Design of composite bridges according to Eurocodes (EN 1994)

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Design process of a bridge

1. Global analysis (see previous presentation)

a. Calculate the internal forces and moments according to Eurocode's principles

b. By modelling the bridge deck (geometry and stiffness to represent the actual behaviour in the best way)

c. And by applying the load cases

2. Section and member analysis

a. Cross-section resistance at ULS (examples N°1 and 2)

b. Cross-section resistance at SLS:

- Stress limitations (example N°3)
- Concrete crack width control (example N°4)

c. Stability (plate or member buckling)

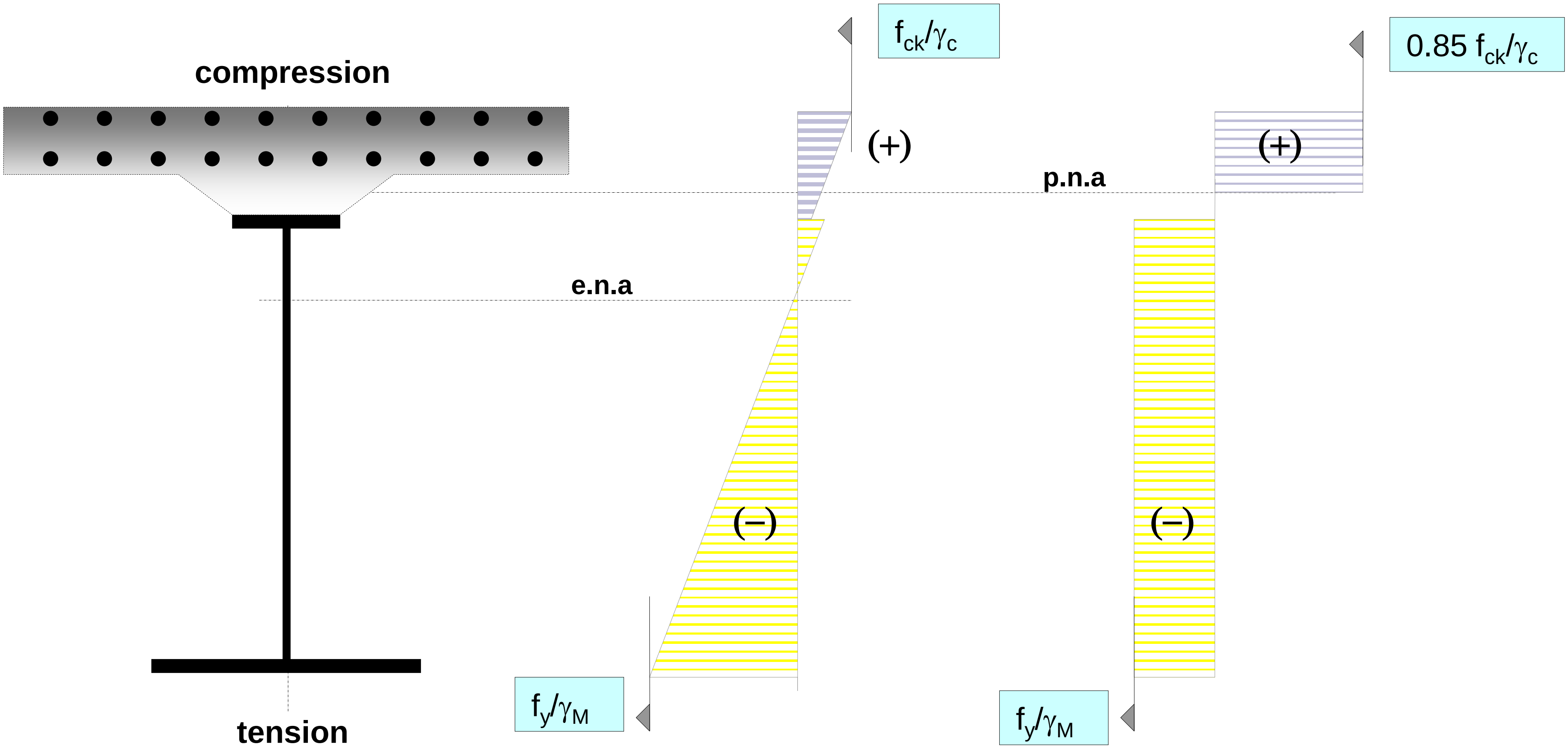
d. Shear connection at the steel–concrete interface (example N°5)

e. Fatigue (example N°6)

