Eurocodes - Background and applications

Dissemination of information workshop

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EN 1994 Part 2

Composite bridges

Joël RAOUL

Laurence DAVAINE

Setra service d'études techniques des routes et autoroutes

Ministry for Ecology, Sustainable development and Spatial Planning (Paris)

Technical Centre for Highways & Motorways

46, avenue Aristide Briand
BP 100
F - 92 225 Bagneux Cedex

Contents : 7 parts

1. Introduction to composite bridges in Eurocode 4
2. Global analysis of composite bridges
3. ULS verifications
4. SLS verifications
5. Connection at the steel–concrete interface
6. Fatigue (connection and reinforcement)
7. Lateral Torsional Buckling of members in compression

All points are illustrated with numerical applications to a twin-girder bridge with upper reinforced concrete slab.

Composite bridges with steel girders under the slab

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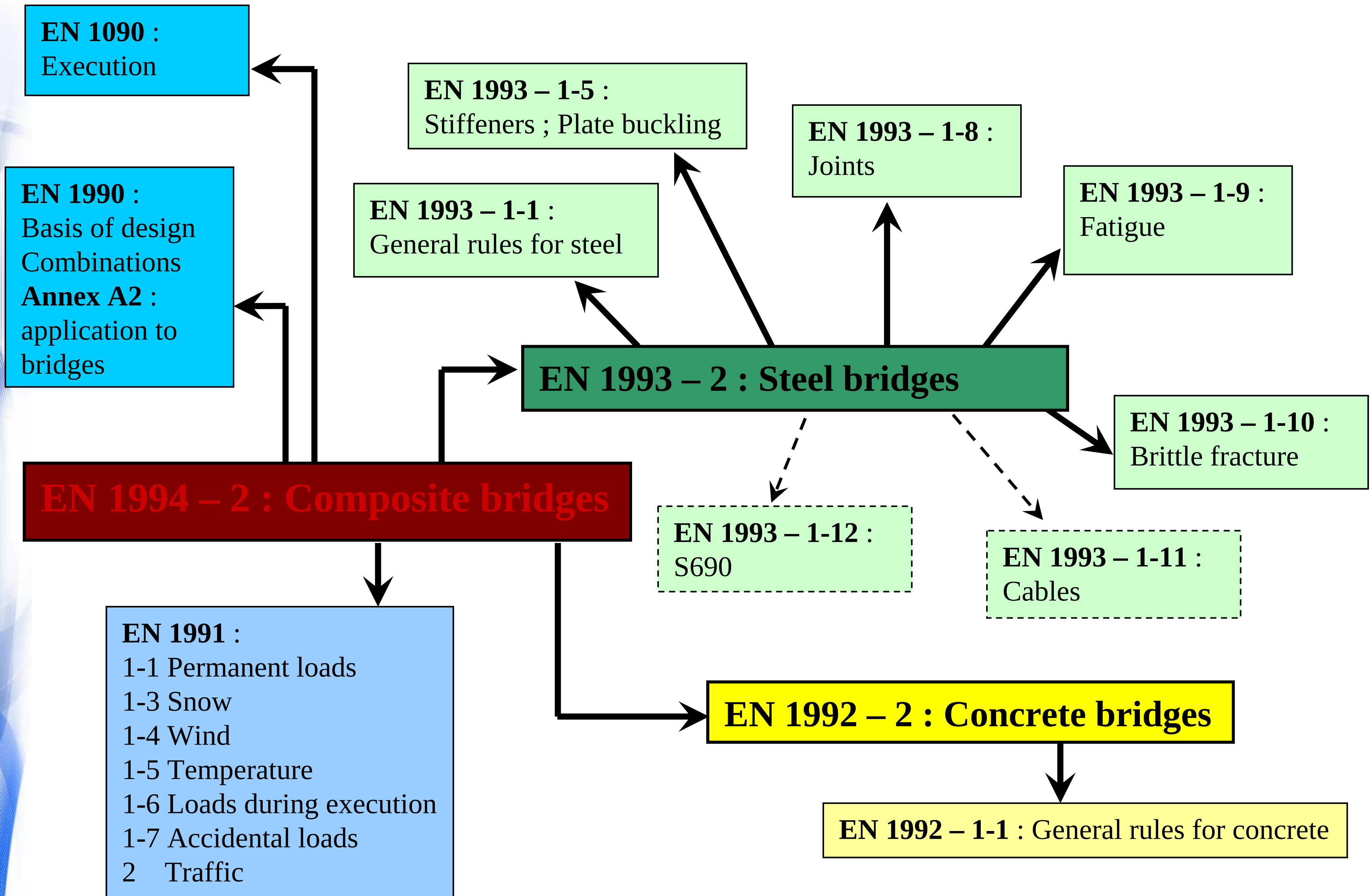
Half through composite bridges

© Sétra

Box-girder bridges

The main others EN called when using EN1994-2



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- **Elastic global analysis without bending redistribution**
- **Second order effect to be considered for structures where**

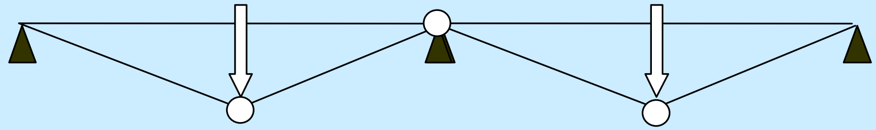
$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed,ULS}} \leq 10$$

In this elastic global analysis, the following points should be taken into account :

- effects of creep and shrinkage of concrete,
- effective width of flanges for shear lag,
- stages and sequence of construction,
- effects of cracking of concrete,
- temperature effects of heat of hydration of cement (only for construction stages).

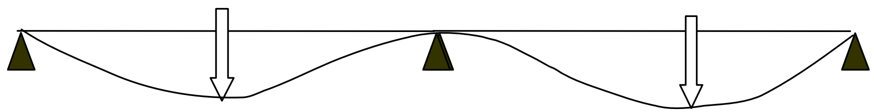
- **Non-linear global analysis may be used (no application rules)**

CLASS 1 sections which can form a plastic hinge with the rotation capacity required for a global plastic analysis

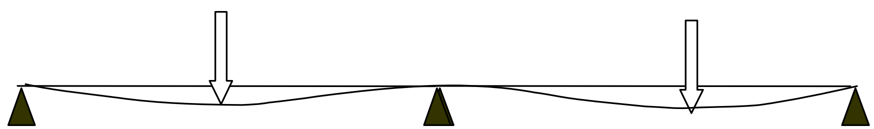


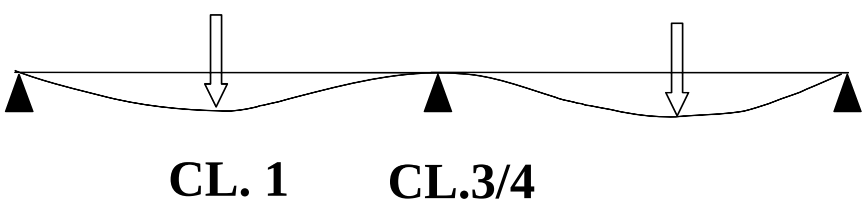
BUILDINGS

CLASS 2 sections which can develop $M_{pl,Rd}$ with limited rotation capacity



CLASS 3 sections which can develop $M_{el,Rd}$

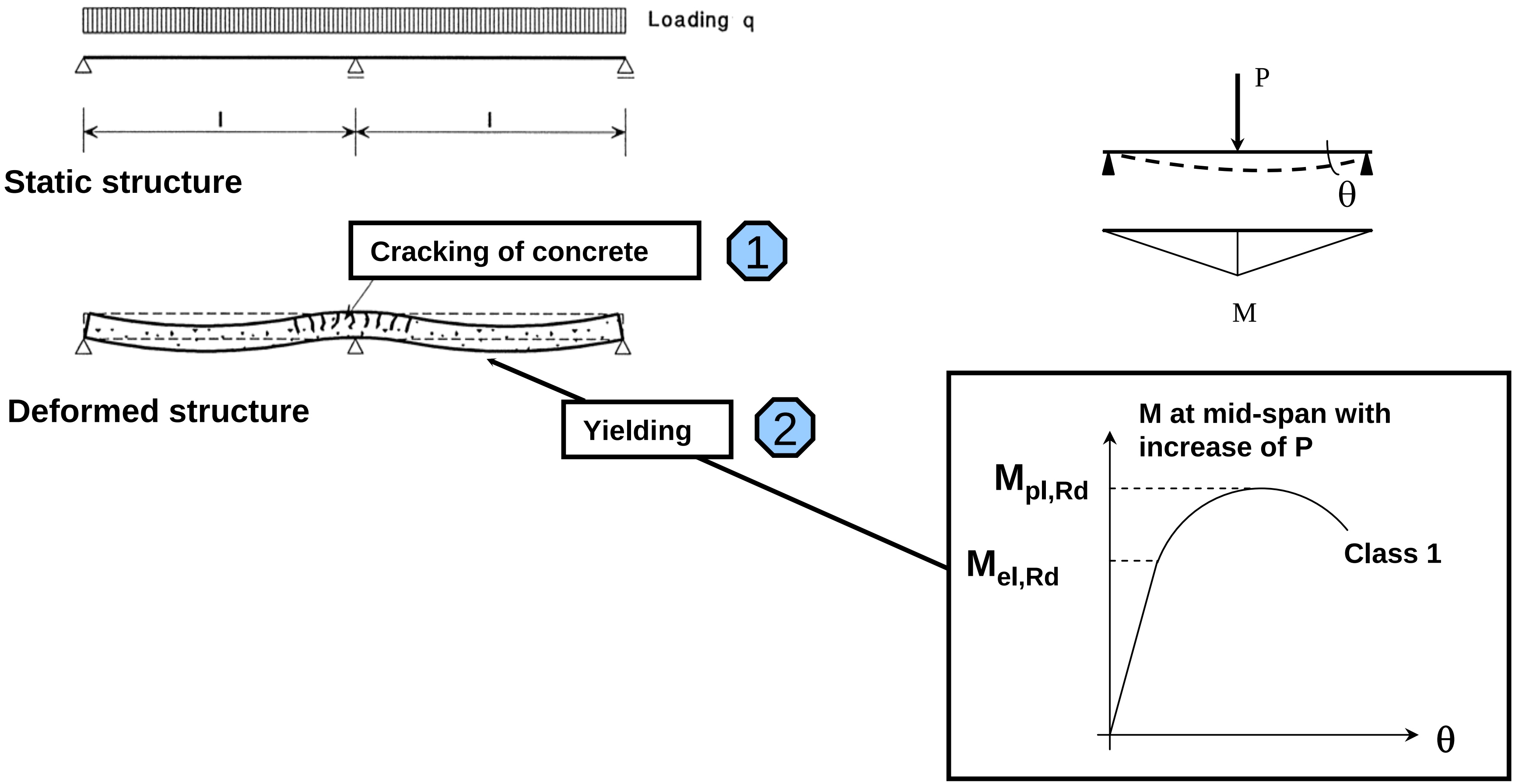




COMPOSITE BRIDGES

In general, non-uniform section (except for small spans)

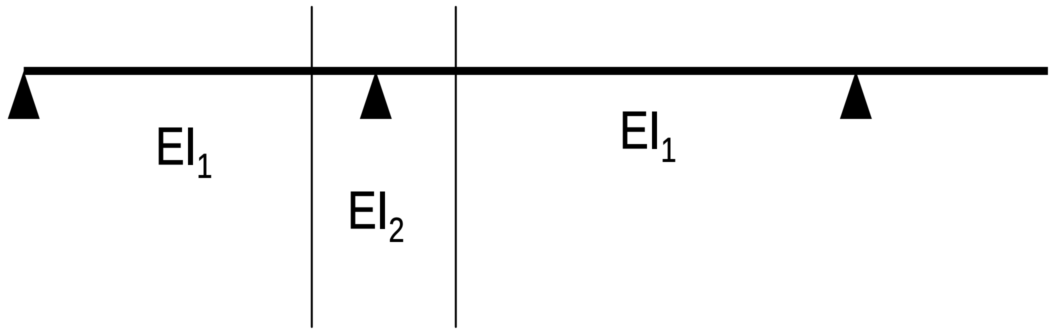
When performing the elastic global analysis, two aspects of the non-linear behaviour are directly or indirectly considered.



Cracked global analysis

1

- Determination of the stresses σ_c in the extreme fibre of the concrete slab under SLS characteristic combination according to a non-cracked global analysis
- In sections where $\sigma_c < -2 f_{ctm}$, the concrete is assumed to be cracked and its resistance is neglected



EI_1 = un-cracked composite inertia (structural steel + concrete in compression)

EI_2 = cracked composite inertia (structural steel + reinforcement)

An additional iteration is not required.

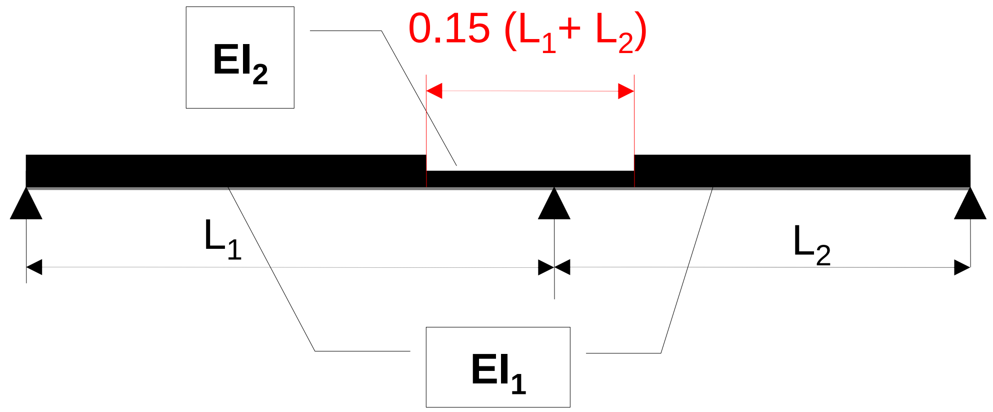
Cracked global analysis

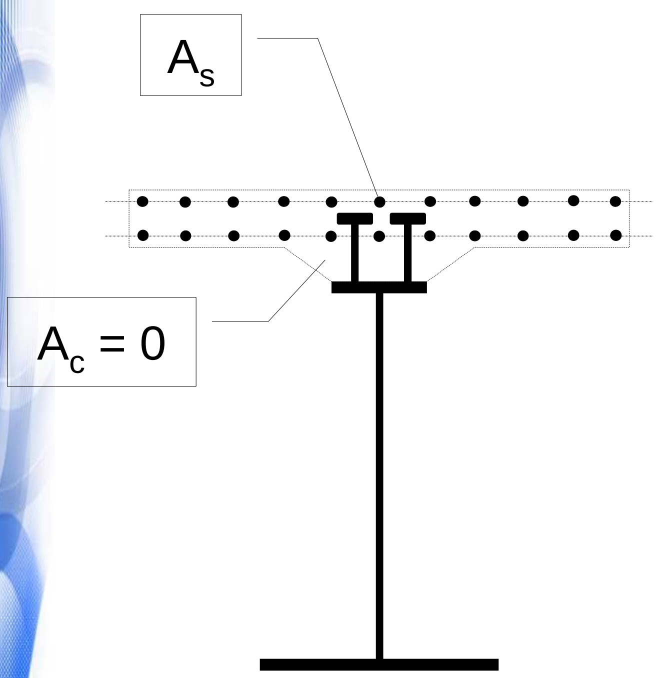
1

<u>Simplified method</u> usable if :

- no pre-stressing by imposed deformation

- $L_{min}/L_{max}>0.6$





In the cracked zones EI_2 :

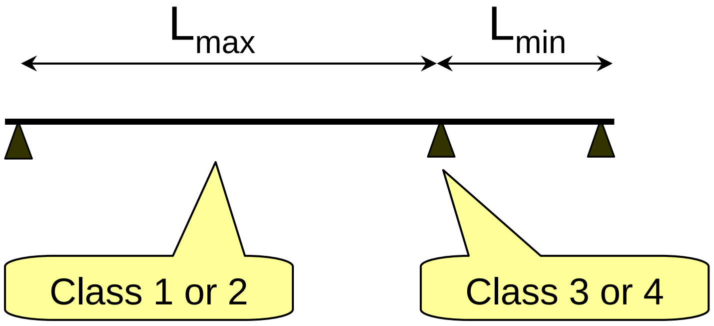
- the resistance of the concrete in tension is neglected
- the resistance of the reinforcement is taken into account

Yielding

2

Yielding at mid-span is taken into account if :

- Class 1 or 2 cross-section at mid-span (and $M_{Ed} > M_{el,Rd}$)
- Class 3 or 4 near intermediate support
- $L_{min}/L_{max} < 0.6$



- Elastic linear analysis with an additional verification for the cross-sections in sagging bending zone (M>0) :

$$\mathbf{M_{Ed} < 0.9\ M_{pl,Rd}}$$

or

- Non linear analysis

- **To calculate the internal forces and moments for the ULS combination of actions**

  – elastic global analysis (except for accidental loads)
    - » linear
    - » non linear (behaviour law for materials in EC2 and EC3)

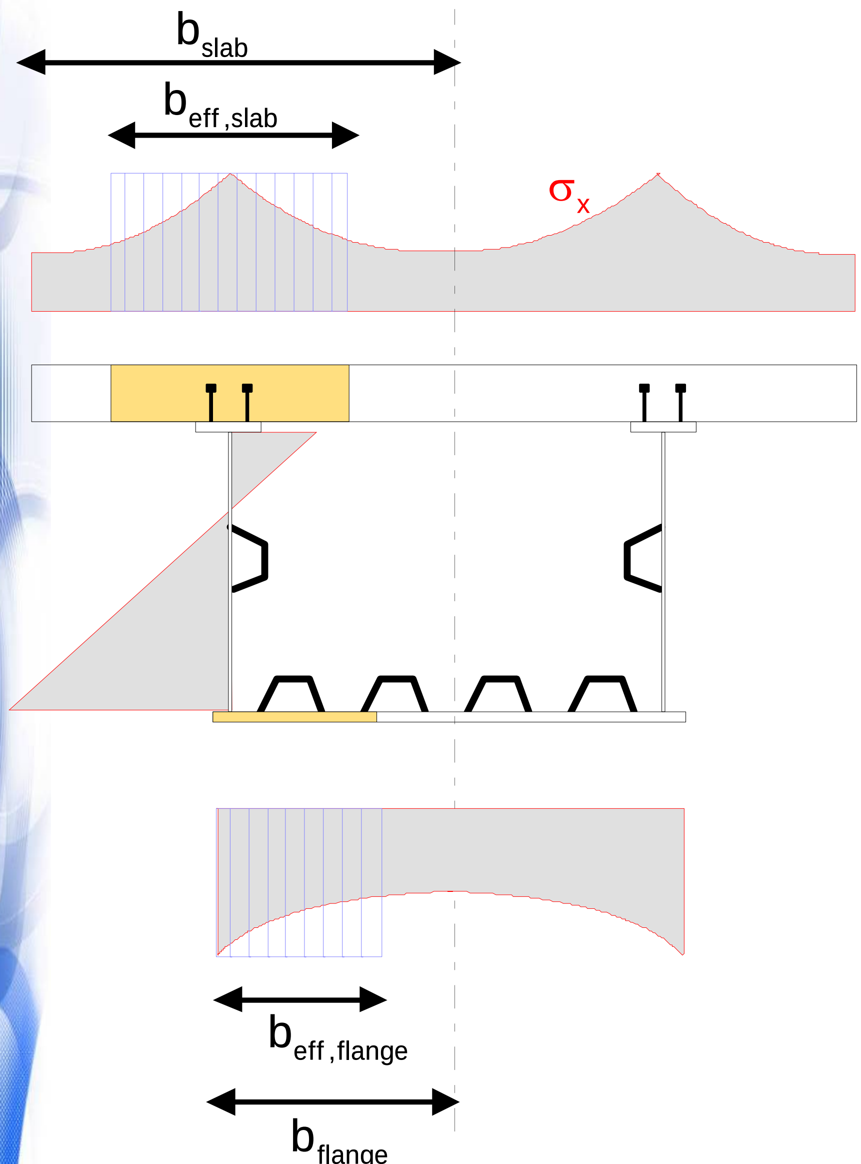
  – cracking of the concrete slab

  – shear lag (in the concrete slab : $L_e/8$ constant value for each span and calculated from the outside longitudinal rows of connectors)

  – neglecting plate buckling (except for an effective^p area of an element ≤ 0.5 * gross area)

- **To calculate the internal forces and moments for the SLS combinations of actions**
  - as for ULS (mainly used for verifying the concrete slab)
- **To calculate the longitudinal shear per unit length (SLS and ULS) at the steel-concrete interface**
  - Cracked global analysis, elastic and linear
  - ***Always*** uncracked section analysis
  - Specific rules for shear connectors design in the elasto-plastic zones for ULS ($M_{el,Rd} < M_{Ed} < M_{pl,Rd}$)

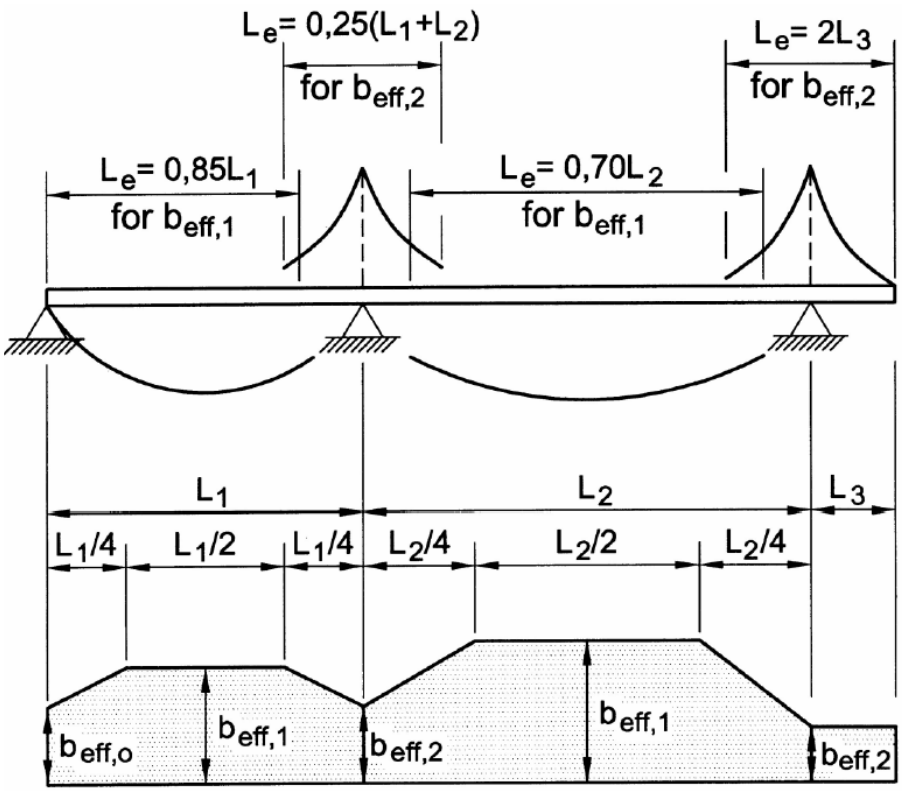
Shear lag in composite bridges



- **Concrete slab ⇒ EN 1994-2**
  - Same effective[s] width b_{eff} for SLS and ULS combinations of actions
- **Steel flange ⇒ EN 1993-1-5**
  - Used for bottom flange of a box-girder bridge
  - Different effective[s] width for SLS and ULS combinations of actions
  - 3 options at ULS (choice to be performed in the National Annex)

Effectives width of the concrete slab – EN1994-2

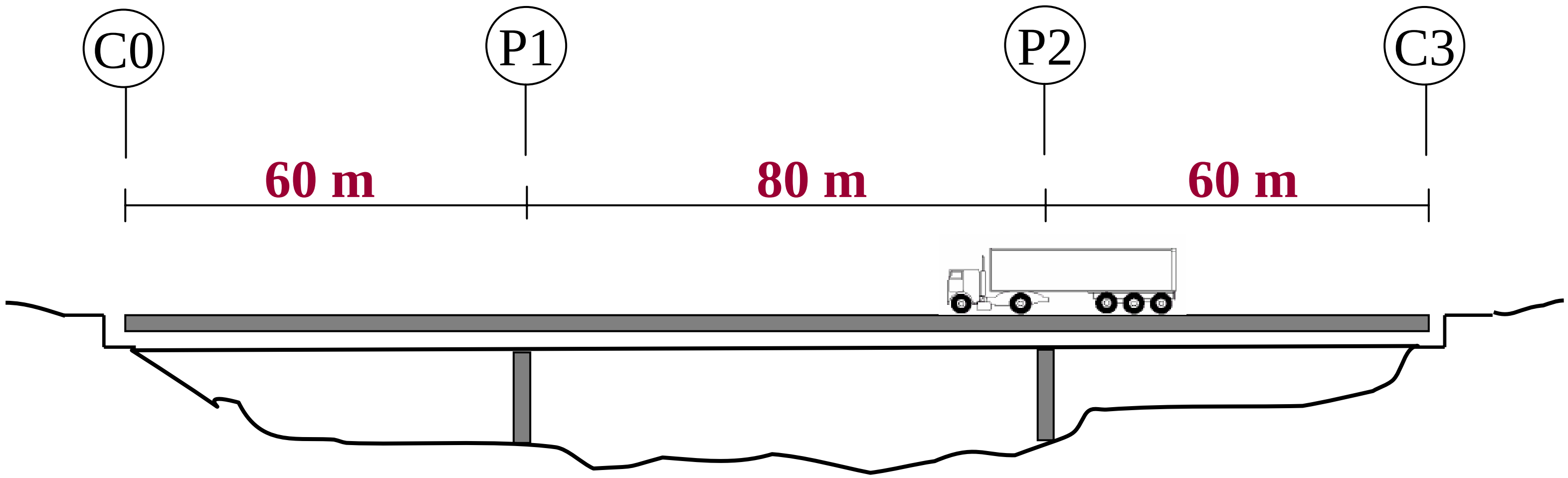
- **Global analysis** : constant for each span for simplification (with a value calculated at mid-span)
- **Section analysis** : variable on both sides of the vertical supports over a length L_i /4

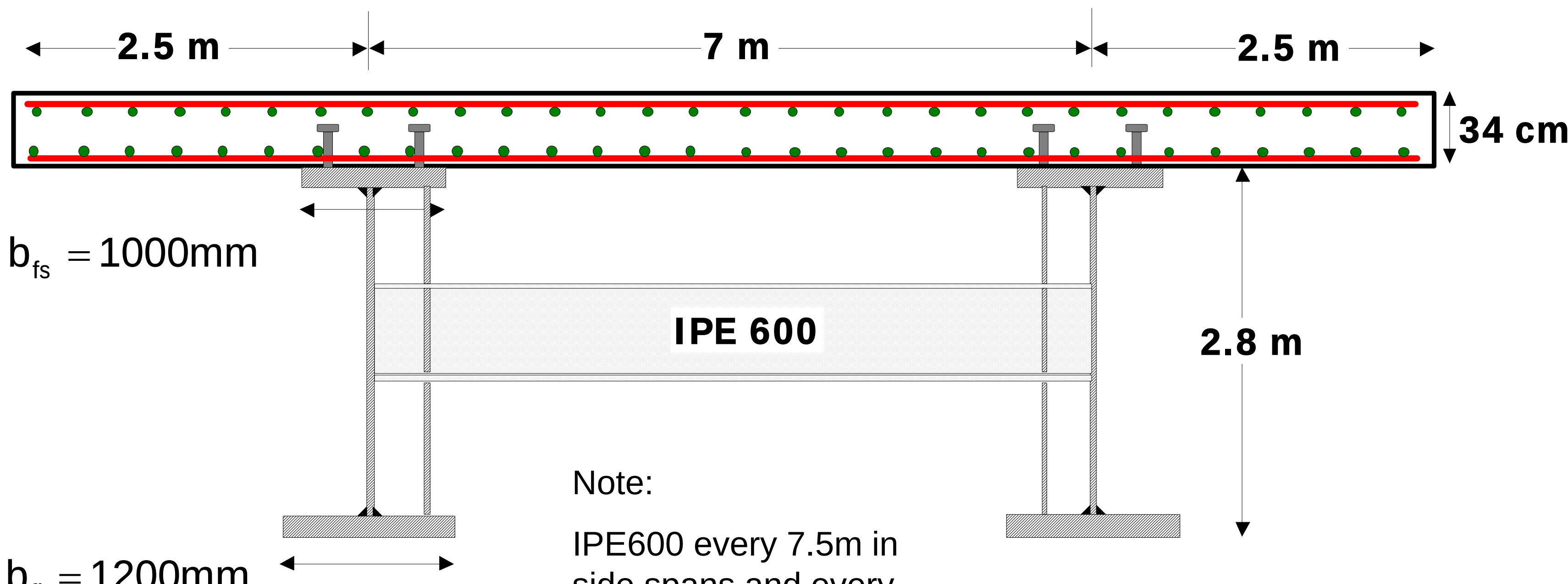


Application to a steel-concrete composite twin girder bridge

Global longitudinal bending

Example : Composite twin-girder road bridge



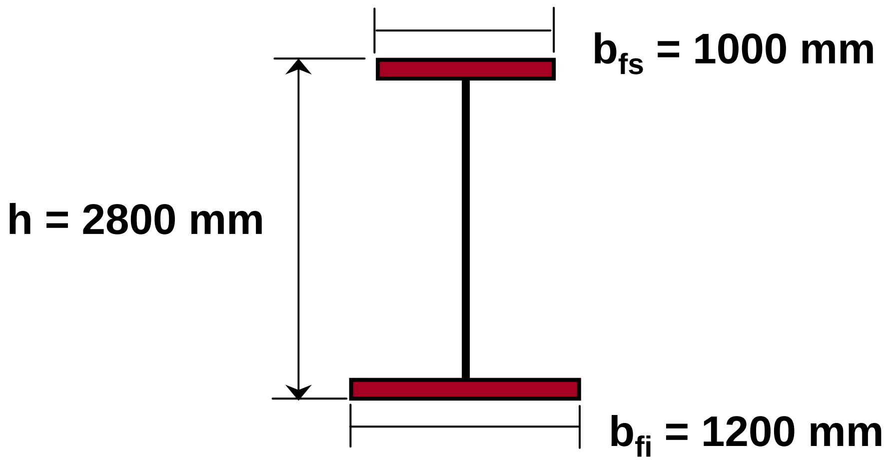


Note:

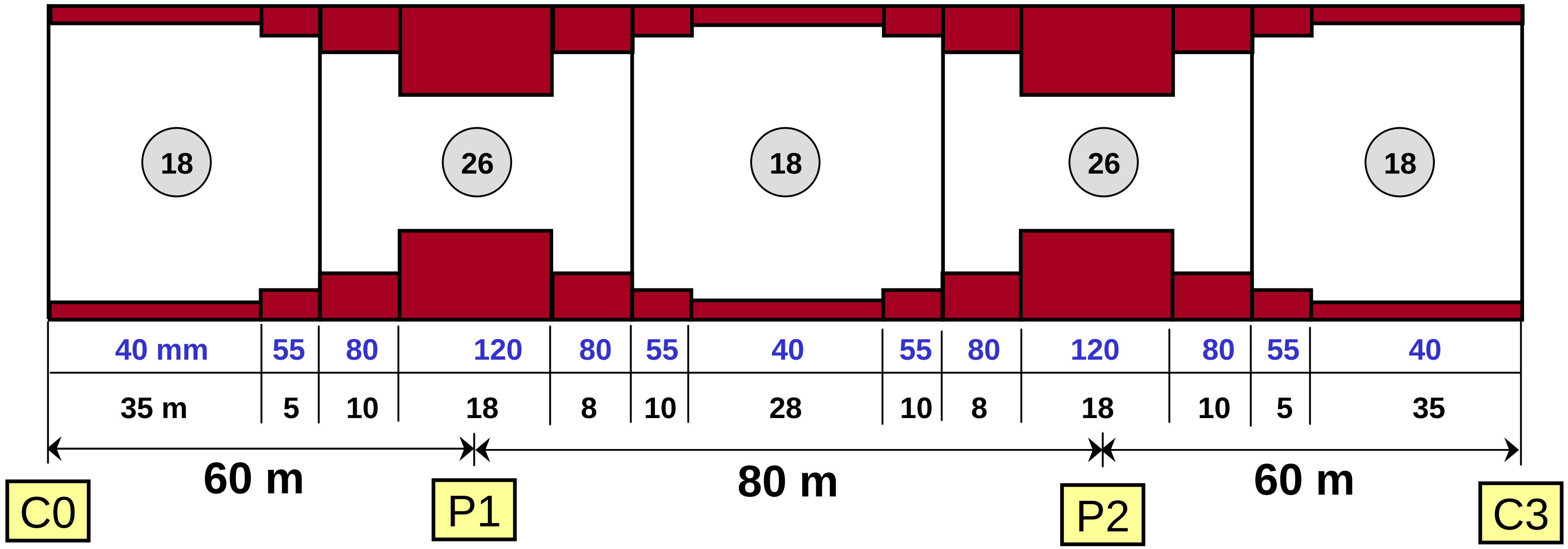
IPE600 every 7.5m in side spans and every 8.0m in central span

Example : Structural steel distribution

Note : Bridge dimensions verified according to Eurocodes (cross-section resistance at ULS, SLS stresses and fatigue)



Longitudinal structural steel distribution of each main girder



Example : Used materials

- Structural steel (EN1993 + EN10025) :
  - S355 N for $t \leq 80$ mm (or S355 K2 for $t \leq 30$ mm)
  - S355 NL for $80 < t \leq 150$ mm

Yield strength f_y (MPa)	thickness t (mm)					
	$t \leq 16$	$16 < t \leq 40$	$40 < t \leq 63$	$63 < t \leq 80$	$80 < t \leq 100$	$100 < t \leq 150$
S 355 N	355	345	335	325		
S 355 NL					315	295

Note : the requirements of EN 1993-1-10 (brittle fracture and through-thickness properties) should also be fulfilled.

