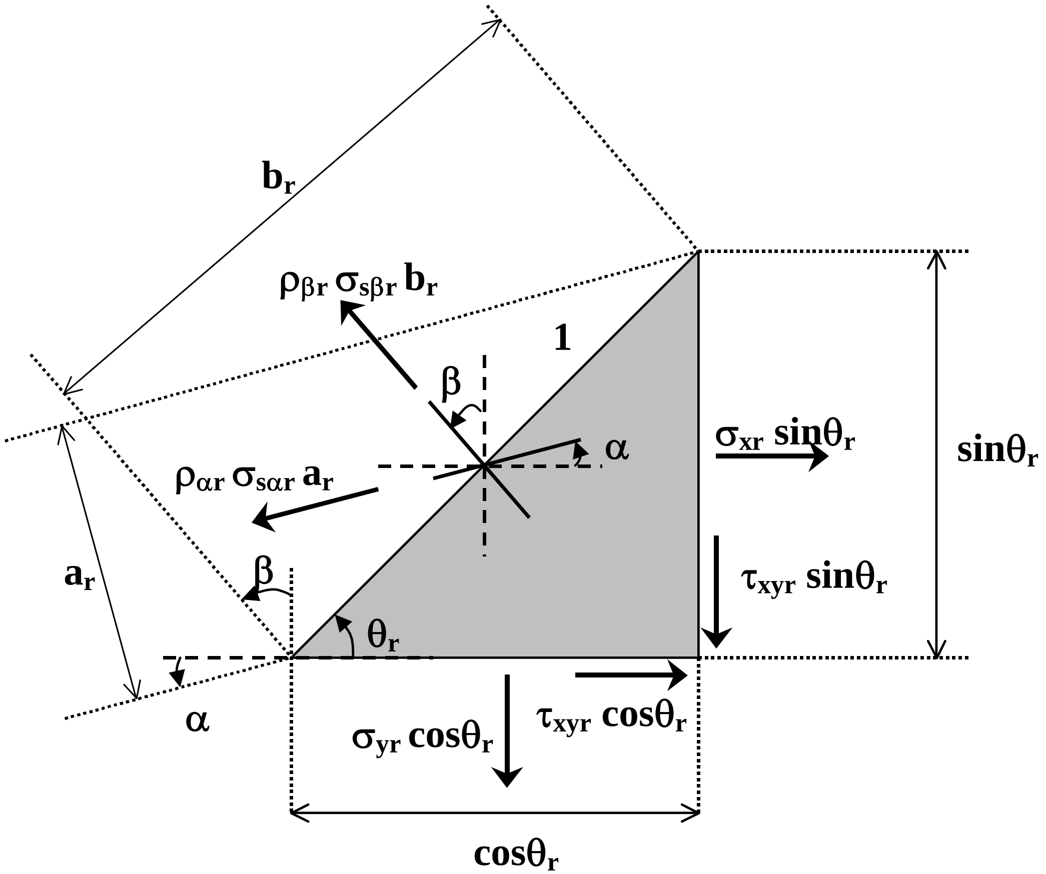
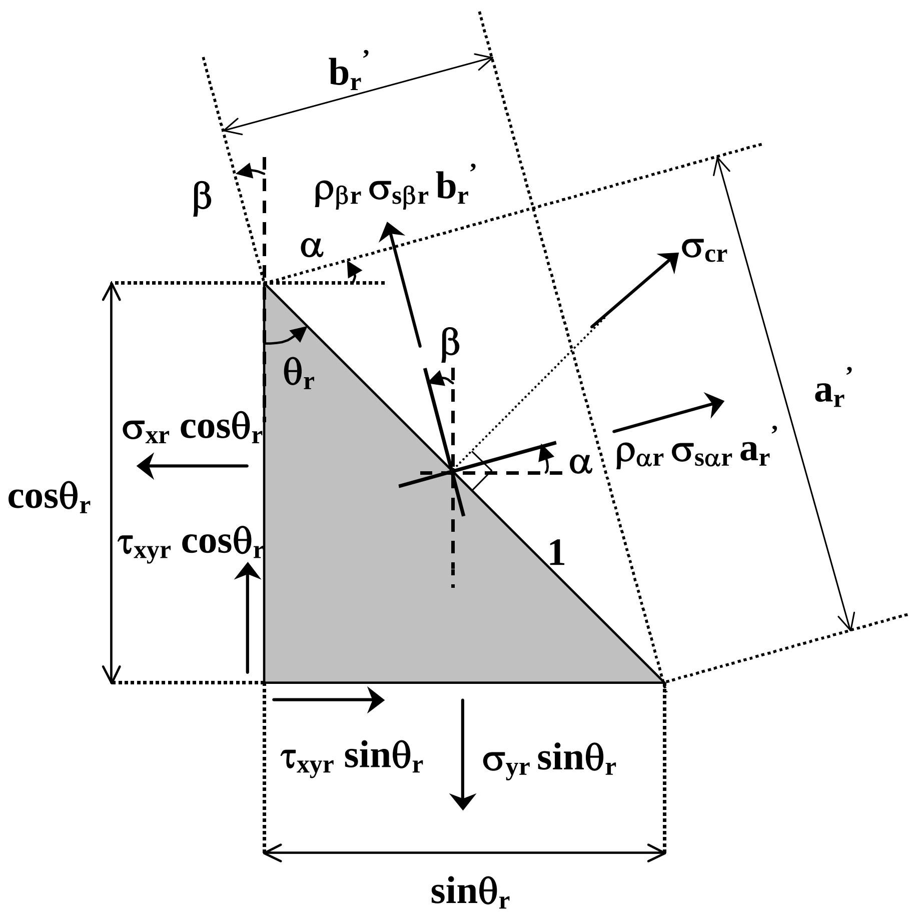


Equilibrium of the section parallel to the compression field

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

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



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)

$k_1 = 0.6$ (recommended value)

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

- 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 (recommended 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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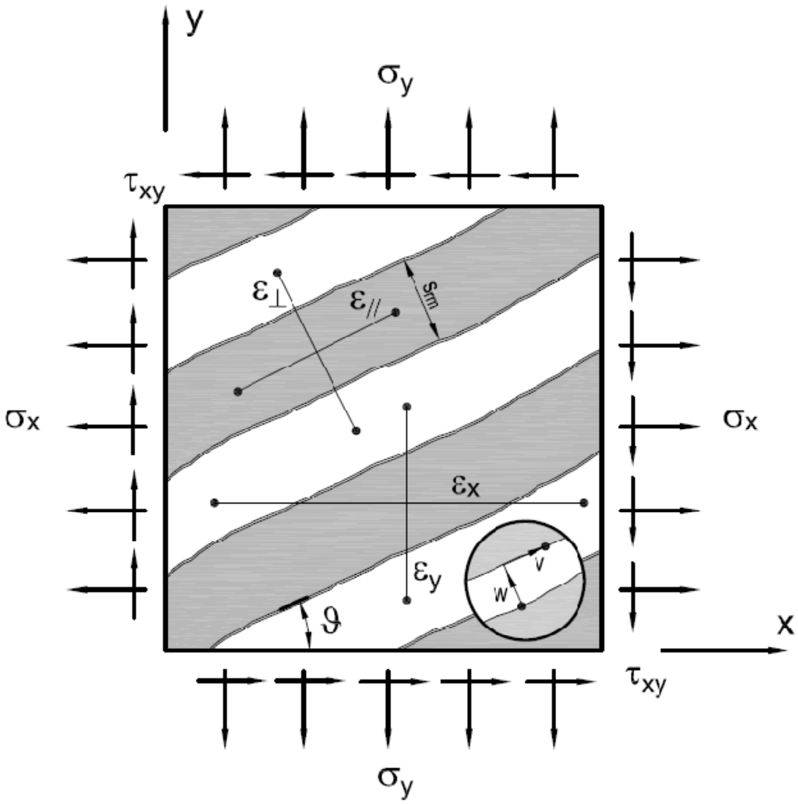
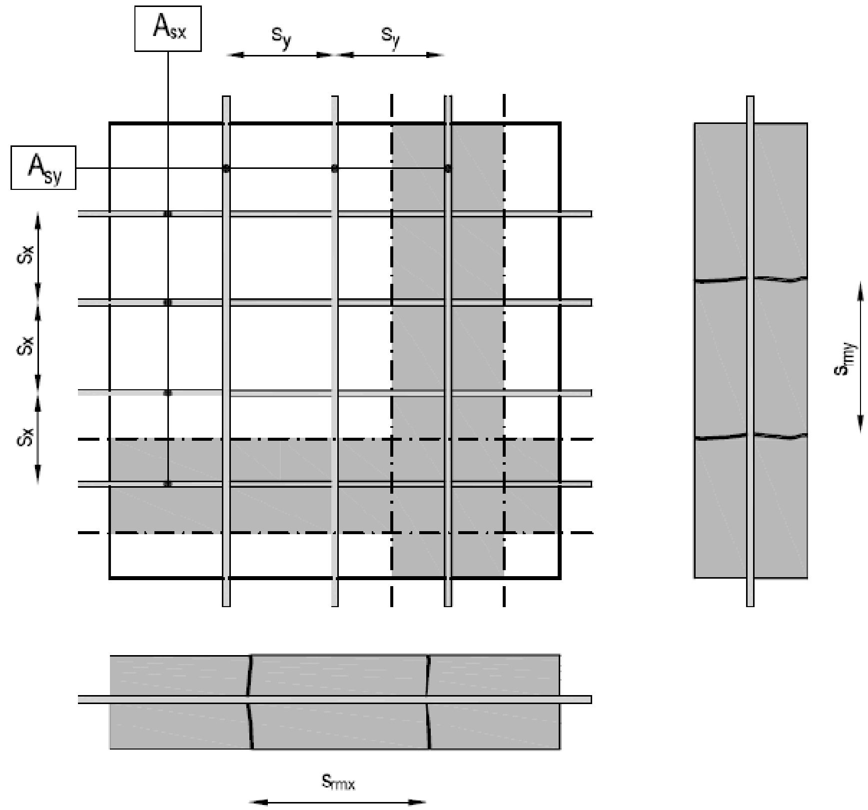




Expressing the compatibility of displacement along the crack, the total strain and the corresponding stresses in reinforcement in x and y directions may be evaluated, as a function of the displacements components w and v, respectively orthogonal and parallel to the crack direction.

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 hydratation*
  - ***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 $t \geq 28$ 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

- Drying shrinkage (RH ≤ 80%)

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

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

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

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

- 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

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

with:

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

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

- 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)*

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 σ the creep strain $\varepsilon_{cc}(t)$, the potential creep strain $\varepsilon_{\propto cc}(t)$ and the creep rate are derived from the whole load history

- 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

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

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

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

- For unloading procedures

$$|\varepsilon_{cc}(t)| > |\varepsilon_{\infty cc}(t)|$$

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 $\varepsilon_{ccMax}(t)$ 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

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

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

- 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 $t_1 \geq t_0$

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

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

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

- 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

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

Thank you for the kind attention

Concrete bridge design (EN1992-2)
Application to the design example

Emmanuel Bouchon
Sétra

Contents

Local justification of the concrete slab
Durability – cover to reinforcement

Minimum cover , c_{min} (EN1992-1-1, 4.4.1.2)

$c_{min} = \max \{c_{min,b}; c_{min,dur} ; 10 \text{ mm}\}$

- $c_{min,b}$ (bond) is given in table 4.2
  $c_{min,b}$ = diameter of bar (max aggregate size ≤ 32 mm
  $c_{min,b}$ = 20 mm on top face of the slab
  $c_{min,b}$ = 25 mm on bottom face at mid span between the steel main girders

- $c_{min,dur}$ (durability) is given in table 4.4N, it depends on :
  the exposure class (table 4.1)
  the structural class (table 4.3N)

Local justification of the concrete slab
Durability – cover to reinforcement

Structural class (table 4.3 N)

Top face of the slab : XC3

Bottom face of the slab : XC4

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 class : 3 (XC2 / XC3) | 4 (XC4)

Local justification of the concrete slab
Durability – cover to reinforcement

$c_{min,dur}$ (table 4.4 N)

Top face of the slab : XC3 – str. class S3

Bottom face of the slab : XC4 – str. class S4

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

Local justification of the concrete slab
Durability – cover to reinforcement

$c_{nom} = c_{min} + \Delta c_{dev}$ (allowance for deviation, expr. 4.1)

Δc_{dev} = 10 mm (recommended value 4.4.1.3 (1)P)

Δc_{dev} may be reduced in certain situations (4.4.1.3 (3))

- in case of quality assurance system with measurements of the concrete cover, the recommended value is:

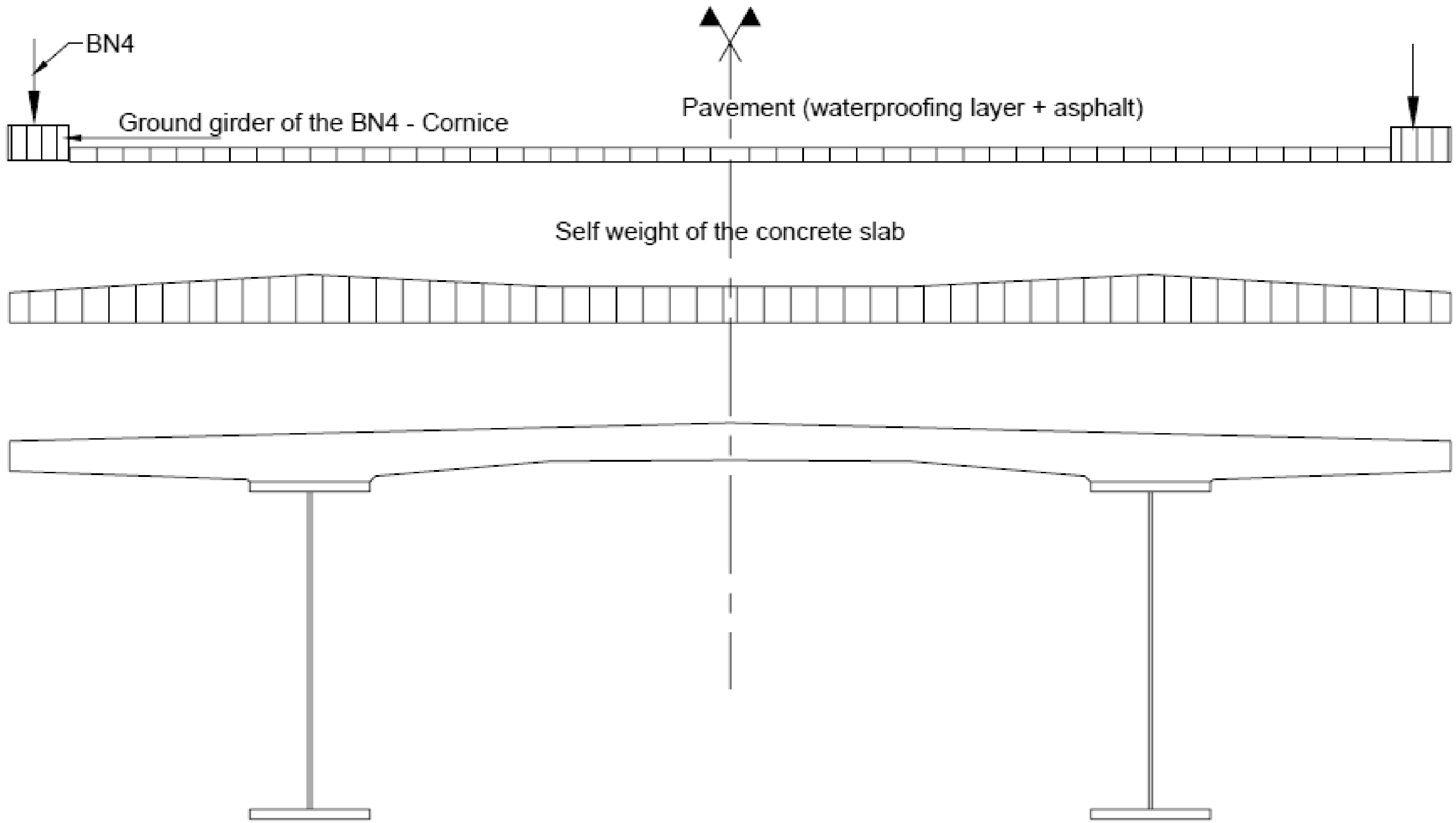
10 mm ≥ Δc_{dev} ≥ 5 mm

Cover (mm)	$c_{min,b}$	$c_{min,dur}$	Δc_{dev}	c_{nom}
Top face of the slab	20	20	10	**30**
Bottom face of the slab	25	30	10	**40**

Local justification of the concrete slab
Verification of the transverse reinforcement