Composite cross-section resistance at ULS

1. **Resistance of the composite cross-sections**
   - **for bending moment M**
   - **for shear force V**
   - **for interaction M+V**
2. *Shear resistance in the concrete slab (EN 1992 and EN 1994)*
3. *Local bending in the concrete slab (EN 1992)*
4. *Punching in the concrete slab (EN 1992)*
5. **Shear connection**
6. **Fatigue**
7. *Member stability (EN 1993)*

ULS cross-section check under M > 0



e.n.a. = elastic neutral axis

p.n.a. = plastic neutral axis

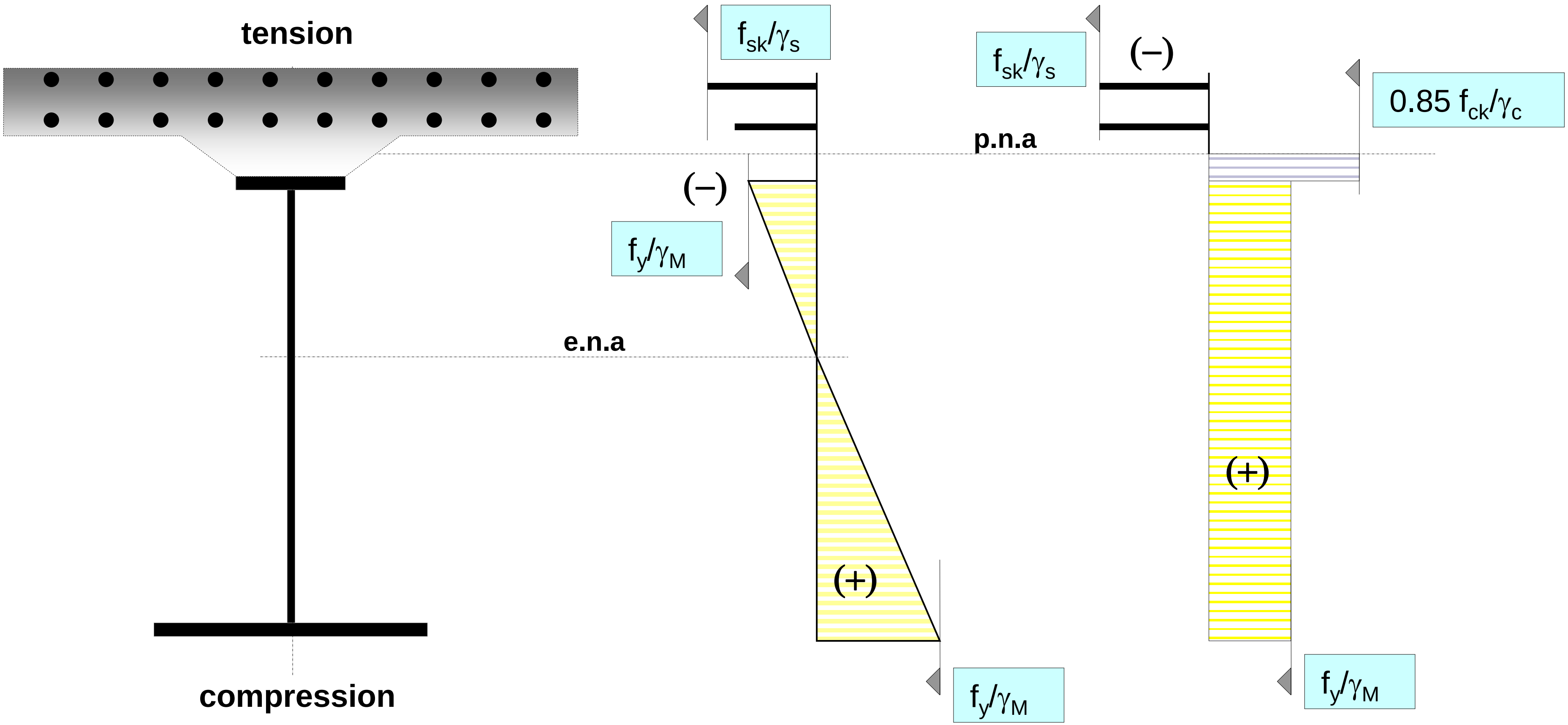
Elastic resistance (for classes 1 to 4)

$$\sigma_{Ed} \leq f_k / \gamma$$

Plastic resistance (for classes 1 and 2)

$$M_{Ed} \leq M_{pl,Rd}$$

ULS cross-section check under M < 0



Elastic resistance (for classes 1 to 4)

$$\sigma_{Ed} \le f_k / \gamma$$

Plastic resistance (for classes 1 and 2)