- Cross bracing and stiffeners : S355
- Shear connectors : headed studs with f_u = 450 MPa
- Reinforcement : high bond bars with f_{sk} = 500 Mpa
- Concrete C35/45 defined in EN1992 : $f_{ck,cyl}$ (at 28 days) = 35 MPa
  $f_{ck,cube}$ (at 28 days) = 45 MPa
  f_{ctm} = -3.2 MPa

Creep - Modular ratios for bridges

$$n_L = n_0.(1+\psi_L \phi_t)$$

$n_0 = \dfrac{E_a}{E_{cm}}$ for short term loading ($\psi_L = 0$)

$\phi_t = \phi(t - t_0)$ creep function defined in EN1992-1-1 with :

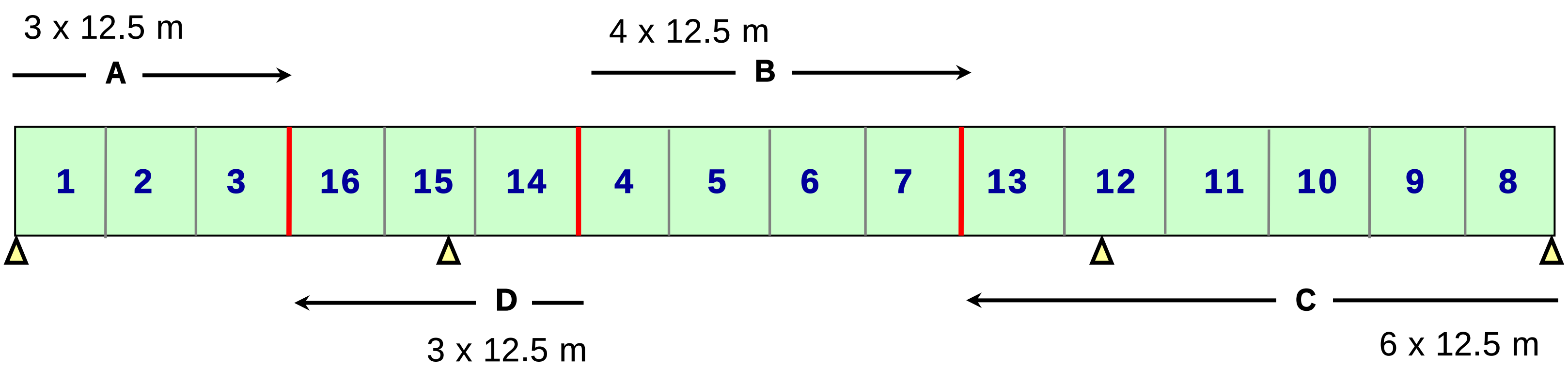
- t = concrete age at the considered instant
- t_0 = mean value of the concrete age when a long-term loading is applied (for instance, permanent loads)
- t_0 = 1 day for shrinkage action

ψ_L correction factor for taking account of the type of loading

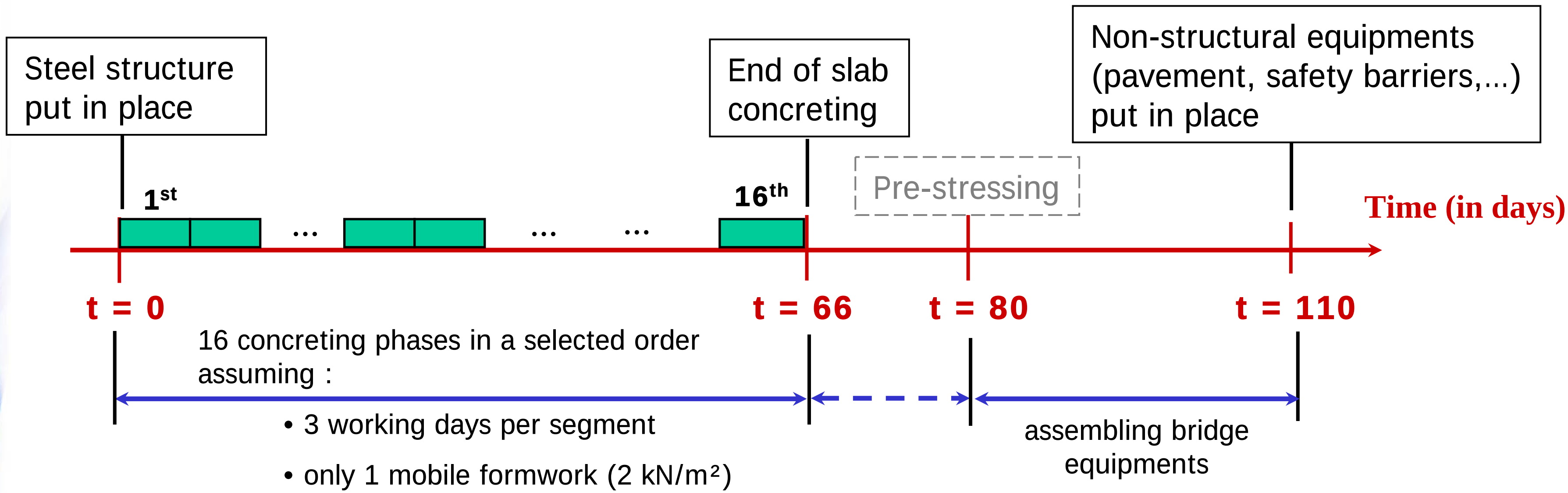
Permanent loads	1.1
Shrinkage	0.55
Pre-stress by imposed deformations (for instance, jacking on supports)	1.5

Example : construction phasing

1. Concreting order of the 12.5-m-long slab segments

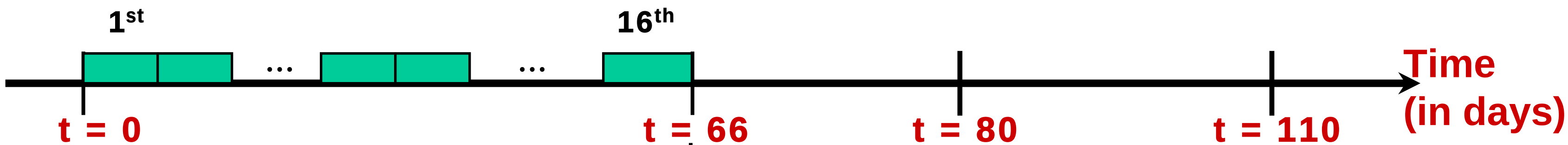


2. Construction timing



<u>Note</u> : **14** days are required in EN1994-2 before introducing pre-stressing by imposed deformations.

Example : age of concrete



Phase 1	3			
Phase 2	8	5		
...	...	...	...	
Phase 16	66	63	...	3

Mean value of the ages of concrete segments :

$$t_0 = \frac{66 + 63 + ... + 3}{16 \text{ phases}} = 35.25 \text{ days}$$

used for all concreting phases (simplification of EN1994-2).

$$\phi_1 = \phi(t = \infty, t_0)$$

$$n_{L,1} = n_0 (1 + 1.1.\phi_1)$$

+ 14 days

$$t_0 = 49.25 \text{ days}$$

$$\phi_2 = \phi(\infty, t_0)$$

$$n_{L,2} = n_0 (1 + 1.5.\phi_2)$$

+ 30 days

$$t_0 = 79.25 \text{ days}$$

$$\phi_3 = \phi(\infty, t_0)$$

$$n_{L,3} = n_0 (1 + 1.1.\phi_3)$$

Note : t_0 = 1 day when shrinkage is applied to a concrete segment.

$$\phi_4 = \phi(\infty, t_0) \qquad n_{L,4} = n_0 (1 + 0.55.\phi_4)$$

EN1992-1-1, Annex B :

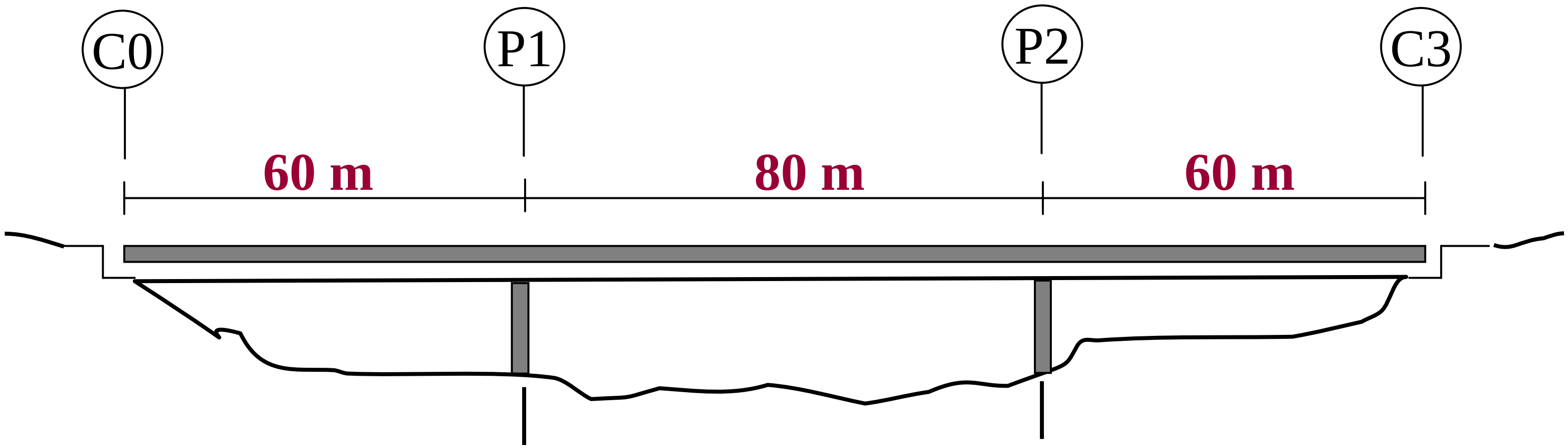
$$\phi(t,t_0) = \phi_0.\beta_c(t-t_0) = \phi_0.\left(\frac{t-t_0}{\beta_H + t - t_0}\right)^{0.3} \xrightarrow{t \longrightarrow +\infty} \phi_0$$

$$\phi_0 = \phi_{RH}.\beta(f_{cm}).\beta(t_0) = \left[1 + \frac{1-\frac{RH}{100}}{0.10.\sqrt[3]{h_0}}.\alpha_1\right].\alpha_2.\left[\frac{16.8}{\sqrt{f_{cm}}}\right].\left[\frac{1}{0.1+t_0^{0.2}}\right]$$

- RH = 80 % (relative humidity)
- h_0 = notional size of the concrete slab = $2A_c/u$
  where u is the part of the slab perimeter which is directly in contact with the atmosphere.
- C35/45 : as f_{cm} = 35+8 > 35 MPa, $\alpha_1 = (35/f_{cm})^{0.7}$, $\alpha_2 = (35/f_{cm})^{0.2}$

Short term loading	Long term loading	
$n_0 = \frac{E_a}{E_{cm}} = 6.16$	Concrete self-weight	$n_{L,1}$ = 15.49
	Shrinkage	$n_{L,4}$ = 15.23
	Pre-stressing	$n_{L,2}$ = 18.09
	Bridge equipments	$n_{L,3}$ = 14.15

Equivalent spans L_e :



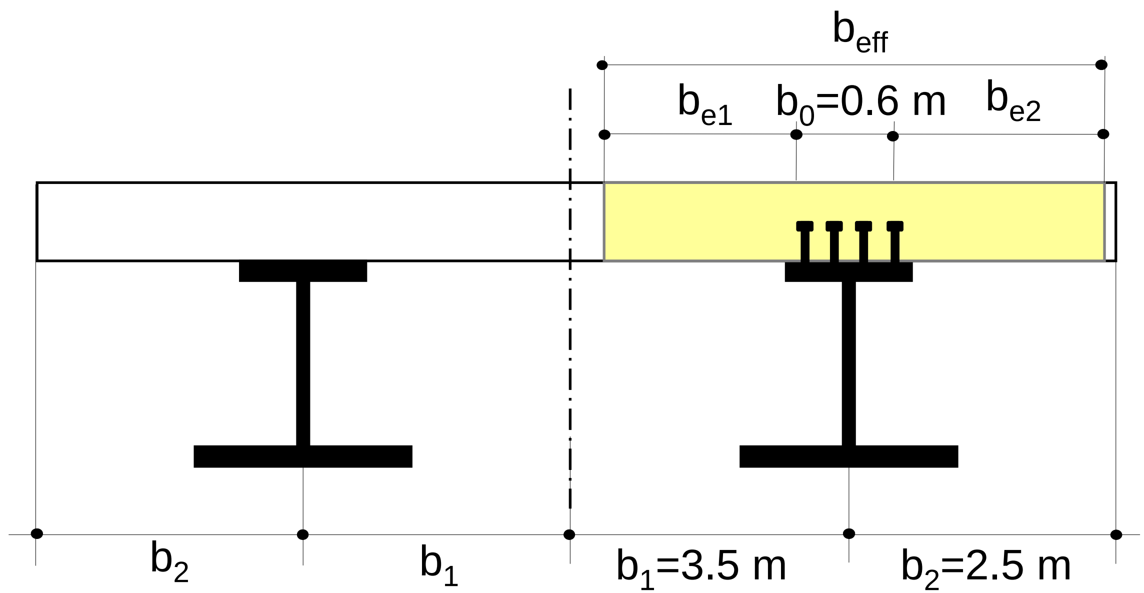
in span	0.85x60 = 51m	0.7x80 = 56m	0.85x60 = 51m
on support	0.25 x (60+80) = 35m	0.25 x (60+80) = 35m	

$b_{eff} = b_0 + \beta_1 . b_{e1} + \beta_2 . b_{e2}$ where:

- $b_{ei} = \min\left(\frac{L_e}{8} ; b_i\right)$
- $\beta_i = 1.0$ except at both end supports where:

$$\beta_i = 0.55 + 0.025 \frac{L_e}{b_{ei}} \leq 1.0$$

Example: shear lag in the concrete slab



	L_e (m)	b_{e1}	b_{e2}	β_1	β_2	b_{eff} (m)
Spans 1 and 3	51	3.2	2.2	/	/	6.0
Span 2	56	3.2	2.2	/	/	6.0
Internal supports P1 and P2	35	3.2	2.2	/	/	6.0
End supports C0 and C4	51	3.2	2.2	0.948	1.129 < 1.0	5.83 < 6.0

=> No reduction for shear lag in the global analysis

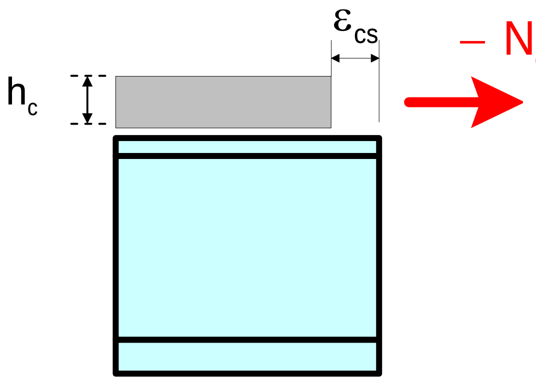
=> Reduction for shear lag in the section analysis :

b_{eff} linearly varies from 5.83m at end supports to 6.0 m at a distance $L_1/4$.

Applied loads on the road bridge example

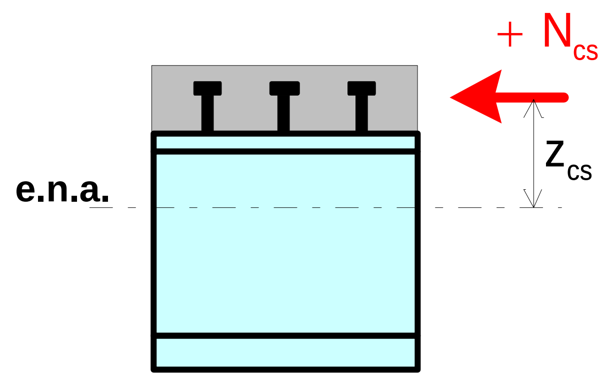
Permanent loads		
G_{max} , G_{min}	Self weight: • structural steel • concrete (by segments in a selected order) • non structural equipments (safety barriers, pavement,...)	EN1991 part 1-1
S	Shrinkage (drying, autogenous and thermal shrinkage strains)	EN1992 part 1-1 EN1994 part 2
	Creep (taken into account through modular ratios)	
P	Possibly, pre-stressing by imposed deformations (for instance, jacking on internal supports)	
Variable loads		
T_k	Thermal gradient	EN1991 part 1-5
UDL, TS	Road traffic (for instance, load model LM1 with uniform design loads UDL and tandem systems TS)	EN1991 part 2
FLM3	Fatigue load model (for instance, the equivalent lorry FLM3)	EN1991 part 2

Effects of shrinkage in a composite bridge



$$-N_{cs} = -E_c \varepsilon_{cs} . b_c h_c$$

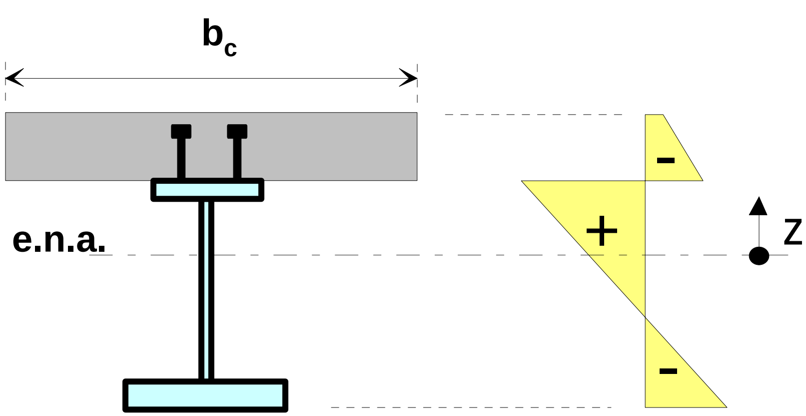
\+



Free shrinkage strain applied on concrete slab only (no steel – concrete interaction)

Shrinkage strain applied on the composite section (after steel – concrete interaction)

1- Auto-equilibrated stress diagram in every section and an imposed rotation due to the bending moment $M_{iso} = N_{cs} z_{cs}$:

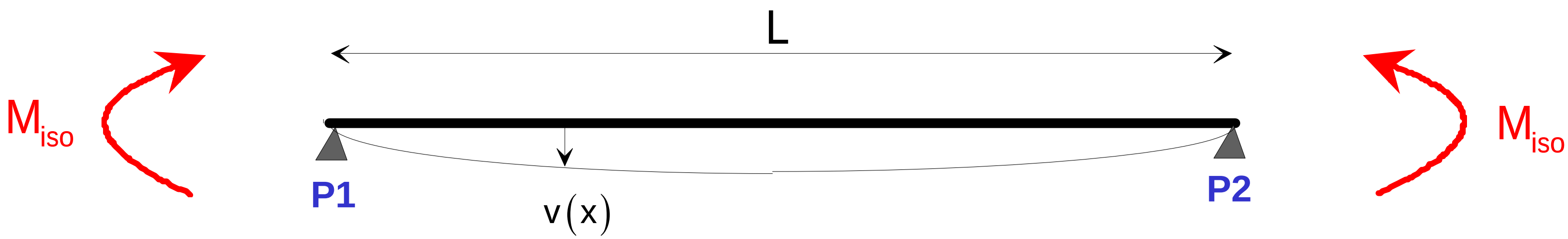


$$\sigma_{concrete} = -E_c \varepsilon_{cs} + \frac{1}{n} . \left[\frac{N_{cs}}{A} + \frac{(N_{cs} z_{cs}).z}{I} \right]$$

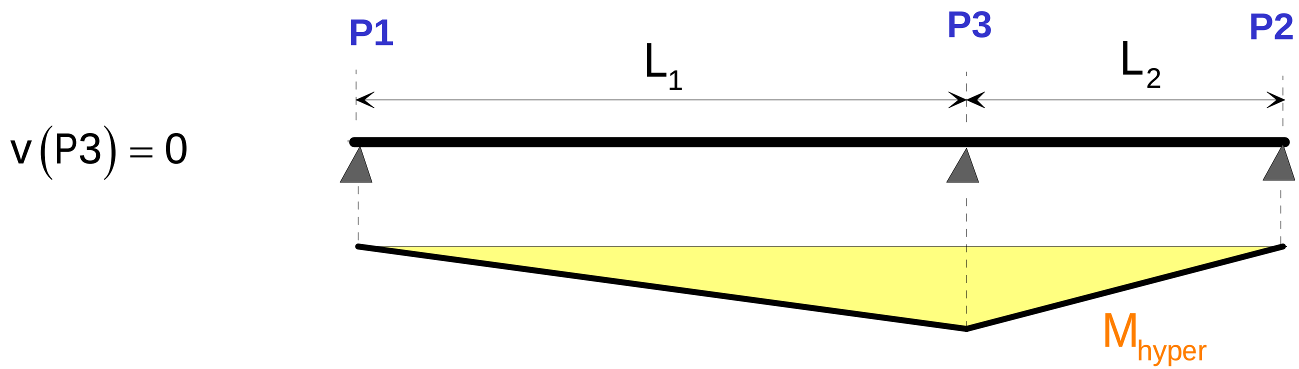
$$\sigma_{steel} = \frac{N_{cs}}{A} + \frac{(N_{cs} z_{cs}).z}{I}$$

Effects of shrinkage in a composite bridge

2- Curvature in an isostatic bridge due to the imposed deformations :



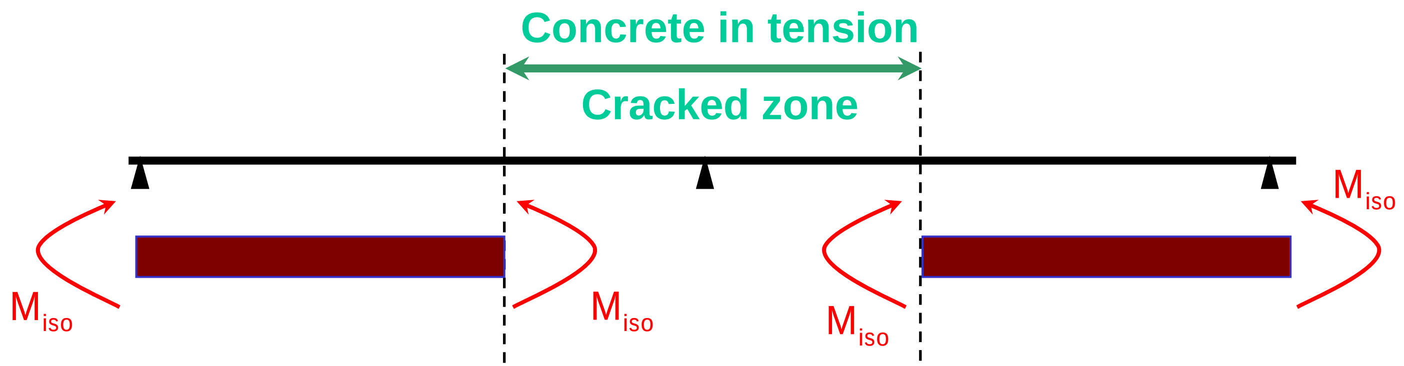
3- Compatibility of deformations to be considered in an hyperstatic bridge :



Effects of shrinkage

- 1+2 = ***isostatic*** (or ***primary***) effects
- 3 = ***hyperstatic*** (or ***secondary***) effects

Shrinkage and cracked global analysis

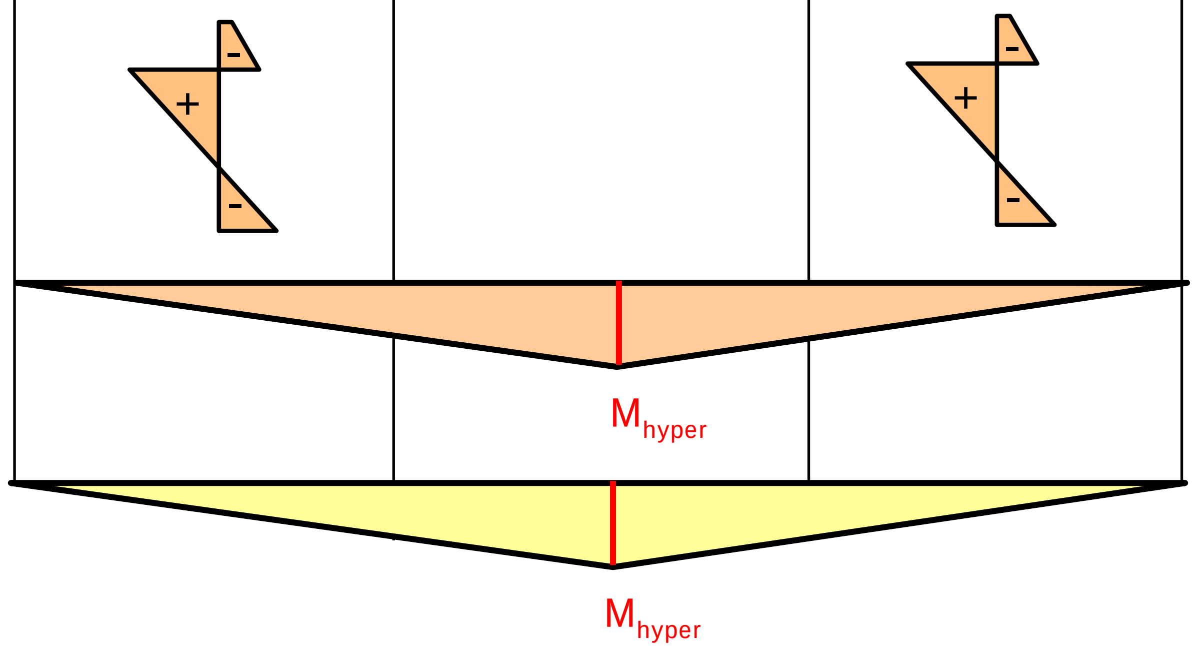


Isostatic effects neglected in cracked zones for calculating hyperstatic effects

SLS combinations	iso + hyper effects	hyper	iso + hyper
ULS combinations	hyper (if class 1)	hyper	hyper (if class 1)