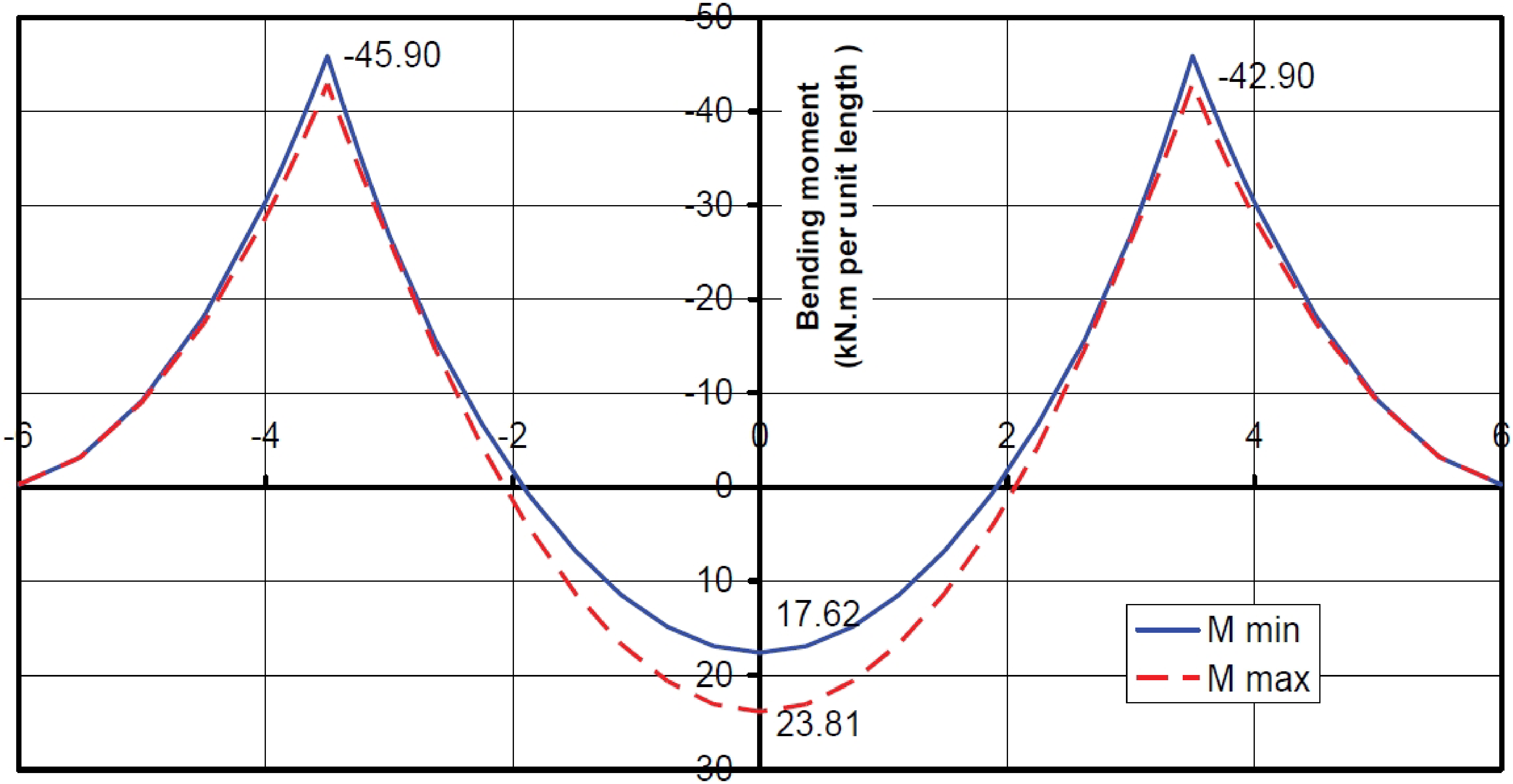
- The verifications of transverse bending and vertical shear are performed on an equivalent beam representing a 1-m-wide slab strip.
- Analysis:
  - For permanent loads, which are uniformly distributed over the length of the deck, the internal forces may be calculated on a simplified model: isostatic beam on two supports.
  - For traffic loads, it is necessary to take into account the 2-dimensional behaviour of the slab. For the design example, the extreme values of transverse internal forces and moments are obtained reading charts established by Setra for the local bending of the slab in twin-girder composite bridges. These charts are derived from the calculation of influence surfaces on a finite element model of a typical composite deck slab.

Analysis - Transverse distribution of permanent loads



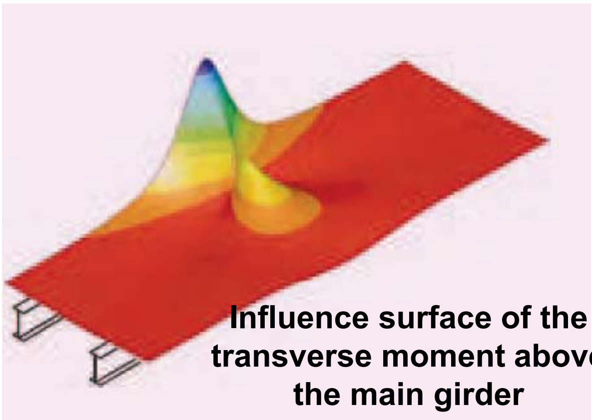
Local justification of the concrete slab
Verification of the transverse reinforcement

Analysis - transverse bending moment envelope due to permanent loads

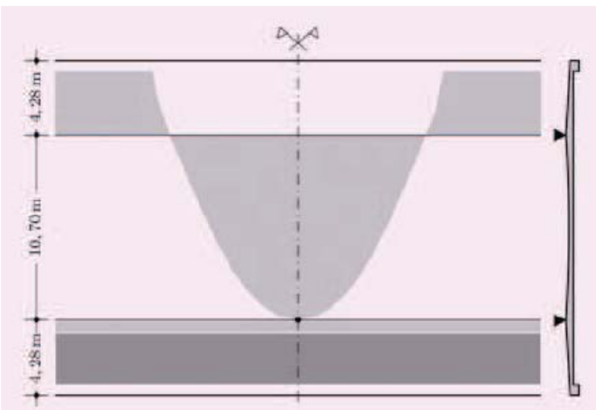


Local justification of the concrete slab
Verification of the transverse reinforcement

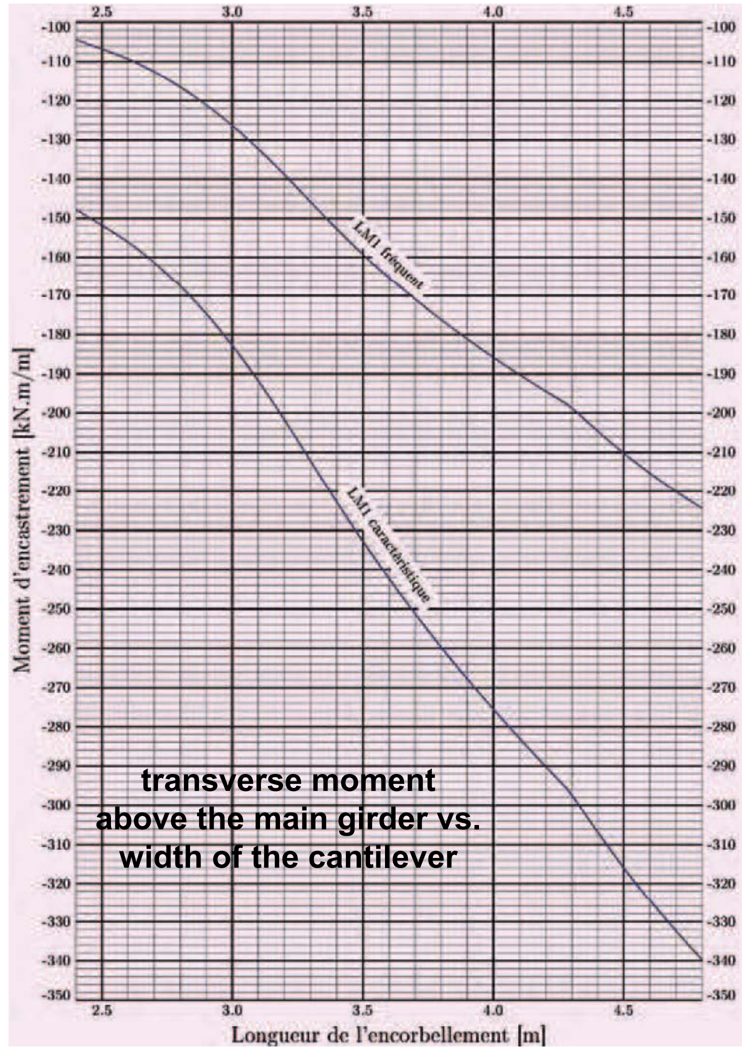
Analysis - Maximum effect of traffic loads



Influence surface of the transverse moment above the main girder



Position of UDL



transverse moment above the main girder vs. width of the cantilever

Combination of actions

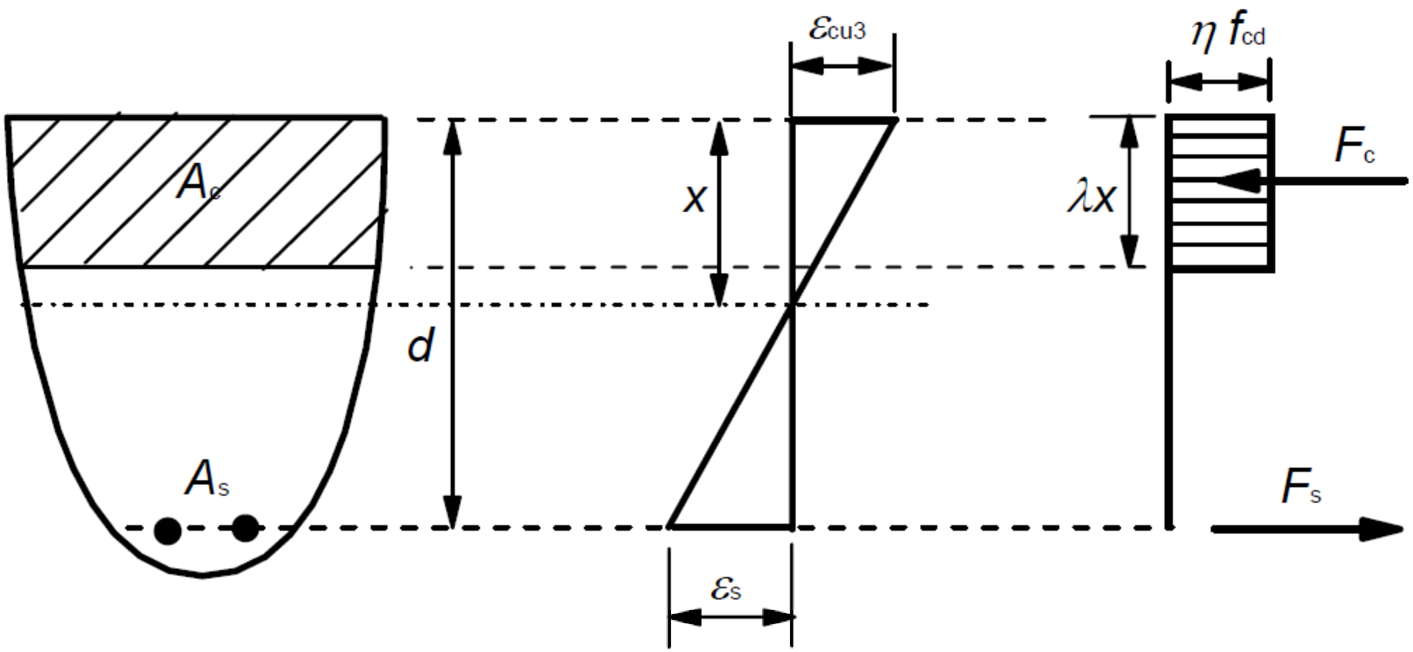
Transverse bending moment

M (kNm/m)	Quasi permanent SLS	Frequent SLS	Characteristic SLS	ULS
Section above the main girder	-46	-156	-204	-275
Section at mid-span	24	132	184	248

Bending resistance at ULS (EN1992-1-1, 6.1)

Stress-strain relationships:

- **for the concrete, a simplified rectangular stress distribution:**
  λ = 0,80 and η = 1,00 as f_{ck} = 35 MPa ≤ 50 MPa
  f_{cd} = 19,8 MPa (with α_{cc} = 0,85 – recommended value)
  ε_{cu3} = 3,5 mm/m

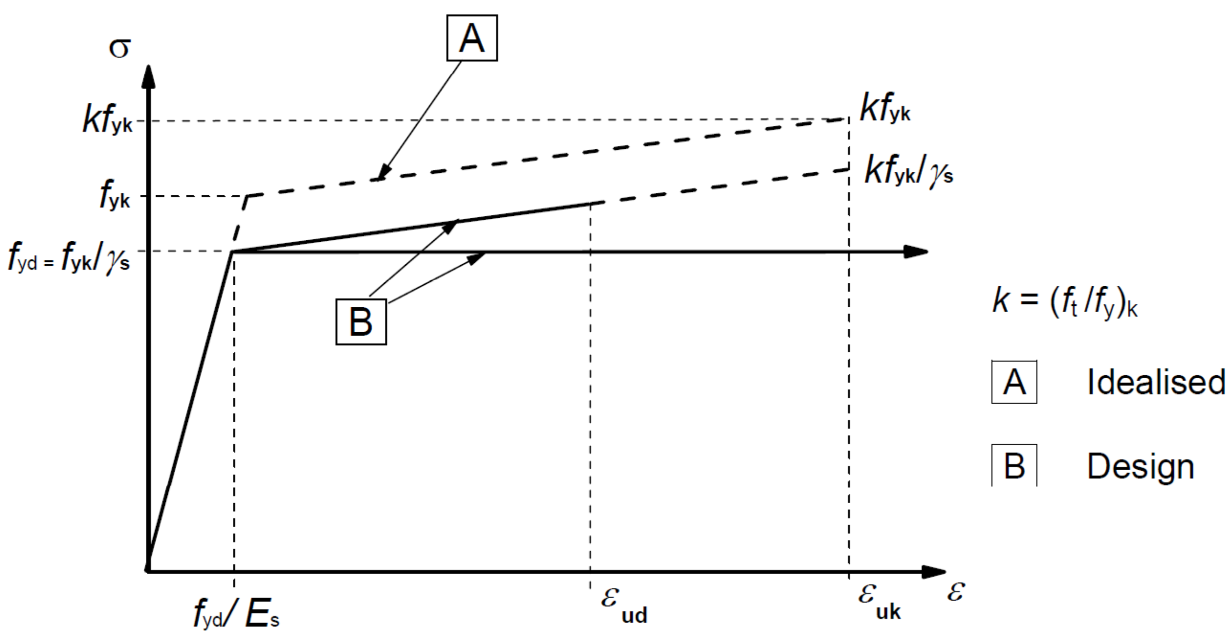


Bending resistance at ULS

Stress-strain relationships:

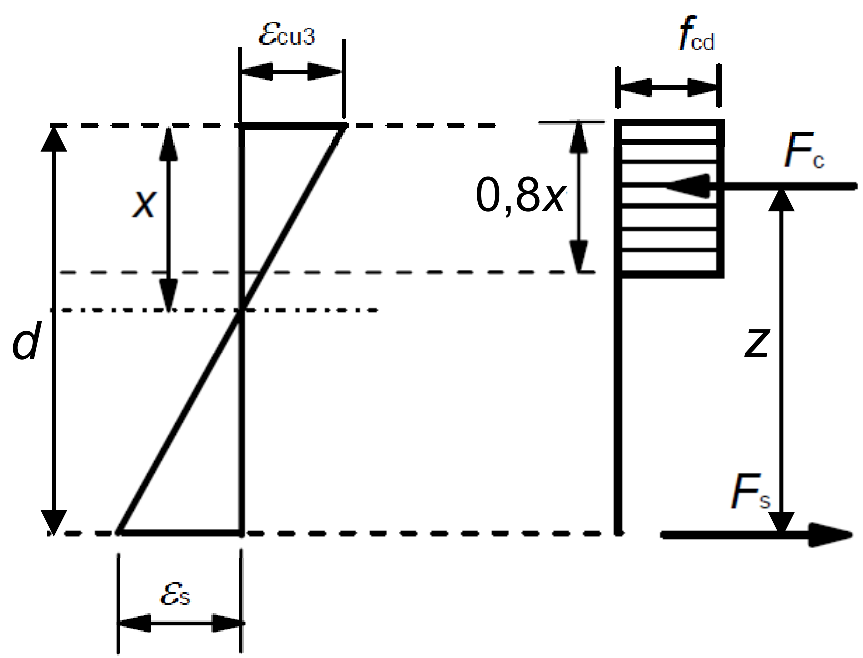
- for the reinforcement, a bi-linear stress-strain relationship with strain hardening (Class B steel bars according to Annex C to EN1992-1-1): f_{yd} = 435 Mpa, k = 1,08 , $\varepsilon_{ud} = 0{,}9.\varepsilon_{uk}$ = 45 mm/m (recommended value)

- for $\varepsilon_s \leq f_{yd} / E_s$ $\sigma_s = E_s \varepsilon_s$
- for $\varepsilon_s \geq f_{yd} / E_s$ $\sigma_s = f_{yd} + (k - 1)\, f_{yd}(\varepsilon_s - f_{yd} / E_s) / (\varepsilon_{uk} - f_{yd} / E_s)$



Bending resistance at ULS

- $\varepsilon_s = \varepsilon_{cu3} (d - x)/x$
- $\sigma_s = f_{yd} + (k - 1)\, f_{yd} (\varepsilon_s - f_{yd} / E_s) / (\varepsilon_{uk} - f_{yd} / E_s)$ (inclined top branch)
- Equilibrium : $N_{Ed} = 0 \Leftrightarrow A_s\sigma_s = 0{,}8 b.x.f_{cd}$ where b = 1 m



- Then, x is the solution of a quadratic equation
- The resistant bending moment is given by:

$$M_{Rd} = 0{,}8 b.x.f_{cd}(d - 0{,}4x)$$
$$= A_s\sigma_s(d - 0{,}4x)$$

Bending resistance at ULS – design example

Section above the main steel girder (absolute values of moments)

- with d = 0,36 m and A_s= 18,48 cm² (φ20 every 0,17 m):
- x = 0,052 m , ε_s = 20,6 mm/m (< ε_{ud}) and σ_s = 448 Mpa
- Therefore M_{Rd} = 0,281 MN.m > M_{Ed} = 0,275 MN.m

Section at mid-span between the main steel girder

- with d = 0,26 m and A_s= 28,87 cm² (φ25 every 0,17 m):
- x = 0,08 m , ε_s = 7,9 mm/m (< ε_{ud}) and σ_s = 439 MPa
- Therefore M_{Rd} = 0,289 MN.m > M_{Ed} = 0,248 MN.m

Local justification of the concrete slab
Verification of the transverse reinforcement

Calulation of normal stresses at SLS

EN 1992-1-1, 7.1(2):

(2) In the calculation of stresses and deflections, cross-sections should be assumed to be uncracked provided that the flexural tensile stress does not exceed $f_{ct,eff}$. The value of $f_{ct,eff}$ maybe taken as f_{ctm} or $f_{ctm,fl}$ provided that the calculation for minimum tension reinforcement is also based on the same value. For the purposes of calculating crack widths and tension stiffening f_{ctm} should be used.