$$M_{Ed} \le M_{pl,Rd}$$

ULS section check under V and interaction M + V

⇨ **Plastic resistance** : ensured by the steel web

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

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

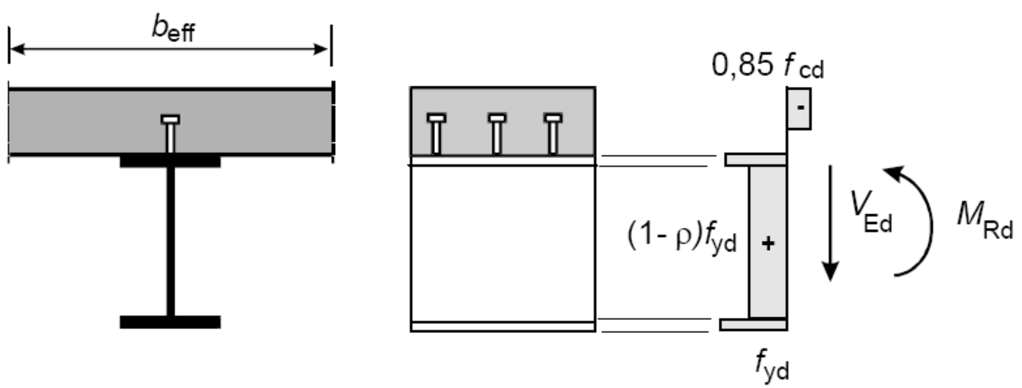
⇨ **Shear buckling resistance** :

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 :**

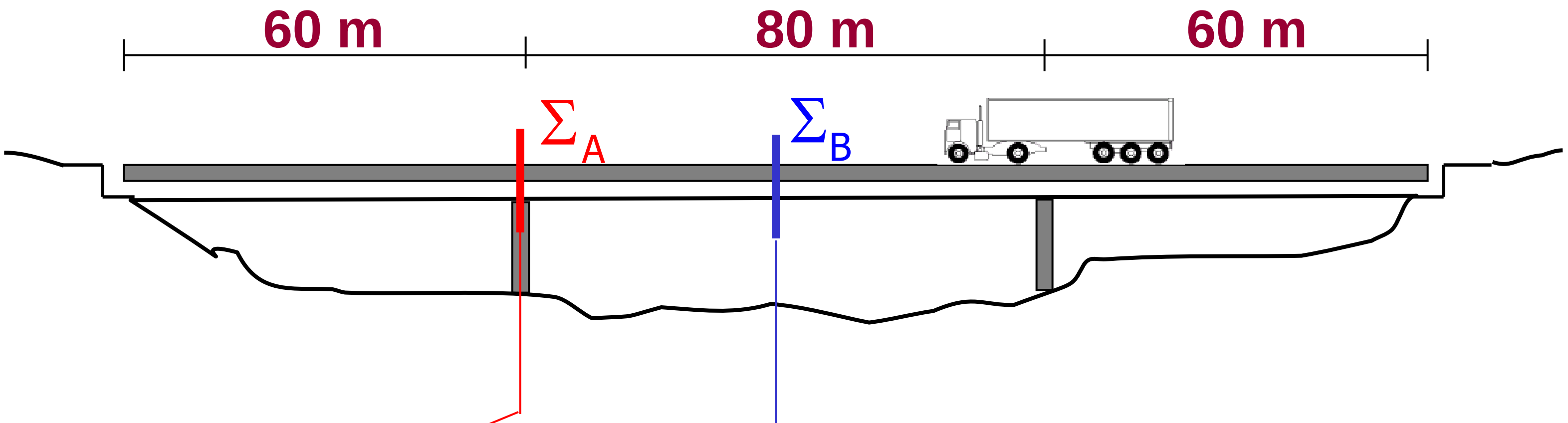
- For Class 1 or 2 sections :
  - if $V_{Ed} < 0.5\ V_{Rd}$ then no interaction occurs
  - if not, the criterion $M_{Ed} < M_{pl,Rd}$ should be verified using a reduced $M_{pl,Rd}$ value



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

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

Worked example : Analysis of 2 different cross-sections



Section Σ_A

Concrete in tension

$M < 0$

Class 3 (elastic section analysis)