Serviceability Limit State

Ultimate Limit State

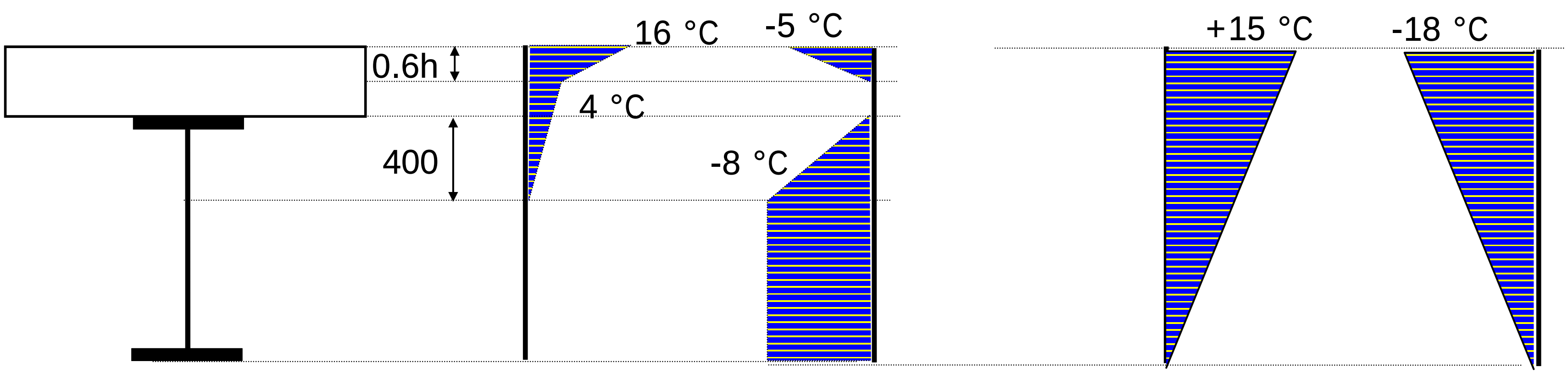


Thermal gradient from EN 1991 part 1-5

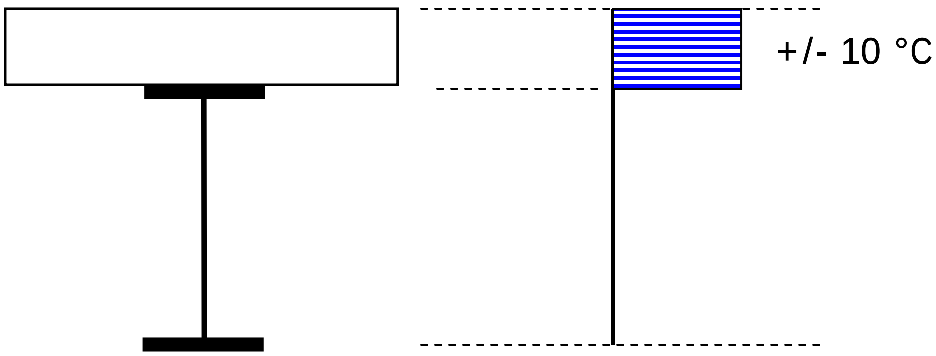
- could be neglected if all cross-sections are in Class 1 or 2

1- Non linear gradients :

2- Linear gradients :

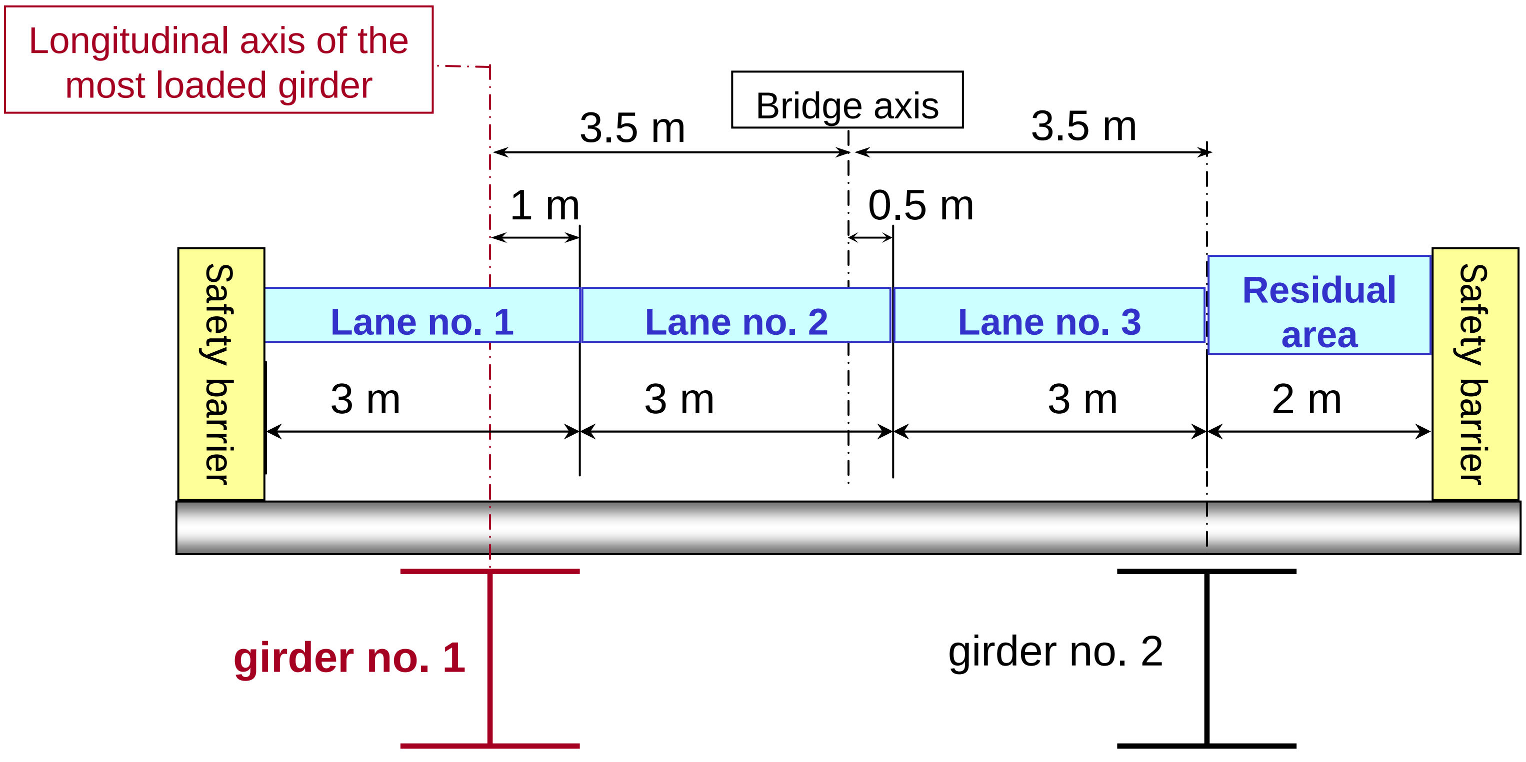


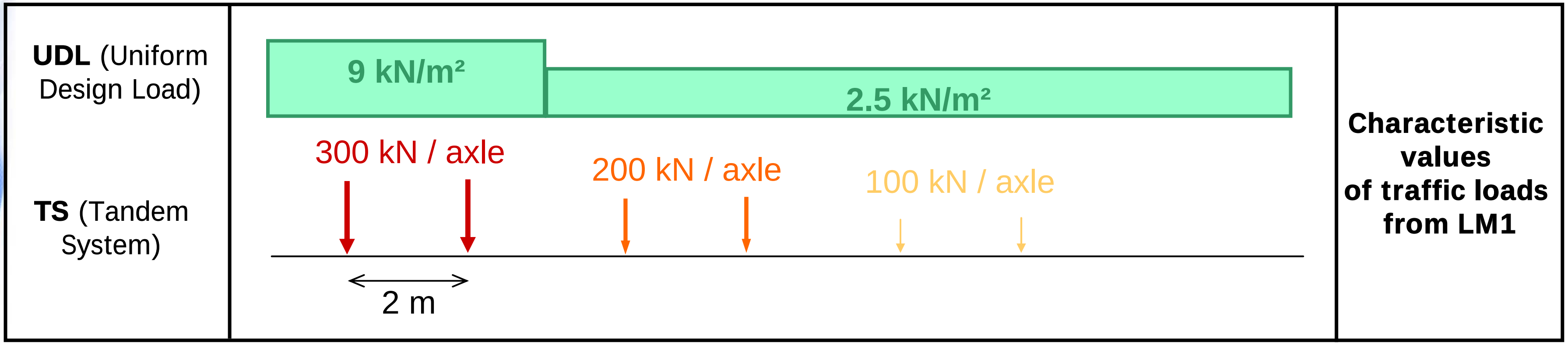
3- Difference +/- 10 °C :



Traffic load LM1 from EN 1991 part 2

Example : Twin-girder composite bridge





For every **permanent design situation**, two limit states of the bridge should be considered :

⇨ Serviceability Limit States (SLS)

- **Quasi permanent SLS**

$G_{max} + G_{min} + S + P + 0.5\ T_k$

- **Frequent SLS**

$G_{max} + G_{min} + S + P + 0.75\ TS + 0.4\ UDL + 0.5\ T_k$

$G_{max} + G_{min} + S + P + 0.6\ T_k$

- **Characteristic SLS**

$G_{max} + G_{min} + S + P + (TS+UDL) + 0.6\ Tk$

$G_{max} + G_{min} + S + P + Q_{lk} + 0.75\ TS + 0.4\ UDL + 0.6\ Tk$

$G_{max} + G_{min} + S + P + Tk + 0.75\ TS + 0.4\ UDL$

⇨ Ultime Limite State (ULS) other than fatigue

$1.35\ G_{max} + G_{min} + S + P + 1.35\ (TS + UDL) + 1.5\ (0.6\ T_k)$

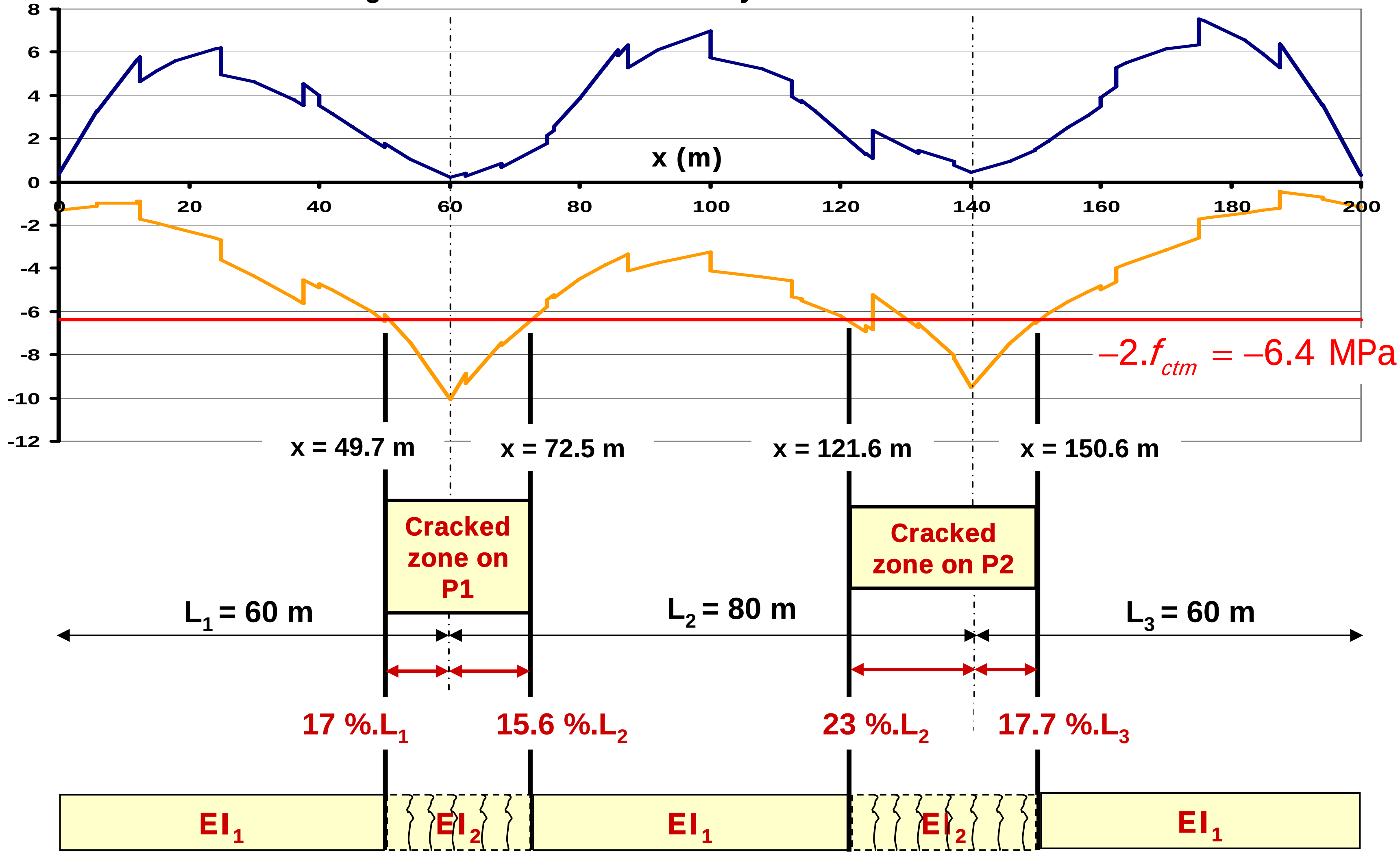
$1.35\ G_{max} + G_{min} + S + P + 1.35\ Q_{lk} + 1.35\ (0.75\ TS + 0.4\ UDL) + 1.5\ (0.6\ T_k)$

$1.35\ G_{max} + G_{min} + S + P + 1.5\ T_k + 1.35\ (0.75\ TS + 0.4\ UDL)$

Un-cracked global analysis

Example : Twin-girder composite bridge

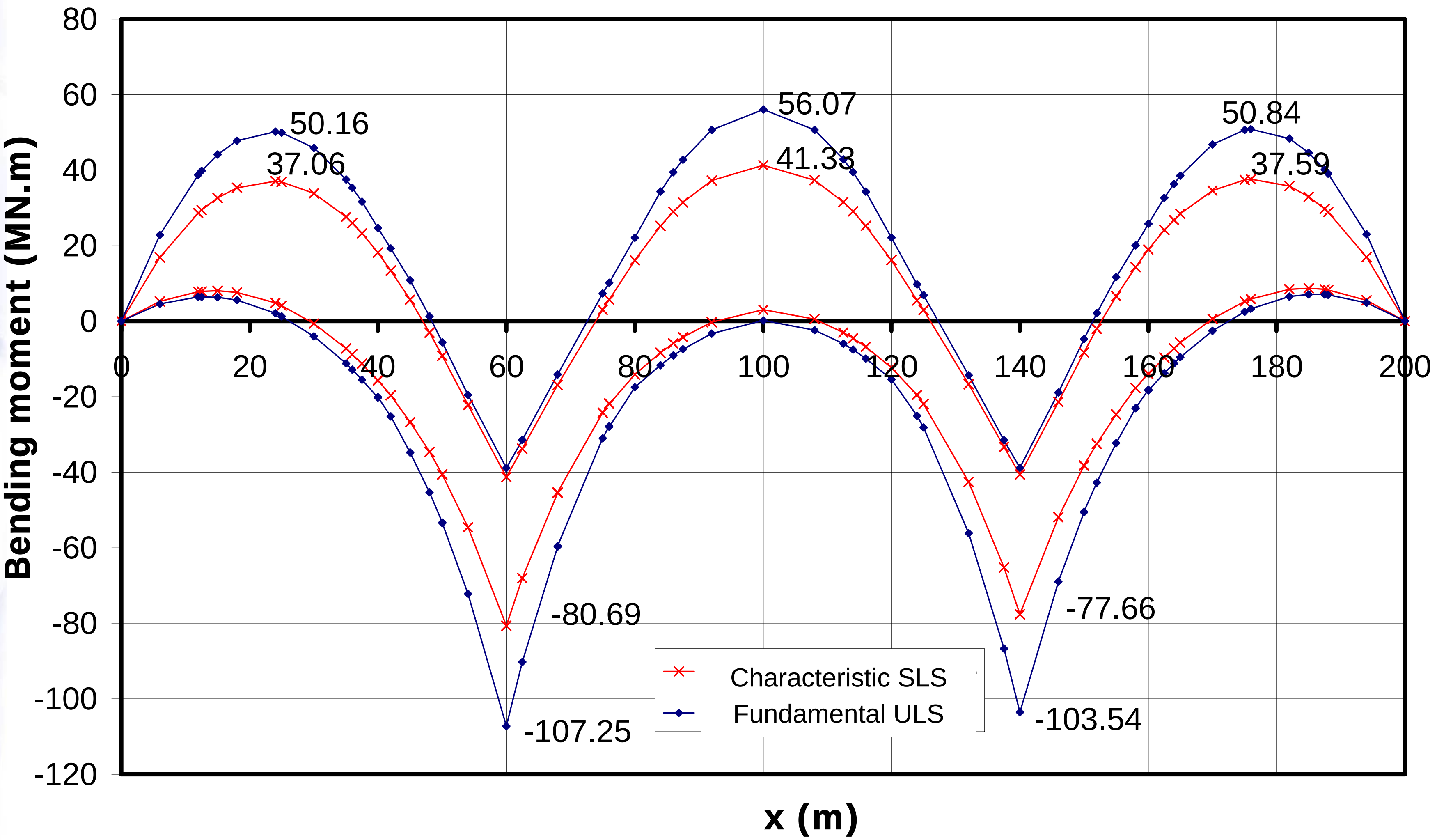
σ (MPa) : Stresses in the extreme fibre of the concrete slab, under Characteristic SLS combination when considering concrete resistance in every cross-section



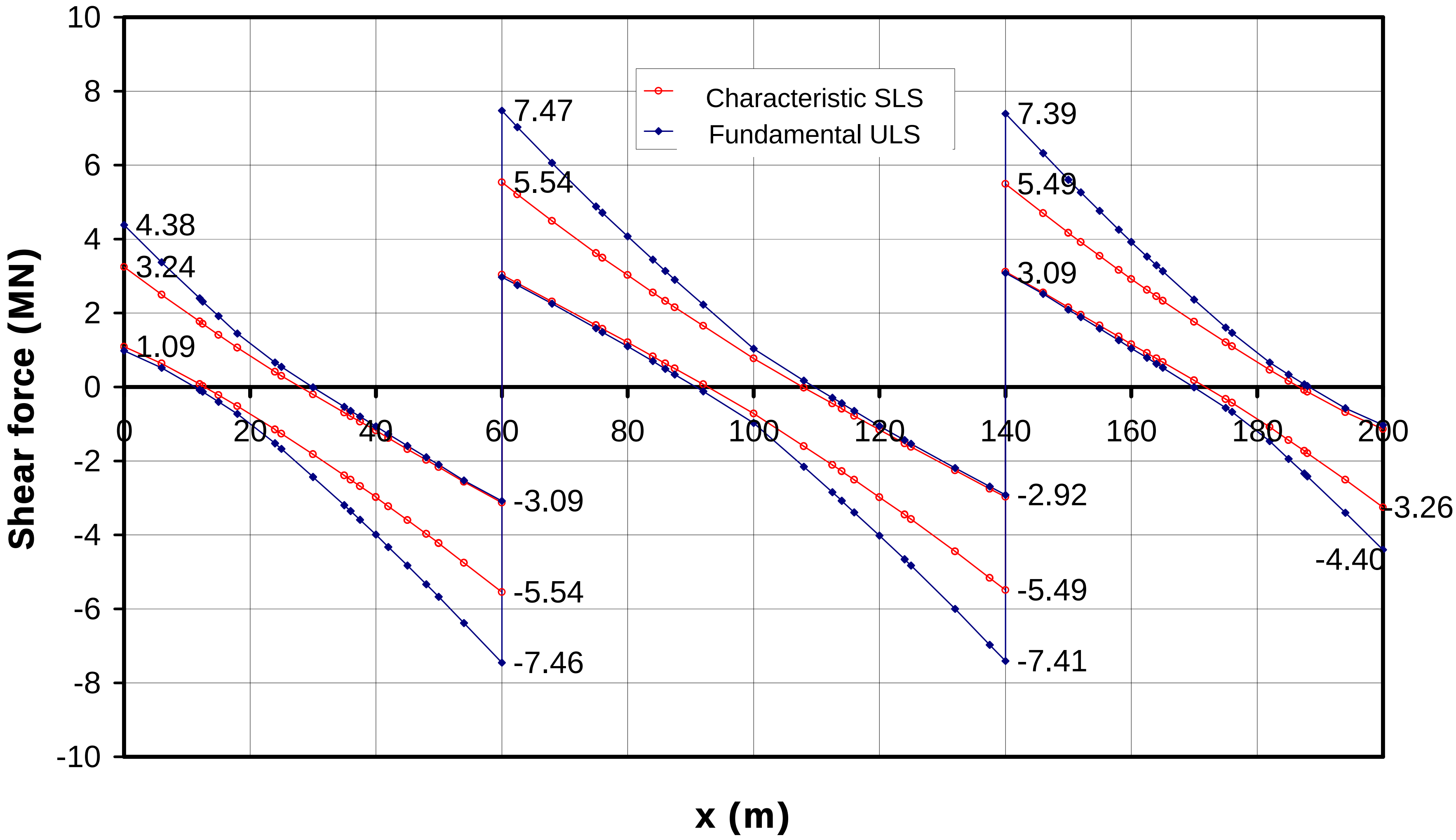
<u>Note</u> : Dissymmetry in the cracked lengths due to sequence of slab concreting.

Cracked global analysis: bending moments

Example : Twin-girder composite bridge



Cracked global analysis: shear forces

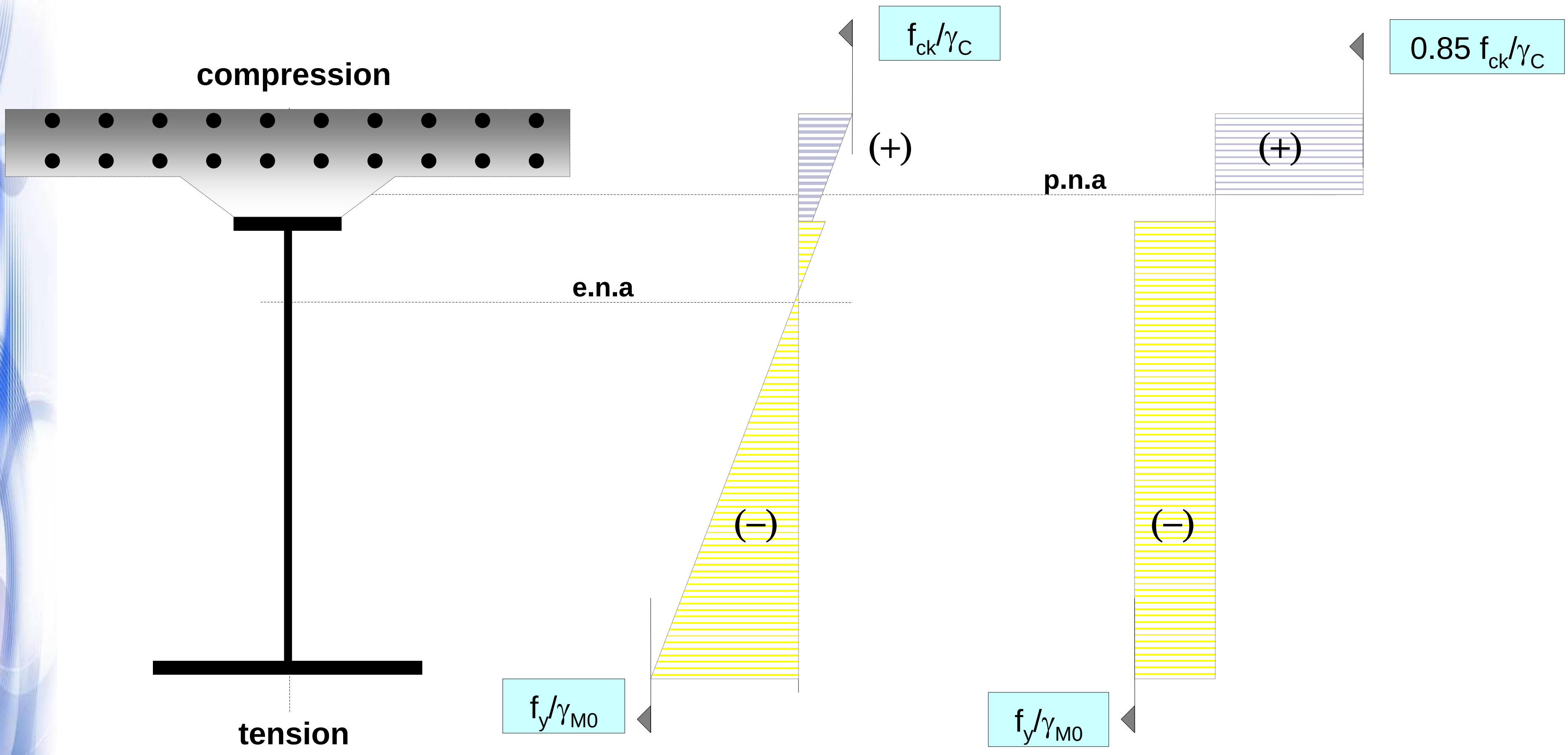


Contents : 7 parts

ULS verifications of a composite bridge

- resistance of the composite cross-sections
  - to bending moment M (EN 1994-2, 6.2.1)
  - to shear force V (EN 1994-2, 6.2.2.1 to 6.2.2.3)
  - to interaction M+V (EN 1994-2, 6.2.2.4)
- shear resistance of the concrete slab (EN 1994-2, 6.2.2.5(3))
- concrete slab (EN 1992)
- shear connection (see below, point 5)
- fatigue ULS (see below, point 6)
- LTB around intermediate supports (see below, point 7)

ULS section resistance to M > 0



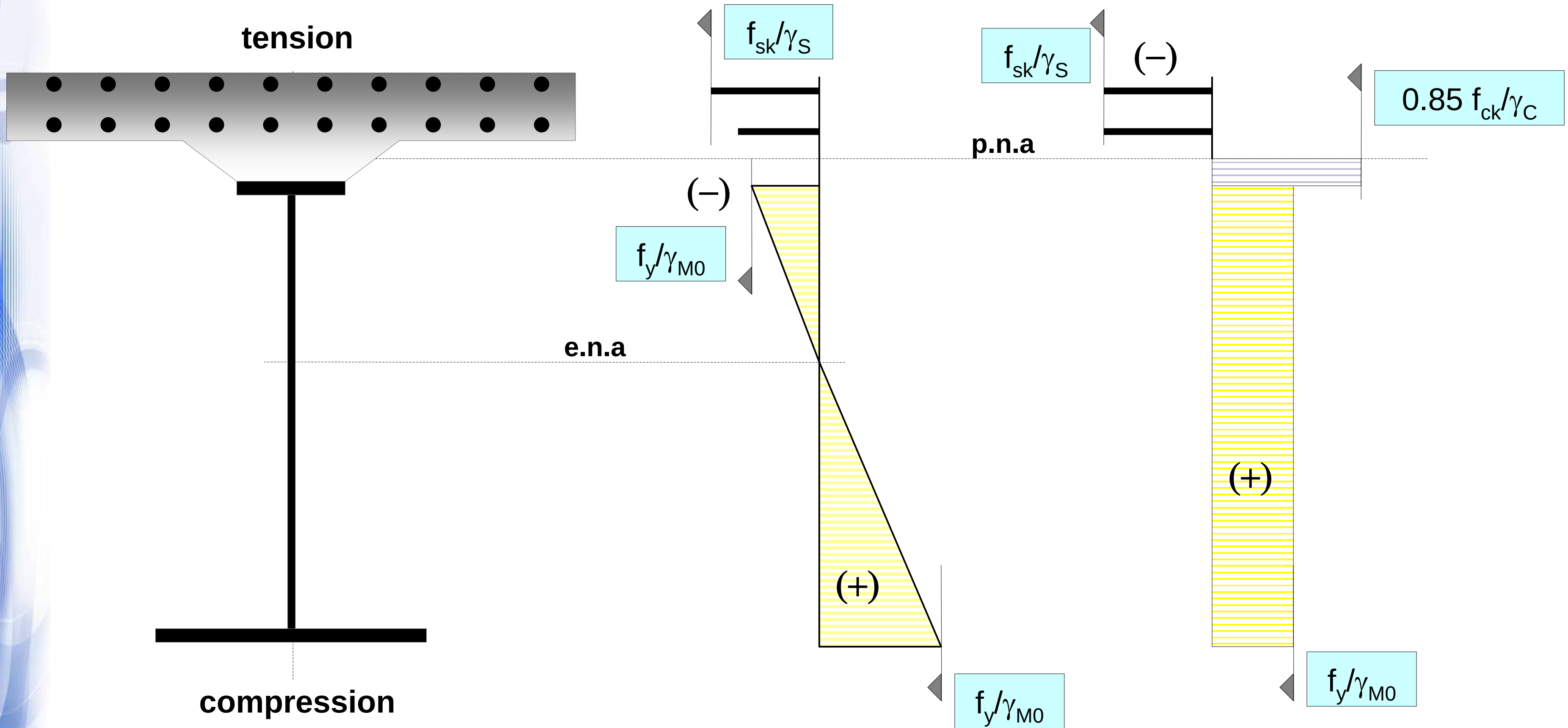
Elastic resistance (for classes 1 to 4)

Plastic resistance (for classes 1 and 2)

e.n.a. = elastic neutral axis

p.n.a. = plastic neutral axis

ULS section resistance to M < 0

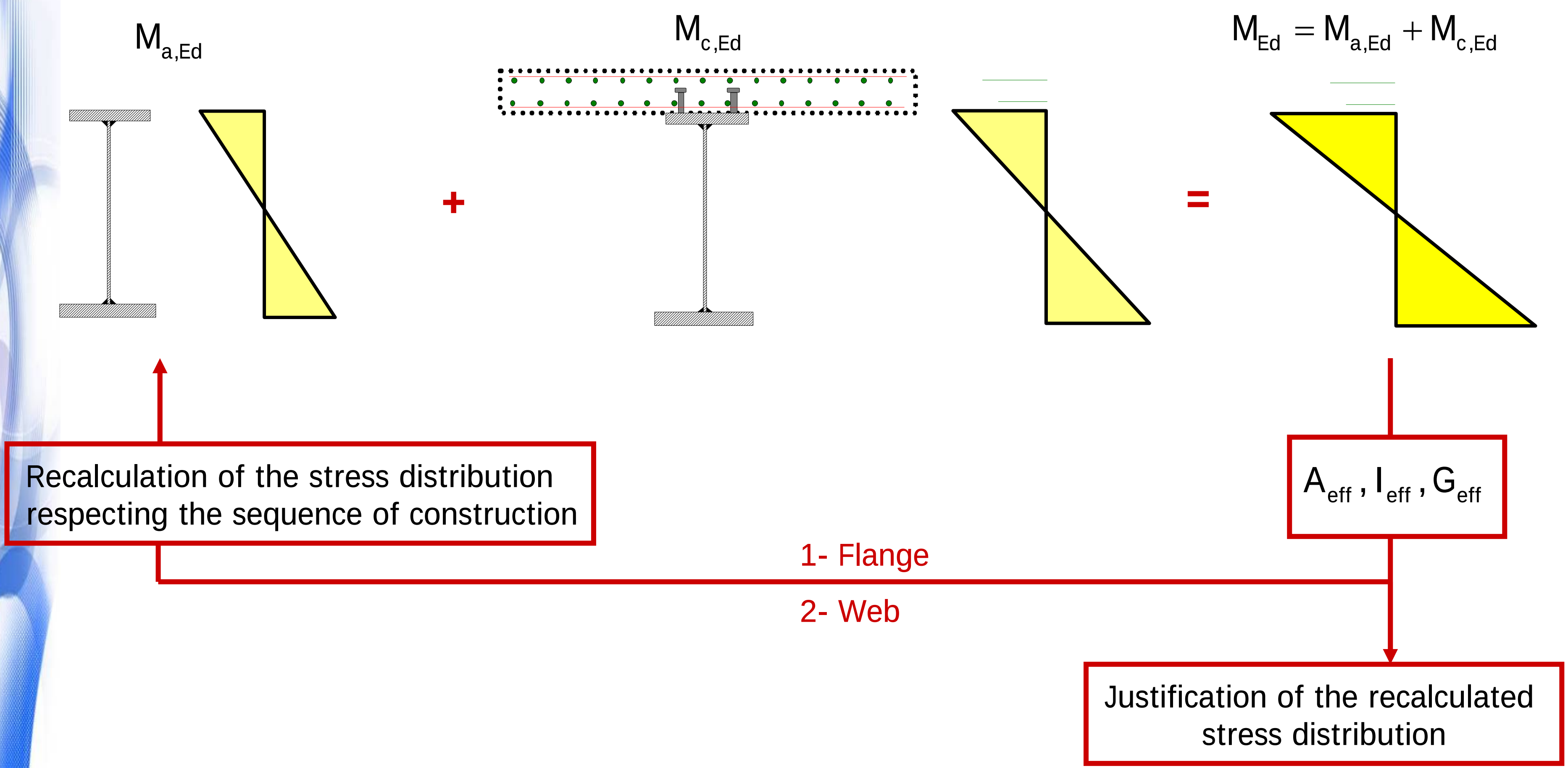


Elastic resistance (for classes 1 to 4)

Plastic resistance (for classes 1 and 2)

Class 4 composite section with construction phases

- Use of the final ULS stress distribution to look for the effective cross-section
- If web and flange are Class 4 elements, the flange gross area is first reduced. The corresponding first effective cross-section is used to re-calculate the stress distribution which is then used for reducing the web gross area.