If the flexural tensile stress is not greater than f_{ctm} (3,2 Mpa for C35/45), then it is not necessary to perform a calculation of normal stresses in the cracked section.

Local justification of the concrete slab
Verification of the transverse reinforcement

Stress limitation under SLS characteristic combination

- **The following limitations should be checked (EN1992-1-1, 7.2(5) and 7.2(2)) :**
  $\sigma_s \leq k_3 f_{yk}$ = 0,8x500 = 400 MPa
  $\sigma_c \leq k_1 f_{ck}$ = 0,6x35 = 21 MPa

where k_1 and k_3 are defined by the National Annex to EN1992-1-1
the recommended values are k_1 = 0,6 and k_3 = 0,8

- **These stress calculations are performed neglecting the tensile concrete contribution. The most unfavourable tensile stresses σ_s in the reinforcement are generally provided by the long-term calculations, performed with a modular ratio *n* (reinforcement/concrete) equal to 15. The most unfavourable compressive stresses σ_c in the concrete are generally provided by the short-term calculations, performed with a modular ratio $n = E_s/E_{cm}$ = 5,9 (E_s = 200 GPa for reinforcing steel and E_{cm} = 34 GPa for concrete C35/45).**

Local justification of the concrete slab
Verification of the transverse reinforcement

Stress limitation under SLS characteristic combination

- **The design example in the section above the steel main girder gives d = 0,36 m, A_s = 18,48 cm^2 and M = 0,204 MN.m.**

Using n = 15, σ_s = 344 MPa < 400 MPa is obtained.
Using n = 5,9, σ_c = 15,6 MPa < 21 MPa is obtained.

- **The design example in the section at mid-span between the steel main girders gives d = 0,26 m, A_s = 28,87 cm^2 and M = 0,184 MN.m.**

Using n = 15, σ_s = 287 MPa < 400 MPa is obtained.
Using n = 5,9, σ_c = 20,0 MPa < 21 MPa is obtained.

Local justification of the concrete slab
Verification of the transverse reinforcement

Crack control (SLS)

- **According to EN1992-2, 7.3.1(105), Table 7.101N, the calculated crack width should not be greater than 0,3 mm under quasi permanent combination of actions, for reinforced concrete, whatever the exposure class.**

<table>
<tr><th>Exposure Class</th><th>Reinforced members and prestressed members with unbonded tendons</th><th>Prestressed members with bonded tendons</th></tr>
<tr><td></td><td>Quasi-permanent load combination</td><td>Frequent load combination</td></tr>
<tr><td>X0, XC1</td><td>0,3[1]</td><td>0,2</td></tr>
<tr><td>XC2, XC3, XC4</td><td rowspan="2">0,3</td><td>0,2[2]</td></tr>
<tr><td>XD1, XD2, XD3 XS1, XS2, XS3</td><td>Decompression</td></tr>
<tr><td colspan="3">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.</td></tr>
</table>

Crack control (SLS)

- **In the design example, transverse bending is mainly caused by live loads, the bending moment under quasi-permanent combination is far lesser than the moment under characteristic combinations. It is the same for the tension in reinforcing steel. Therefore, ther is no problem with the control of cracking.**
- **The concrete stresses due to transverse bending under quasi permanent combination, are as follows:**
- **above the steel main girder:**

  M = - 46 kNm/m σ_c = ± 1,7 MPa
- **at mid-span between the main girders:**

  M = 24 kNm/m σ_c = ± 1,5 MPa
- **Since $\sigma_c > - f_{ctm}$, the sections are assumed to be uncracked (EN1992-1-1, 7.1(2)) and there is no need to check the crack openings. A minimum amount of bonded reinforcement is required in areas where tension is expected (EN1992-1-1, 7.3.2)**

Local justification of the concrete slab
Verification of the transverse reinforcement

Crack control (SLS)

- **Minimum reinforcement areas (EN1992-1-1 and EN1992-2 , 7.3.2)**

(1)P If crack control is required, a minimum amount of bonded reinforcement is required to control cracking in areas where tension is expected. The amount may be estimated from equilibrium between the tensile force in concrete just before cracking and the tensile force in reinforcement at yielding or at a lower stress if necessary to limit the crack width

$$A_{s,min}\sigma_s = k_c\, k\, f_{ct,eff}\, A_{ct} \qquad (7.1)$$

- $A_{s,min}$ is the minimum area of reinforcing steel within the tensile zone…
- A_{ct} is the area of concrete within tensile zone...
- σ_s is the absolute value of the maximum stress permitted in the reinforcement immediately after formation of the crack...
- $f_{ct,eff}$ is the mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur: $f_{ct,eff} = f_{ctm}$ or lower,…
- k is the coefficient which allows for the effect of non-uniform self-equilibrating which lead to a reduction of restraint forces…
- k_c is a coefficient which takes account of the stress distribution within the section immediately prior to cracking and of the change of the lever arm…

Crack control (SLS) - design example

- **Minimum reinforcement areas (EN1992-1-1 and EN1992-2 , 7.3.2)**

$$A_{s,min}\sigma_s = k_c\, k\, f_{ct,eff}\, A_{ct} \qquad (7.1)$$

For the design example:

- $A_{ct} = bh/2$ (b = 1 m ; h = 0,40 m above main girder and 0,32 m at mid-span)
- $\sigma_s = f_{yk}$ (lower value only when control of cracking is ensured by limiting bar size or spacing according to 7.3.3)
- $f_{ct,eff}$ = 3,2 MPa (= f_{ctm})
- k = 0,65 (flanges with width ≥ 800 mm)
- k_c = 0,4 (expression 7.2 with σ_c – mean stress of the concrete = 0)

The following areas of reinforcement are obtained:

- **$A_{s,min}$ = 5,12 cm²/m on top face of the slab above the main girder**
- **$A_{s,min}$ = 4,10 cm²/m on bottom face of the slab at mid-span**

Resistance to vertical shear force - ULS

- The shear force calculations are not detailed. The maximum shear force at ULS is obtained in the section located above the steel main girder by applying the traffic load model LM1 between the two steel main girders. This gives V_{Ed} = 235 kN to be resisted by a 1-m-wide slab strip.
- The resistance to vertical shear – without specific shear reinforcement – is obtained by using the formula (6.2a) in EN1992-2:

$$V_{Rd,c} = b_w d \left\{ k_1 \sigma_{cp} + \max \left[C_{Rd,c} k \left(100 \rho_l f_{ck}\right)^{1/3} ; v_{min} \right] \right\} \qquad (2)$$

where:

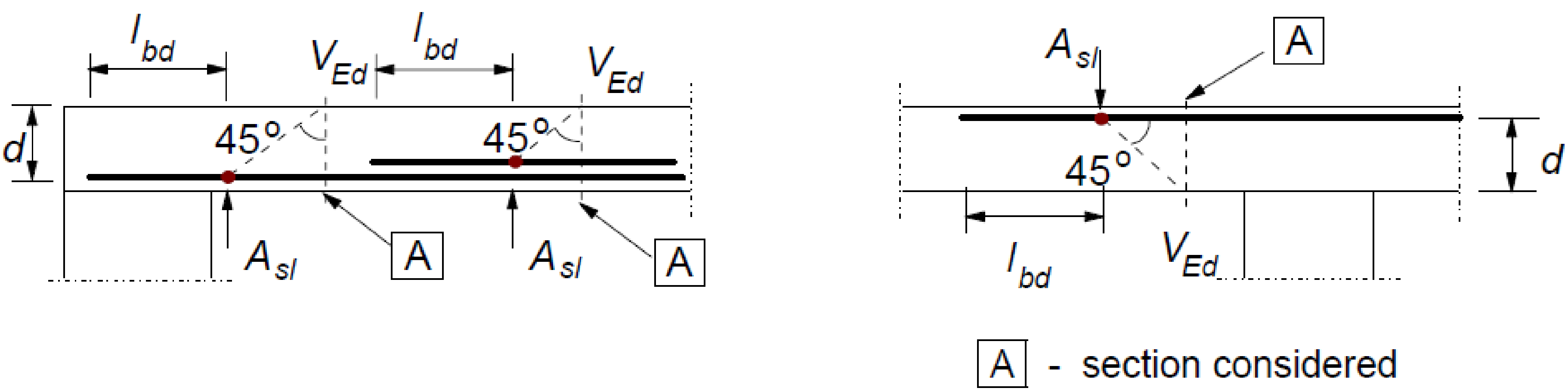
f_{ck} is given in MPa

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2,0 \quad \text{with } d \text{ in mm}$$

$$\rho_l = \frac{A_{sl}}{b_w d} \leq 0,02$$

Resistance to vertical shear force - ULS

- A_{sl} is the area of reinforcement in tension (see Figure 6.3 in EN1992-1-1 for the provisions that have to be fulfilled by this reinforcement). For the design example, A_{sl} represents the transverse reinforcing steel bars of the upper layer in the studied section above the steel main girder. b_w is the smallest width of the studied section in the tensile area. In the studied slab b_w = 1000 mm in order to obtain a resistance $V_{Rd,c}$ to vertical shear for a 1-m-wide slab strip (rectangular section).



Local justification of the concrete slab
Verification of the transverse reinforcement

Resistance to vertical shear force - ULS

- $\sigma_{cp} = \dfrac{N_{Ed}}{A_c} \leq 0{,}2 f_{cd}$ in MPa. This stress is equal to zero where there is no normal force (which is the case in the transverse slab direction in the example).
- The values of $C_{Rd,c}$ and k_1 can be given by the National Annex to EN1992-2. The recommended ones are used:

$$C_{Rd,c} = \frac{0{,}18}{\gamma_c} = 0{,}12$$

$$k_1 = 0{,}15$$

- $v_{min} = 0{,}035.k^{3/2} f_{ck}^{1/2}$

Resistance to vertical shear force – ULS

Design example

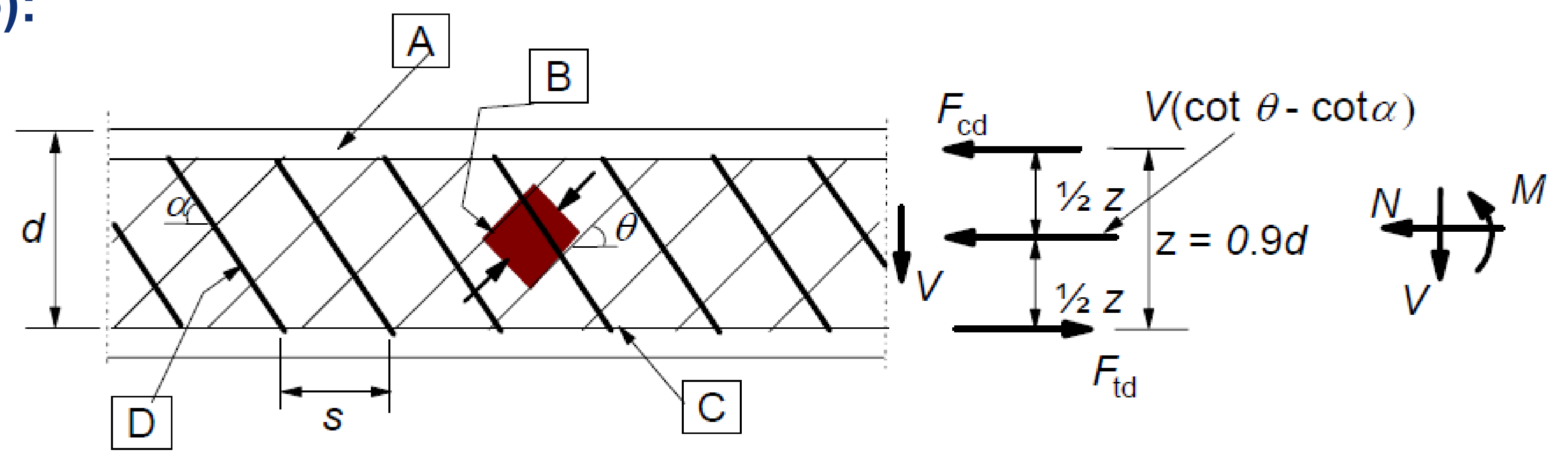
- The design example in the studied slab section above the steel main girder gives successively:
  f_{ck} = 35 MPa; $C_{Rd,c}$ = 0,12; d = 360 mm ;
  $k = 1 + \sqrt{\frac{200}{360}} = 1{,}75$; A_{sl} = 1848 mm² ; b_w = 1000 mm
  $\rho_l = \frac{1848}{1000\text{x}360} = 0{,}51\%$
- $C_{Rd,c}k(100\rho_l f_{ck})^{1/3}$ = 0,55 MPa
  $\sigma_{cp} = 0$
- $v_{min} = 0{,}035\text{x}1{,}75^{3/2}\text{x}35^{1/2}$ = 0,48 MPa < 0,55 Mpa

The shear resistance without shear reinforcement is:

$V_{Rd,c} = C_{Rd,c}k(100\rho_l f_{ck})^{1/3}b_w d$ = 198 kN / m < V_{Ed} = 235 kN / m

Resistance to vertical shear force – ULS

- **According to EN1992, shear reinforcement is needed in the slab, near the main girders. With vertical shear reinforcement, the shear design is based on a truss model (EN1992-1-1 and 1992-2, 6.2.3, fig. 6.5):**



A - compression chord, B - struts, C - tensile chord, D - shear reinforcement

Resistance to vertical shear force – ULS

- **For vertical reinforcement ($\alpha = 90°$), the resistance V_{Rd} is the smaller value of:**

$$V_{Rd,s} = (A_{sw}/s).z.f_{ywd}.\cot\theta \qquad \text{and}$$
$$V_{Rd,max} = \alpha_{cw}\, b_w\, z\, \nu_1\, f_{cd}/(\cot\theta + \tan\theta)$$

where:

- z is the inner lever arm (**z = 0,9d** may normally be used for members without axial force)
- θ is the angle of the compression strut with the horizontal, must be chosen such as **$1 \leq \cot\theta \leq 2{,}5$**
- A_{sw} is the cross-sectional area of the shear reinforcement
- s is the spacing of the stirrups
- f_{ywd} is the design yield strength of the shear reinforcement
- ν_1 is a strength reduction factor for concrete cracked in shear, the recommended value of ν_1 is $\nu = 0{,}6(1-f_{ck}/250)$
- α_{cw} is a coefficient taking account of the interaction of the stress in the compression chord and any applied axial compressive stress; the recommended value of **α_{cw} is 1** for non prestressed members.

Local justification of the concrete slab
Verification of the transverse reinforcement

Resistance to vertical shear force – ULS

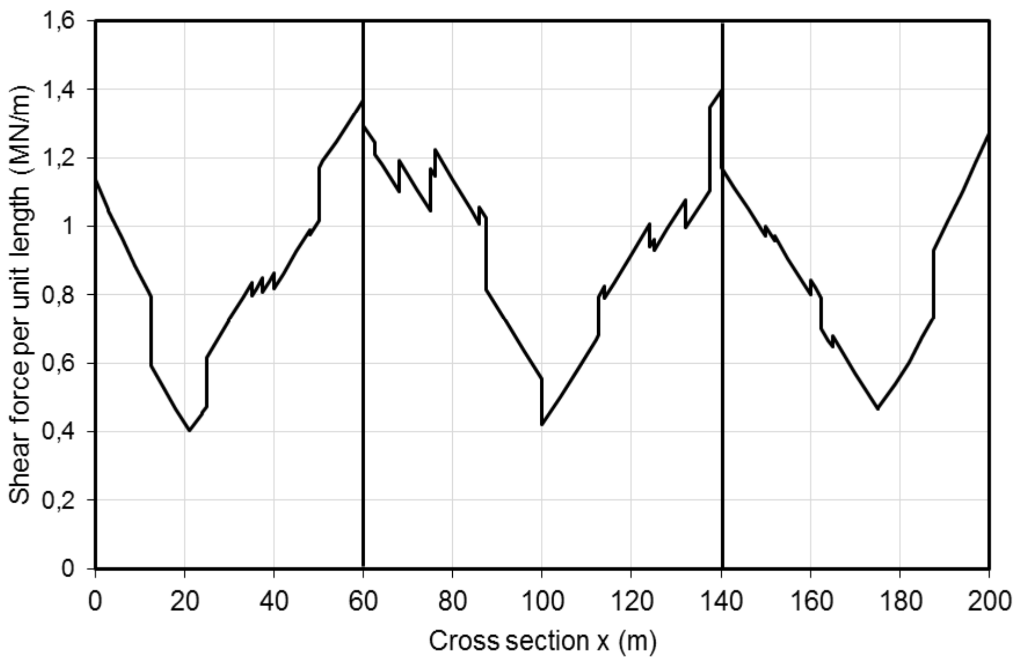
- **In the design example, choosing cotθ = 2,5, with a shear reinforcement area A_{sw}/s = 6,8 cm^2/m for a 1-m-wide slab strip:**

  $V_{Rd,s}$ = 0,00068x(0,9x0,36)x435x2,5 = 240 kN/m > V_{Ed}

  $V_{Rd,max}$ = 1,0x1,0x(0,9x0,36)x0,6x(1 – 35/250)x35/(2,5+0,4) = 2,02 MN/m > V_{ed}

Resistance to longitudinal shear stress - ULS

The longitudinal shear force per unit length at the steel/concrete interface is determined by an elastic analysis at characteristic SLS and at ULS. The number of shear connectors is designed thereof, to resist to this shear force per unit length and thus to ensure the longitudinal composite behaviour of the deck.