M_{ULS} = -109.35 MN.m

V_{ULS} = 8.12 MN

Section Σ_B

Concrete in compression

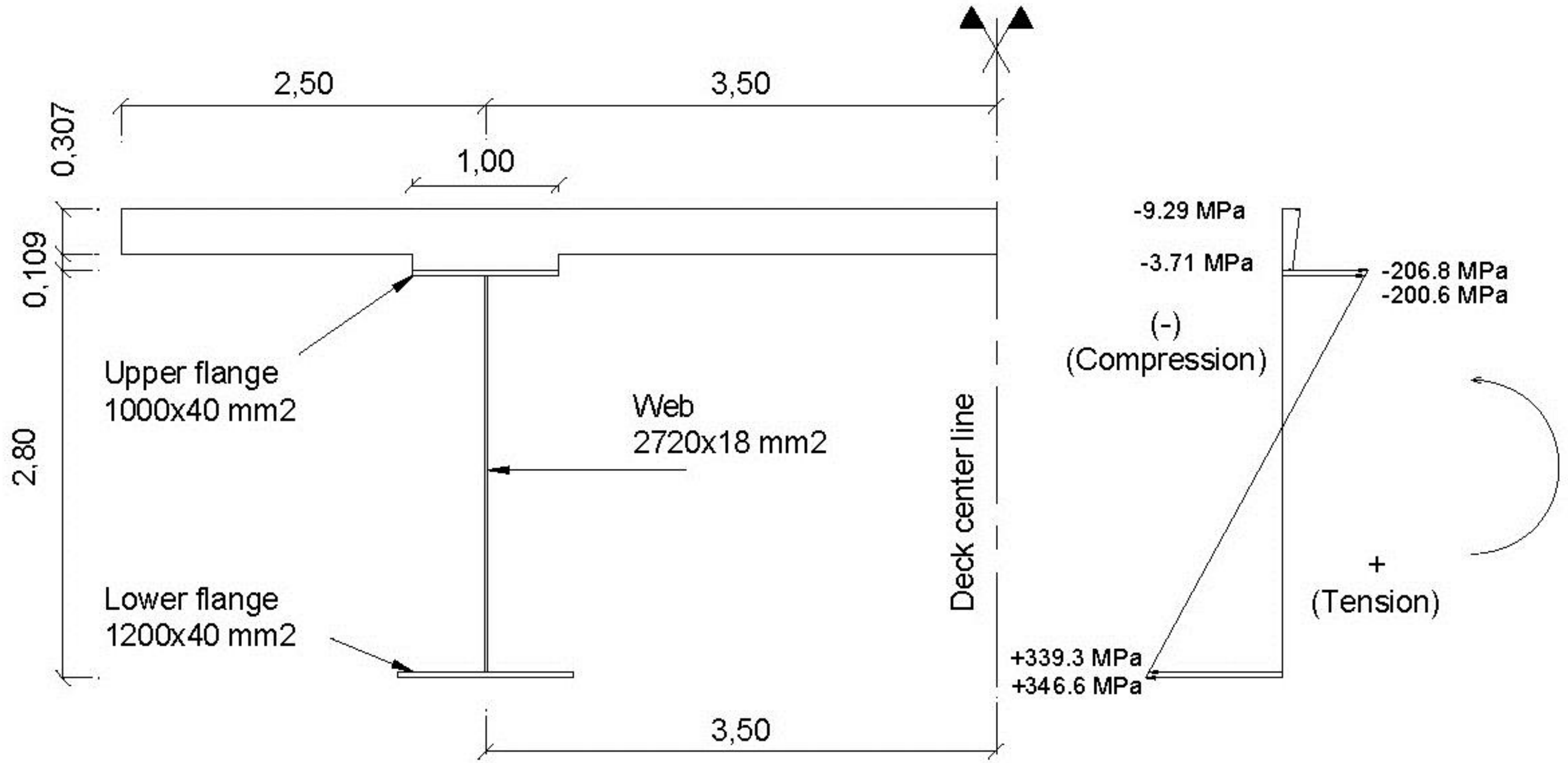
$M > 0$

Class 1 (plastic section analysis)