⇨ Plastic resistance : ensured by the steel web

$V_{pl,a,Rd}$ is calculated by using Eurocode 3 part 1-1.

$$V_{Rd} = V_{pl,a,Rd} = A_V \cdot \frac{f_y}{\gamma_{M0}\sqrt{3}}$$

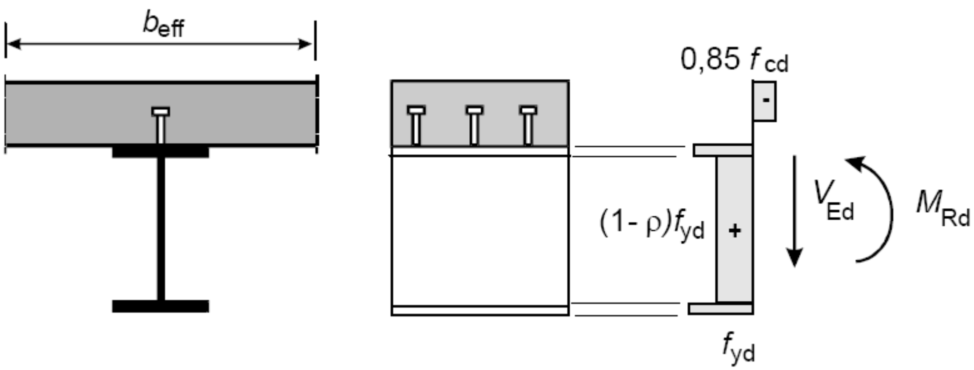
⇨ Shear buckling resistance :

See Eurocode 3 part 1-5.

$$V_{Rd} = V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t_w}{\gamma_{M1}\sqrt{3}}$$

⇨ Interaction between M and V :

- For Class 1 or 2 sections :
  - If $V_{Ed} < 0.5 V_{Rd}$, no interaction occurs.
  - If not, the criterion $M_{Ed} < M_{pl,Rd}$ is verified using a reduced $M_{pl,Rd}$ value



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

- For Class 3 or 4 sections : See Eurocode 3 part 1-5.

ULS shear resistance in the concrete slab

- **For the solid slab of a composite bridge:**

$V_{Ed} \leq V_{Rd,c}$ => Shear reinforcement (A_{st} for b = 1 m) is not necessary (nor the minimum shear reinforcement area according to EN1992-2,9.2.2)

$$V_{Rd,c} = \left[C_{Rd,c} k (100 \rho_l f_{ck})^{1/3} + k_1 \sigma_{cp}\right] b h_c \geq (v_{min} + k_1 \sigma_{cp}) b h_c$$

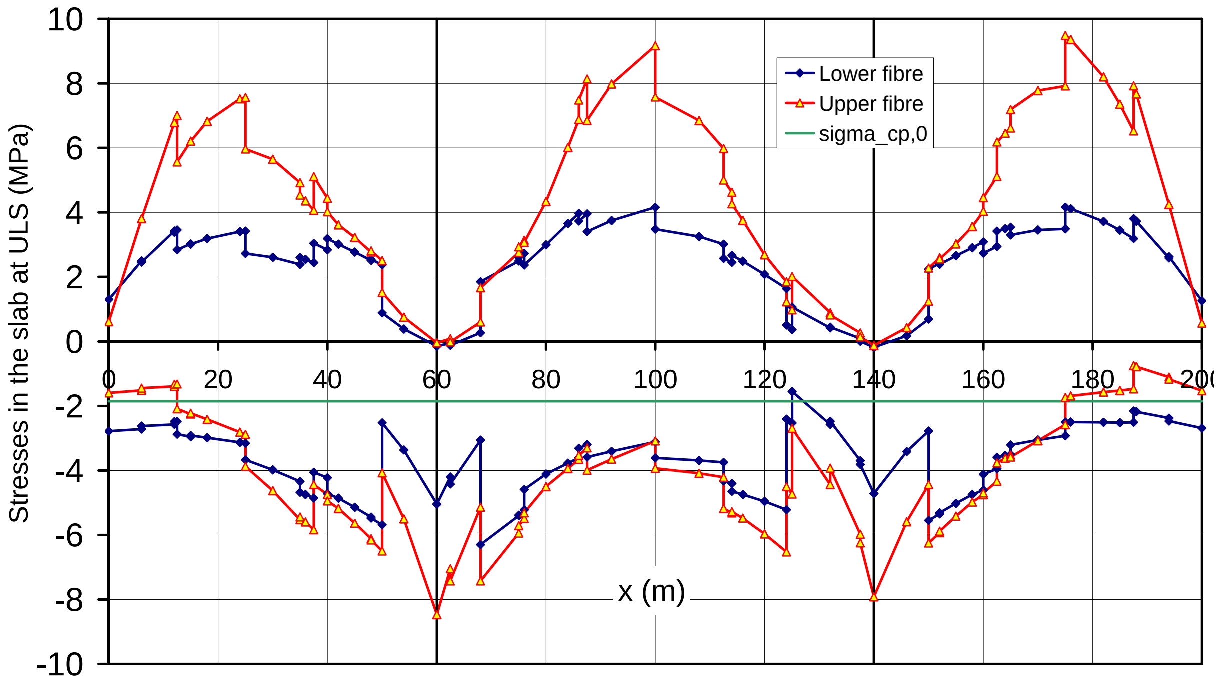
- **If the concrete flange is in tension :** $C_{Rd,c} = \frac{0.15}{\gamma_C} = 0.12$ $k_1 = 0.12$

$$\sigma_{cp,0} = -1.85 \text{ MPa} \leq \sigma_{cp} = \frac{N_{Ed}}{b h_c}$$

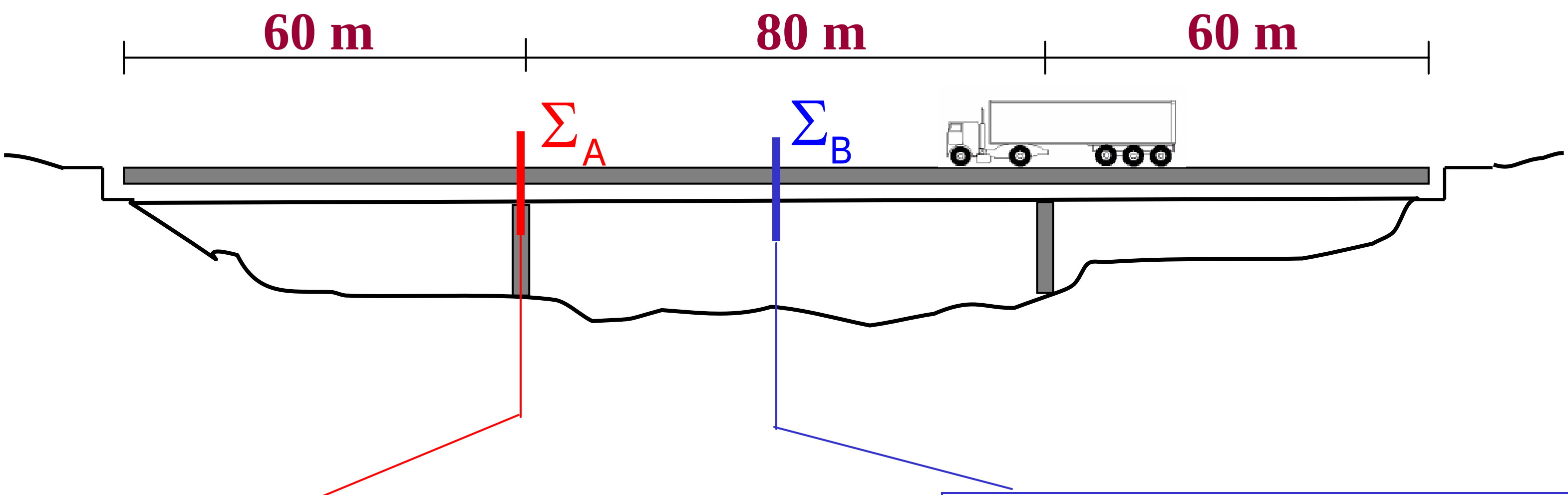
$$v_{min} = 0.035 . k^{1.5} \sqrt{f_{ck}}$$

$$k = 1 + \sqrt{\frac{200}{h_c}}$$

$$\rho_l = \frac{A_{st}}{b h_c}$$



Example: Analysis of 2 different cross-sections



Section Σ_A

Concrete in tension

$M<0$

Class 3 (elastic section analysis)

M_{ULS} = -107.25 MN.m

V_{ULS} = 7.47 MN

Section Σ_B

Concrete in compression

$M>0$

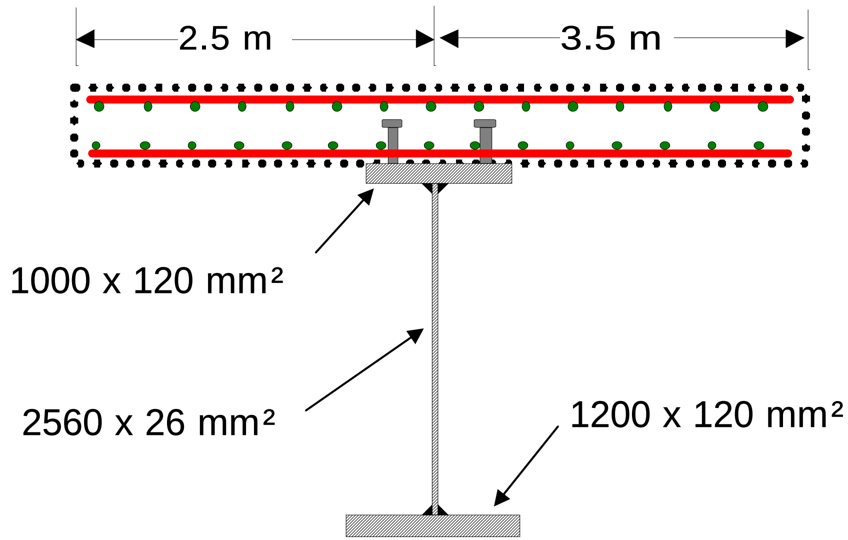
Class 1 (plastic section analysis)

M_{ULS} = +56.07 MN.m

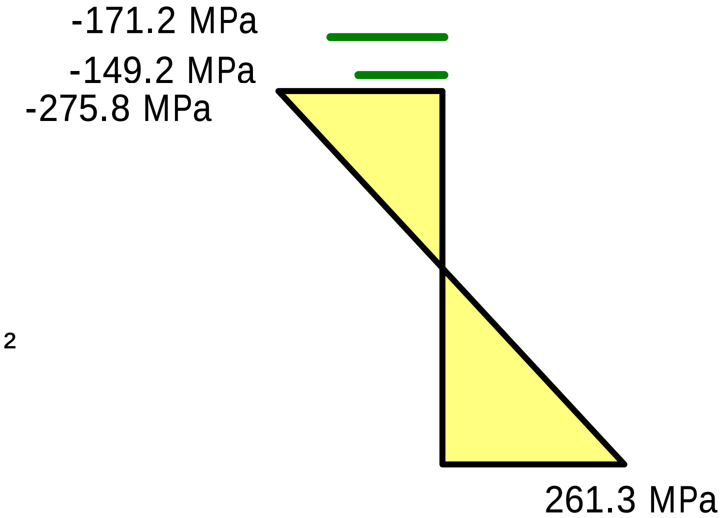
V_{ULS} = 1.04 MN

Example: Cross-section Σ_A under bending

Example : Twin-girder composite bridge



Stress diagram under bending



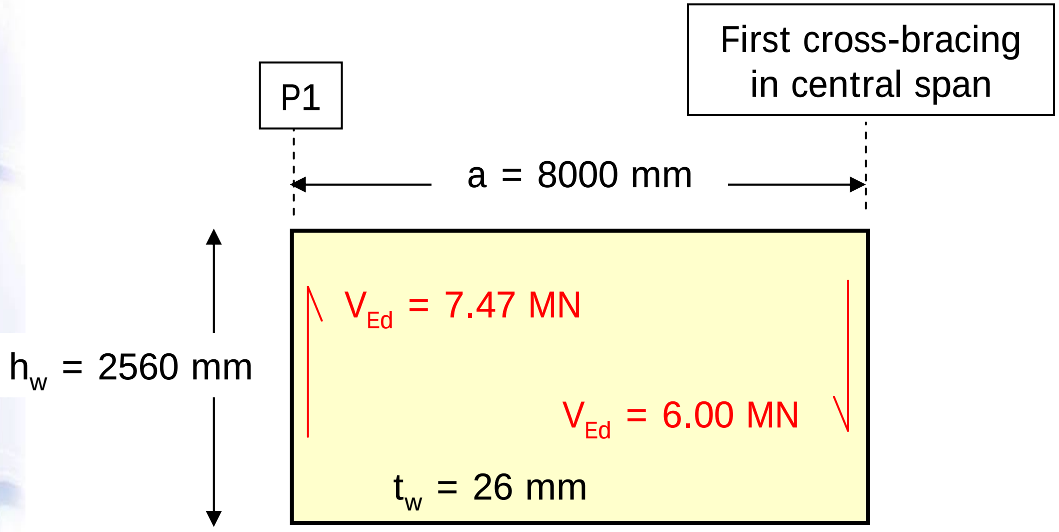
Elastic section analysis :

$$-434.8\text{ MPa} = -\frac{f_{sk}}{\gamma_S} \leq \sigma_{reinf.}$$

$$-295\text{ MPa} = -\frac{f_y}{\gamma_{M0}} \leq \sigma_{steel,sup}$$

$$\sigma_{steel,inf} \leq \frac{f_y}{\gamma_{M0}} = 295\text{ MPa}$$

Example: Cross-section Σ_A under shear force



$$k_\tau = 5.34 + 4\left(\frac{h_w}{a}\right)^2 = 5.75$$

$$\frac{h_w}{t_w} \geq \frac{31\varepsilon}{\eta}\sqrt{k_\tau}$$

Shear buckling to be considered:

$$V_{Rd} = V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t_w}{\gamma_{M1}\sqrt{3}}$$

Contribution of the web $V_{bw,Rd}$	Contribution of the flange $V_{bf,Rd}$
$\tau_{cr} = k_\tau \sigma_E = 19.58$ MPa $\bar{\lambda}_w = \sqrt{\frac{f_{yw}}{\tau_{cr}\sqrt{3}}} = 1.33 \geq 1.08$ $\chi_w = \frac{1.37}{0.7 + \bar{\lambda}_w} = 0.675$ $V_{bw,Rd} = \chi_w \frac{f_{yw}}{\gamma_{M1}\sqrt{3}} h_w t_w = 8.14$ MN	$V_{bf,Rd} = 0.245$ MN can be neglected.

$\frac{V_{Ed}}{V_{Rd}} \geq 0.5$ so the M+V interaction should be checked, and as the section is in Class 3, the following criterion should be applied (EN1993-1-5) :

$$\bar{\eta}_1 + \left[1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right]\left[2\bar{\eta}_3 - 1\right]^2 \leq 1.0$$

at a distance $h_w/2$ from internal support P1.

$M_{f,Rd} = 117.3$ MN.m : design plastic resistance to bending of the effective composite section excluding the steel web (EN 1994-2, 6.2.2.5(2)).

$M_{pl,Rd} = 135.6$ MN.m : design plastic resistance to bending of the effective composite section.

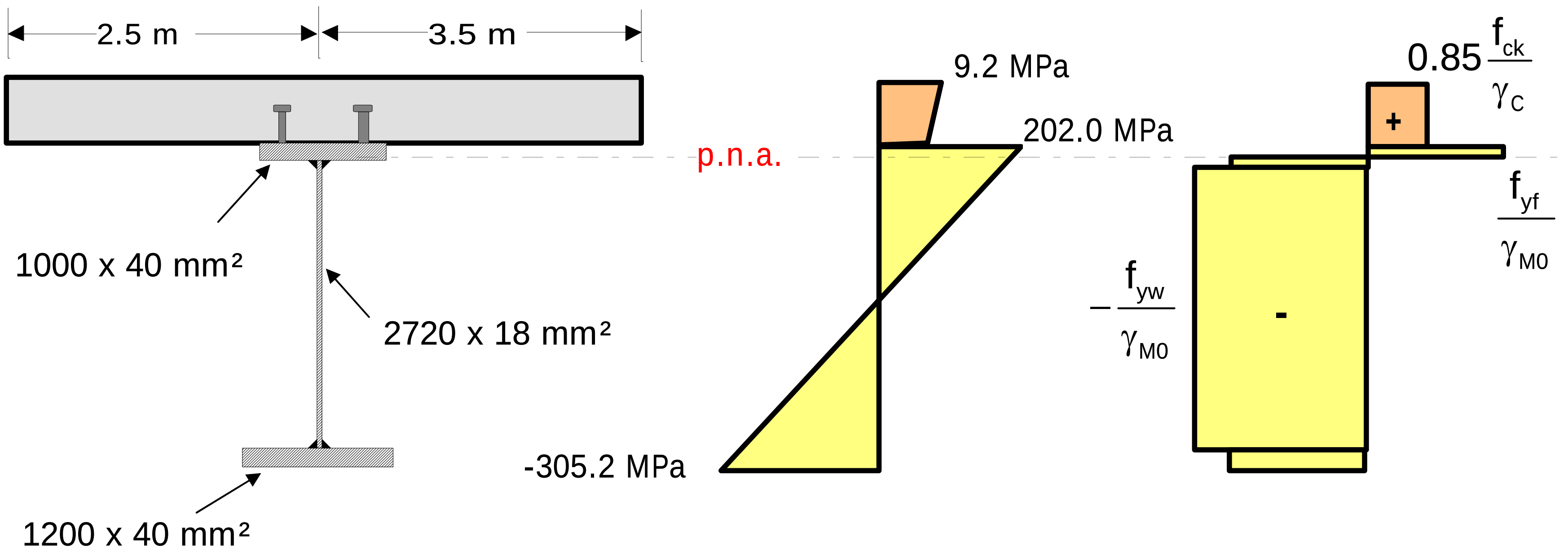
$$\bar{\eta}_3 = \frac{V_{Ed}}{V_{bw,Rd}} = 0.89$$

$$\bar{\eta}_1 = \frac{M_{Ed}}{M_{pl,Rd}} = 0.73 \leq \frac{M_{f,Rd}}{M_{pl,Rd}} = 0.86$$

As $M_{Ed} < M_{f,Rd}$, the flanges alone can be used to resist M whereas the steel web resists V.

=> No interaction !

Example: Cross-section Σ_B (Class 1)



Plastic section analysis under bending : $M_{Ed} = 56.07 \leq M_{pl,Rd} = 79.59$ MN.m

$k_\tau = 5.34 + 4\left(\frac{h_w}{a}\right)^2 = 5.80$ and $\frac{h_w}{t_w} \geq \frac{31\varepsilon}{\eta}\sqrt{k_\tau}$, so the shear buckling has to be considered:

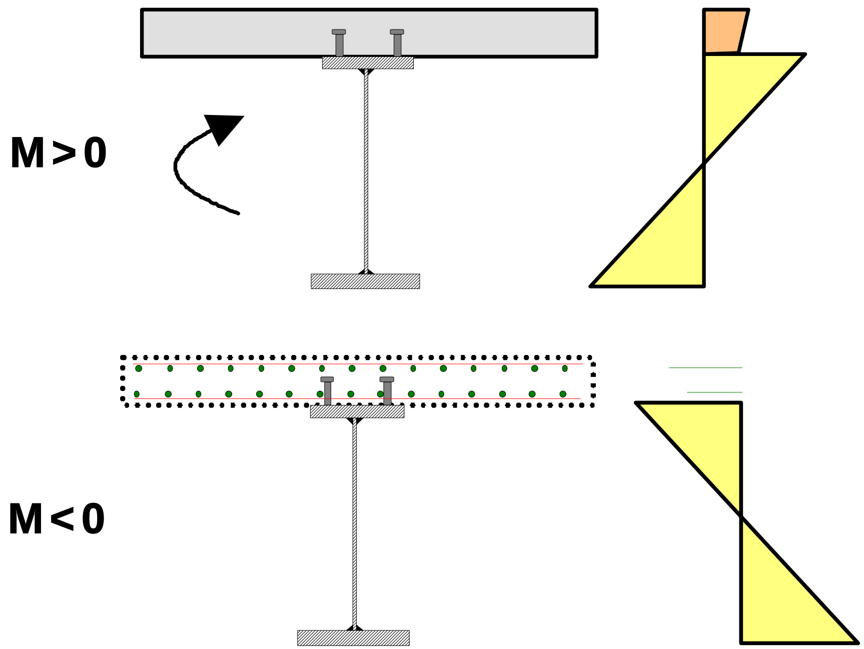
$$V_{Ed} = 2.21 \text{ MN } \leq V_{Rd} = V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \approx V_{bw,Rd} = 4.44 \text{ MN} \leq \frac{\eta f_{yw} h_w t_w}{\gamma_{M1}\sqrt{3}} = 10.64 \text{ MN}$$

$\frac{V_{Ed}}{V_{Rd}} \leq 0.5$ => No M+V interaction !

Contents : 7 parts

SLS verifications in a composite bridge

- Limitation of stresses in cross-sections at characteristic SLS



$\sigma_c \leq 0.6.f_{ck}$ (concrete in compression)

$\sigma_a \leq 1.0.f_{yk}$

$\sigma_s \leq 0.8.f_{sk}$ (reinforcement in tension)

$\sigma_a \leq 1.0.f_{yk}$

- Crack width control
- Limitations of deflections (national regulations)
- Web breathing (fatigue phenomenon, see EN1993-2)

<u>Note</u> : for shear connectors, see section 5 below

Crack width control

1. Minimum reinforcement required

- in cross-sections where tension exists in the concrete slab for characteristic SLS combinations of actions

- estimated from equilibrium between tensile force in concrete just before cracking and tensile force in the reinforcement (at yielding or at a lower stress level if necessary to limit the crack width)

2. Control of cracking due to direct loading

The design crack width w_k should be limited to a maximum crack width w_{max} by limiting :

- bar spacing $s \leq s_{max}$

- **or** bar diameter $\Phi \leq \Phi_{max}$

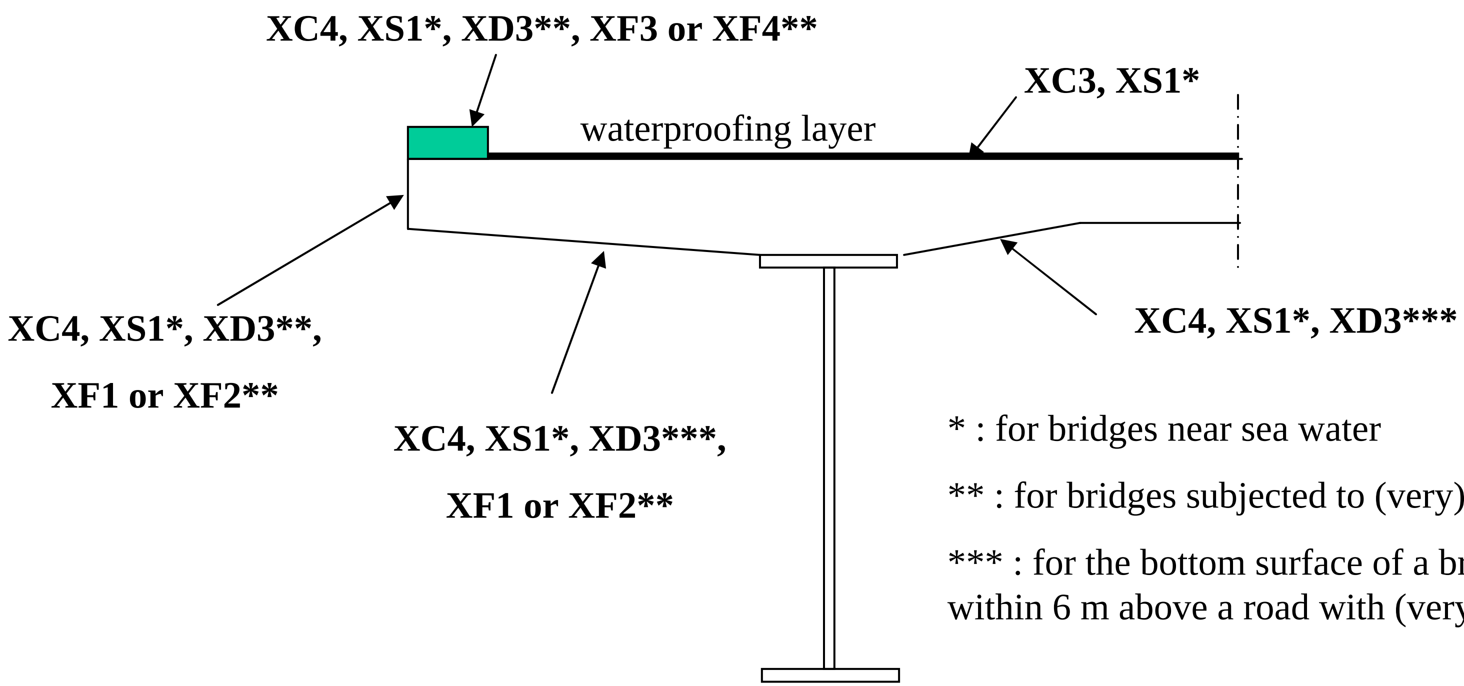
w_{max} depends on the exposure class of the considered concrete face

s_{max} and Φ_{max} depend on the calculated stress level $\boldsymbol{\sigma_s = \sigma_{s,0} + \Delta\sigma_s}$ in the reinforcement and on the design crack width w_k

3. Control of cracking due to indirect loading

For instance, concrete shrinkage.