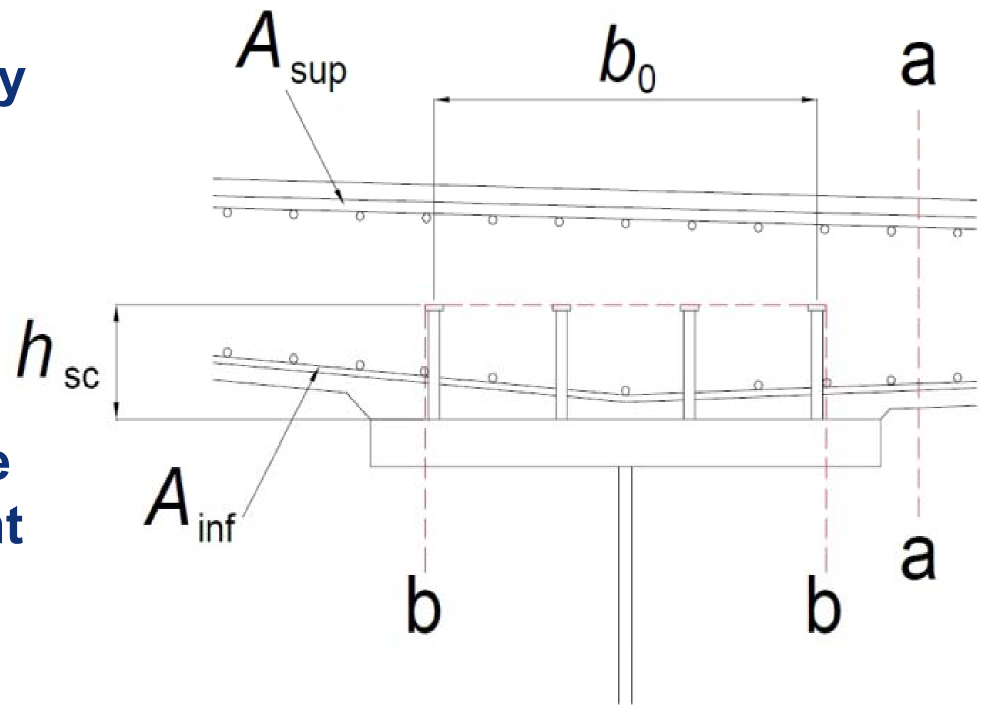
Local justification of the concrete slab
Verification of the transverse reinforcement

Resistance to longitudinal shear stress - ULS

At ULS this longitudinal shear stress should also be resisted to for any potential surface of longitudinal shear failure within the slab.
Two potential surfaces of shear failure are defined in EN1994-2, 6.6.6.1, fig 6.15:

- **surface a-a holing only once by the two transverse reinforcement layers,**
  As = Asup + Ainf
  there are 2 surfaces a-a.

- **surface b-b holing twice by the lower transverse reinforcement layers, As = 2.Ainf**

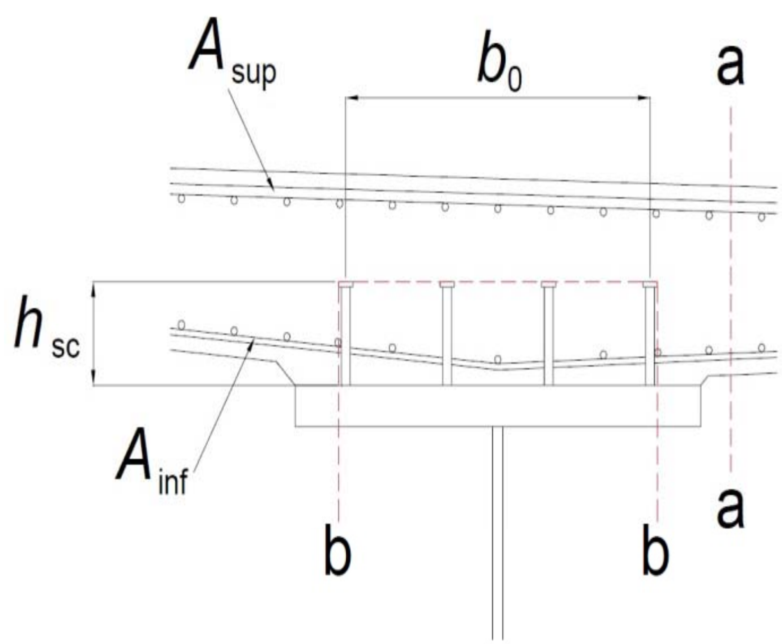


Resistance to longitudinal shear stress - ULS

The maximum longitudinal shear force per unit length resisted to by the shear connectors is equal to 1,4 MN/m. This value is used here for verifying shear failure within the slab. The shear force and on each potential failure surface is as follows

- **surface a-a, on the cantilever side : 0,59 MN/m**
- **surface a-a, on the central slab side : 0,81 MN/m**

- **surface b-b : 1,4 MN/m**



Local justification of the concrete slab
Verification of the transverse reinforcement

Resistance to longitudinal shear stress - ULS

Failure surfaces a-a:

- **The shear resistance is determined according to EN1994-2, 6.6.6.2(2), which refers to EN1992-1-1, 6.2.4, fig. 6.7 (see below), the resulting shear stress is :**

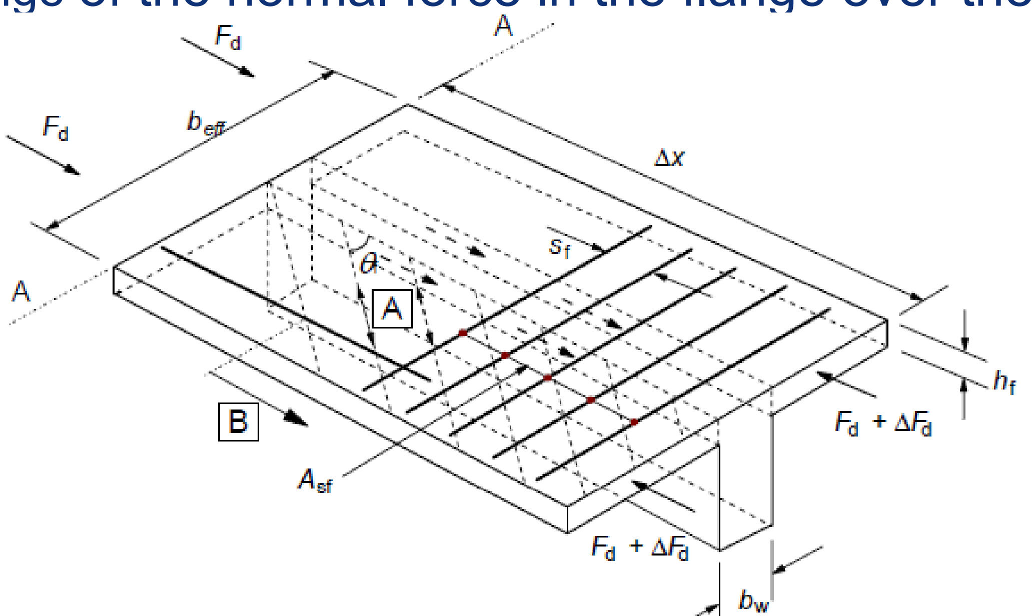
$$v_{Ed} = \Delta F_d/(h_f \cdot \Delta x)$$

where:

h_f is the thickness of flange at the junctions

Δx is the length under consideration, see Figure 6.7

ΔF_d is the change of the normal force in the flange over the length Δx.



A - compressive struts B - longitudinal bar anchored beyond this projected point

Local justification of the concrete slab
Verification of the transverse reinforcement

Resistance to longitudinal shear stress - ULS

- The calculation is made only for the surface a-a on the central slab side, where the longitudinal shear is higher than on cantilever side.
  Shear stress: $v_{Ed} = \Delta F_d/(h_f \cdot \Delta x) = 0{,}81/0{,}40 = 2{,}03$ MPa ($h_f = 0{,}40$ m)

Two different verifications should be carried out:

- the transverse reinforcement should be designed to resist to the tensile force:

$$v_{Ed} h_f \tan\theta_f \leq \frac{A_s}{s} f_{yd}$$

where s is the spacing between the transverse reinforcing steel bars and A_s is the corresponding area within the 1-m-wide slab strip.

- the crushing should be prevented in the concrete compressive struts:

$$v_{Ed} \leq \nu f_{cd} \sin\theta_f \cos\theta_f$$

with $\nu = 0{,}6\left(1 - \frac{f_{ck}}{250}\right)$ and f_{ck} in MPa (strength reduction factor for the concrete cracked in shear)

Resistance to longitudinal shear stress - ULS

As the concrete slab is in tension in the longitudinal direction of the deck, the angle θ_f for the concrete compressive strut should be limited to cotan θ_f = 1,25 i.e. θ_f = 38,65°.

- **For the design example, above the steel main girder, A_s/s = 30,3 cm²/m. The previous criterion is thus verified:**

$$A_s/s \geq \frac{v_{Ed}\, h_f}{f_{yd}\, \text{cotan}\, \theta_f} = 0{,}81/(435 \text{x} 1{,}25) = 14{,}9 \text{ cm}^2/\text{m}$$

- **The second criterion is also verified for the failure surface a-a.**

$$v_{Ed} = 2{,}03 \text{ MPa} \leq \nu . f_{cd} . \sin\theta_f . \cos\theta_f = 6{,}02 \text{ MPa.}$$

Resistance to longitudinal shear stress - ULS

Failure in shear plane b-b

- The length of this shear surface is calculated by encompassing the studs as closely as possible within 3 straigth lines (see Figure 4(a)):
  $h_f = 2h_{sc} + b_0 + \phi_{head} = 2x0{,}200 + 0{,}75 + 0{,}035 = 1{,}185$ m.
- The shear stress for the surface b-b of shear failure is equal to:
  $v_{Ed} = 1{,}4/1{,}185 = 1{,}18$ Mpa

For the design example, the two previous criteria are justified:

$A_s/s = 23{,}65$ cm²/m (two layers of high bond bars with a 16 mm diameter and a spacing $s = 170$ mm)

$$A_s/s \geq \frac{v_{Ed} h_f}{f_{sd}.\text{cotan}\,\theta_f} = 1{,}4/(435x1{,}25) = 25{,}75 \text{ cm2/m}$$

$v_{Ed} = 1{,}18 \text{ MPa} \leq \nu.f_{cd}.\sin\theta_f.\cos\theta_f = 6{,}02$ MPa

Local justification of the concrete slab
Verification of the transverse reinforcement

Interaction shear/transverse bending - ULS

The traffic load models are such that they can be arranged on the pavement to provide a maximum longitudinal shear flow and a maximum transverse bending moment simultaneously. EN1992-2, 6.2.4 (105) sets the following rules to take account of this concomitance:

- **the criterion for preventing the crushing in the compressive struts is verified with a height h_f reduced by the depth of the compressive zone considered in the transverse bending assessment (as this concrete is worn out under compression, it cannot simultaneously take up the shear stress);**
- **the total reinforcement area should be not less than $A_{flex} + A_{shear}/2$ where A_{flex} is the reinforcement area needed for the pure bending assessment and A_{shear} is the reinforcement area needed for the pure longitudinal shear flow.**

Local justification of the concrete slab
Verification of the transverse reinforcement

Interaction shear/transverse bending - ULS

Crushing in the compressive struts

The compression in the struts is much lower than the limit. The reduction in h_f is not a problem therefore.

- **shear plane a-a:**

  $h_{f,red} = h_f - x_{ULS}$ = 0,40-0,05 = 0,35 m

  $v_{Ed,red} = v_{Ed}.h_f/h_{red}$ = 0,81/0,35 = 2,31 MPa ≤ 6,02 MPa

- **shear plane b-b:**

  $h_{f,red} = h_f - 2x_{ULS}$ = 1,185 – 2x0,.05 = 1,085 m

  $v_{Ed,red} = v_{Ed}.h_f/h_{red}$ = 1,4/1,085 = 1,29 MPa ≤ 6,02 Mpa

Total reinforcement area:

A_{flex} = 18,1 cm²/m required

A_{shear} = 14,9 cm²/m required

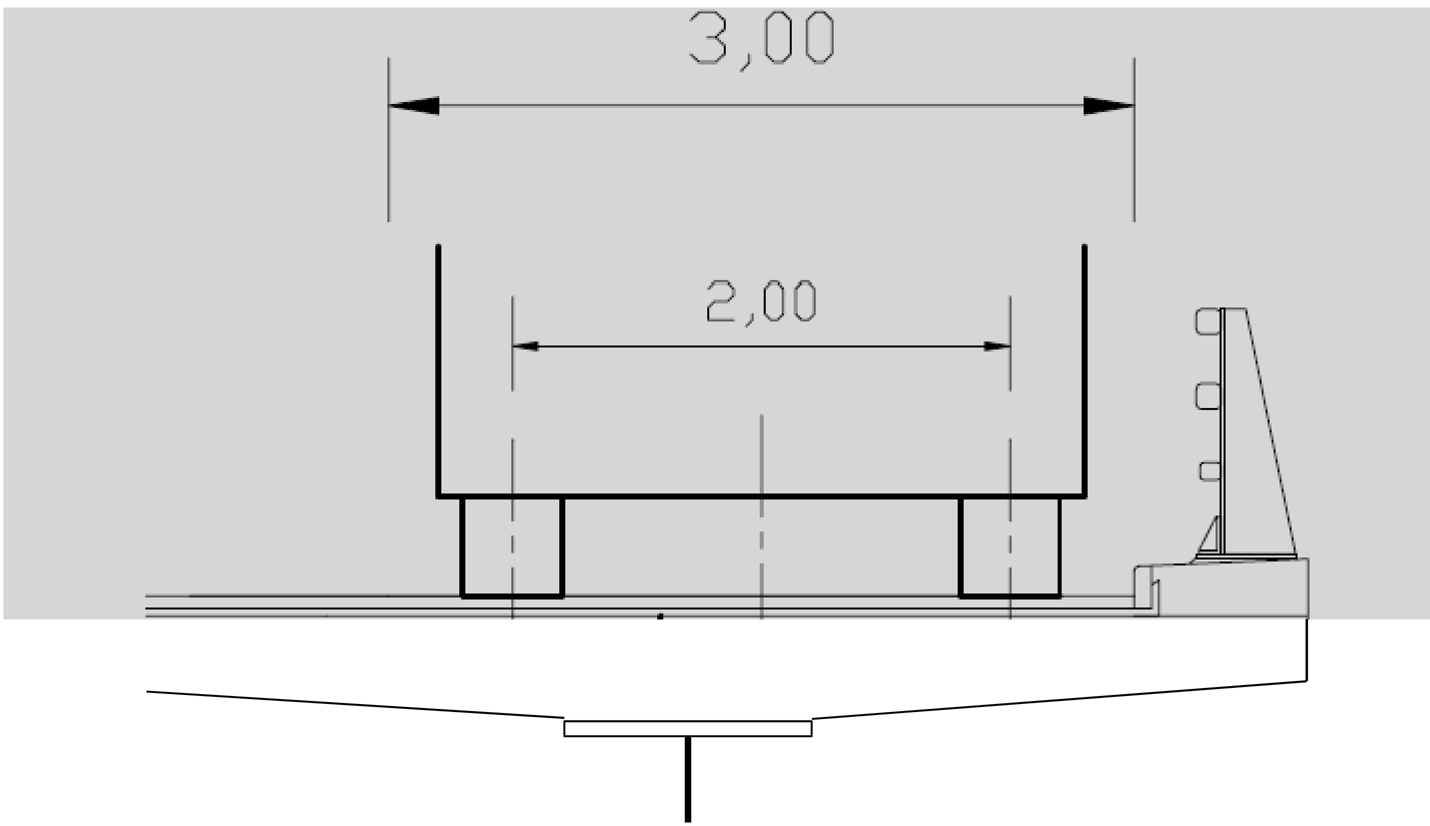
A_{shear} /2 + A_{flex} = 25,6 cm²/m

A_s = 30,3 cm²/m : the criterion is saatisfied

Local justification of the concrete slab
Verification of the transverse reinforcement