M_{ULS} = +63.9 MN.m

V_{ULS} = 1.25 MN

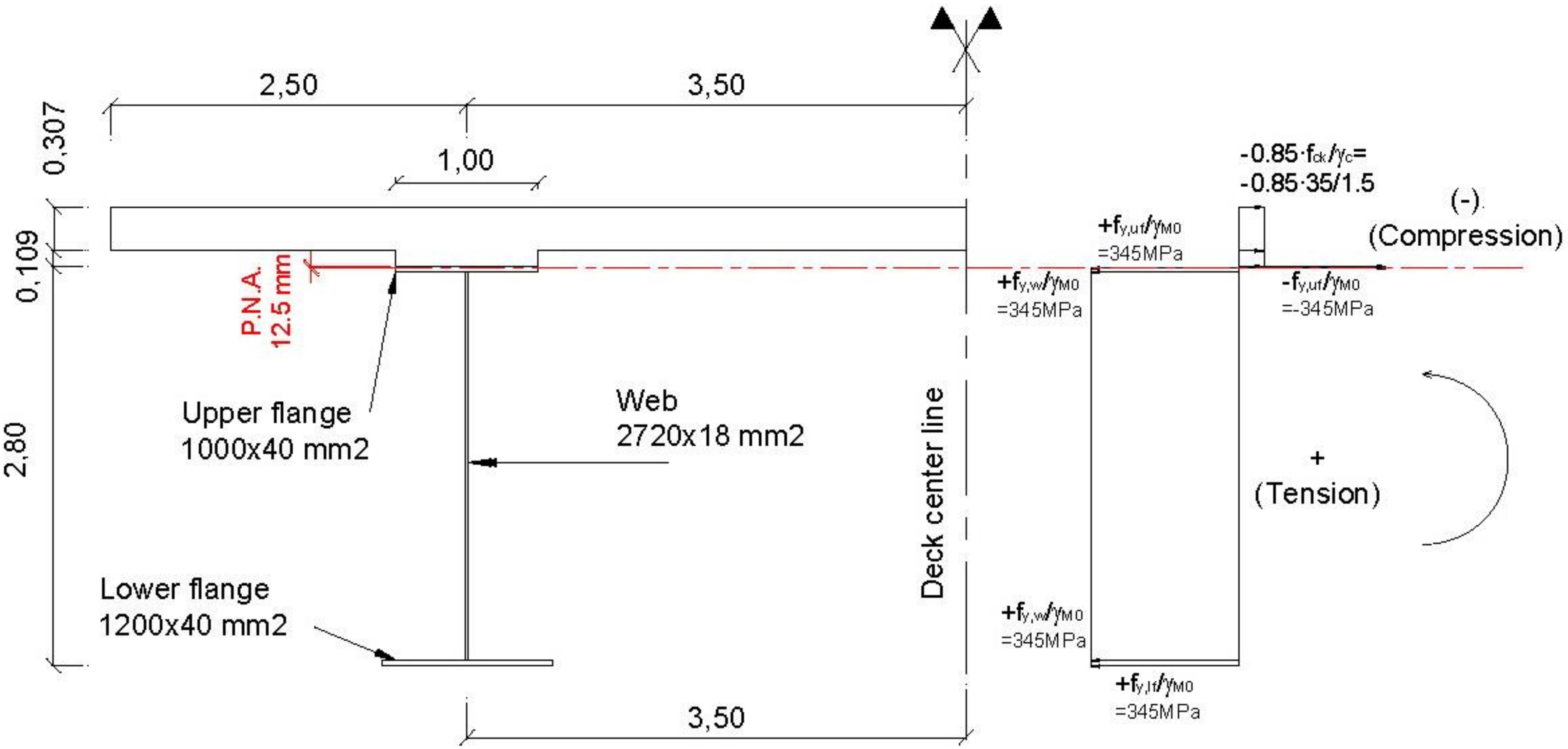
Example 1 Section Σ_B at mid-span P1-P2



- Concrete slab in compression
- Stresses calculated with the **un-cracked** composite mechanical properties and obtained by adding the various steps coming from the construction phases

Example 1 Section Σ_B at mid-span P1-P2

- Lower flange in tension: Class 1
- Composite upper flange connected following EN1994 recommendations: Class 1
- To classify the steel web, we need to determine the position of the Plastic Neutral Axis (PNA)



<u>Example 1</u> Section Σ_B at mid-span P1-P2

- Design plastic resistance of the concrete section in compression:

$$F_c = A_c \frac{0.85 f_{ck}}{\gamma_c} = 1.948\text{m}^2 \cdot \frac{0.85 \cdot 35\text{MPa}}{1.5} = 38.65\,\text{MN}$$

The reinforcing steel bars in compression are neglected.

- Design plastic resistance of the structural steel section in tension :

$$F_a = A_a \frac{f_y}{\gamma_{M0}} = 0.137\text{m}^2 \cdot \frac{345\text{MPa}}{1.0} = 47.25\,\text{MN} \quad \geq F_c$$

- $F_a > F_c$ indicates that the PNA is located in the steel section and its location comes from the internal axial forces equilibrium :

$$\frac{F_a - F_c}{2 b_{fs} f_y / \gamma_{M0}} = 12.5\,\text{mm} \leq t_{fs} = 40\,\text{mm}$$

<u>Example 1</u> Section Σ_B at mid-span P1-P2

⇒ The whole steel web is in tension and therefore in Class 1.

⇒ With every element in Class 1, the cross-section is also in Class 1.