Exposure classes for composite bridges (durability)



* : for bridges near sea water

** : for bridges subjected to (very) frequent salting

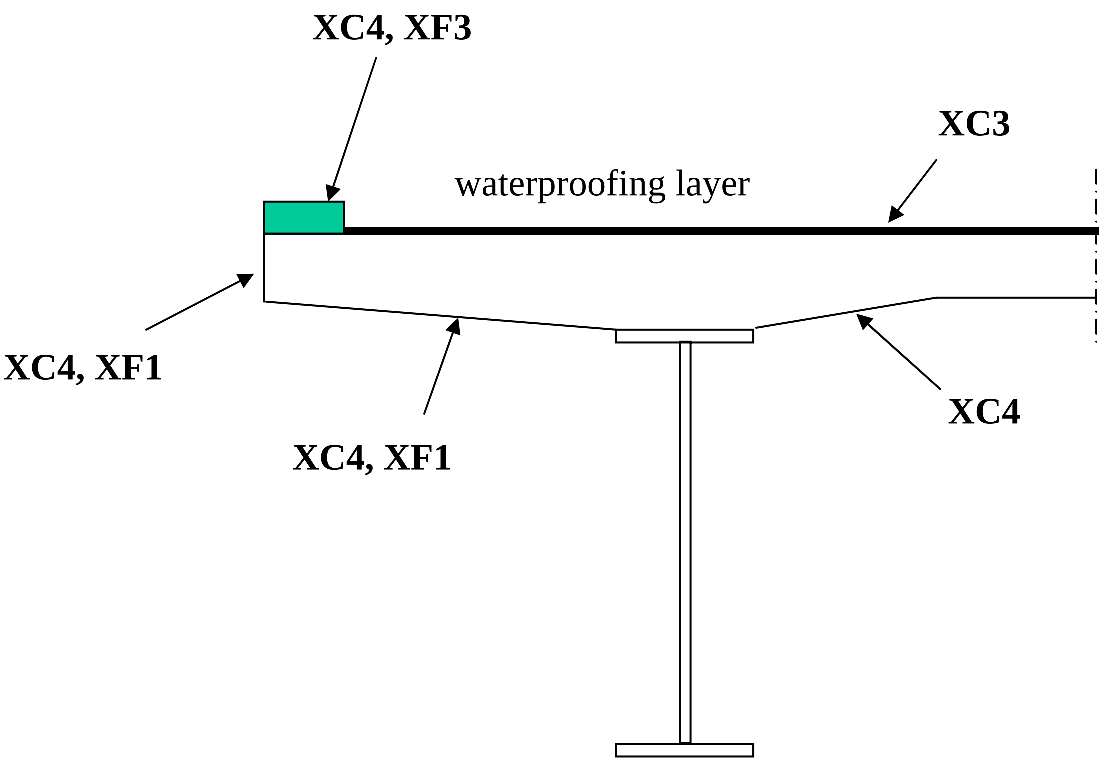
*** : for the bottom surface of a bridge deck located within 6 m above a road with (very) frequent salting

	Class	Description of the environment
	XO	No risk of corrosion or attack of concrete
Risk of corrosion of reinforcement	XC1 to XC4	Corrosion induced by carbonation
	XD1 to XD3	Corrosion induced by chlorides
	XS1 to XS3	Corrosion induced by chlorides from sea water
Attack to concrete	XF1 to XF4	Freeze/thaw attack
	XA1 to XA3	Chemical attack
	XM	Mechanical abrasion

Exposure classes for composite bridges (durability)

Example : Twin-girder composite bridge

Hypothesis : Bridge in a low-level frost area



The choice of exposure classes leads to define :

- a minimum resistance for concrete (according to EN1992 and EN206), for instance C30/37
- a concrete makeup (maximum E/C ratio, minimum cement content) according to EN206
- a structural class (S1 to S6) for every face of the slab, chosen according to Table 4.3 in EN1992 and to the retained concrete
- a minimum concrete cover for every face of the slab according to the exposure class and the structural class

Maximum crack width w_{max}

Recommended values defined in EN1992-2 (concrete bridges) :

Table 7.101N — Recommended values of w_{max} and relevant combination rules

Exposure Class	Reinforced members and prestressed members without bonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
X0, XC1	0,3[a]	0,2
XC2, XC3, XC4	0,3	0,2[b]
XD1, XD2, XD3 XS1, XS2, XS3		Decompression

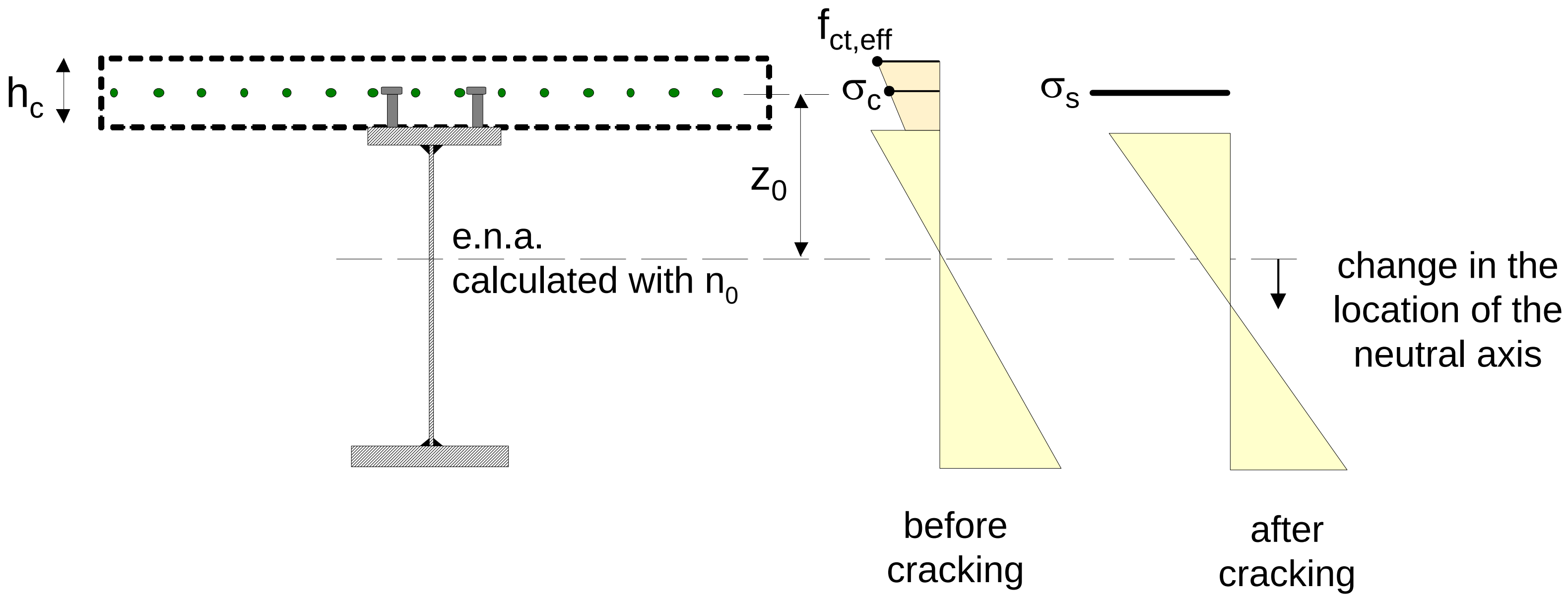
[a] For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed.

[b] For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.

The stress level $\sigma_{s,0}$ in the reinforcement is calculated for the **quasi-permanent SLS** combination of actions (in case of reinforced concrete slab).

The tension stiffening effect $\Delta\sigma_s$ should be taken into account.

Minimum reinforcement



$$k_s k . \left[k_c f_{ct,eff} A_{ct} \right] = \sigma_s A_s$$

$$k_c = \frac{1}{1+\frac{h_c}{2z_0}} + 0.3 \leq 1.0$$

stress distribution within the tensile concrete height h_c before cracking (including indirect loading) + change in the location of the neutral axis at cracking time

$k_s = 0.9$

reduction of the normal force in the concrete slab due to initial cracking and local slip of the shear connection

$k = 0.8$

effect of non-uniform shape in the self-equilibrating stresses within h_c

$f_{ct,eff} = f_{ctm}$ and $\sigma_s = f_{sk}$ give the minimum reinforcement section $A_{s,min}$.

The elastic neutral axis is located in the steel web for every section of the bridge, so A_{ct} is the slab section : A_{ct} = 6 x 0.34 = 2.04 m²

h_c = 0.34 m

z_0 = 0.52 m

$$k_c = \min\left[\frac{1}{1+\frac{h_c}{2z_0}} + 0.3; 1.0\right] = 1.0$$

$f_{ct,eff} = f_{ctm}$ = -3.2 Mpa

f_{sk} = 500 MPa

$A_{s,min}$ = 94 cm² which means a minimum reinforcement ratio $\rho_{s,min} = 0.46\%$

For the design, the following reinforcement ratios have been considered :

- Top layer : high bonded bars with ϕ = 16 mm and s = 130 mm, so $\rho_{s,top} = 0.46\%$
- Bottom layer : high bonded bars with ϕ = 16 mm and s = 130 mm, so $\rho_{s,bottom} = 0.46\%$

We verify : $\rho_{s,top} + \rho_{s,bottom} = 0.92\% \geq \rho_{s,min}$

Crack width control

A_{st} is put in place through **n** high bonded bars of diameter ϕ per meter.

Diameter ϕ*

(Table 7.1)

Steel stress σ_s (N/mm^2)	Maximum bar diameter ϕ^* (mm) for design crack width w_k		
	w_k=0.4mm	w_k=0.3mm	w_k=0.2mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

$$\Phi = \Phi^* \frac{f_{ct,eff}}{2.9\ \text{MPa}}$$

or

Spacing s = 1/n

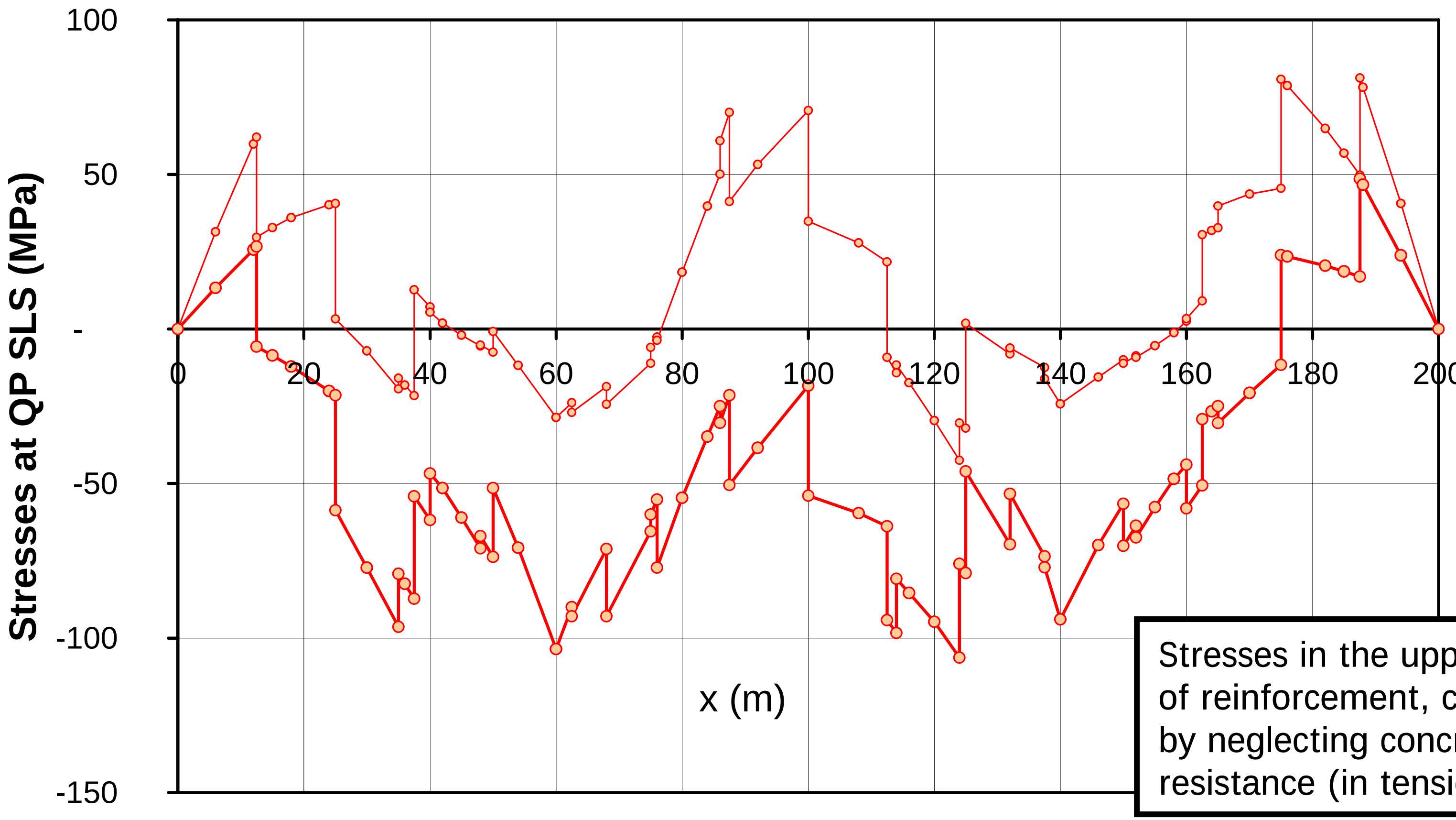
(Table 7.2)

Steel stress σ_s (N/mm^2)	Maximum bar spacing (mm) for design crack width w_k		
	w_k=0.4mm	w_k=0.3mm	w_k=0.2mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

Example : crack width control for direct loading

The stress level σ_s due to direct loading at quasi-permanent SLS combinations of actions can be calculated :

- Top and bottom layers : A_{st} with ϕ = 16 mm and s = 130 mm, so $\rho_{s,top} = \rho_{s,bottom} = 0.46\%$
- $\sigma_{s,0}$ = 106 Mpa (maximum tension) at quasi-permanent SLS in the top layer



Stresses in the upper layer of reinforcement, calculated by neglecting concrete resistance (in tension).

Example : crack width control for direct loading

- Tension stiffening effect : $\Delta\sigma_s = 0.4 \dfrac{f_{ctm}}{\rho_s \alpha_{st}}$
- in the considered cross-section (where $\sigma_{s,0}$ is maximum) :

$$\alpha_{st} = \frac{AI}{A_a I_a} = 1.31 \qquad \rho_s = 0.92\% \quad \text{(Reinforcement ratio)}$$

- $\Delta\sigma_s = 0.4 \dfrac{f_{ctm}}{\rho_s \alpha_{st}} = 106.2 \text{ MPa}$
- $\sigma_s = \sigma_{s,0} + \Delta\sigma_s = 212.2 \text{ MPa}$

- $\Phi^*_{max} = 22.3 \text{ mm}$ (interpolation in Table 7.1 of EN 1994-2)
- $\Phi = 16 \text{ mm} \leq \Phi_{max} = \Phi^*_{max} 3.2/2.9 = 24.6 \text{ mm}$

or

- $s_{max} = 235 \text{ mm}$ (interpolation in Table 7.2 of EN 1994-2)
- $s = 130 \text{ mm} \leq s_{max} = 235 \text{ mm}$

The stress level σ_s due to indirect loading (for instance, concrete shrinkage) can not be calculated in the reinforcement.

In the sections where the concrete slab is in tension for characteristic SLS combinations of actions, σ_s is estimated using :

$$\sigma_s = k_s k k_c f_{ct,eff} \frac{A_{ct}}{A_s} = 0.9 \; 0.8 \; 1.0 \; 3.2 \; \frac{2.04}{0.92\% \; 2.04} = 250.4 \text{ MPa}$$

The reinforcement layers are designed using high bonded bars with ϕ = 16 mm.

$\phi^* = \phi \, f_{ct,eff}/f_{ct,0} = 2.9/3.2 = 14.5$ mm

The interpolation in Table 7.1 from EN 1994-2 gives : $\sigma_{s,max}$ = 255 Mpa

We verify :

$$\sigma_s = 250.4 \text{ Mpa} < \sigma_{s,max} = 255 \text{ Mpa}$$

Contents : 7 parts

Steel-concrete connection

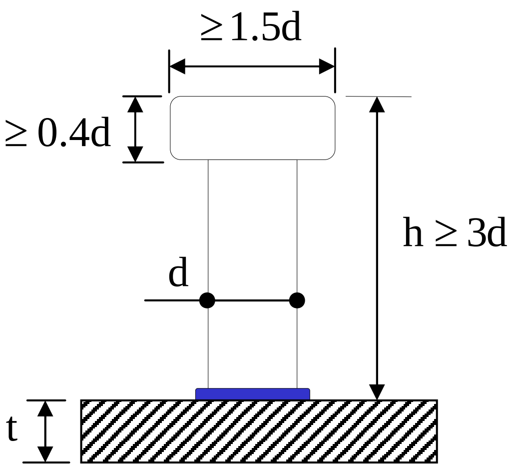
Objective :

Transmit the longitudinal shear force $v_{L,Ed}$ per unit length of the steel-concrete interface

Performed by the use of shear connectors (only studs in EN1994) and transverse reinforcement

- Full interaction required for bridges
- Elastic resistance design of the shear connectors at SLS and at ULS
- Plastic resistance design of the shear connectors at ULS in Class 1 or 2 cross sections where $M_{el,Rd} \leq M_{Ed} \leq M_{pl,Rd}$
- Shear connectors locally added due to concentrated longitudinal shear force (for instance, shrinkage and thermal action at both bridge deck ends or cable anchorage)
- ULS design of transverse reinforcement to prevent longitudinal shear failure or splitting in the concrete slab

Resistance of the headed stud shear connector



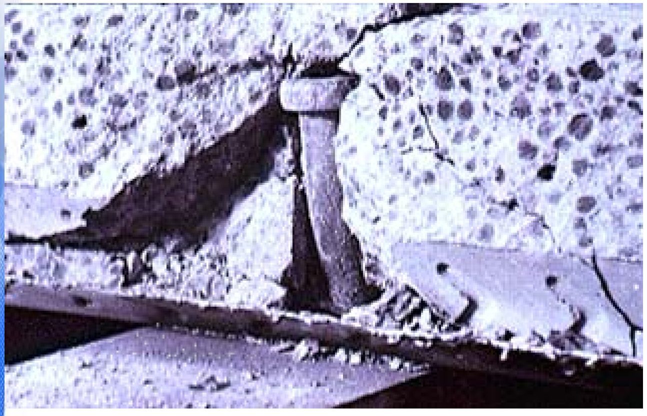
$16 \le d \le 25mm$

$$P_{Rk} = \min\left[P_{Rk}^{(1)} ; P_{Rk}^{(2)} \right]$$

- Shank shear resistance : $P_{Rk}^{(1)} = 0.8 f_u . \left\{ \frac{\pi d^2}{4} \right\}$
- Concrete crushing : $P_{Rk}^{(2)} = 0.29 \alpha d^2 \sqrt{f_{ck} E_{cm}}$

if $3 \le \frac{h}{d} \le 4$, then $\alpha = 0.2 \left(\frac{h}{d} + 1 \right)$

else $\alpha = 1$



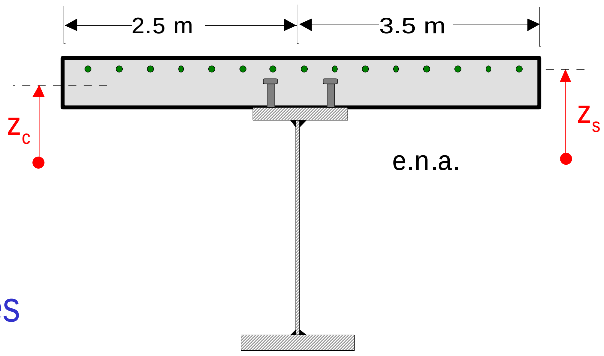
Limit State	Design resistance	National Annex
U.L.S.	$P_{Rd} = \frac{P_{Rk}}{\gamma_V}$	$\gamma_V = 1.25$
S.L.S.	$k_s . P_{Rd}$	$k_s = 0.75$

• SLS and ULS elastic design using the shear flow $v_{L,Ed}$ at the steel-concrete interface, which is calculated with an ***uncracked*** behaviour of the cross sections.

Shear force from cracked global analysis

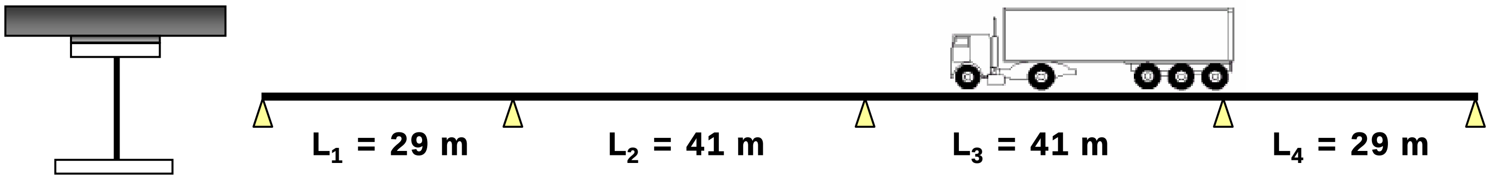
$$v_{L,Ed}(x) = V_{Ed}(x).\frac{A_c z_c + A_s z_s}{I}$$

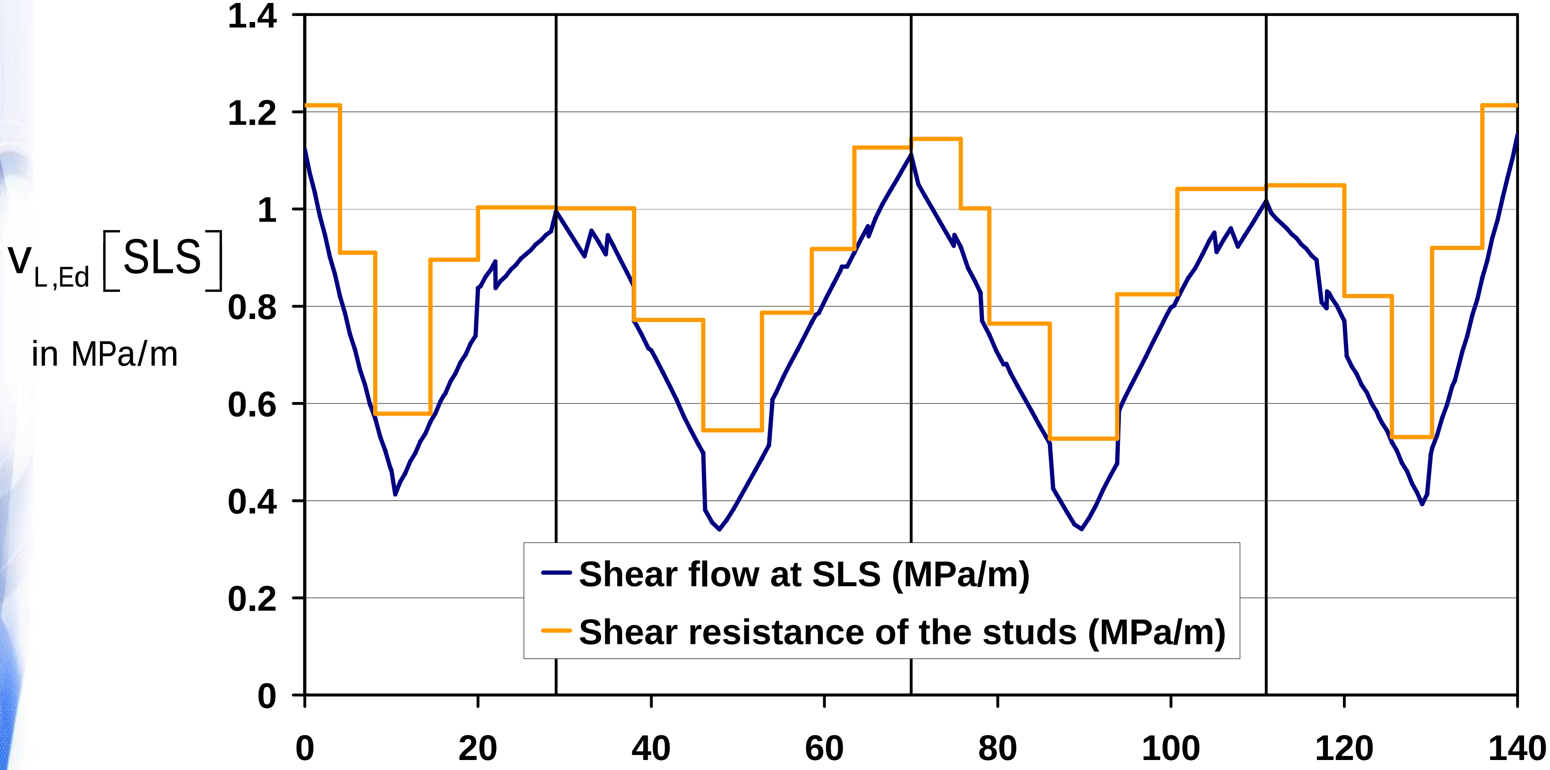
Uncracked mechanical properties

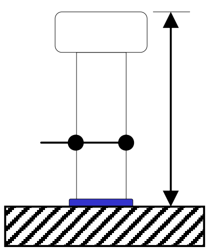


SLS	ULS
For a given length l_i of the girder (to be chosen by the designer), the N_i shear connectors are uniformly distributed and satisfy :	For a given length l_i of the girder (to be chosen by the designer), the N_i^* shear connectors are uniformly distributed and satisfy :
$v_{L,Ed}^{SLS}(x) \leq \frac{N_i}{l_i}.\{k_s P_{Rd}\}$	$v_{L,Ed}^{ULS}(x) \leq 1.1\frac{N_i^*}{l_i}.P_{Rd}$
$(0 \leq x \leq l_i)$	$\int_0^{l_i} v_{L,Ed}^{ULS}(x)dx \leq N_i^*.P_{Rd}$

Example : SLS elastic design of connectors







Studs with :

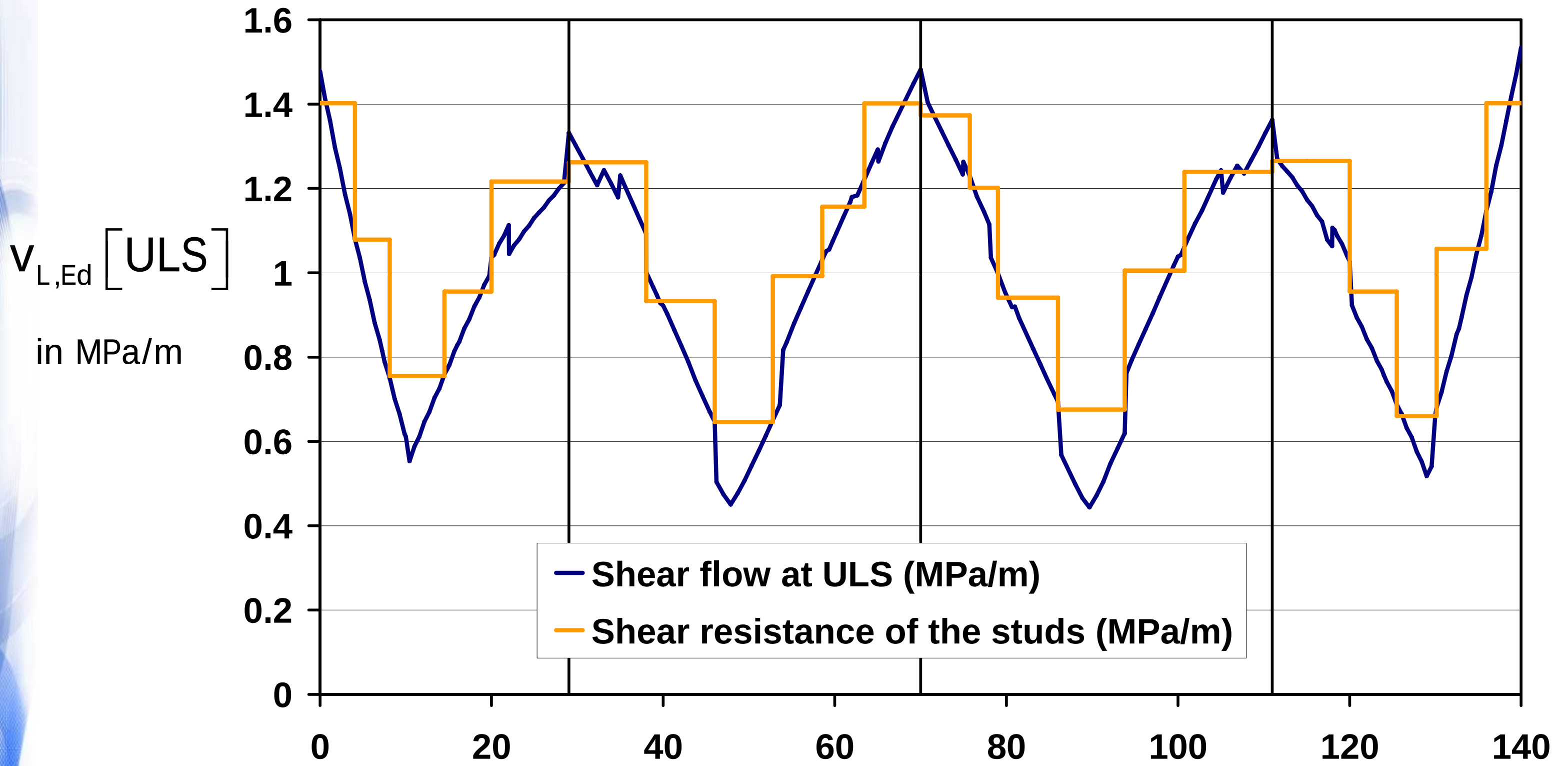
d = 22 mm

h = 150 mm

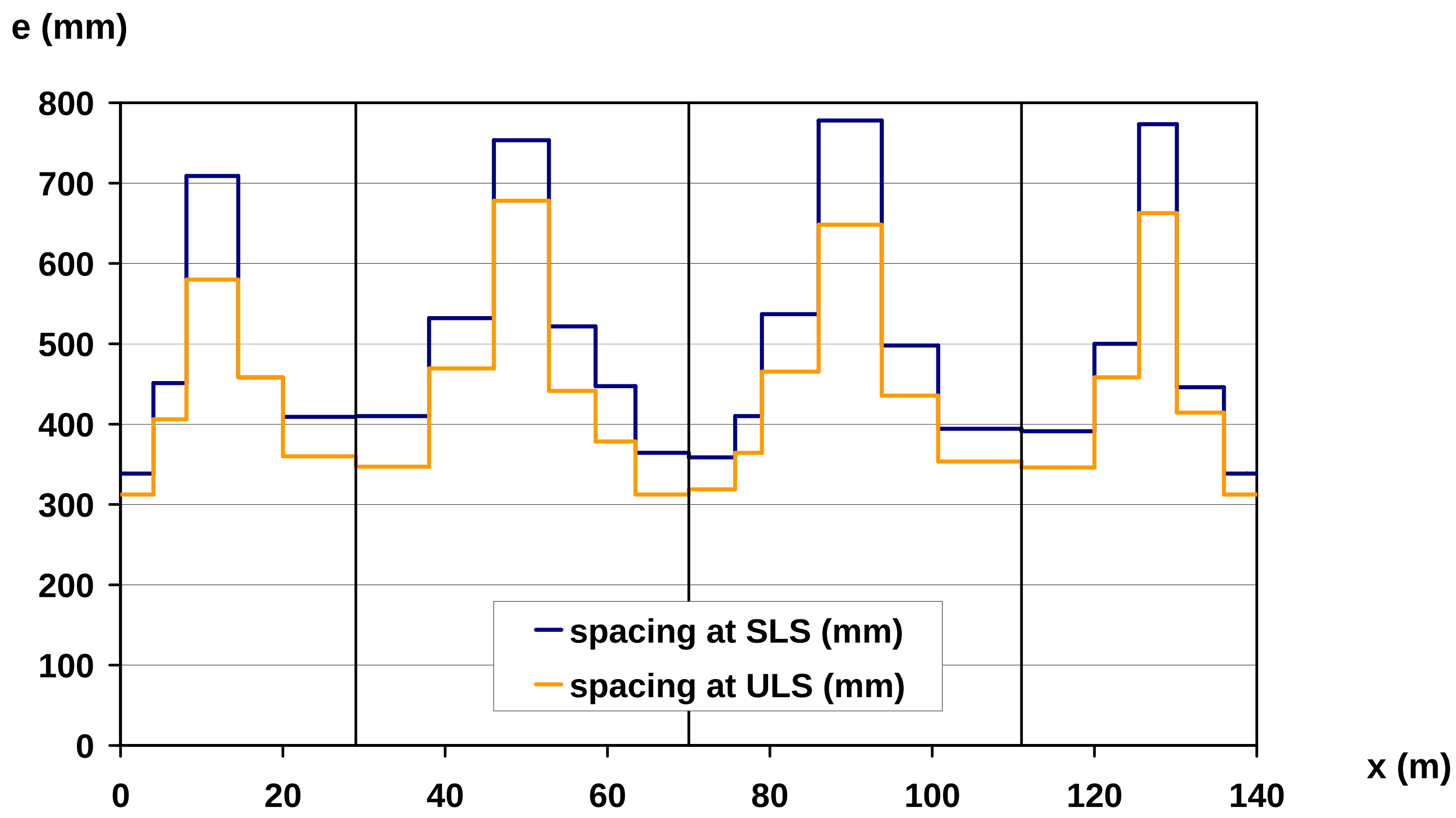
in S235

Example : ULS elastic design of connectors

- Using the same segment lengths l_i as in SLS calculation and the same connector type