ULS of fatigue – transverse bending

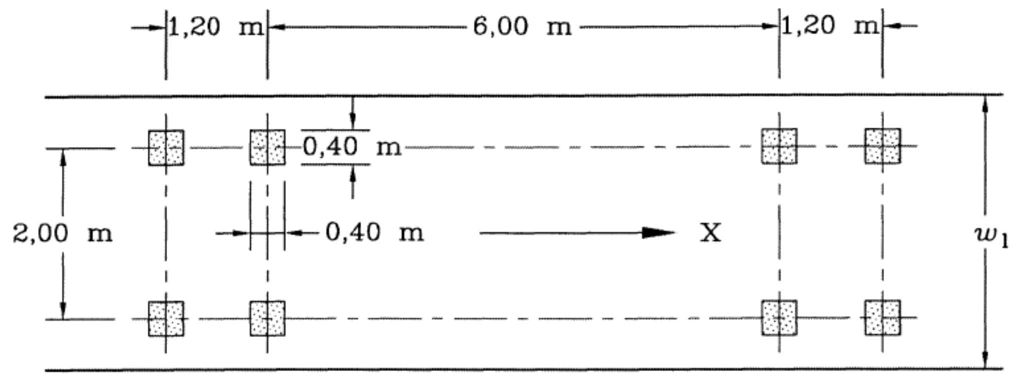
The slow lane is assumed to be close to the safety barrier and the the fatigue load is centered on this lane



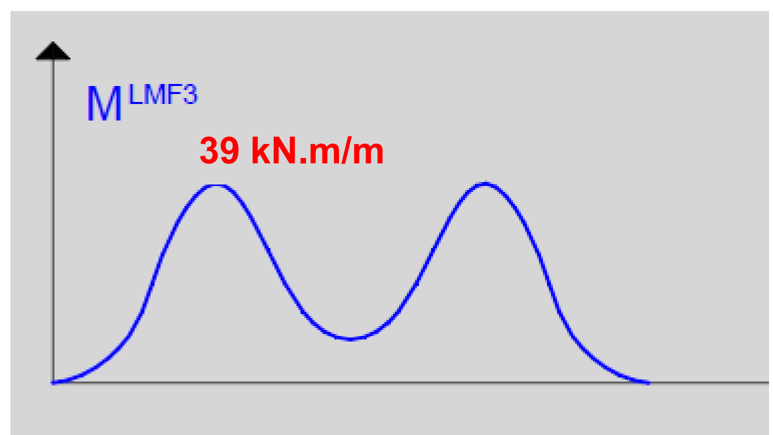
ULS of fatigue – transverse bending

Fatigue load model FLM3 is used. Verification are performed by the damage equivalent stress range method (EN1992-1-1, 6.8.5 and EN1992-2, Annex NN)

FLM3 (Axle loads 120 kN)



Variation of transverse bending moment above main steel girder during the passage of FLM 3



$\Delta\sigma_s$**(FLM3) = 63 Mpa**

Local justification of the concrete slab
Verification of the transverse reinforcement

ULS of fatigue – transverse bending

Damage equivalent stress range method

EN1992-1-1, 6.8.5:

(3) For reinforcing or prestressing steel and splicing devices adequate fatigue resistance should be assumed if the Expression (6.71) is satisfied:

$$\gamma_{F,fat} \cdot \Delta\sigma_{S,equ}(N^*) \leq \frac{\Delta\sigma_{Rsk}(N^*)}{\gamma_{s,fat}} \tag{6.71}$$

where:

$\Delta\sigma_{Rsk}(N^*)$ is the stress range at N^* cycles from the appropriate S-N curves given in Figure 6.30.

Note: See also Tables 6.3N and 6.4N.

$\Delta\sigma_{S,equ}(N^*)$ is the damage equivalent stress range for different types of reinforcement and considering the number of loading cycles N^*. For building construction $\Delta\sigma_{S,equ}(N^*)$ may be approximated by $\Delta\sigma_{S,max}$.

$\Delta\sigma_{S,max}$ is the maximum steel stress range under the relevant load combinations

ULS of fatigue – transverse bending

Damage equivalent stress range method

$$\gamma_{F,fat} \cdot \Delta\sigma_{S,equ}(N^*) \leq \frac{\Delta\sigma_{Rsk}(N^*)}{\gamma_{s,fat}}$$

- $\gamma_{F,fat}$ is the partial factor for fatigue load (EN1992-1-1, 2.4.2.3) . The recommended value is 1,0
- $\Delta\sigma_{Rsk}$ **(*N***) = 162,5 MPa (EN1992-1-1, table 6.3N)
- $\gamma_{s,fat}$ is the partial factor for reinforcing steel (EN1992-1-1, 2.4.2.4). The recommended value is 1,15.

Local justification of the concrete slab
Verification of the transverse reinforcement

ULS of fatigue – transverse bending

Annex NN – NN.2.1 (102)

$$\Delta\sigma_{s,equ} = \Delta\sigma_{s,Ec}.\lambda_s$$

where

$\Delta\sigma_{s,Ec} = \Delta\sigma_s(1{,}4.FLM3)$ (stress range due to 1,4 times FLM3, in the case of pure bending, it is equal to 1,4 $\Delta\sigma_s(FLM3)$. For a verification of fatigue on intermediate supports of continuous bridges, the axle loads of FLM3 are multiplied by 1,75

λ_s is the damage coefficient.

$$\lambda_s = \varphi_{fat}.\lambda_{s,1}.\ \lambda_{s,2}.\ \lambda_{s,3}.\ \lambda_{s,4}$$

where φ_{fat} is a dynamic magnification factor

$\lambda_{s,1}$ takes account of the type of member and the length of the influence line or surface

$\lambda_{s,2}$ takes account of the volume of traffic

$\lambda_{s,3}$ takes account of the design working life

$\lambda_{s,4}$ takes account of the number of loaded lanes

Local justification of the concrete slab
Verification of the transverse reinforcement

ULS of fatigue – transverse bending

Annex NN – NN.2.1 (104)

$\lambda_{s,1}$ is given by figure NN.2, curve 3c) . In the design example, the length of the influence line is 2,5 m. Therefore $\lambda_{s,1} \approx 1,1$

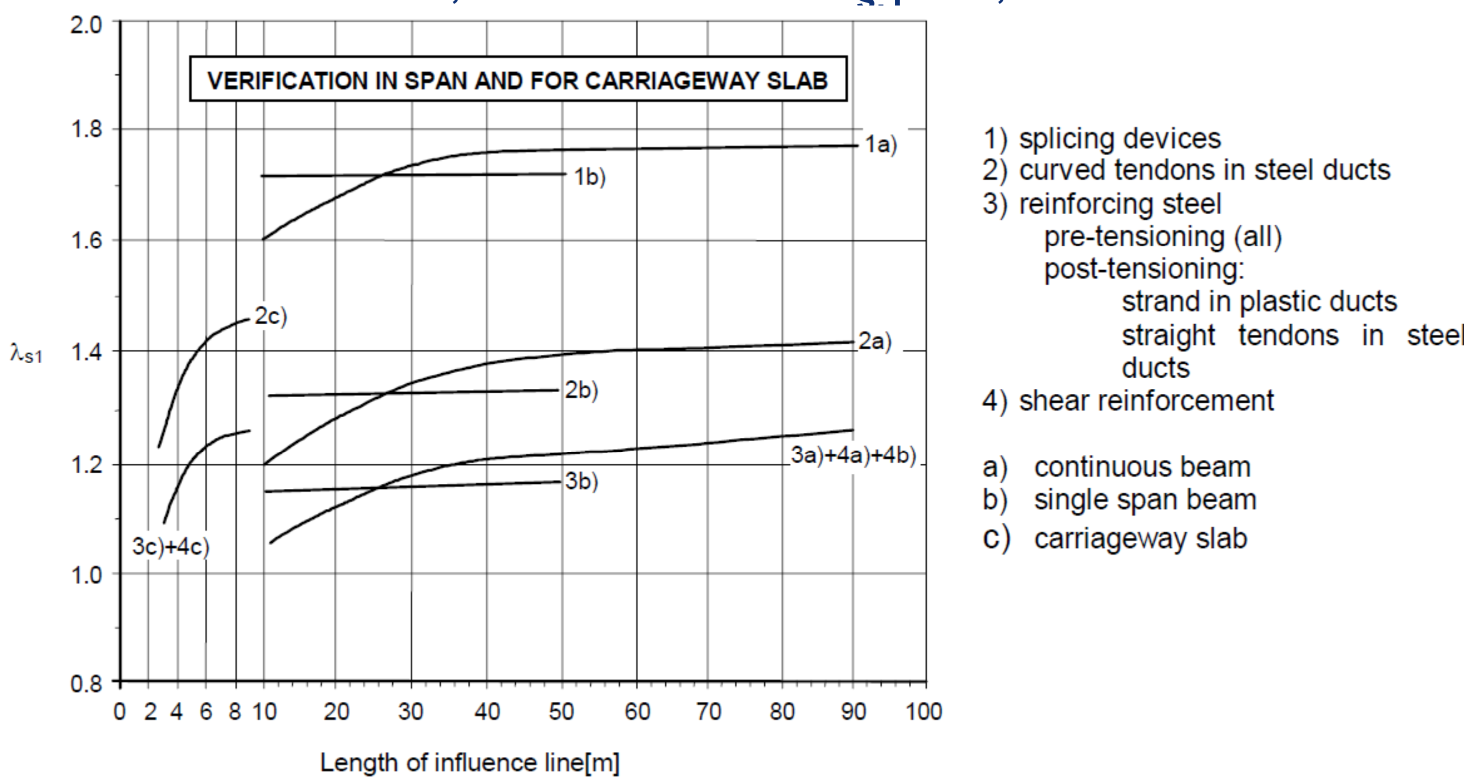


Figure NN.2: $\lambda_{s,1}$ value for fatigue verification in span and for local elements

Local justification of the concrete slab
Verification of the transverse reinforcement

ULS of fatigue – transverse bending

Annex NN – NN.2.1 (105)

$$\lambda_{s,2} = \overline{Q}\sqrt[k_2]{\frac{N_{obs}}{2.0}} \qquad \text{(NN.103)}$$

where:

N_{obs} number of lorries per year according to EN 1991-2, Table 4.5

k_2 slope of the appropriate S-N-Line to be taken from Tables 6.3N and 6.4N of EN 1992-1-1

$\overline{Q}$ factor for traffic type according to Table NN.1

Table NN.1 – Factors for traffic type

$\overline{Q}$ - factor for	Traffic type (see EN 1991-2 Table 4.7)		
	Long distance	Medium distance	Local traffic
$k_2 = 5$	1.0	0.90	0.73
$k_2 = 7$	1.0	0.92	0.78
$k_2 = 9$	1.0	0.94	0.82

$k_2 = 9$ (table 6.3 N); $N_{obs} = 0,5.10^6$ (EN1991-2, table 4.5); Q = 0,94

$\lambda_{s,2}$ **= 0,81**

ULS of fatigue – transverse bending

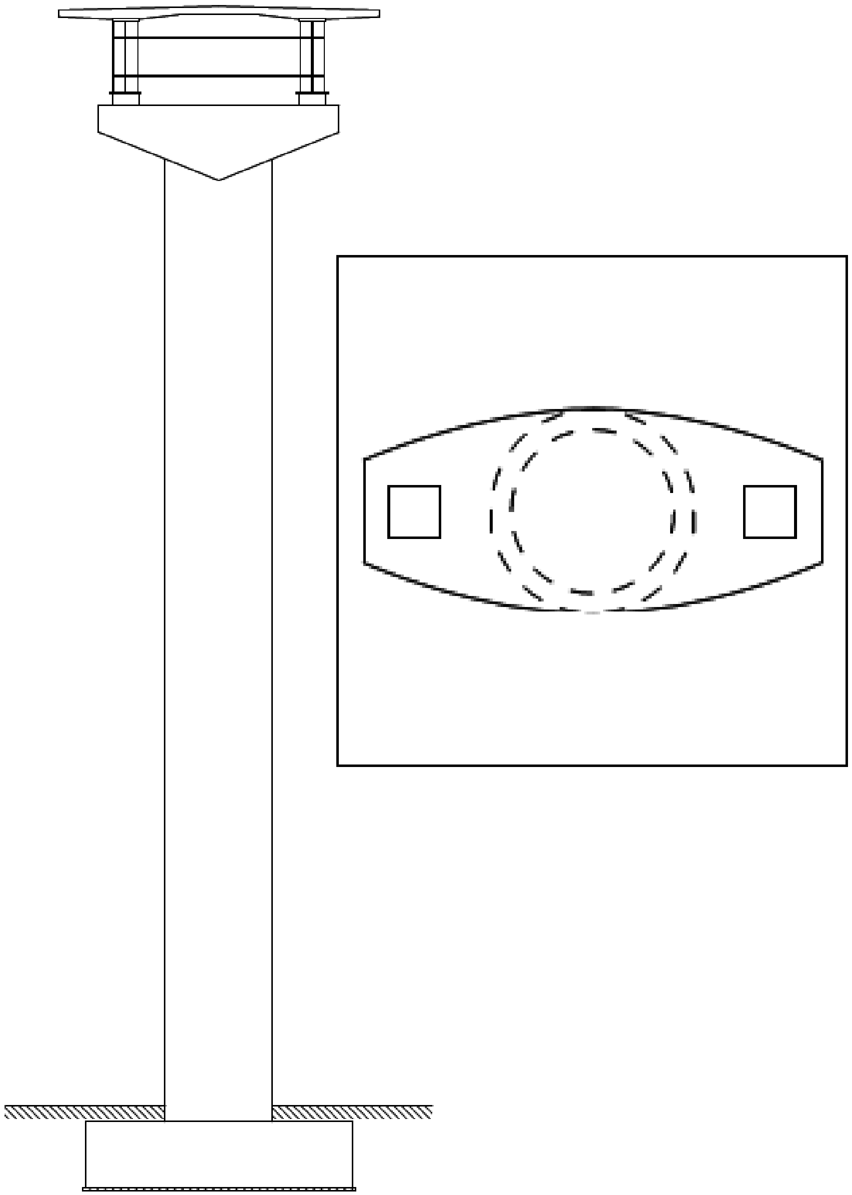
Annex NN – NN.2.1 (106) and (107)

$\lambda_{s,3}$ **= 1** (design working life = 100 years)
$\lambda_{s,4}$ **= 1** (different from one if more than one lane are loaded)
φ_{fat} **=** 1,0 except for the areas close to the expansion joints
where φ_{fat} = 1,3

It comes:
λ_s = 0,89 (1,16 near the expaxsion joints)
$\Delta\sigma_{s,Ec}$ = 1,4x63 = 88 MPa

$\Delta\sigma_{s,eq}$ = 78 MPa (102 near the expansion joints)
$\Delta\sigma_{Rsk} / \gamma_{s,fat}$ = 162,5/1,15 = 141 MPa > 102 MPa
The resistance of reinforcement to fatigue under transverse bending is verified

Second order effects in the high piers



Pier height : 40 m

Pier shaft

- **external diameter : 4 m**
- **wall thickness : 0,40**
- **longitudinal reinforcement: 1,5%**
- A_c **= 4,52 m²**
- I_c **= 7,42 m⁴**
- A_s **= 678 cm²**
- I_s **= 0,110 m⁴**

Pier head:

- **volume : 54 m³**
- **Weight : 1,35 MN**

Concrete

- **C35/45**
- f_{ck} **= 35 MPa**
- E_{cm} **= 34000 MPa**

Second order effects in the high piers

Forces and moments on top of the piers are calculated assuming that the inertia of the piers is equal to 1/3 of the uncracked inertia. Two ULS combinations are taken into account:

- **Comb 1: 1,35G + 1,35(UDL + TS) + 1,5(0,6F_{wkT}) (transverse direction)**
- **Comb 2: 1,35G + 1,35(0,4UDL+ 0,75TS + braking) + 1,5(0,6T_k) (longitudinal direction)**

	F_z (vertical)	F_y (trans.)	F_x (long.)	M_x(trans.)
G	14,12 MN	0	0	0
UDL	3,51 MN	0	0	8,44 MN.m
TS	1,21 MN	0	0	2,42 MN.m
Braking	0	0	0,45 MN	0
F_{wkT} (wind on trafic)	0	0,036 MN	0	0,11 MN.m
T_K	0	0	0,06 MN	0
Comb 1	**25,43 MN**	**0,032 MN**	**0**	**14,76 MN.m**
Comb 2	**22,18 MN**	**0**	**0,66 MN**	**7,01 MN.m**

Second order effects in the high piers

The second order effects are analysed by a simplified method: EN1992-1-1, 5.8.7 - method based on nominal stiffness. The analysis is performed in longitudinal direction.