⇒ **PLASTIC SECTION ANALYSIS COULD BE CARRIED OUT.**

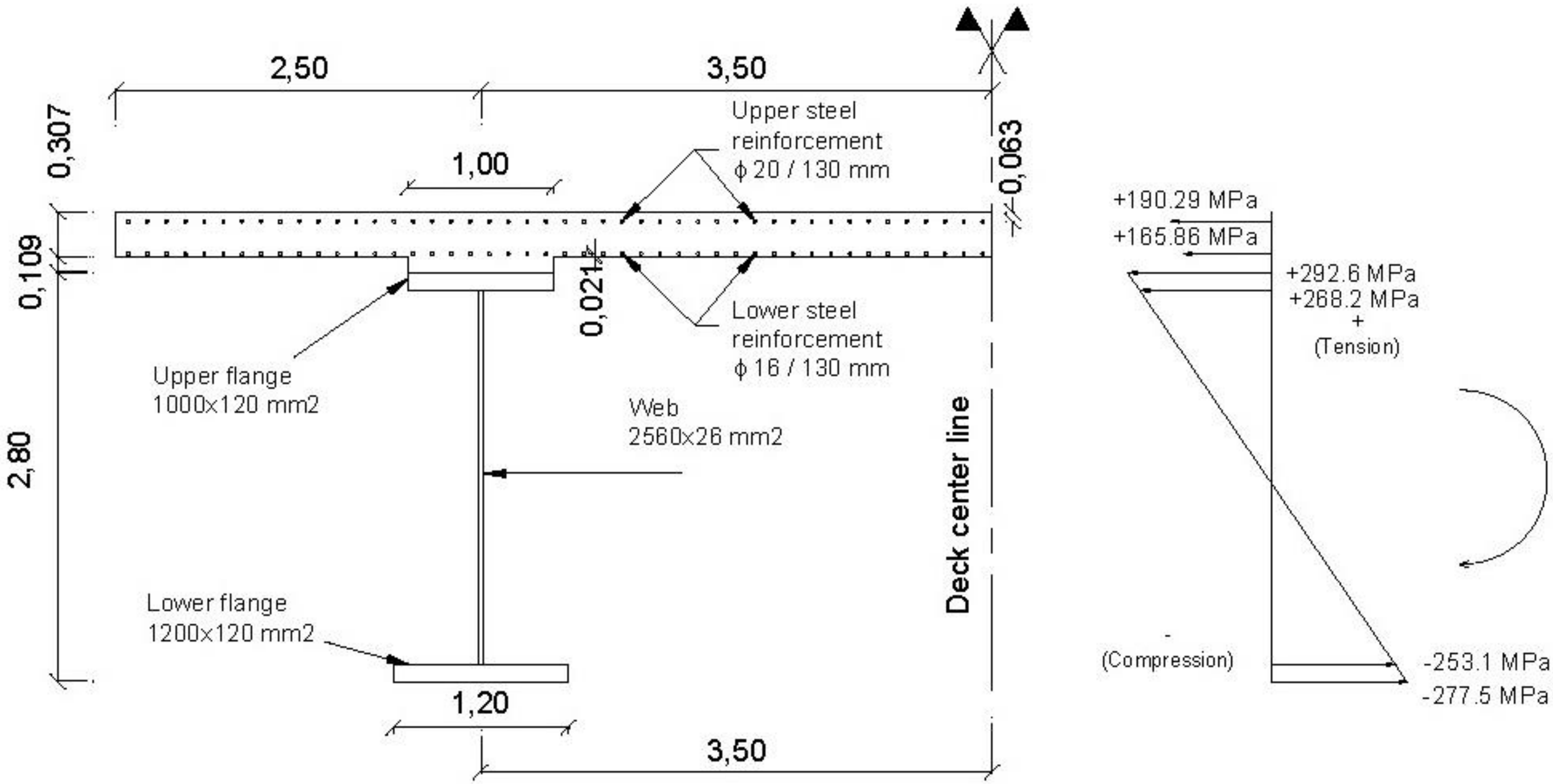
The plastic design bending resistance is calculated from the PNA location:

$$M_{pl,Rd} = +\ 79.6\ \text{MN.m}$$

The cross-section positive bending check is satisfied:

$$M_{Ed} = 63.9\ \text{MN.m} \leq M_{pl,Rd} = 79.6\ \text{MN.m} \quad \text{OK !}$$

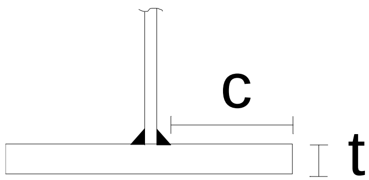
<u>Example 2</u> Section Σ_A at internal support P1