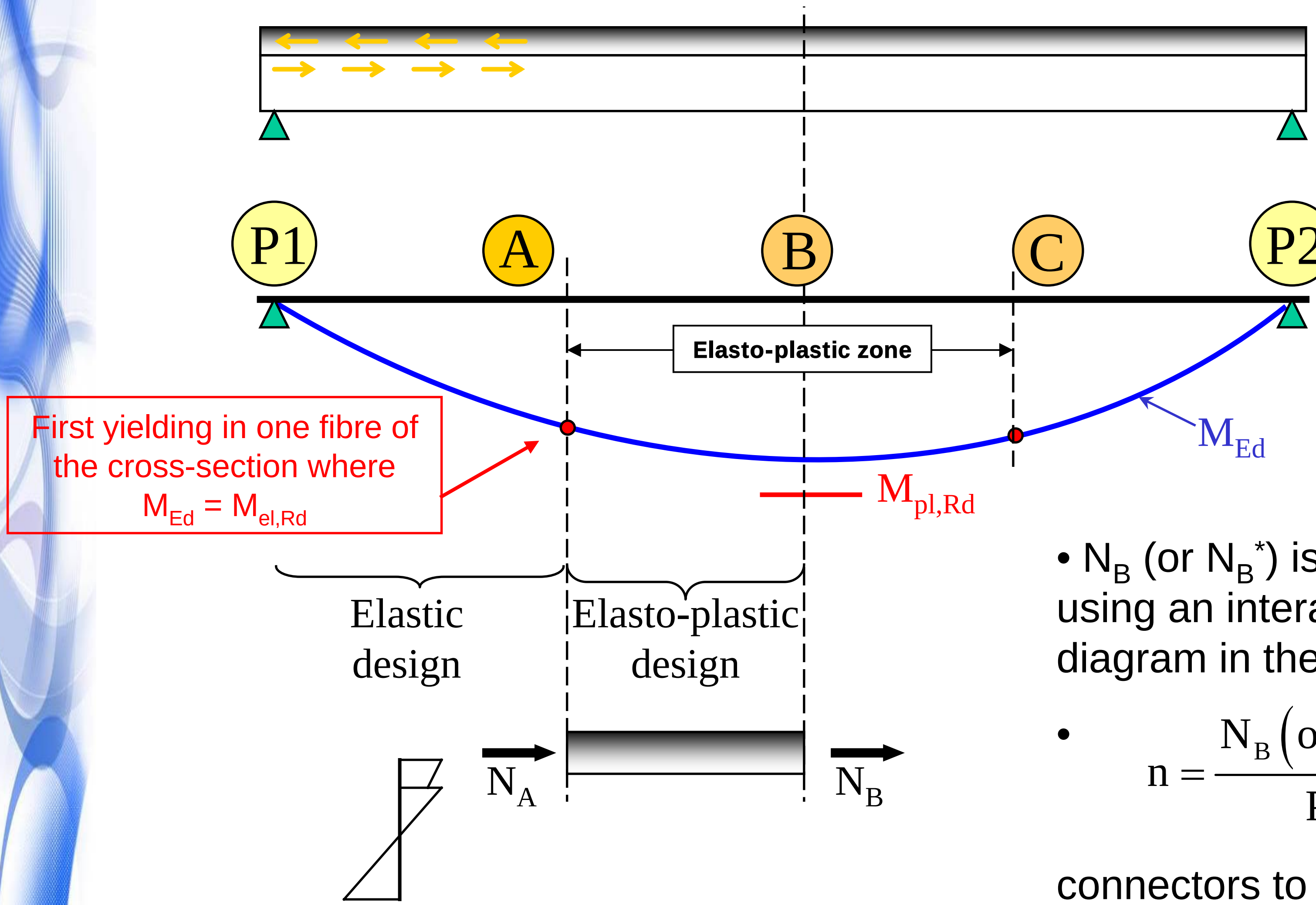


Example : longitudinal spacing of studs rows



=> Elastic design governed by ULS.

• Eventually adding shear connectors in the **elasto-plastic zones** where $M_{pl,Rd} > M_{Ed} > M_{el,Rd}$



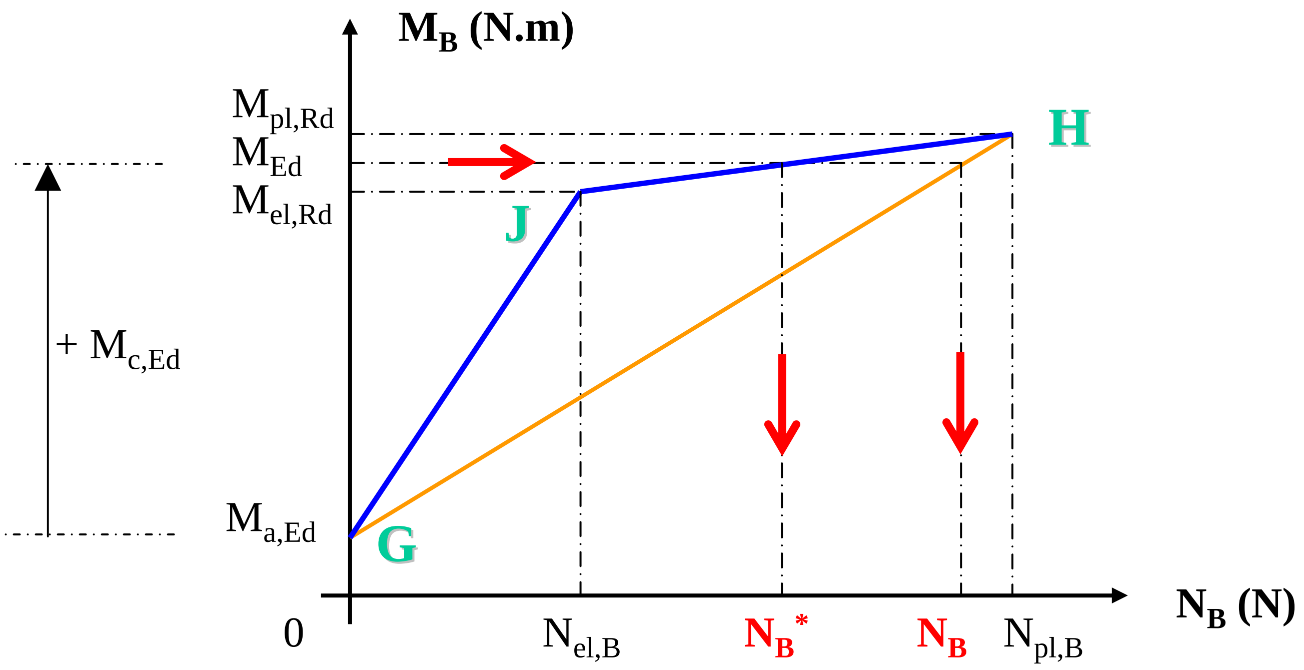
• N_B (or N_B^*) is determined by using an interaction M-N diagram in the section B.

• $$n = \frac{N_B \left(\text{or } N_B^*\right) - N_A}{P_{Rk}/\gamma_V}$$

connectors to put between sections A and B.

Interaction diagram in the cross-section B

• **Two options** : simplified diagram (straight line GH) / more precise diagram (broken line GJH)

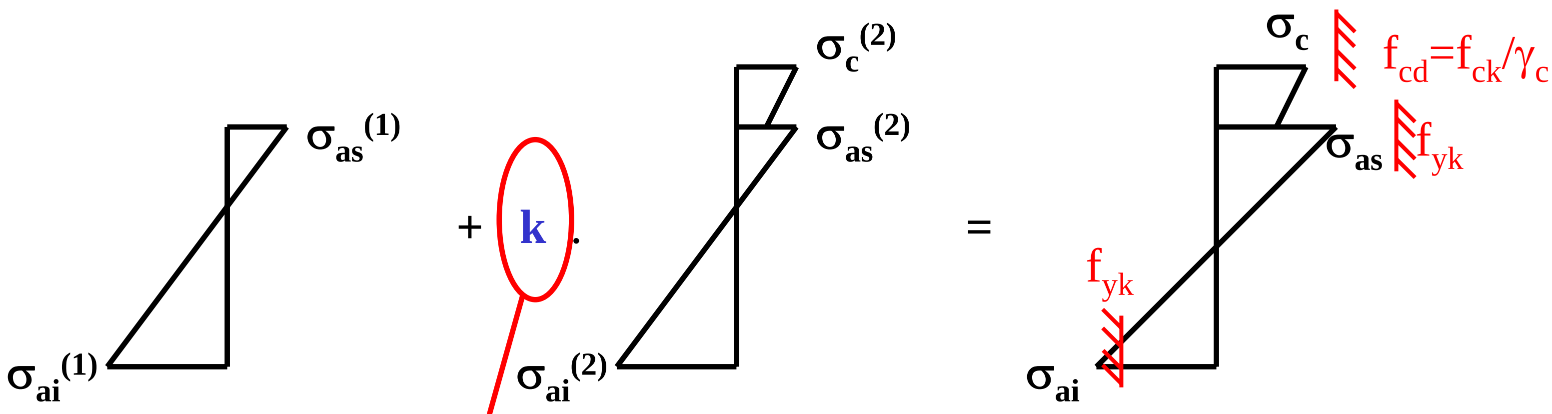


• Plastic resistance of the concrete slab (within the effective width) to compressive normal force :

$$N_{pl,B} = \frac{0.85.f_{ck}}{\gamma_C}.b_{eff}.h_c$$

Elastic resistance moment in the section B

$$M_{a,Ed} \quad + \quad M_{c,Ed} \quad = \quad M_{Ed}$$



Step 1 : stress diagram for load cases applied to the structure **before** concreting Section B

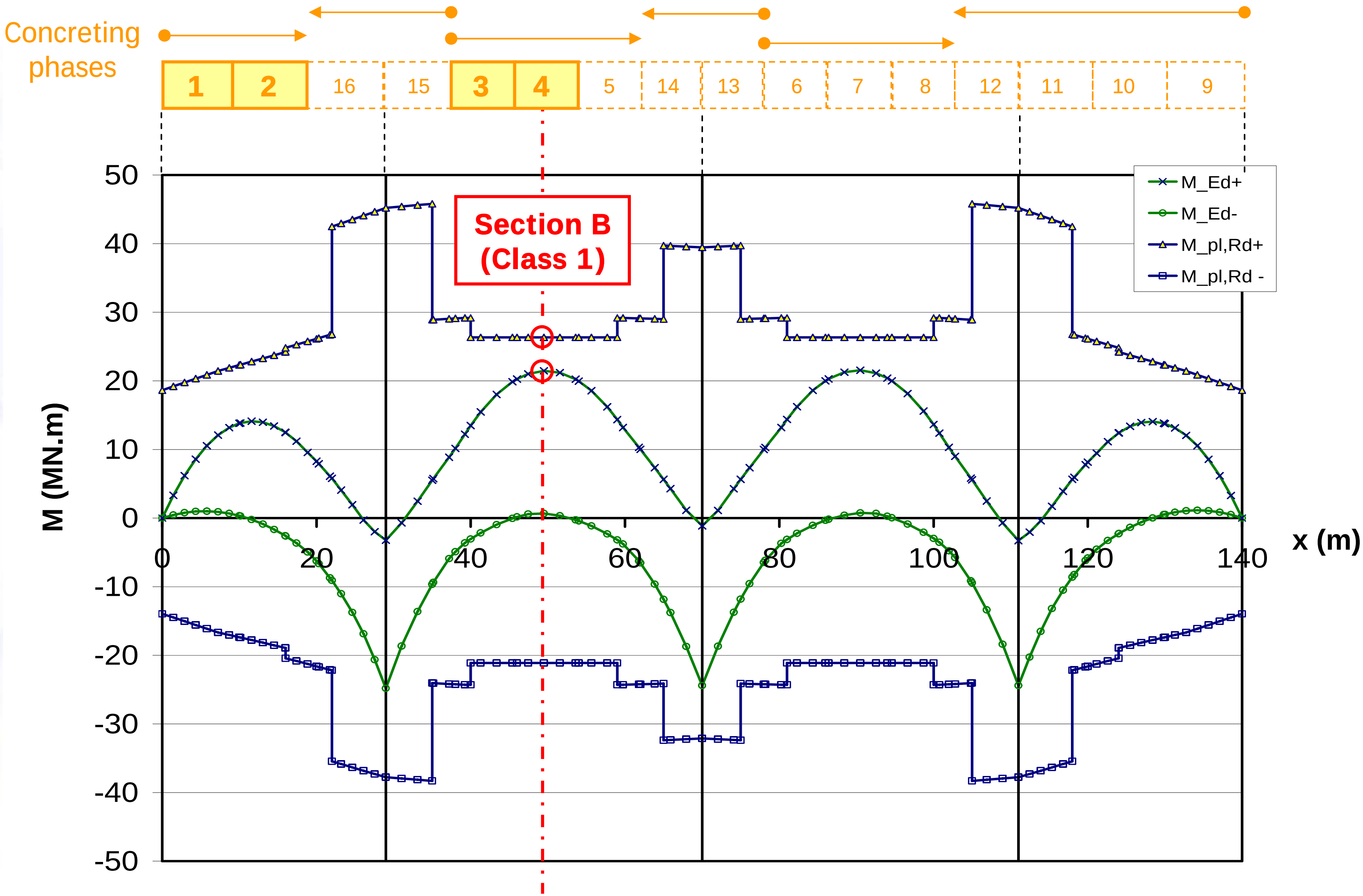
Step 2 : stress diagram for load cases applied to the structure **after** concreting Section B

Step 3 : ULS stress diagram in Section B (if yielding is reached in the extreme bottom fibre)

k (< 1) is the maximum value for keeping step 3 within its yield strength limits. => $M_{el,Rd} = M_{a,Ed} + \mathbf{k}.\ M_{c,Ed}$

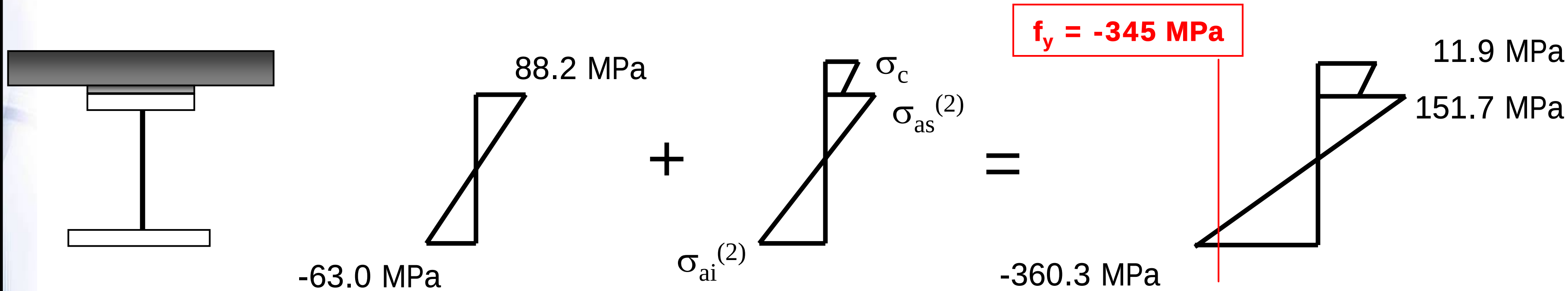
(For instance, $\sigma_{ai}^{(1)} + \mathbf{k}.\sigma_{ai}^{(2)} = f_{yk}$)

Example : Bending moment in section B



$M_{a,Ed}$(B) = 2.7 MN.m -----> M_{Ed}(B) = 22.3 MN.m < $M_{pl,Rd}$ (B) = 25.7 MN.m

Example : Normal stresses in section B



$M_{a,Ed}$(B) = 2.7 MN.m **$M_{c,Ed}$(B)** **M_{Ed}(B) = 22.3 MN.m**

$M_{c,Ed}$(B) = 22.3 – 2.7 = 19.6 MN.m

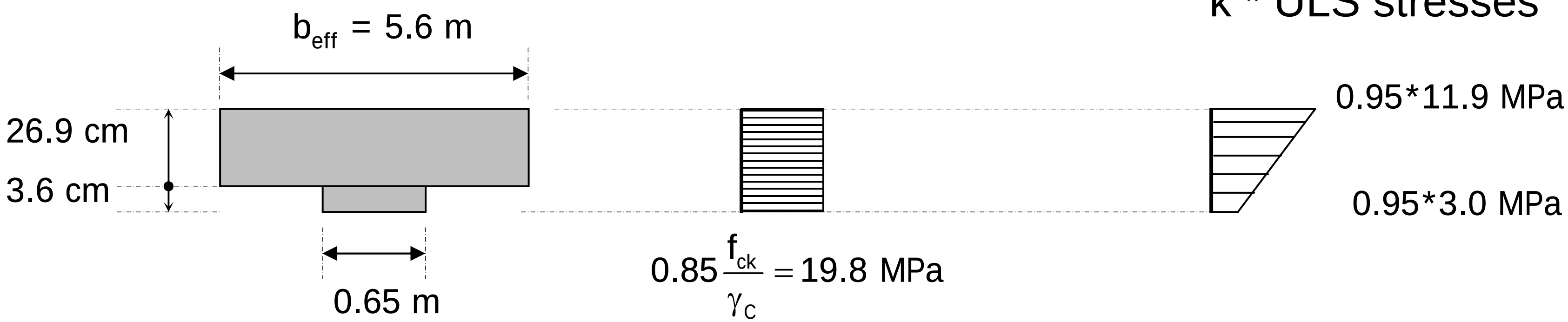
$\sigma_{ai}^{(2)}$ = (-360.3) – (-63.0) = -297.3 Mpa

k is defined by $k = \dfrac{f_y - (-63.0)}{\sigma_{ai}^{(2)}} = 0.95 \leq 1.0$

$M_{el,Rd}$ is then defined by $M_{el,Rd}$ = $M_{a,Ed}$ + k. $M_{c,Ed}$ = 21.3 MN.m

Example : Interaction diagram in section B

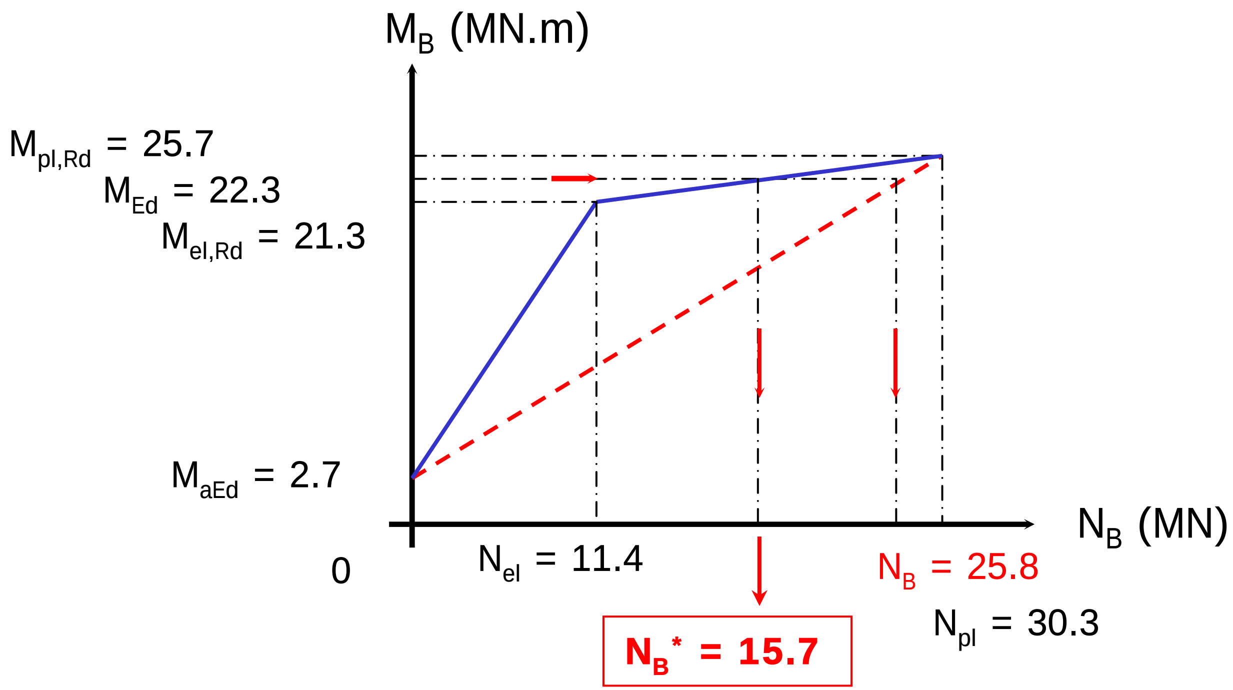
k * ULS stresses



$$0.85\frac{f_{ck}}{\gamma_C} = 19.8\ \text{MPa}$$

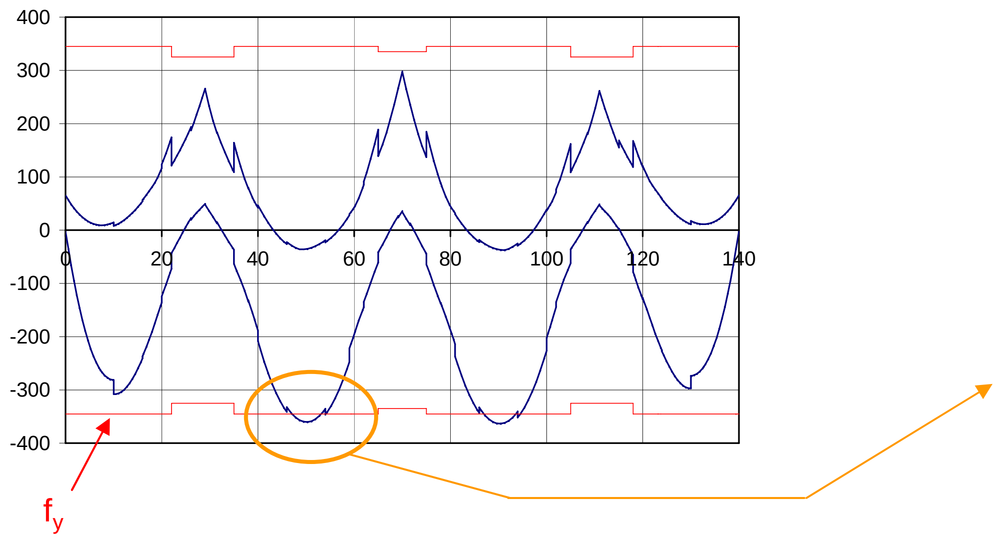
$$N_{pl} = 0.85\frac{f_{ck}}{\gamma_C}.A_{c,eff} = 30.3\ \text{MPa}$$

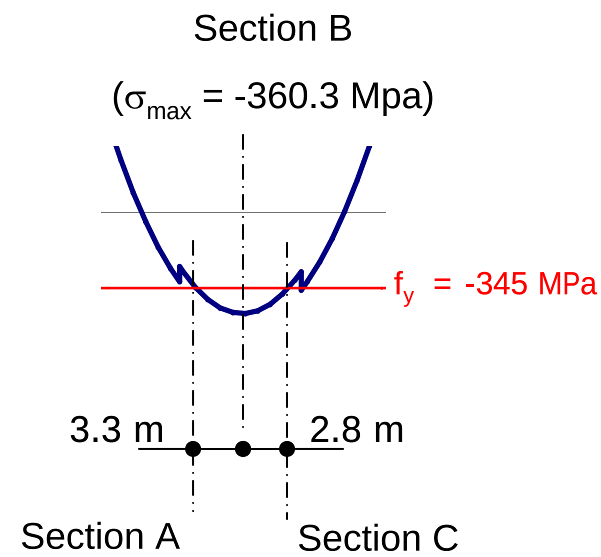
$N_{el} = 11.4$ MN

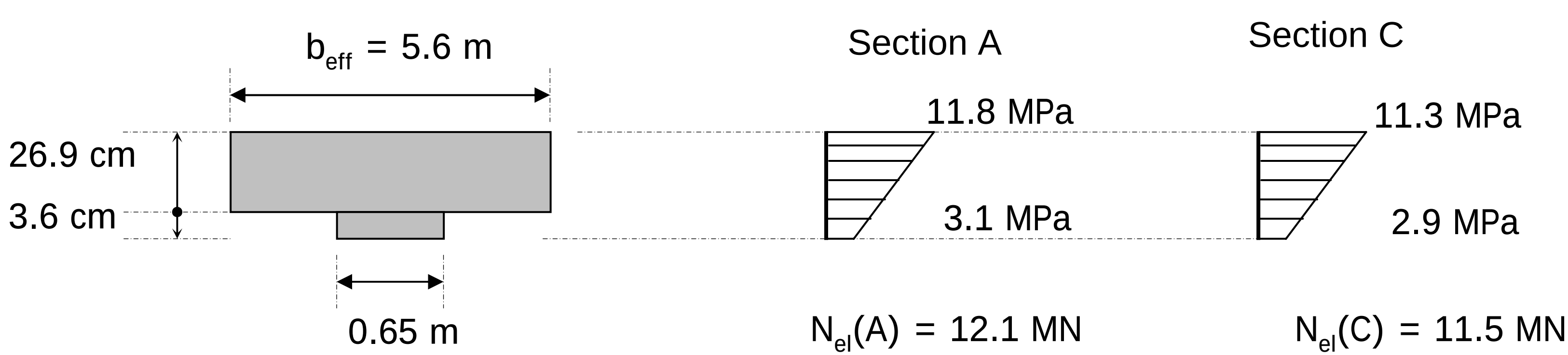


Example : Limits of the elasto-plastic zone

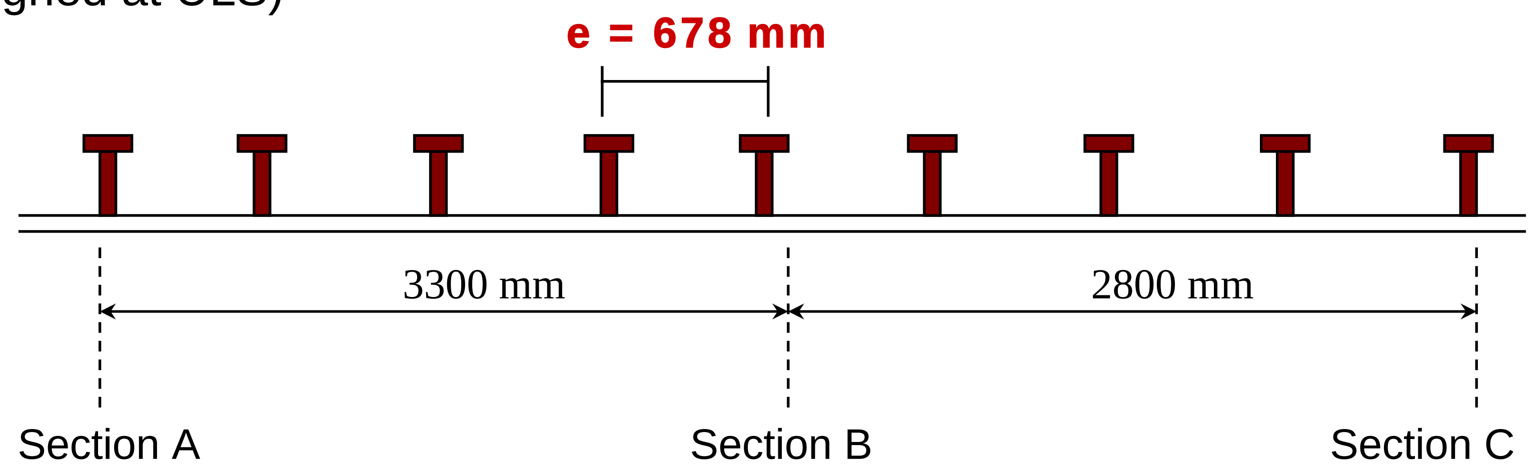
ULS Stresses (MPa) in the bottom steel flange