Geometric imperfection (EN1992-1-1, 5.2(5):

$$\theta_l = \theta_0 \alpha_h$$

where

θ_0 = 1/200 (recommended value)

$\alpha_h = 2/l^{1/2}$; $2/3 \leq \alpha_h \leq 1$

l is the height of the pier = 40 m

θ_l = 0,0016 resulting in a moment under permanent combination M_{0Eqp} = 1,12 MN.m at the base of the pier

Second order effects in the high piers

First order moment at the base of the pier:

- $M_{0Ed} = 1{,}35\ M_{0Ed} + 1{,}35\ F_z\,(0{,}4\text{UDL} + 0{,}75\text{TS}).l.\theta_l + 1{,}35\ F_x(\text{braking}).l + 1{,}5(0{,}6F_x(\text{T}_k).l$

$$M_{0Ed} = 28{,}2\ \text{MN.m}$$

- Effective creep ratio (EN 1992-1-1, 5.8.4 (2)):

$$\varphi_{ef} = \varphi_{(\infty,t0)} \cdot M_{0Eqp} / M_{0Ed} \qquad (5.19)$$

where:

$\varphi_{(\infty,t0)}$ is the final creep coefficient according to 3.1.4
M_{0Eqp} is the first order bending moment in quasi-permanent load combination (SLS)
M_{0Ed} is the first order bending moment in design load combination (ULS)

$$\varphi_{ef} = 2.(1{,}12/28{,}2) = 0{,}08$$

Second order effects in the high piers

Nominal stiffness (EN1992-1-1, 5.8.7.2 (1)

$$EI = K_c E_{cd} I_c + K_s E_s I_s \tag{5.21}$$

where:

E_{cd} is the design value of the modulus of elasticity of concrete, see 5.8.6 (3)
I_c is the moment of inertia of concrete cross section
E_s is the design value of the modulus of elasticity of reinforcement, 5.8.6 (3)
I_s is the second moment of area of reinforcement, about the centre of area of the concrete
K_c is a factor for effects of cracking, creep etc, see 5.8.7.2 (2) or (3)
K_s is a factor for contribution of reinforcement, see 5.8.7.2 (2) or (3)

$E_{cd} = E_{cm}/\gamma_{cE}$ = 34000/1,2 = 28300 MPa
I_c = 7,42 m^4
E_s = 200000 MPa
I_s = 0,110 m^4
K_s = 1

Second order effects in the high piers

Nominal stiffness (EN1992-1-1, 5.8.7.2 (1)

$K_c = k_1 k_2 / (1 + \varphi_{ef})$

where:

- ρ is the geometric reinforcement ratio, A_s/A_c
- A_s is the total area of reinforcement
- A_c is the area of concrete section
- φ_{ef} is the effective creep ratio, see 5.8.4
- k_1 is a factor which depends on concrete strength class, Expression (5.23)
- k_2 is a factor which depends on axial force and slenderness, Expression (5.24)

$$k_1 = \sqrt{f_{ck} / 20} \text{ (MPa)}$$

$$k_2 = n \cdot \frac{\lambda}{170} \leq 0{,}20$$

where:

- n is the relative axial force, $N_{Ed} / (A_c f_{cd})$
- λ is the slenderness ratio, see 5.8.3

Nominal stiffness (EN1992-1-1, 5.8.7.2 (1)

ρ = 0,015
k_1 = 1,32
N_{Ed} = 22,18
n = 22,18/(4,52.19,8) = 0,25
$\lambda = l_0/i$; $l_0 = 1{,}43.l$ = 57,20 m (taking into account the rigidity of the second pier) ; $i = (I_c/A_c)^{0,5}$ = 1,28 m
λ = 45
k_2 = 0,25.(45/170) = 0,066

K_c= 1,32x0,066/1,08 = 0,081

EI = 39200 MN.m^2 (= $Ei_{uncracked}$/6)

Second order effects in the high piers

.Moment magnification factor (EN1992-1-1, 5.8.7.3)

(1) The total design moment, including second order moment, may be expressed as a magnification of the bending moments resulting from a linear analysis, namely:

$$M_{\mathrm{Ed}} = M_{0\mathrm{Ed}}\left[1+\frac{\beta}{(N_{\mathrm{B}}/N_{\mathrm{Ed}})-1}\right] \qquad (5.28)$$

where:

- $M_{0\mathrm{Ed}}$ is the first order moment; see also 5.8.8.2 (2)
- β is a factor which depends on distribution of 1st and 2nd order moments, see 5.8.7.3 (2)-(3)
- N_{Ed} is the design value of axial load
- N_B is the buckling load based on nominal stiffness

(2) For isolated members with constant cross section and axial load, the second order moment may normally be assumed to have a sine-shaped distribution. Then

$$\beta = \pi^2 / c_0 \qquad (5.29)$$

Second order effects in the high piers

Moment magnification factor (EN1992-1-1, 5.8.7.3)

M_{0Ed} = 28,2 MN.m
β = 0,85 (c_0 = 12)
$N_B = \pi^2 EI/l_0^2$ = 118 MN
N_{Ed} = 26 MN (mean value on the height of the pier)

$M_{Ed} = 1{,}23\ M_{0Ed}$ = 33,3 MN.m

Thank you for your kind attention

Application of external prestressing to steel-concrete composite two girder bridge

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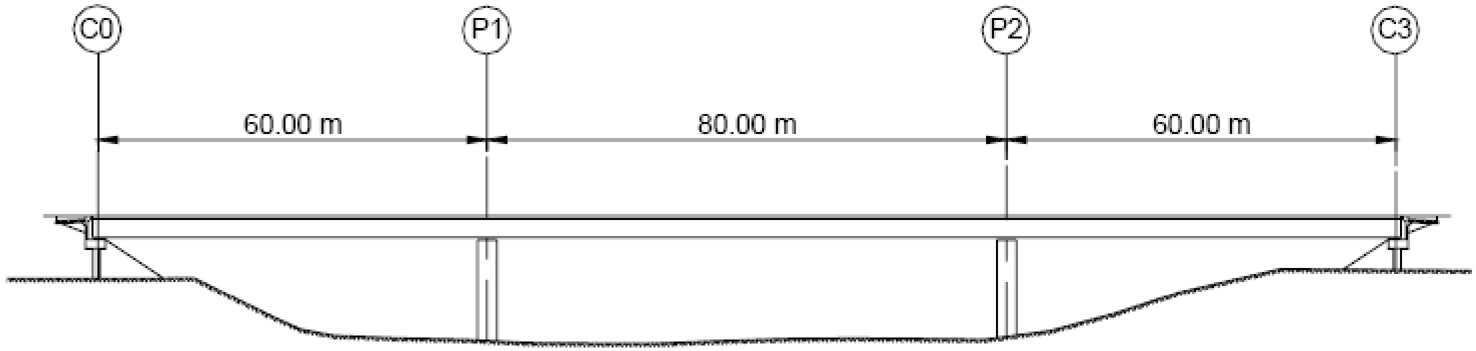
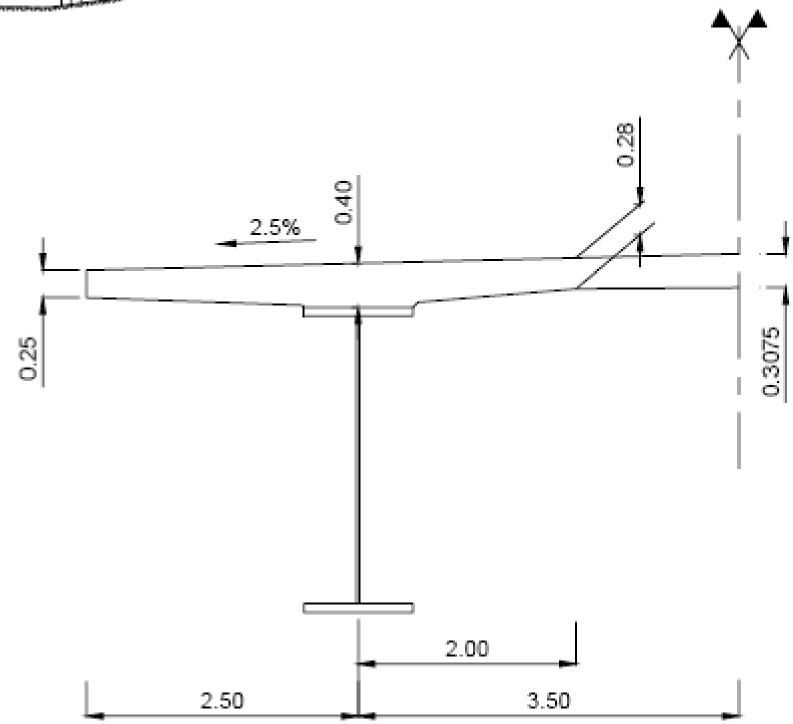
Application of external prestressing to steel-concrete composite two girder bridge

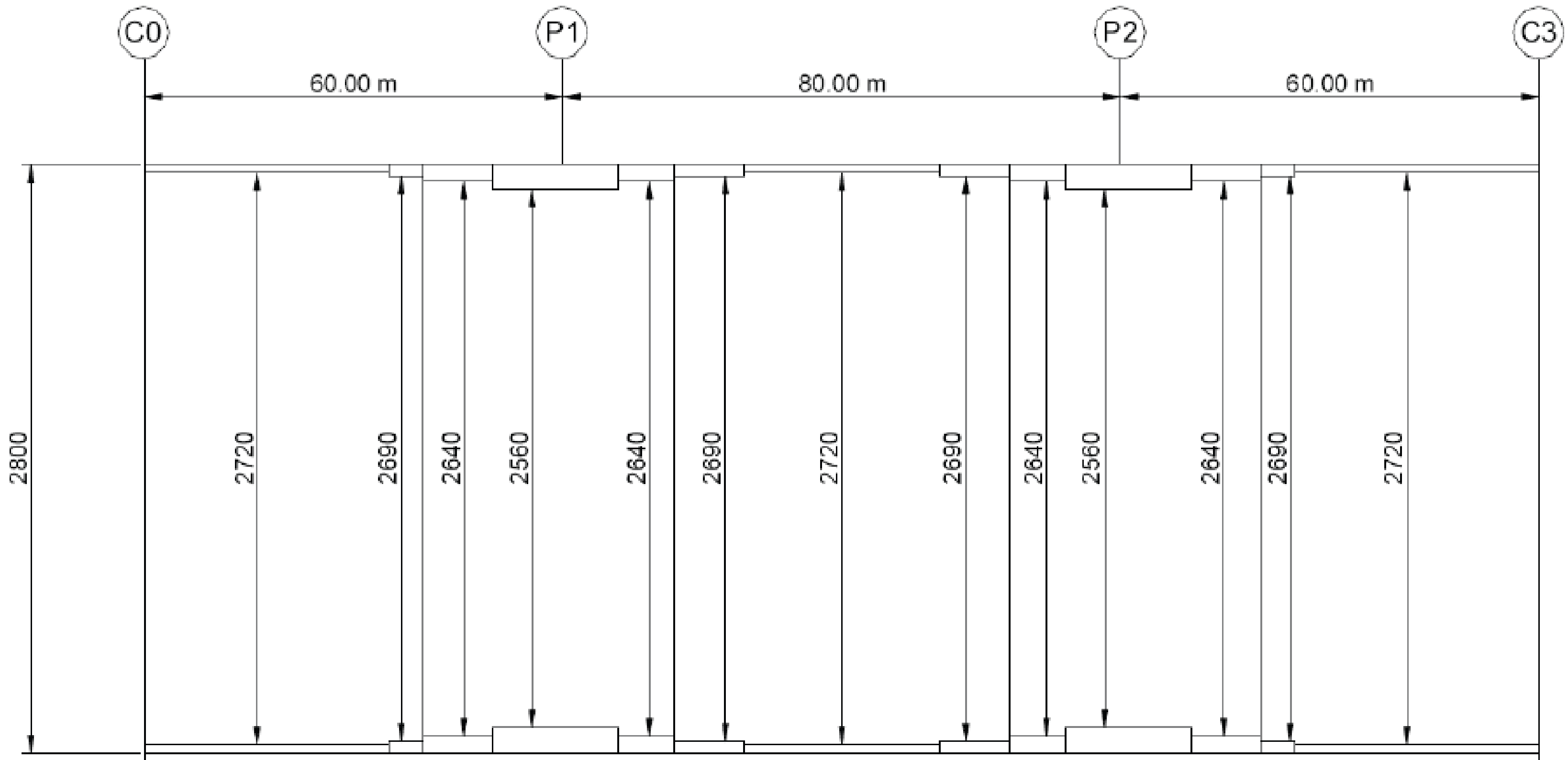




Span Distribution
&
Basic geometry of cross section

Application of external prestressing to steel-concrete composite two girder bridge

Structural steel distribution for main girder



UPPER FLANGE Constant width 1000 mm	35000 x 40	5000 x 55	10000 x 80	18000 x 120	8000 x 80	10000 x 55	28000 x 40	10000 x 55	8000 x 80	18000 x 120	10000 x 80	5000 x 55	35000 x 40
WEB	18 x 40000		26 x 36000			18 x 48000			26 x 36000			18 x 40000	
LOWER FLANGE Constant width 1200 mm	35000 x 40	5000 x 55	10000 x 80	18000 x 120	8000 x 80	10000 x 55	28000 x 40	10000 x 55	8000 x 80	18000 x 120	10000 x 80	5000 x 55	35000 x 40

The not prestressed original solution has been compared with 4 different solutions with external prestressing.

Comparison has been done taking into account only:

1. SLU bending and axial force verification during construction phases and in service
2. SLU Fatigue verification
3. SLS crack control

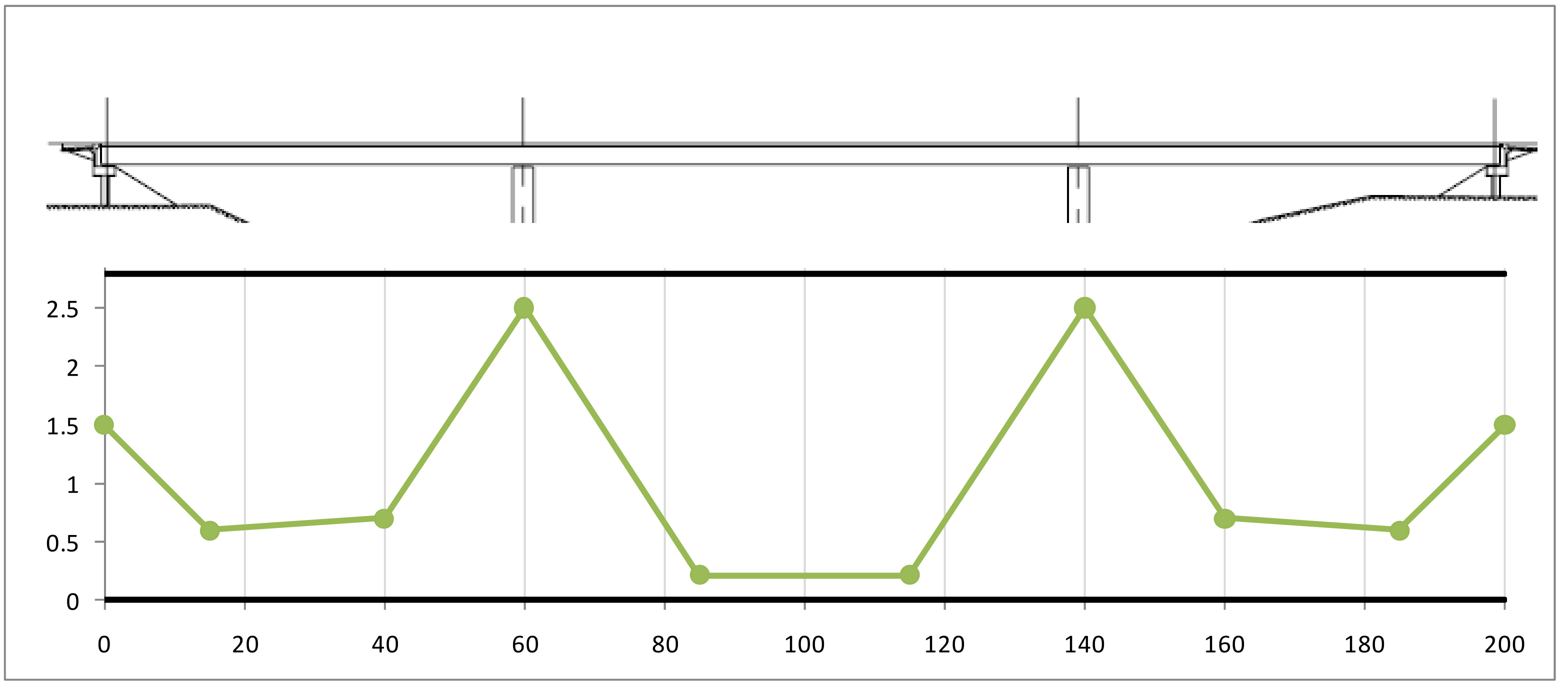
Goals of the work:

1. Reduction of steel girder
2. Reduction of slab longitudinal ordinary steel
3. Avoiding cracks in the upper slab

Application of external prestressing to steel-concrete composite two girder bridge