- Concrete slab in tension
- Stresses are calculated with the **<u>cracked</u>** composite mechanical properties and obtained by summing the various steps coming from the construction phases

Example 2 Section Σ_A at internal support P1

- Upper flange in tension : Class 1
- Lower flange in compression and classified according to EC3 : Class 1



$$\frac{c}{t} = 4.8 \leq 9\sqrt{\frac{235}{f_y}} = 8$$

- The web is partially in compression.
- Based on the plastic stress blocks, we look at the Plastic Neutral Axis (assumed to be located within the web depth). It is obtained by equilibrating the internal axial forces applied to each part of the cross-section:

$$A_s\frac{f_{sk}}{\gamma_s} + A_{f,top}\frac{f_{yf}}{\gamma_{M0}} + xt_w\frac{f_{yw}}{\gamma_{M0}} = (h_w - x)t_w\frac{f_{yw}}{\gamma_{M0}} + A_{f,\inf}\frac{f_{yf}}{\gamma_{M0}}$$

Reinforcing steel bars

Top flange

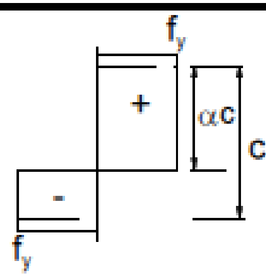
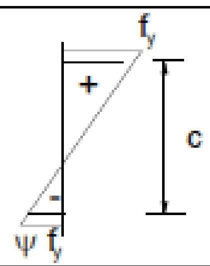
Upper web part in tension

Lower web part in compression

Bottom flange

$x = 1.1\text{m}$ meaning that 57 % of the web is in compression .

<u>Example 2</u> Section Σ_A at internal support P1

Class	Part subject to bending and compression
Stress distribution in parts (compression positive)	
1	when $\alpha > 0{,}5$: $c/t \le \frac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \frac{36\varepsilon}{\alpha}$
2	when $\alpha > 0{,}5$: $c/t \le \frac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \frac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)	
3	when $\psi > -1$: $c/t \le \frac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \le -1^{*)}$: $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

α = 0.57 (% of web depth in compression)

$c/t = h_w/t_w$ = 2560/26 = 98.5 (web slenderness)

$$\varepsilon = \sqrt{\frac{235}{f_y}} = 0.82$$

c/t = 98.5 >> 58.6
The web is at least in Class 3.

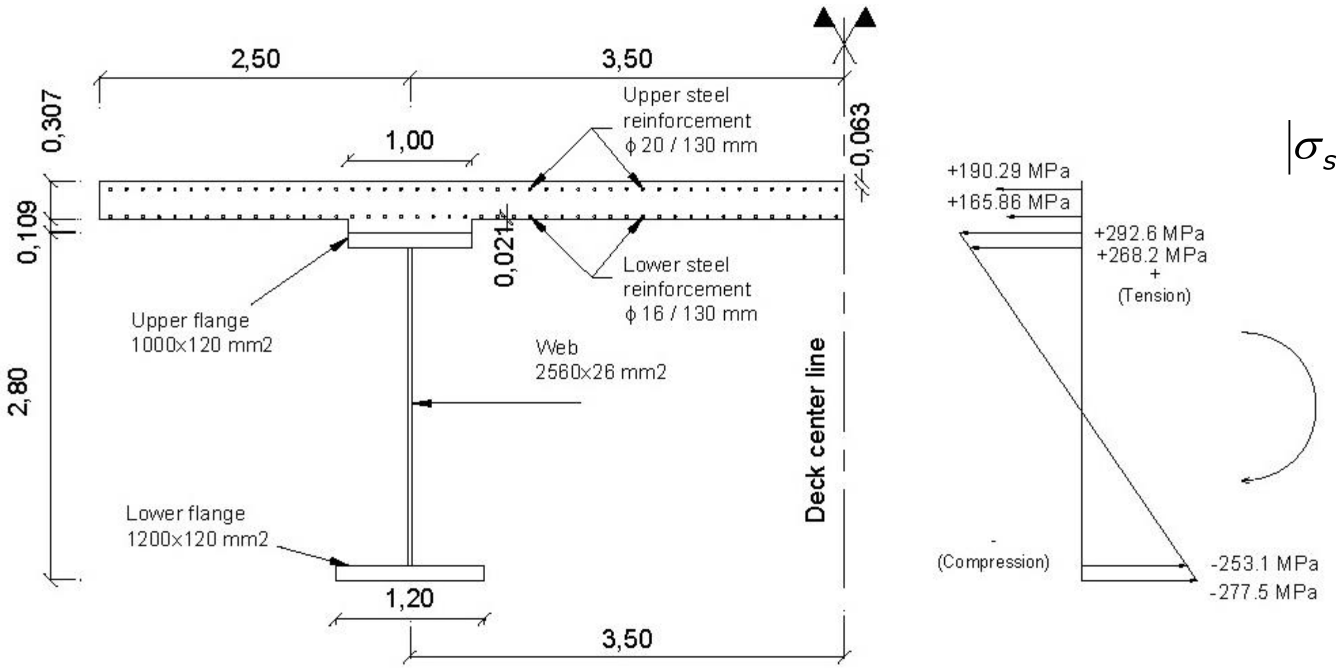
$$\psi = \frac{\text{tension}}{\text{compression}} = \frac{268.2\,\text{MPa}}{-253.1\,\text{MPa}} = -1.06$$