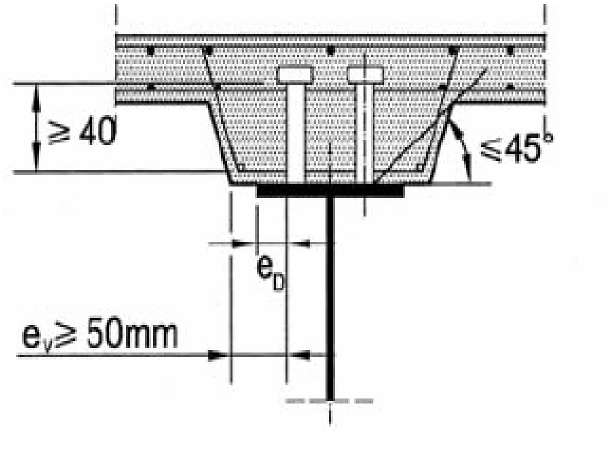


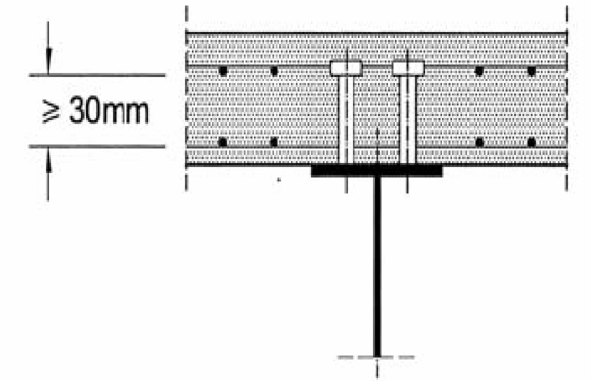
• 9 rows with 4 studs and a longitudinal spacing equal to 678 mm (designed at ULS)



Simplified interaction diagram	(25.8-12.1)/(4x0.1095) = 28 rows spacing = 3300/28 = **118 mm**	(25.8-11.5)/(4x0.1095) = 33 rows spacing = 2800/33 = **84 mm** (which is even lower than 5d=110 mm !)
More precise interaction diagram	(15.7-12.1)/(4x0.1095) = 9 rows spacing = 3300/9 = **367 mm**	(15.7-11.5)/(4x0.1095) = 10 rows spacing = 2800/10 = **280 mm**

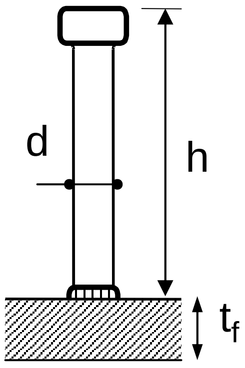
- for solid slabs :





- to allow a correct welding of the connector : $25\text{ mm} \leq e_D$

- and if the used shear connectors are studs :



- $d \leq 2.5.t_f$
- $d \leq 1.5.t_f$ for a structural steel flange in tension, subjected to fatigue
- $h \geq 3d$
- $\Phi_{head} \geq 1.5d$
- $h_{head} \geq 0.4d$

⇨ Longitudinal spacing between shear connectors rows

– to insure the composite behaviour in all cross-sections :

e_{max} = min (800 mm; 4 h)

where h is the concrete slab thickness

– if the structural steel flange in compression which is connected to the concrete slab, is a class 3 or 4 element :

- to avoid buckling of the flange between two studs rows : $e_{max} \leq 22t_f \sqrt{\frac{235}{f_y}}$
- to avoid buckling of the cantilever e_D-long part of the flange : $e_D \leq 9t_f \sqrt{\frac{235}{f_y}}$

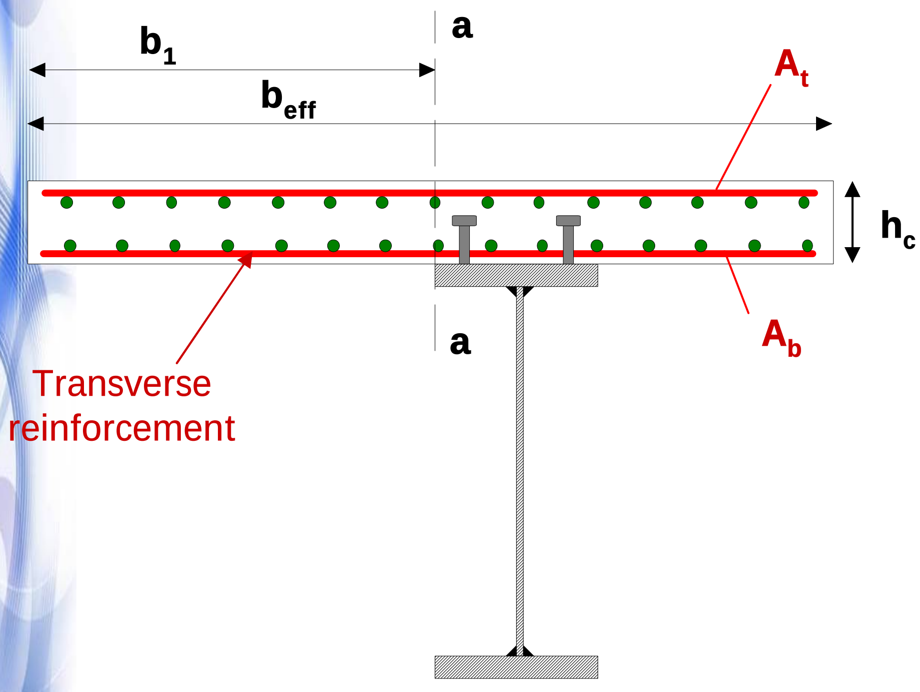
– and if the used shear connectors are studs : $5.d \leq e_{min}$

⇨ Transversal spacing between adjacent studs

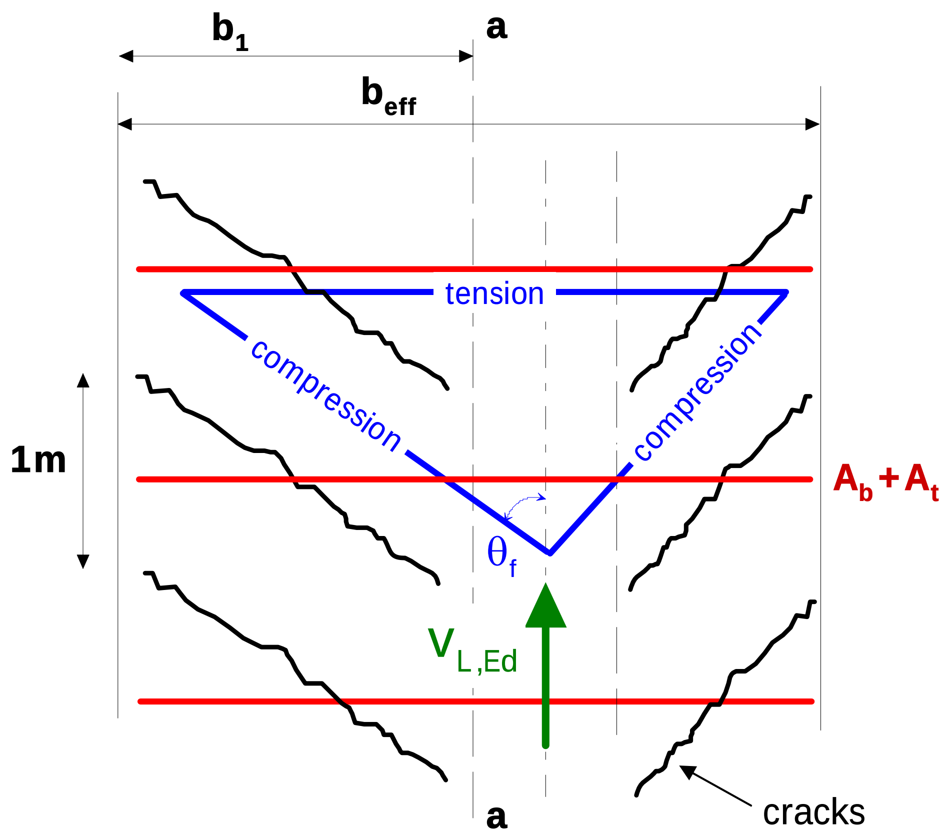
$e_{trans,min} \geq 2.5.d$ for solid slabs

$e_{trans,min} \geq 4.d$ in other cases

Truss model for transverse reinforcement which supplements the shear strength of the concrete on potential surface of failure (a-a for instance)



$$v_{aa} = v_{L,Ed}\left[ULS\right].\frac{b_1}{b_{eff}}$$



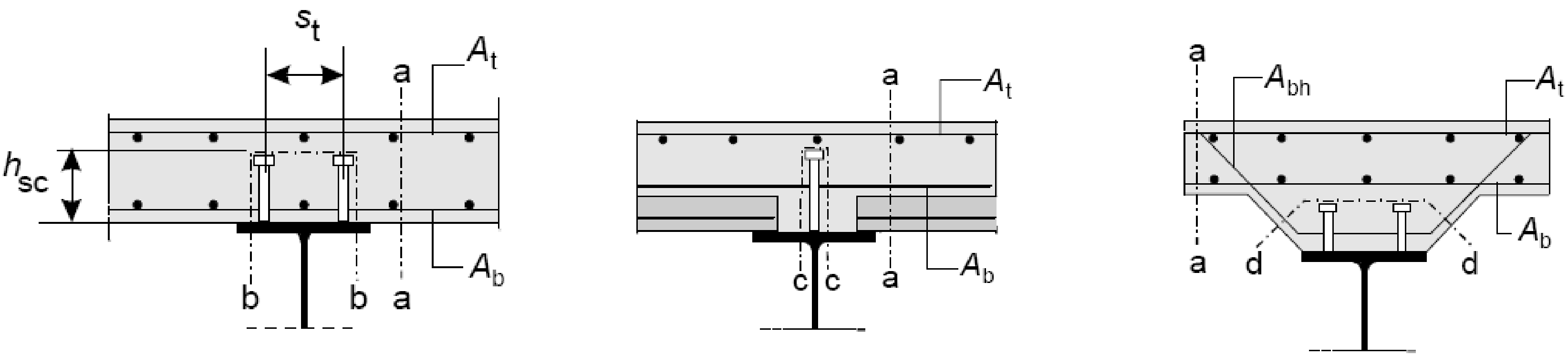
Transverse reinforcement for solid slabs

- tension in reinforcement : $v_{aa}.h_c.(1m).\tan\theta_f \le (A_b + A_t).f_{sd}$

- compression in concrete struts : $v_{aa} \le 0.6\left(1 - \frac{f_{ck}}{250}\right) f_{cd}.\sin\theta_f \cos\theta_f$

- for slab in tension at ULS : $1.0 \le \cot\theta_f \le 1.25$ (or $38.6° \le \theta_f \le 45°$)
- for slab in compression at ULS : $1.0 \le \cot\theta_f \le 2.0$ (or $26.5° \le \theta_f \le 45°$)

- Other potential surfaces of shear failure defined in EN1994-2 :



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Fatigue ULS in a composite bridge

In a composite bridge, fatigue verifications shall be performed for :

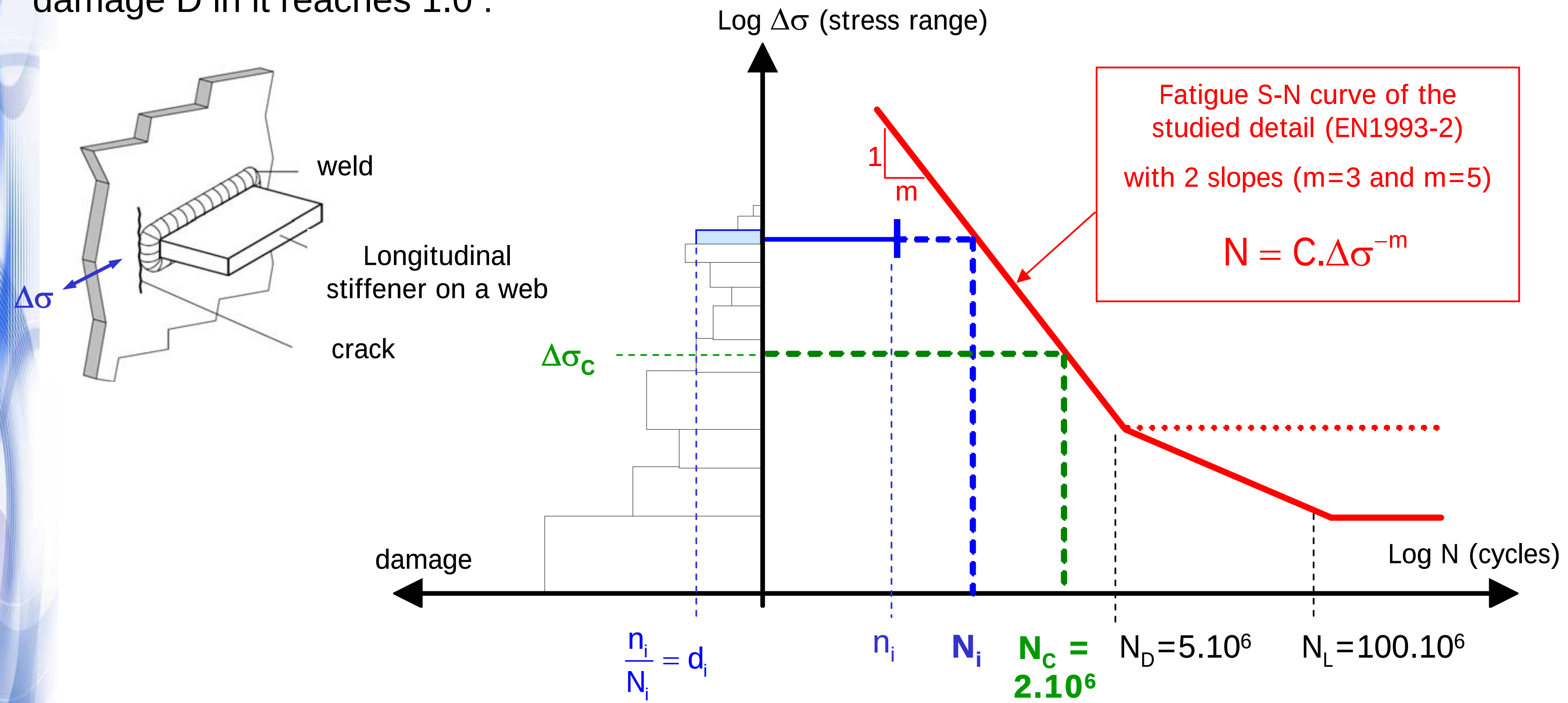
- the ***structural steel*** details of the main girder (see EN1993-2 and EN1993-1-9)
- the slab ***concrete*** (see EN1992-2)
- the slab ***reinforcement*** (see EN1994-2)
- the shear ***connection*** (see EN1994-2)

Two assessment methods in the Eurocodes which differ in the partial factor γ_{Mf} for fatigue strength in the structural steel :

Assessment method (National Choice)	Consequence of detail failure for the bridge	
	Low consequence	High consequence
Damage tolerant Required regular inspections and maintenance for detecting and repairing fatigue damage during the bridge life	$\gamma_{Mf} = 1.0$	$\gamma_{Mf} = 1.15$
Safe life No requirement for regular in-service inspection for fatigue damage	$\gamma_{Mf} = 1.15$	$\gamma_{Mf} = 1.35$

Damage equivalent stress range $\Delta\sigma_E$

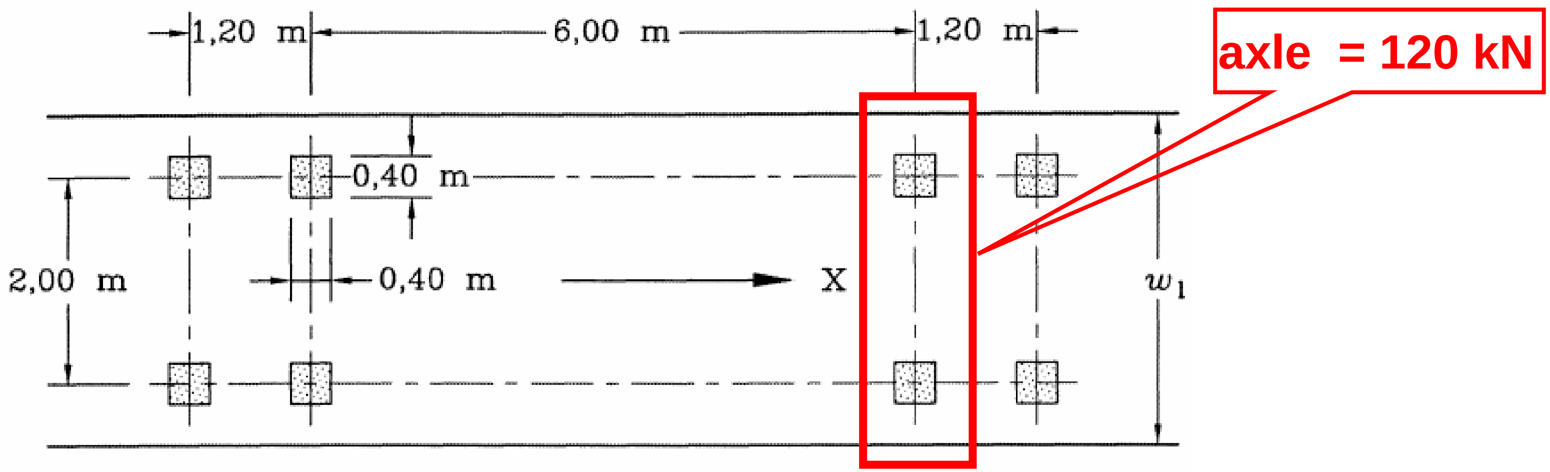
In a given structural detail of the bridge which is subjected to repeated fluctuations of stresses due to traffic loads, a fatigue crack could initiate and propagate. The detail fails when the damage D in it reaches 1.0 :



Total damage in the detail : $D = \sum \frac{n_i}{N_i}$

In term of D, the actual traffic $(n_i, \Delta\sigma_i)_i$ is equivalent to $n_E = \Sigma\, n_i$ cycles of the unique equivalent stress range $\Delta\sigma_E$.

Fatigue Load Model 3 « equivalent lorry » (FLM3)



- 2.10^6 FLM3 lorries are assumed to cross the bridge per year and per slow lane defined in the project
- every crossing induces a stress range $\Delta\sigma_p = |\sigma_{max,f} - \sigma_{min,f}|$ in a given structural detail
- the equivalent stress range $\Delta\sigma_E$ in this detail is obtained as follows :

$$\Delta\sigma_E = \lambda\Phi.\Delta\sigma_p$$

where :

- λ is the damage equivalence factor
- Φ is the damage equivalent impact factor (= 1.0 as the dynamic effect is already included in the characteristic value of the axle load)

Damage equivalence factor λ

In a **structural steel** detail (in EN 1993-2):

$\lambda=\lambda_1 \lambda_2 \lambda_3 \lambda_4 < \lambda_{max}$

which represents the following parameters :

- λ_1 : influence of the loaded lengths, defined in function of the bridges spans (< 80 m) and the shape of the influence line for the internal forces and moments
- λ_2 : influence of the traffic volume
- λ_3 : life time of the bridge (λ_3=1 for 100 years)
- λ_4 : influence of the number of loaded lanes
- λ_{max} : influence of the constant amplitude fatigue limit $\Delta\sigma_D$ at 5.10^6 cycles

For **shear connection** (in EN1994-2): $\lambda_v = \lambda_{v,1} . \lambda_{v,2} . \lambda_{v,3} . \lambda_{v,4}$

For **reinforcement** (in EN1992-2): $\lambda_s = \varphi_{fat} . \lambda_{s,1} . \lambda_{s,2} . \lambda_{s,3} . \lambda_{s,4}$

For **concrete** in compression (in EN1992-2 and only defined for railway bridges):

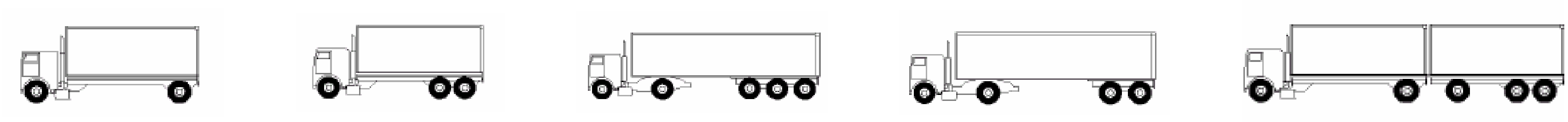
$$\lambda_c = \lambda_{c,0} . \lambda_{c,1} . \lambda_{c,2,3} . \lambda_{c,4}$$

Example : Damage equivalence factor λ_v

- for road bridges (with L< 100 m) : $\lambda_{v,1} = 1.55$

- hypothesis for the traffic volume in the example (based for instance on the existing traffic description in EN 1991 part 2):

$N_{obs} = 0.5.10^6$ lorries per slow lane and per year with the following distribution



$Q_1 = 200$ kN	$Q_2 = 310$ kN	$Q_3 = 490$ kN	$Q_4 = 390$ kN	$Q_5 = 450$ kN
40%	10%	30%	15%	5%

Mean value of lorries weight : $$Q_{ml} = \left(\frac{\sum n_i Q_i^5}{\sum n_i} \right)^{1/5} = 407 \text{ kN}$$

$$\lambda_{v,2} = \frac{Q_{ml}}{480} \left(\frac{N_{obs}}{0.5.10^6} \right)^{(1/8)} = \frac{407}{480} = 0.848$$

- bridge life time = 100 years, so $\lambda_{v,3} = 1.0$

- only 1 slow lane on the bridge, so $\lambda_{v,4} = 1.0$

$$\lambda_v = 1.314$$

Stress range $\Delta\sigma_p = |\sigma_{max,f} - \sigma_{min,f}|$ in the structural steel

Basic combination of non-cyclic actions	+	Fatigue loads
$G_{max}\ (\text{or } G_{min}) + 1.0\ (\text{or } 0.0)S + 0.6T_k$		FLM3
In every section : $M_{max}\ (\text{or } M_{min}) = M_{a,Ed} + M_{c,Ed}$		$M_{FLM3,max}$ and $M_{FLM3,min}$

- Bending moment in the section where the structural steel detail is located :

$$M_{Ed,max,f} = M_{a,Ed} + M_{c,Ed} + M_{FLM3,max} \qquad M_{Ed,min,f} = M_{a,Ed} + M_{c,Ed} + M_{FLM3,min}$$

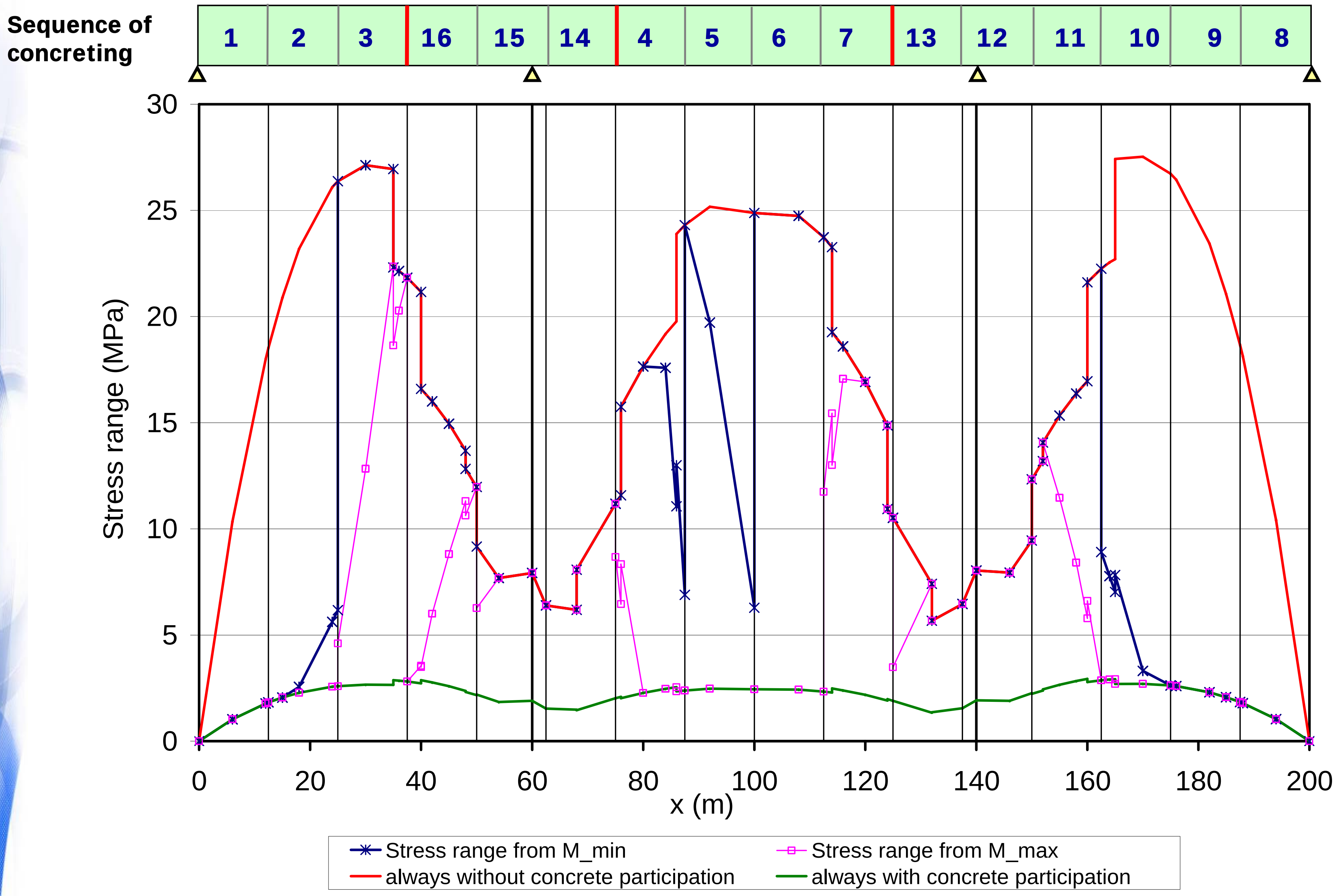
- Corresponding stresses in the concrete slab (participating concrete) :

$$\sigma_{c,Ed,max,f} = M_{c,Ed}\left(\frac{v_1}{I_1}\right)_{n_L} + M_{FLM3,max}\left(\frac{v_1}{I_1}\right)_{n_0} \qquad \sigma_{c,Ed,min,f} = M_{c,Ed}\left(\frac{v_1}{I_1}\right)_{n_L} + M_{FLM3,min}\left(\frac{v_1}{I_1}\right)_{n_0}$$

Case 1	$\sigma_{c,Ed,max,f} > 0$ $\sigma_{c,Ed,min,f} > 0$	$\Delta\sigma_p = \left[M_{a,Ed}\frac{v_a}{I_a} + M_{c,Ed}\frac{v_1}{I_1} + M_{FLM3,max}\frac{v_1}{I_1}\right] - \left[M_{a,Ed}\frac{v_a}{I_a} + M_{c,Ed}\frac{v_1}{I_1} + M_{FLM3,min}\frac{v_1}{I_1}\right] = \Delta M_{FLM3}\frac{v_1}{I_1}$
Case 2	$\sigma_{c,Ed,max,f} < 0$ $\sigma_{c,Ed,min,f} < 0$	$\Delta\sigma_p = \Delta M_{FLM3}\frac{v_2}{I_2}$
Case 3	$\sigma_{c,Ed,max,f} > 0$ $\sigma_{c,Ed,min,f} < 0$	$\Delta\sigma_p = M_{c,Ed}\left(\frac{v_1}{I_1} - \frac{v_2}{I_2}\right) + M_{FLM3,max}\frac{v_1}{I_1} + M_{FLM3,min}\frac{v_2}{I_2}$

Stress range $\Delta\sigma_p$ for the upper face of the upper steel flange

Example : Twin-girder composite bridge



Stress range $\Delta\sigma_{s,p} = |\sigma_{s,max,f} - \sigma_{s,min,f}|$ in the reinforcement

Case 1	$\sigma_{c,Ed,max,f} > 0$ $\sigma_{c,Ed,min,f} > 0$	$\Delta\sigma_{s,p} = \Delta M_{FLM3}\dfrac{v_1}{I_1}$
Case 2	$\sigma_{c,Ed,max,f} < 0$ $\sigma_{c,Ed,min,f} < 0$	$\Delta\sigma_{s,p} = \left[\left\|M_{c,Ed} + M_{FLM3,min}\right\|\dfrac{v_2}{I_2} + \Delta\sigma_{s,f}\right]\left(1 - \dfrac{M_{c,Ed} + M_{FLM3,max}}{M_{c,Ed} + M_{FLM3,min}}\right)$
Case 3	$\sigma_{c,Ed,max,f} > 0$ $\sigma_{c,Ed,min,f} < 0$	$\Delta\sigma_{s,p} = \left\|\left(M_{c,Ed} + M_{FLM3,max}\right)\dfrac{v_1}{I_1} - \left[\left\|M_{c,Ed} + M_{FLM3,min}\right\|\dfrac{v_2}{I_2} + \Delta\sigma_{s,f}\right]\right\|$

- influence of the tension stiffening effect

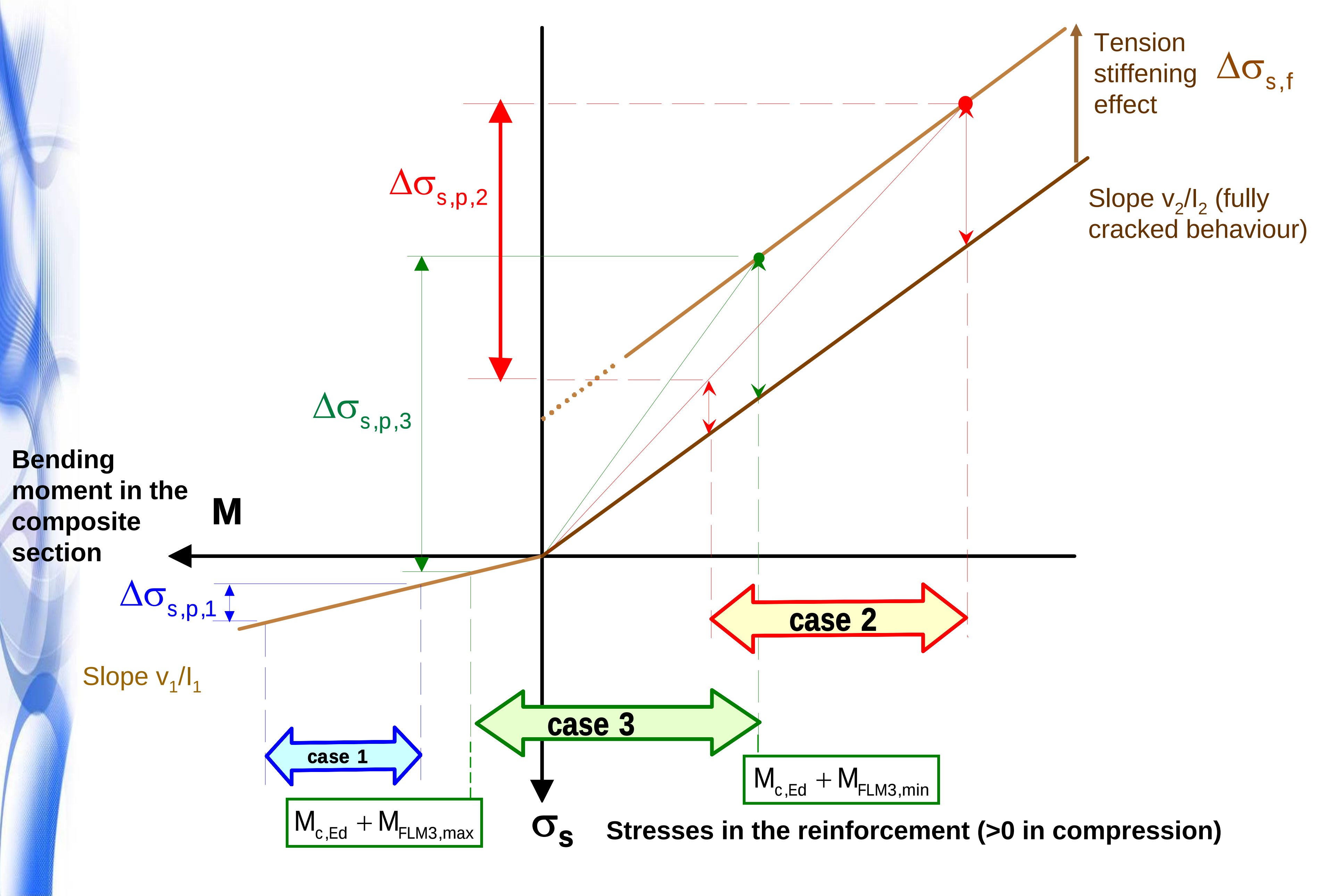
$$\Delta\sigma_{s,f} = 0.2\frac{f_{ctm}}{\alpha_{st}\rho_s}$$

Fatigue : 0.2
SLS verifications : 0.4

$$\alpha_{st} = \frac{AI}{A_a I_a} \qquad \rho_s = \frac{A_{s,eff}}{A_{c,eff}}.100$$

- in case 3, $M_{c,Ed}$ is a sum of elementary bending moments corresponding to different load cases with different values of v_1/I_1 (following n_L).