1st prestressing layout
4x22 ϕ0.6” strand tendons on green layout

100% prestressing applied to steel girder



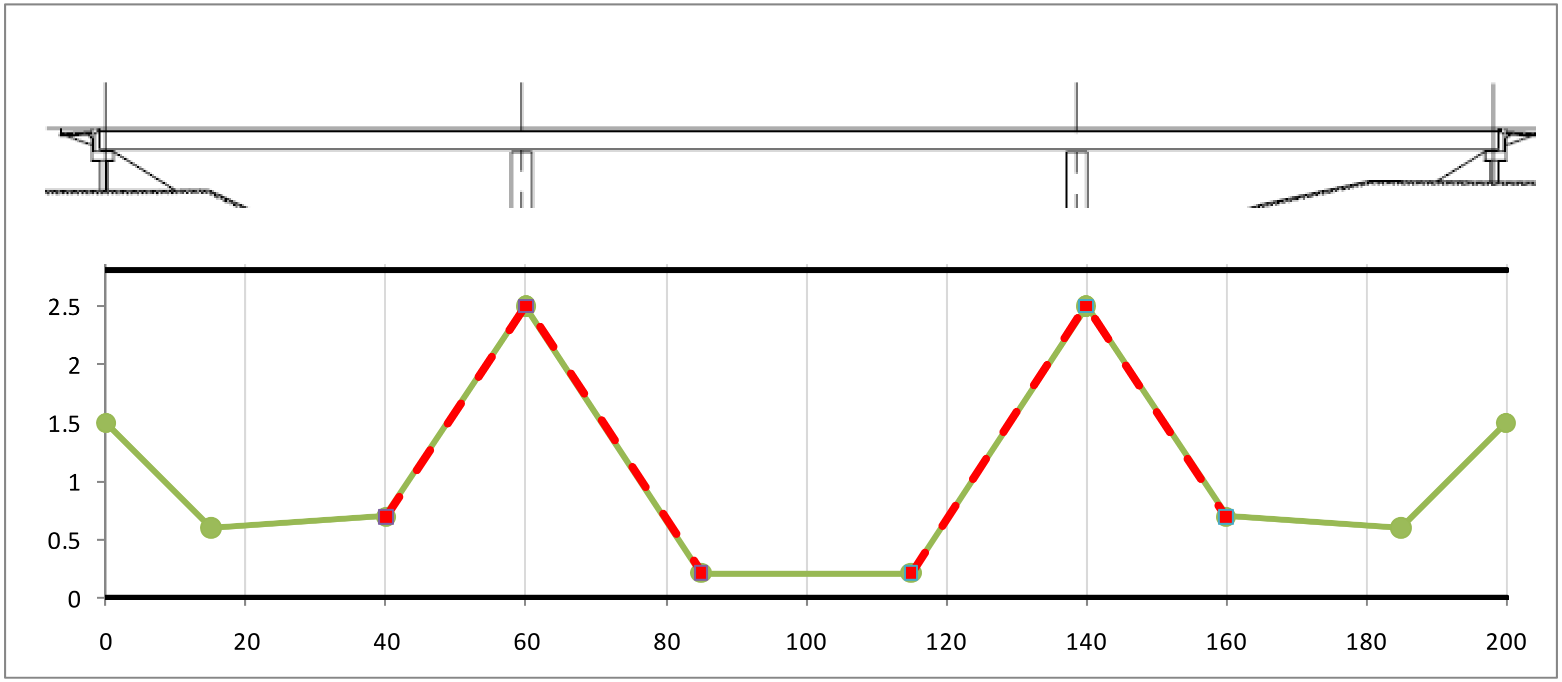
Application of external prestressing to steel-concrete composite two girder bridge

2nd prestressing layout

2x22 ϕ0.6” strand tendons on green layout

2x22 ϕ0.6” strand tendons red dashed layout

100% prestressing applied to steel girder



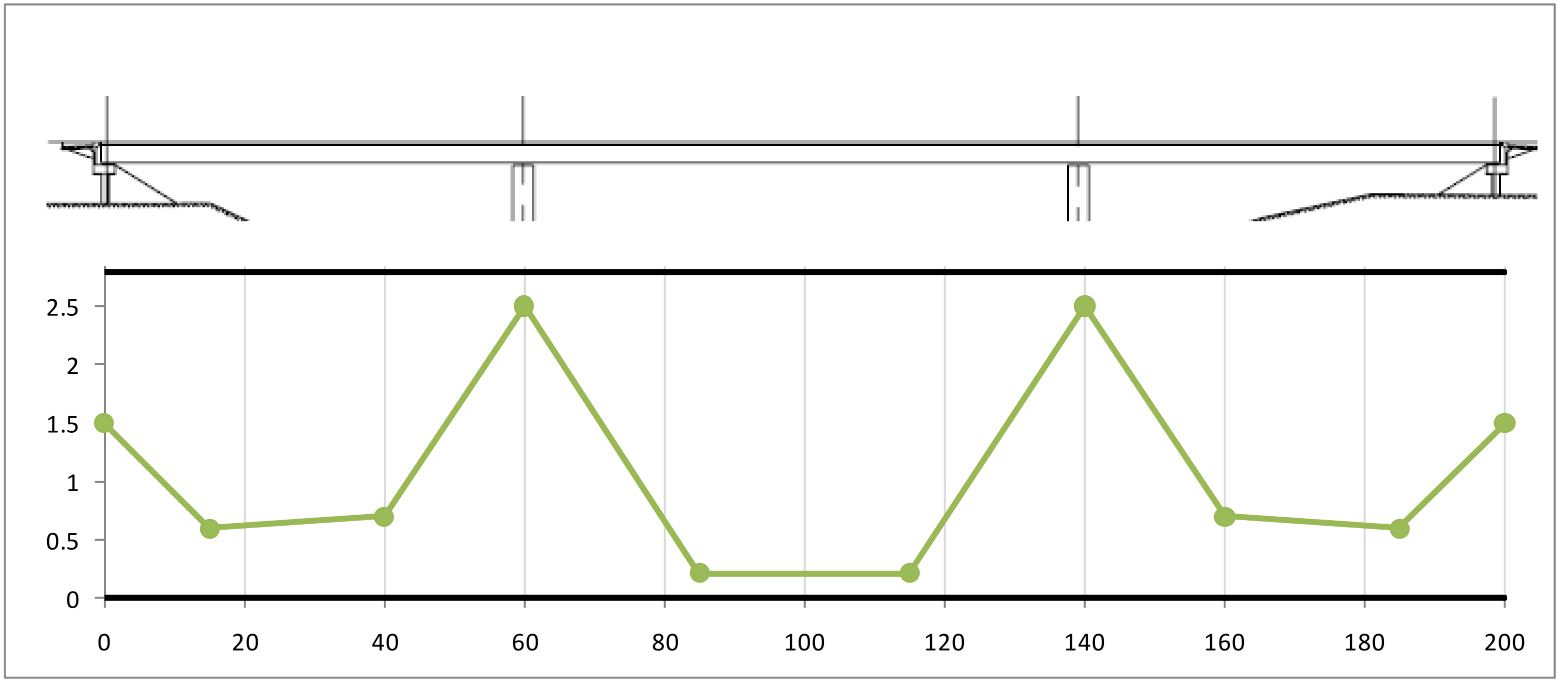
Application of external prestressing to steel-concrete composite two girder bridge

3rd prestressing layout

4x22 ϕ0.6” strand tendons on green layout

50% prestressing applied to steel girder +

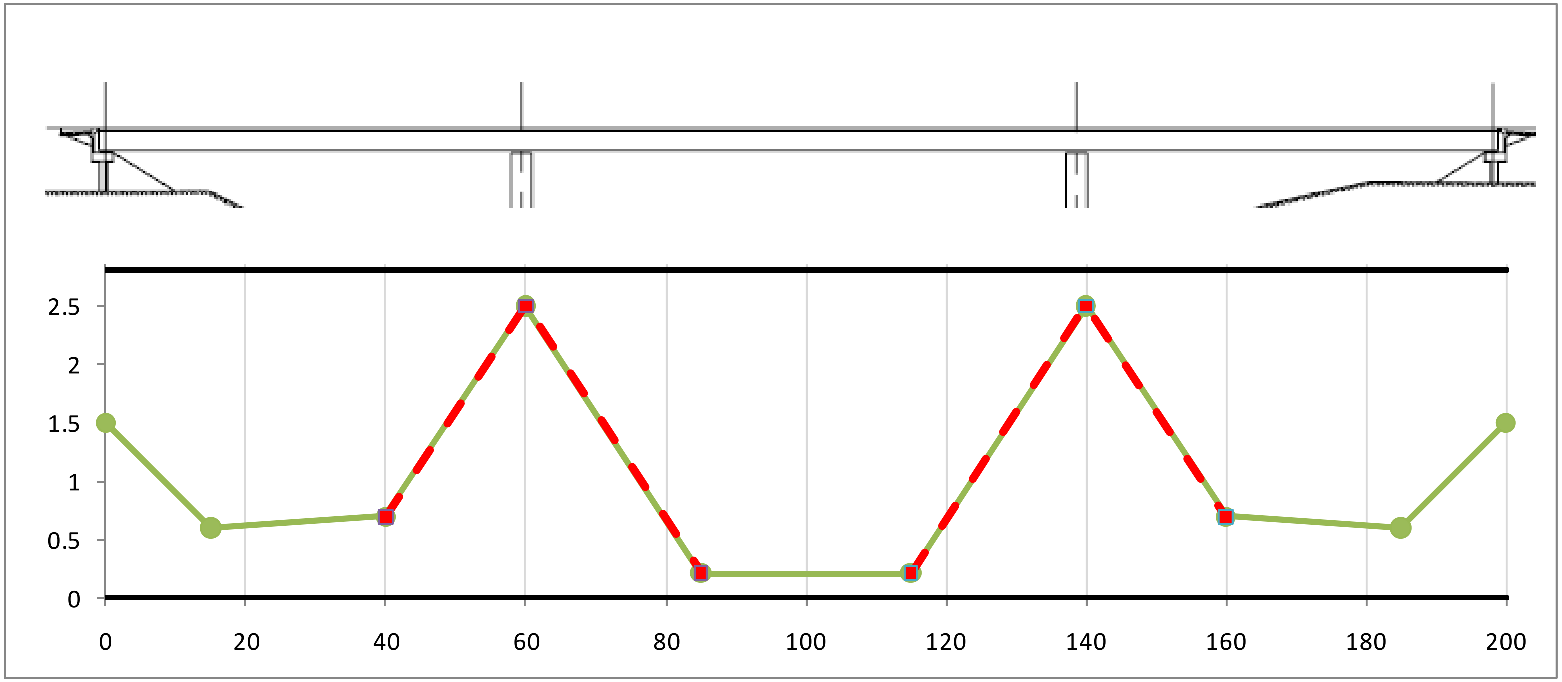
50% prestressing applied to steel composite section



Application of external prestressing to steel-concrete composite two girder bridge

4th prestressing layout
2x22 ϕ0.6" strand tendons on green layout
2x22 ϕ0.6" strand tendons red dashed layout

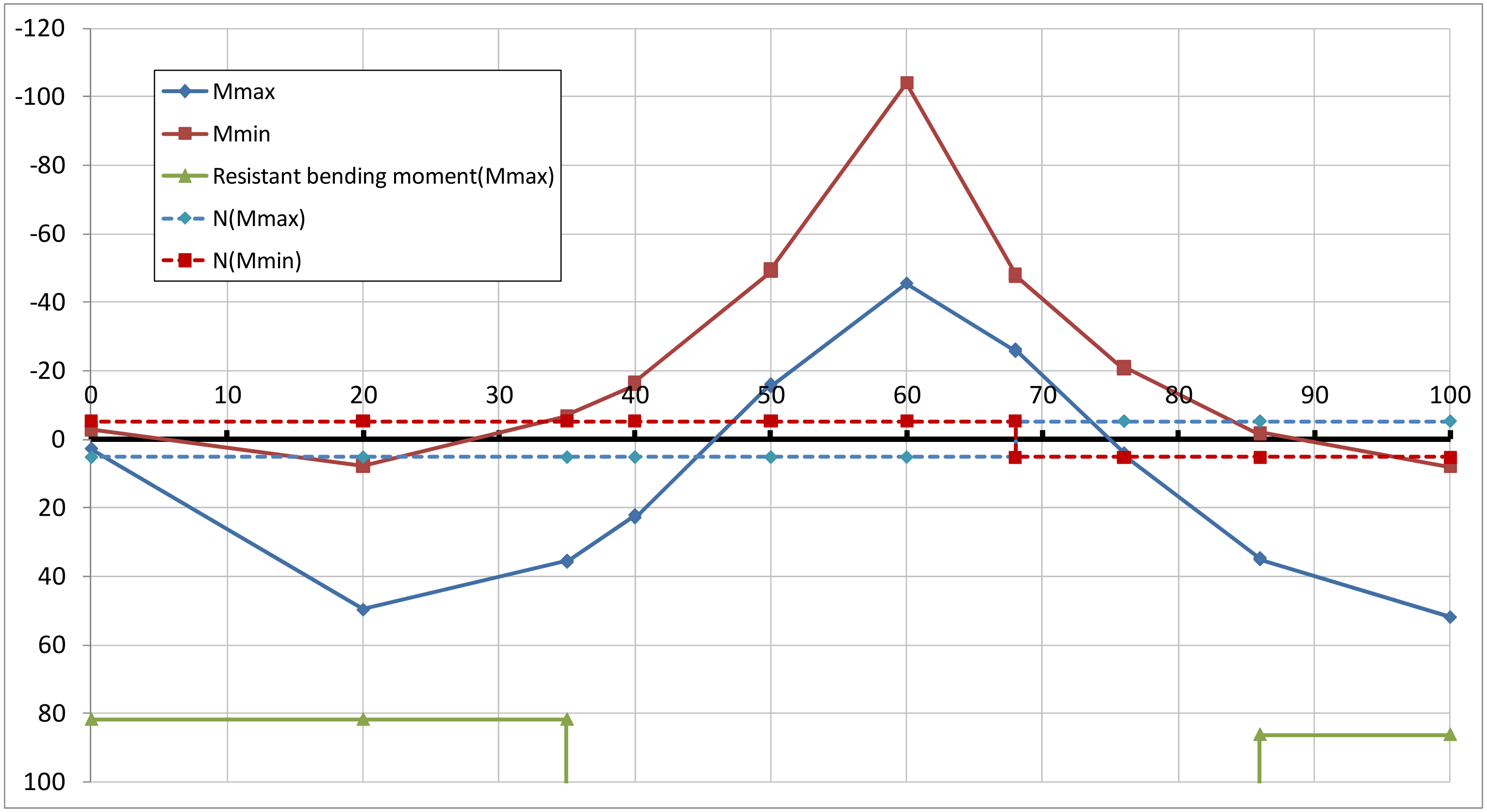
50% prestressing applied to steel girder +
50% prestressing applied to composite section

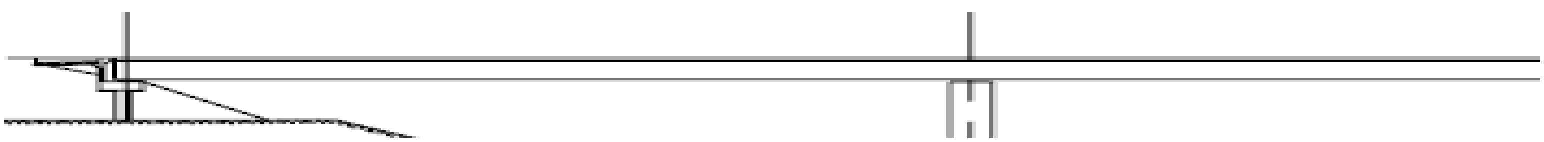


Application of external prestressing to steel-concrete composite two girder bridge

SLU internal actions on NOT prestressed girder at t_0

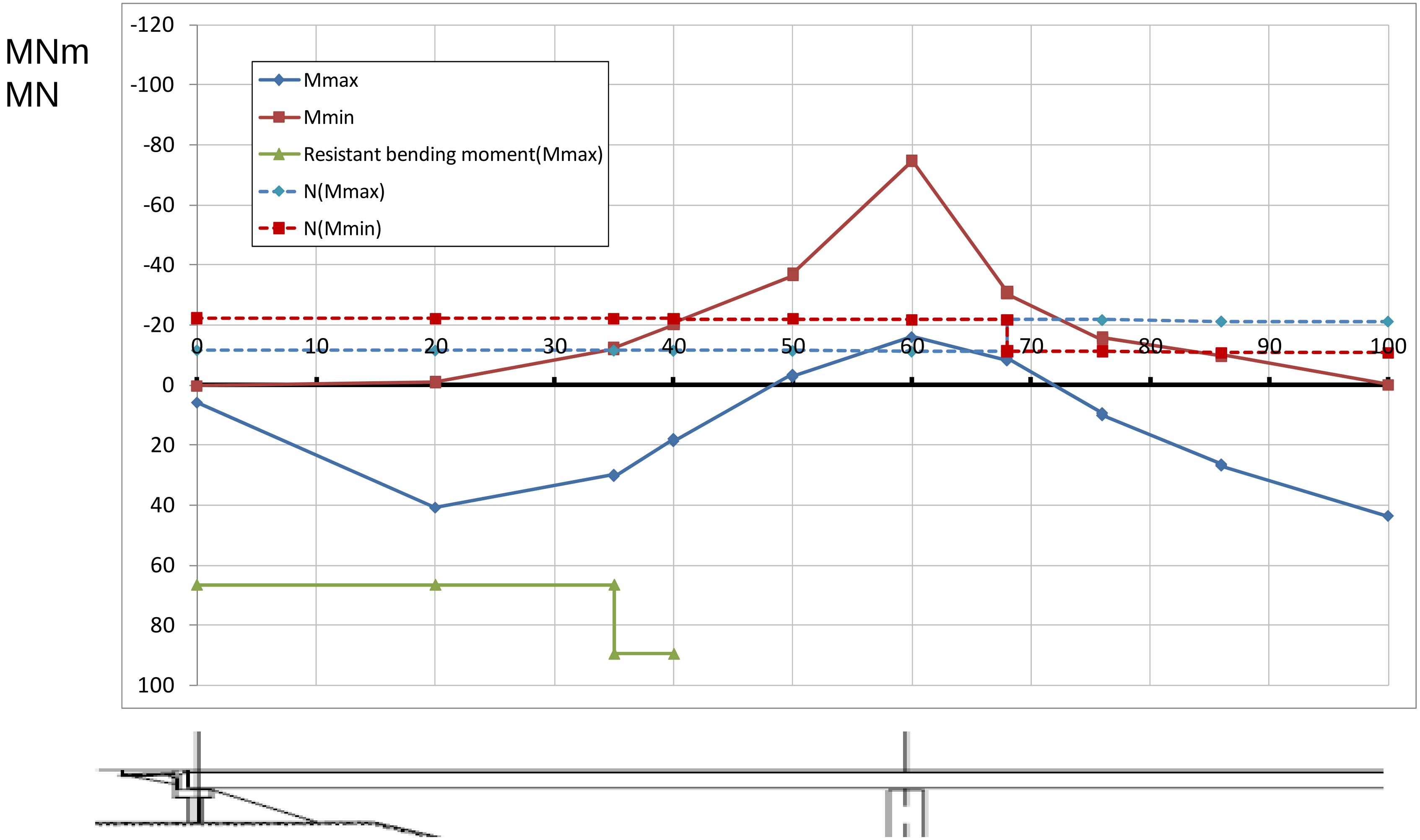
MNm
MN



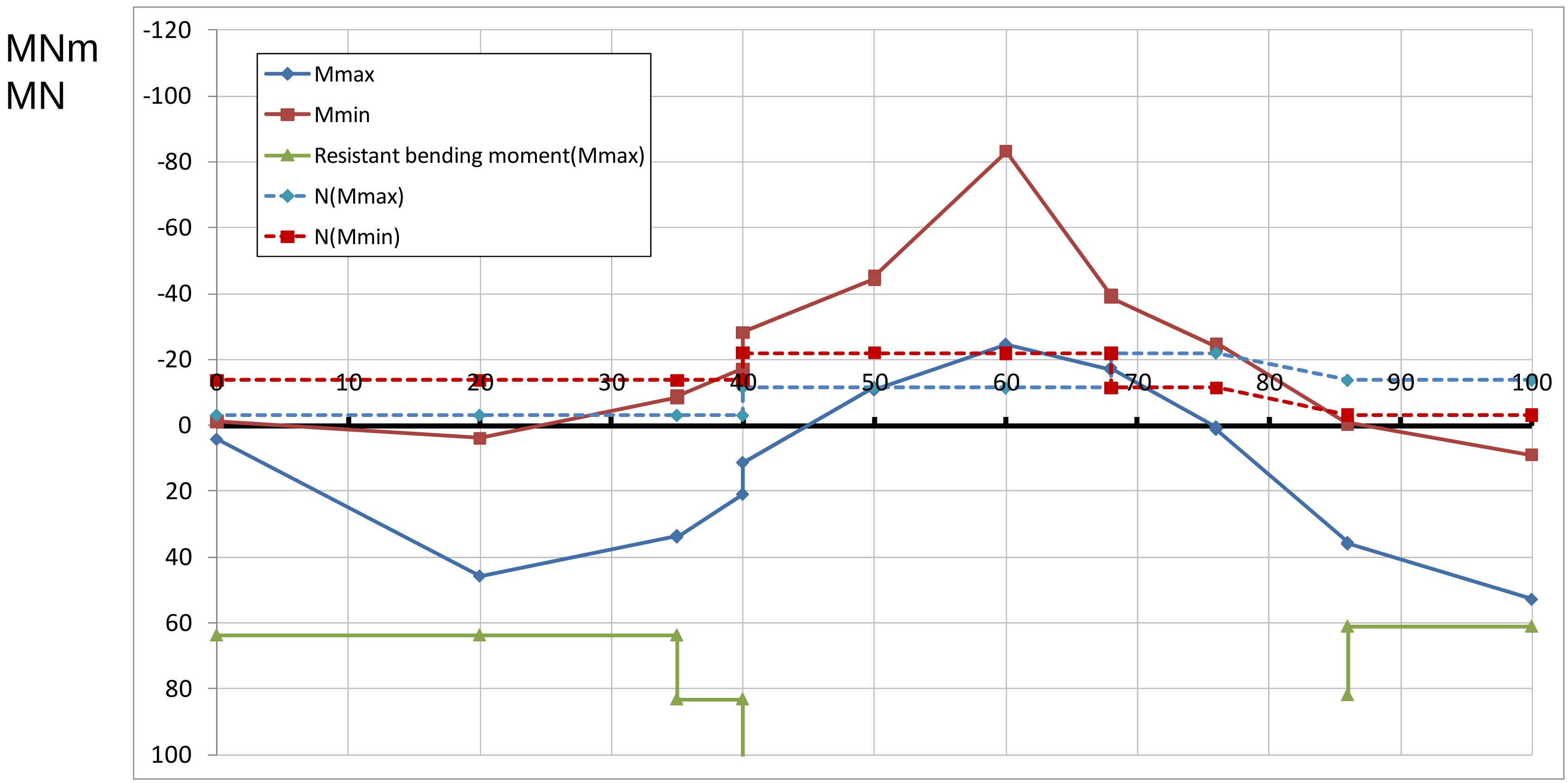


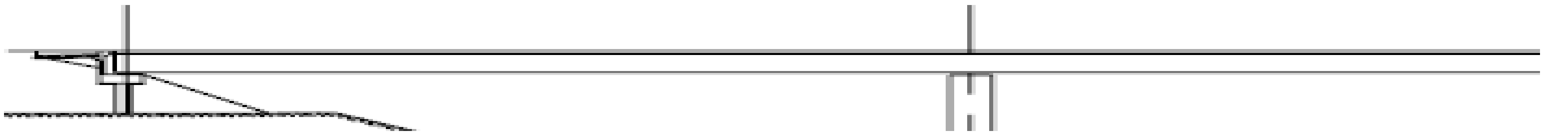
Application of external prestressing to steel-concrete composite two girder bridge

SLU internal actions with 1st prestressing layout at t_0



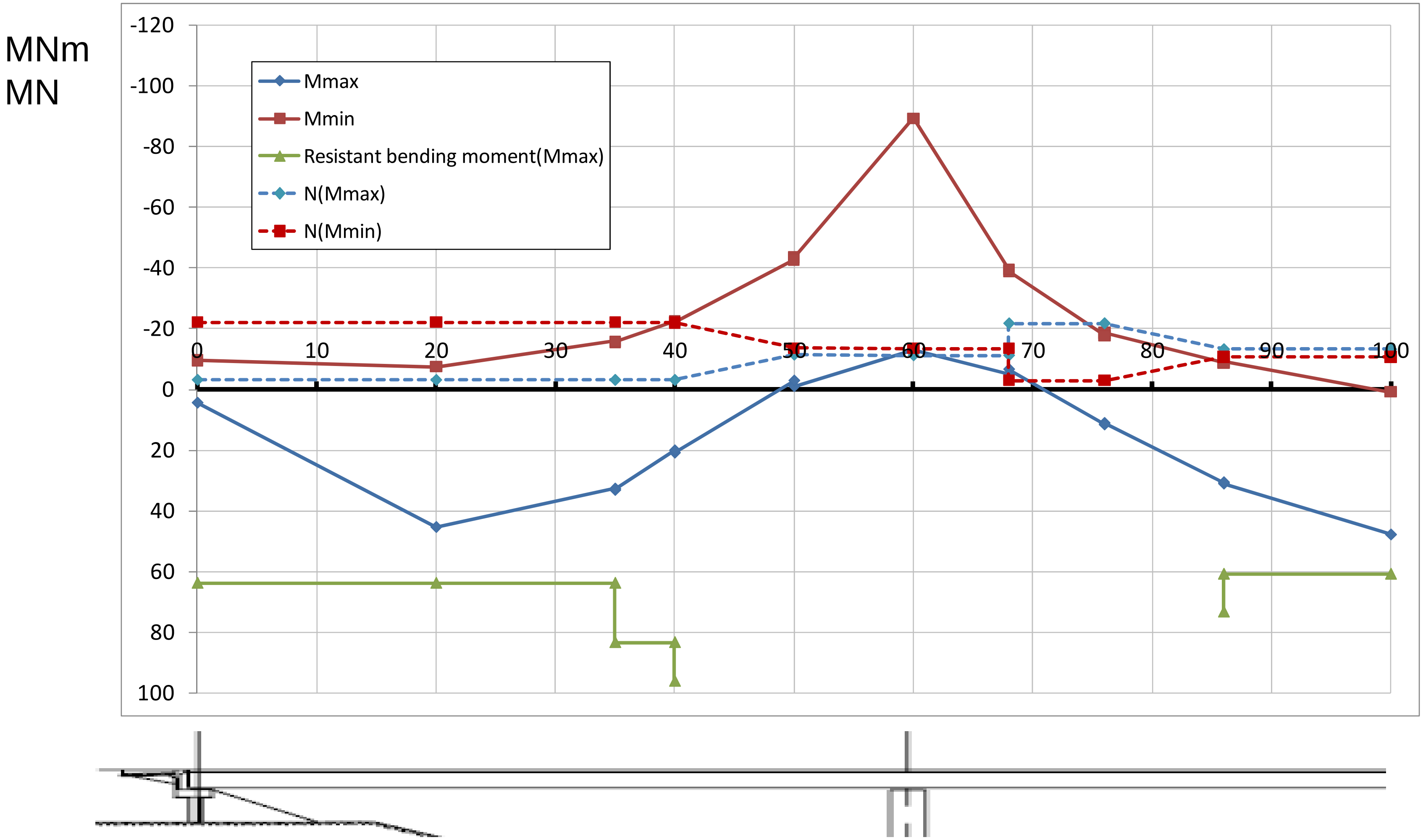
SLU actions with 2nd prestressing layout at t_0





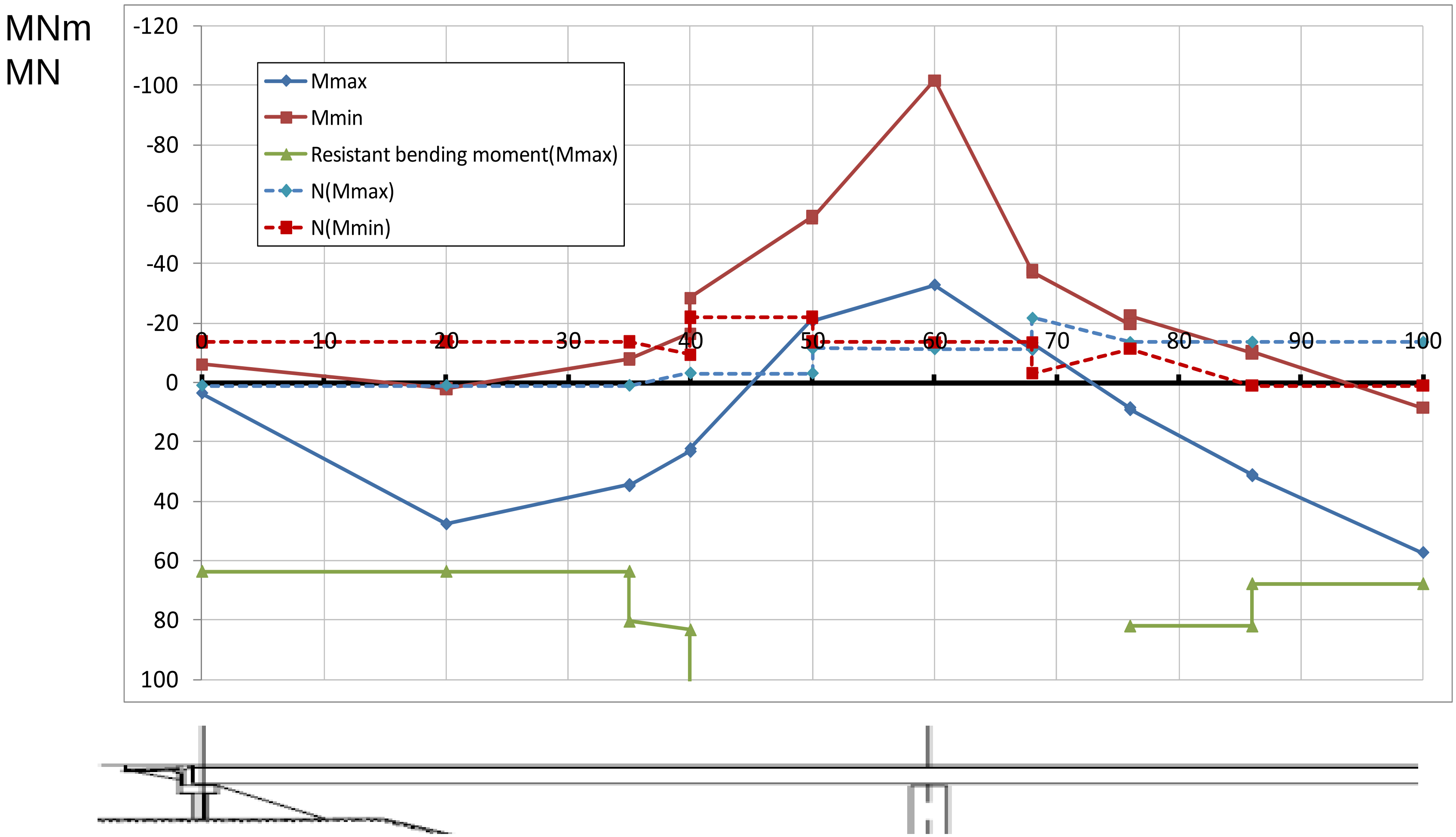
Application of external prestressing to steel-concrete composite two girder bridge

SLU internal actions with 3rd prestressing layout at t_0



Application of external prestressing to steel-concrete composite two girder bridge

SLU internal actions with 4th prestressing layout at t_0



Conclusions

The following quantities have been obtained

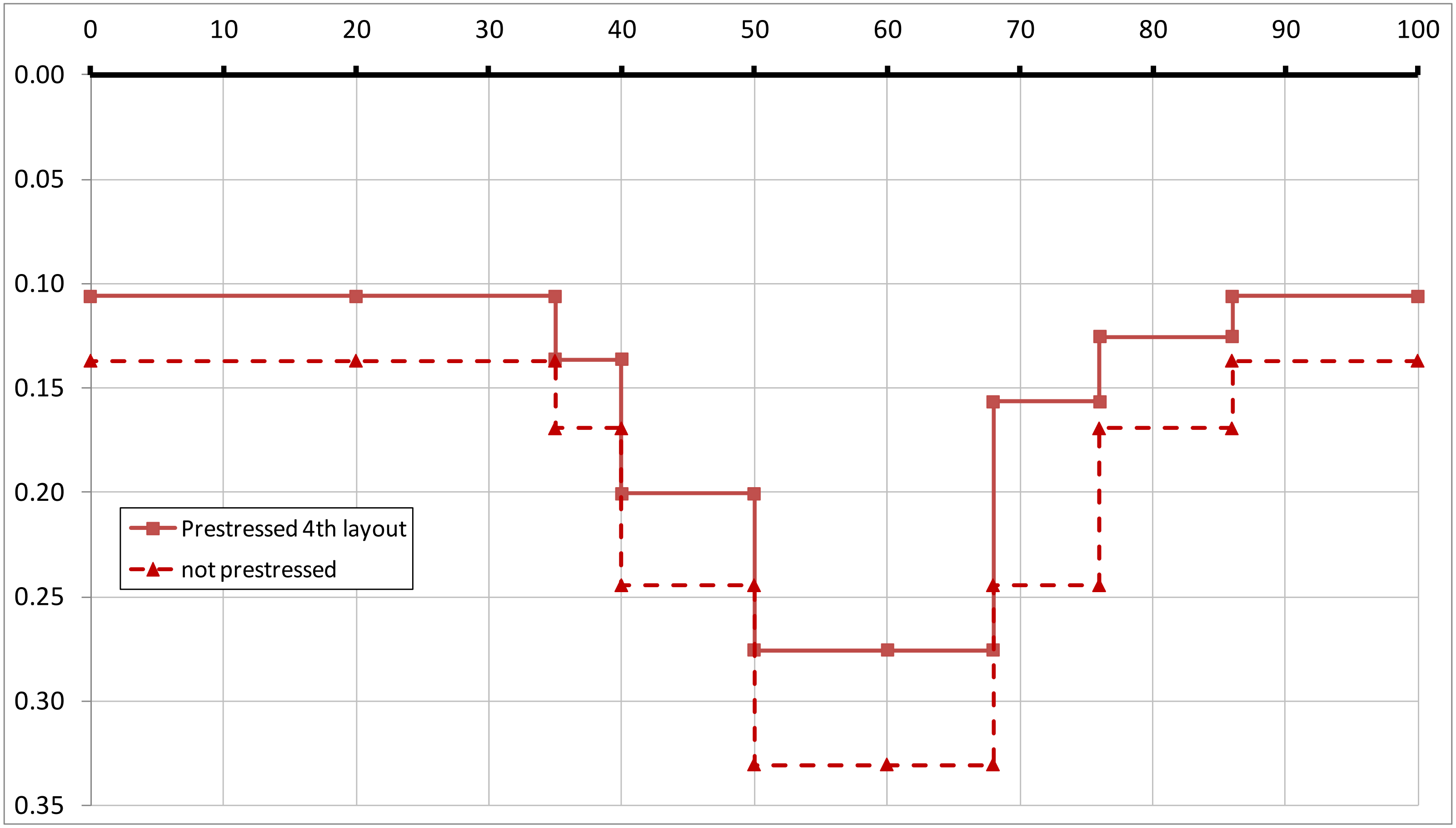
Quantities		**No prestr.**	**Prestressing layout**			
			1	**2**	**3**	**4**
Girder steel	[kg/m²]	256	202	194	190	201
Prestressing steel	[kg/m²]	0	16.0	11.6	16.0	11.6
Slab longitudinal ordinary reinforcement	[kg/m²]	51	51	51	13.1	13.1

Top slab in SLS frequent combination	[status]	cracked	cracked	cracked	**Compres-sed**	**Compres-sed**

$$A_{s,min} = 0{,}26 \frac{f_{ctm}}{f_{yk}} b_t d \qquad \text{but not less than } 0{,}0013 b_t d$$

Conclusions

Steel cross section [m^2]: comparison between not prestressed solution and 4th layout solution



Thank you for the kind attention