Tension stiffening effect



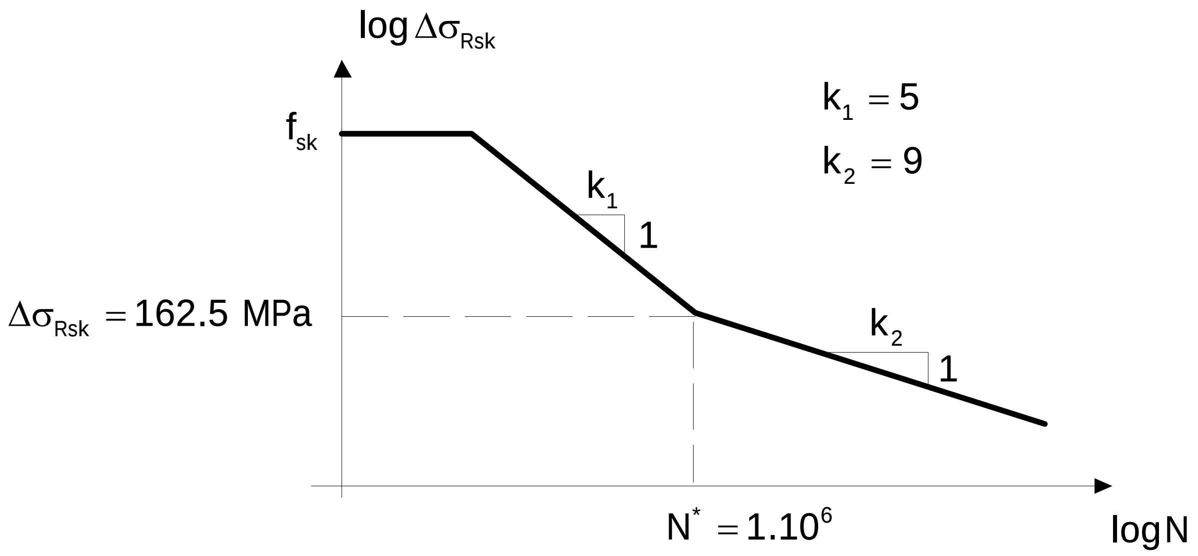
- In a structural steel detail :

$$\gamma_{Ff}\ \Delta\sigma_E \leq \frac{\Delta\sigma_c}{\gamma_{Mf}} \qquad \gamma_{Ff}\ \Delta\tau_E \leq \frac{\Delta\tau_c}{\gamma_{Mf}}$$

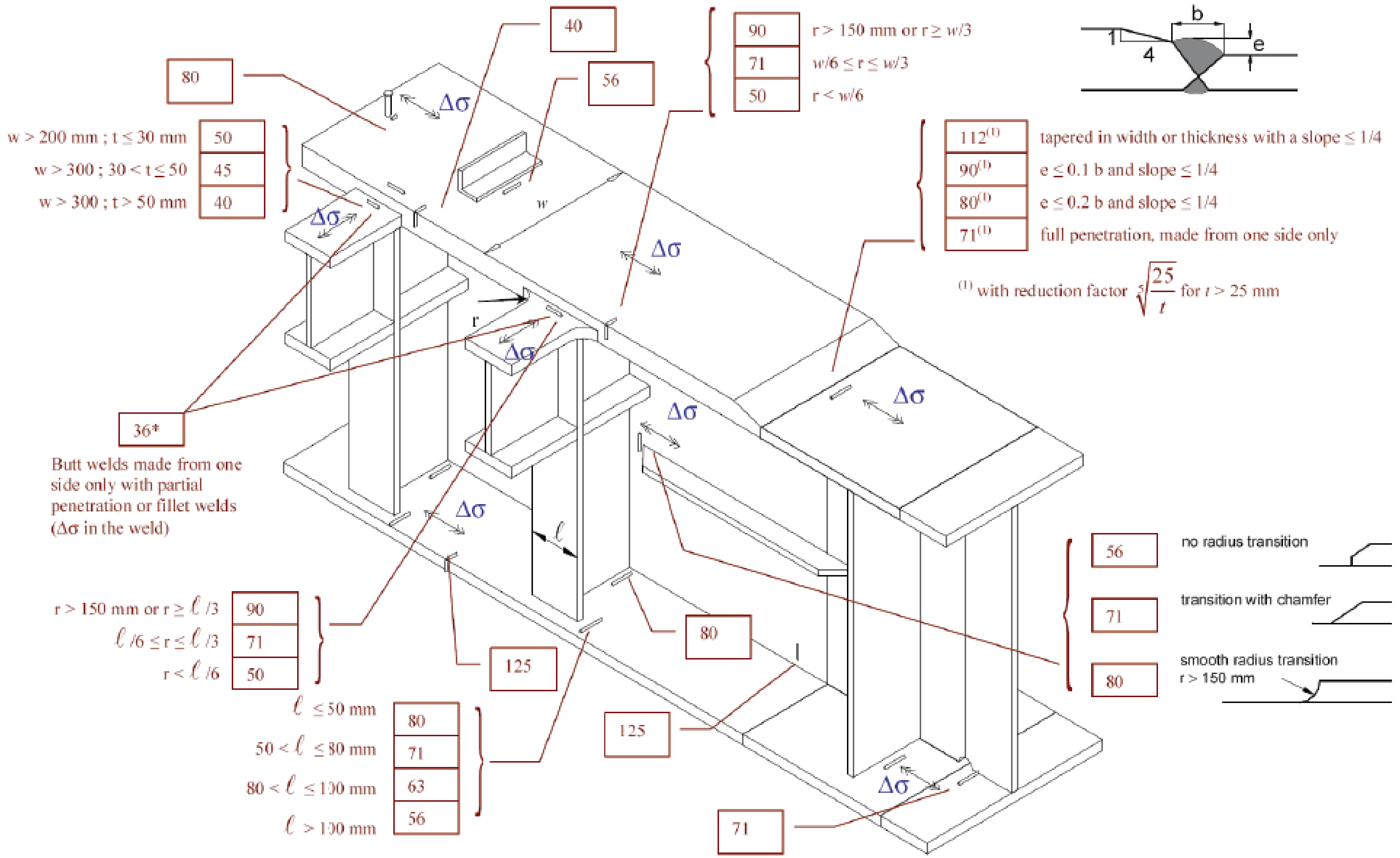
$$\left(\frac{\gamma_{Ff}\Delta\sigma_E}{\Delta\sigma_C/\gamma_{Mf}}\right)^3 + \left(\frac{\gamma_{Ff}\Delta\tau_E}{\Delta\tau_C/\gamma_{Mf}}\right)^5 \leq 1.0$$

- In the reinforcement :

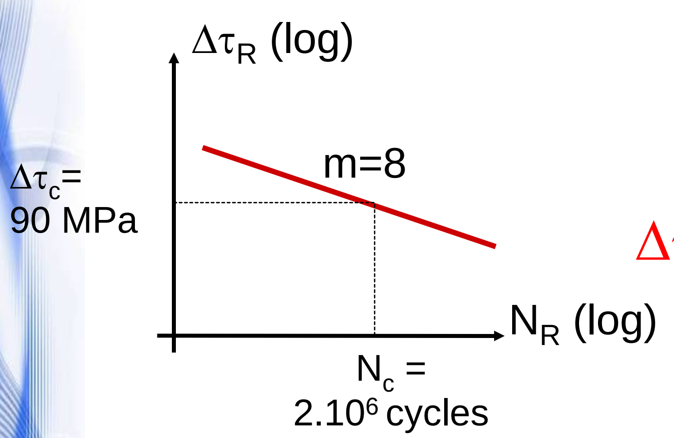
$$\gamma_{F,fat}\ \Delta\sigma_E \leq \frac{\Delta\sigma_{Rsk}}{\gamma_{S,fat}} \qquad \gamma_{S,fat} = 1.15$$

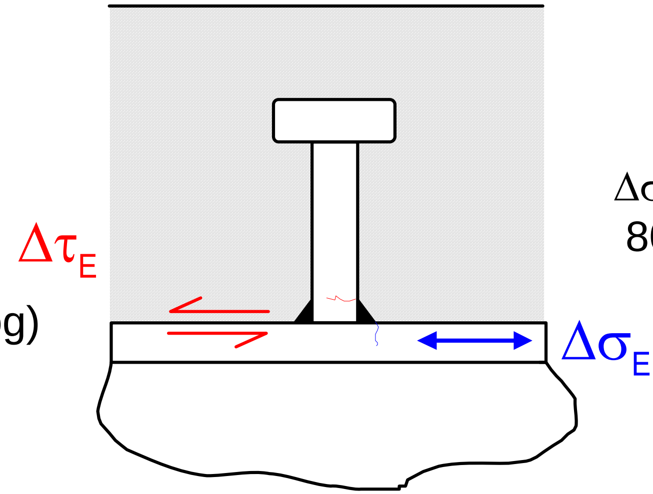


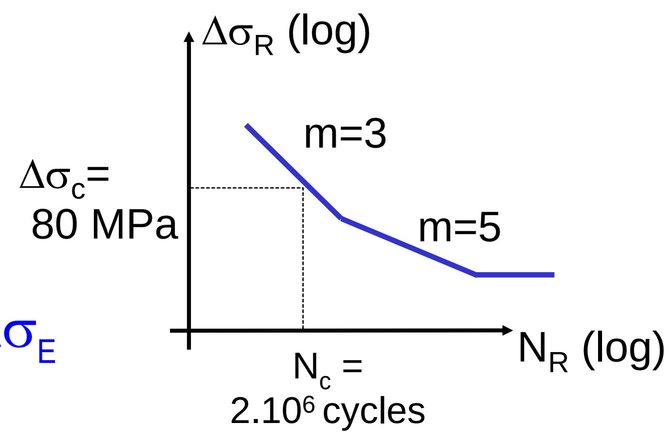
Classification of typical structural details



Fatigue verifications for shear connectors







$$(\Delta\tau_R)^m N_R = (\Delta\tau_C)^m N_C$$

1. **For a steel flange in compression at fatigue ULS :**

$$\gamma_{Ff}\ \Delta\tau_E \leq \frac{\Delta\tau_c}{\gamma_{Mf,s}}$$

with the recommended values :

$$\gamma_{Ff} = 1.0$$
$$\gamma_{Mf,s} = 1.0$$

2. **For a steel flange in tension at fatigue ULS :**

$$\gamma_{Ff}\ \Delta\sigma_E \leq \frac{\Delta\sigma_c}{\gamma_{Mf}} \qquad \gamma_{Ff}\ \Delta\tau_E \leq \frac{\Delta\tau_c}{\gamma_{Mf,s}} \qquad \frac{\gamma_{Ff}\Delta\sigma_E}{\Delta\sigma_C/\gamma_{Mf}} + \frac{\gamma_{Ff}\Delta\tau_E}{\Delta\tau_C/\gamma_{Mf,s}} \leq 1.3$$

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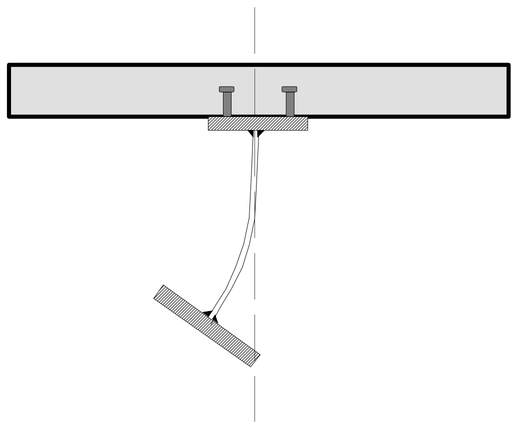
To verify the LTB in the lower bottom flange (which is in compression around internal supports), two approaches are available :



1. Bridge with uniform cross-sections in Class 1,2 or 3 and an un-stiffened web (except on supports) : U-frame model

2. Bridge with non-uniform cross-sections : general method from EN1993-2, 6.3.4
   - 6.3.4.1 : General method
   - 6.3.4.2 : Simplified method (Engesser's formula for σ_{cr})

$$\bar{\lambda}_{LT} = \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} \quad \text{with} \quad \alpha_{ult} = \frac{f_y}{\sigma_a} \quad \text{and} \quad \alpha_{cr} = \frac{\sigma_{cr}}{\sigma_a}$$

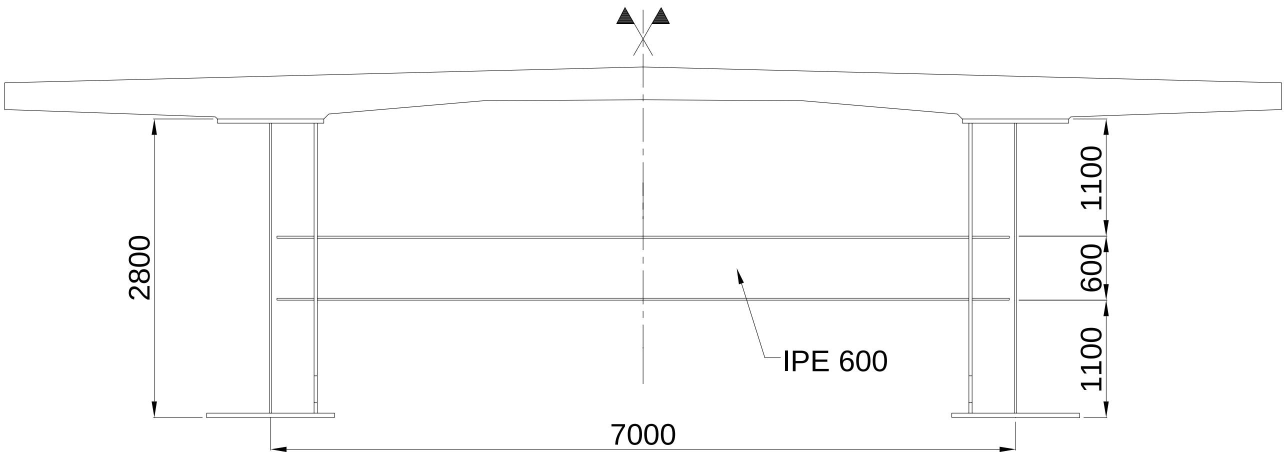
$$\chi_{LT} = f\left(\bar{\lambda}_{LT}\right)$$

$$\frac{\chi_{LT}\alpha_{ult}}{\gamma_{M1}} \geq 1.0 \ ?$$

Example : lateral restraints

Lateral restraints are provided on each vertical support (piles) and in cross-sections where cross bracing frames are provided:

- Transverse bracing frames every 7.5 m in end spans and every 8.0 m in central span



Cross section with transverse bracing frame in span

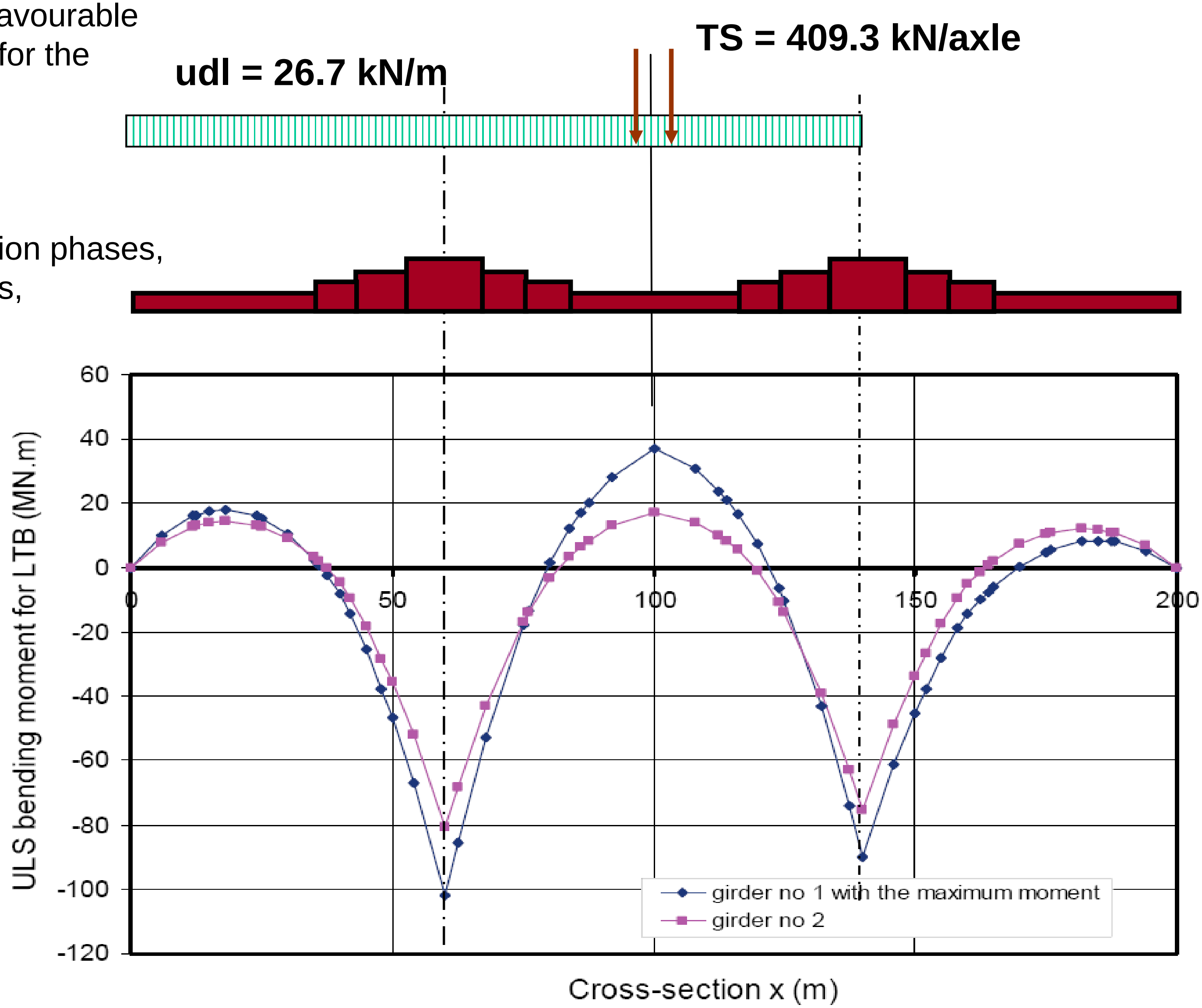
- A frame rigidity evaluated to $\mathbf{C_d = 20.3\ MN/m}$ (spring rate)

Maximum bending at support P1 under traffic

Traffic loads (with unfavourable transverse distribution for the girder n°1)

+

Dead loads (construction phases, cracked elastic analysis, shrinkage)



M_{Ed} = -102 MN.m

$N_{Ed} = M_{Ed} / h$

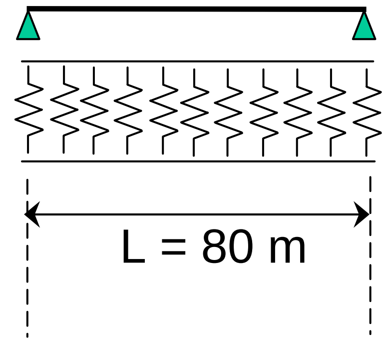
= 38 MN

Elastic critical load for lateral flange buckling

• EN 1993-2, 6.3.4.2 : ENGESSER

- N_{Ed} = constant = N_{max}
- I = constant = I_{max}

$$I = \frac{t_f b_f^3}{12} = \frac{120.1200^3}{12}$$



c = C_d/a

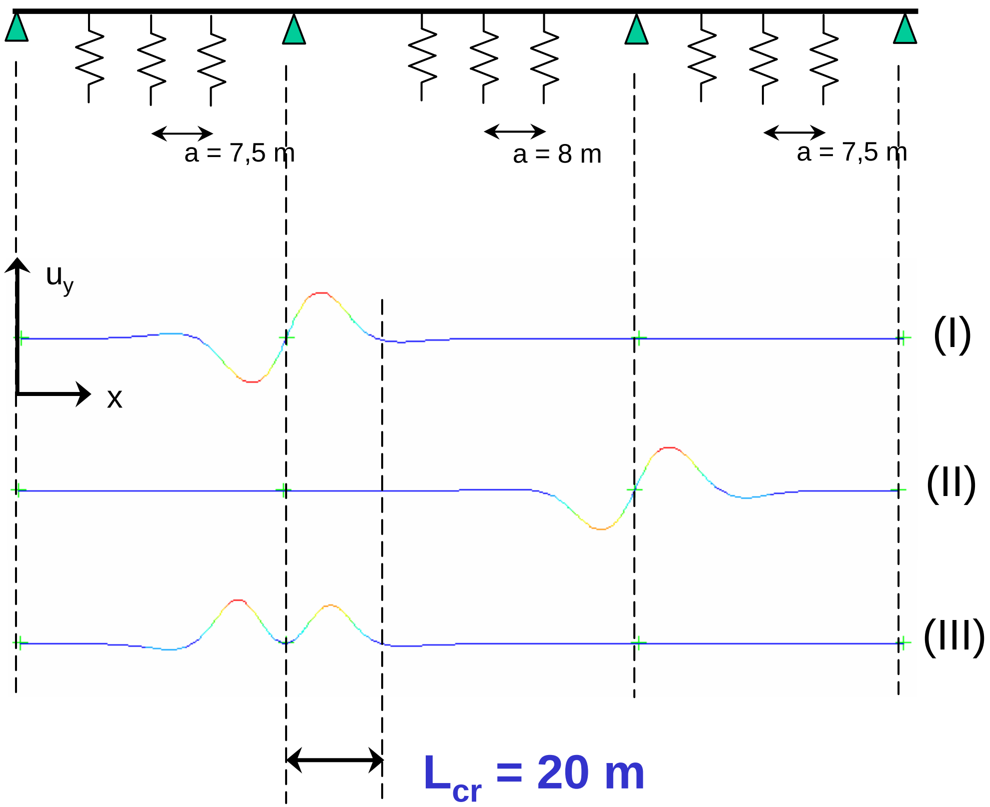
$$N_{cr} = 2\sqrt{EIc} = 192 \text{ MN}$$

$$\alpha_{cr} = N_{cr}/N_{Ed} = \mathbf{5.1} < 10$$

• EN 1993-2, 6.3.4.1: General method

- I and N_{Ed} are variable
- discrete elastic lateral support, with rigidity C_d

$$\alpha_{cr} = N_{cr}/N_{Ed} = \mathbf{8.9} \text{ (Mode I at P1)}$$
$$= \mathbf{10.3} \text{ (Mode II at P2)}$$
$$= \mathbf{17.5} \text{ (Mode III at P1)}$$

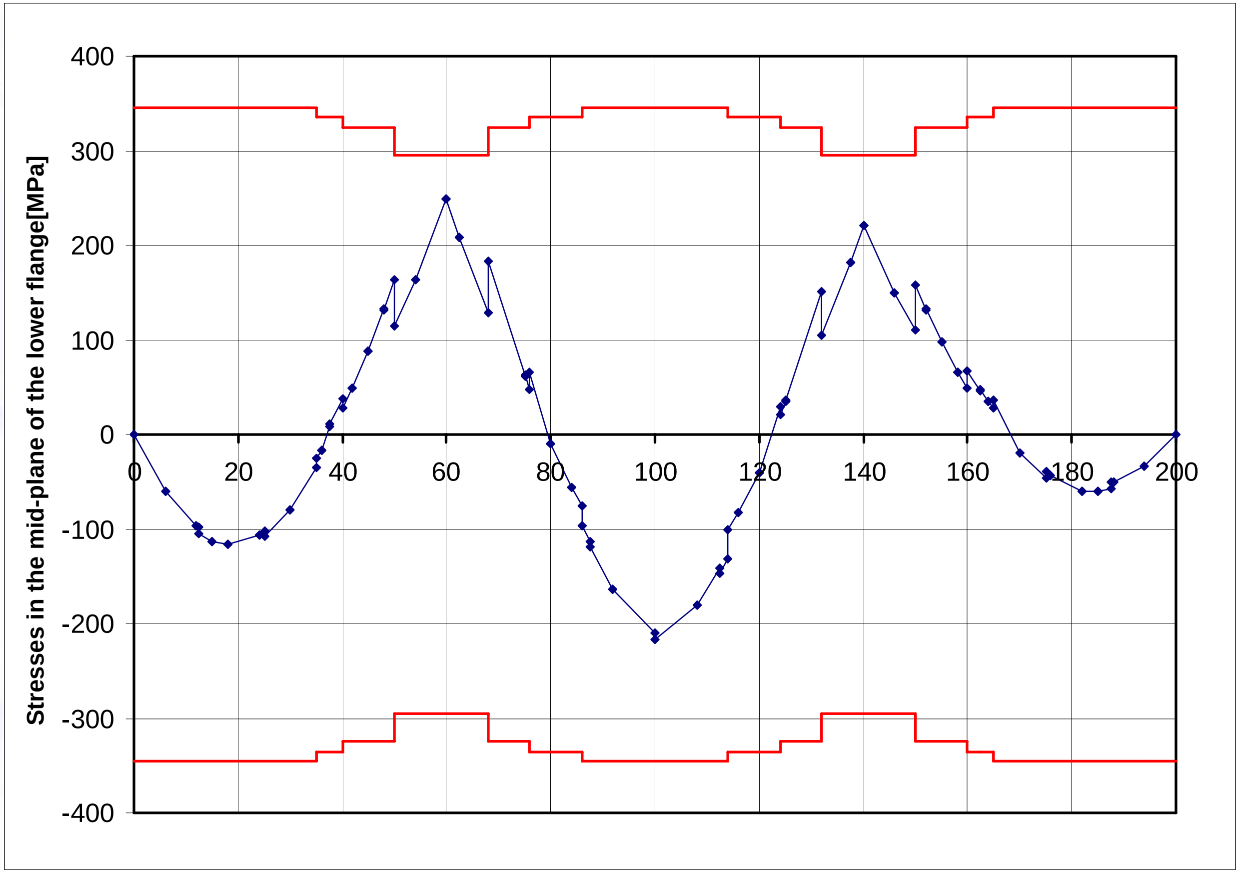


L_{cr} = 20 m

EN1993-2, 6.3.4.1 (general method)

Example : Twin-girder composite bridge

First order stresses in the mid plane of the lower flange (compression at support P1)



$$\alpha_{\text{ult,k}} = \min\left[\frac{f_{\text{yf}}}{\sigma_{\text{f}}}\right] = \frac{295}{249} = 1.18$$

$$\bar{\lambda}_{\text{op}} = \sqrt{\frac{\alpha_{\text{ult,k}}}{\alpha_{\text{cr,op}}}} = \sqrt{\frac{1.18}{8.9}} = 0.37 \geq 0.2$$

Using buckling curve d:

$$\chi_{\text{op}} = 0.875 \leq 1.0$$

$$\chi_{\text{op}} \frac{\alpha_{\text{ult,k}}}{\gamma_{\text{M1}}} = \frac{1.036}{1.1} = 0.94 > 1.0$$

NO !

More information about the numerical design example by downloading the PDF guidance book :

“Eurocodes 3 and 4 – Application to steel-concrete composite road bridges”

on the Sétra website :

http://www.setra.equipement.gouv.fr/In-English.html

Thank you for your kind attention