c/t = 98.5 < 108.5
The web is a Class 3 element.

Example 2 Section Σ_A at internal support P1

Class 3 cross-section => **ELASTIC SECTION ANALYSIS** should be performed !

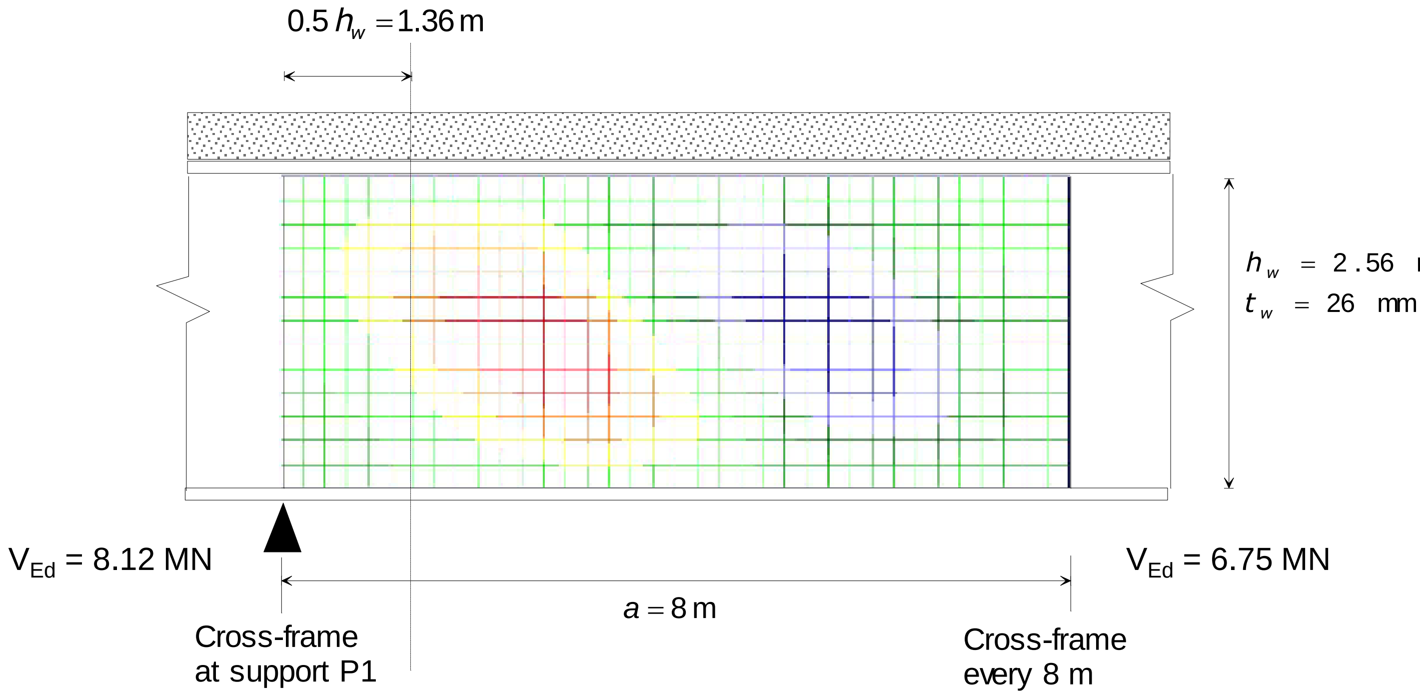
At ULS this check could be carried out in the mid-plane of the flanges instead of using the extreme fibre of the steel I-section.



$$|\sigma_s| \le \frac{f_{sk}}{\gamma_s} = \frac{500}{1.15} = 435\,\text{MPa}$$

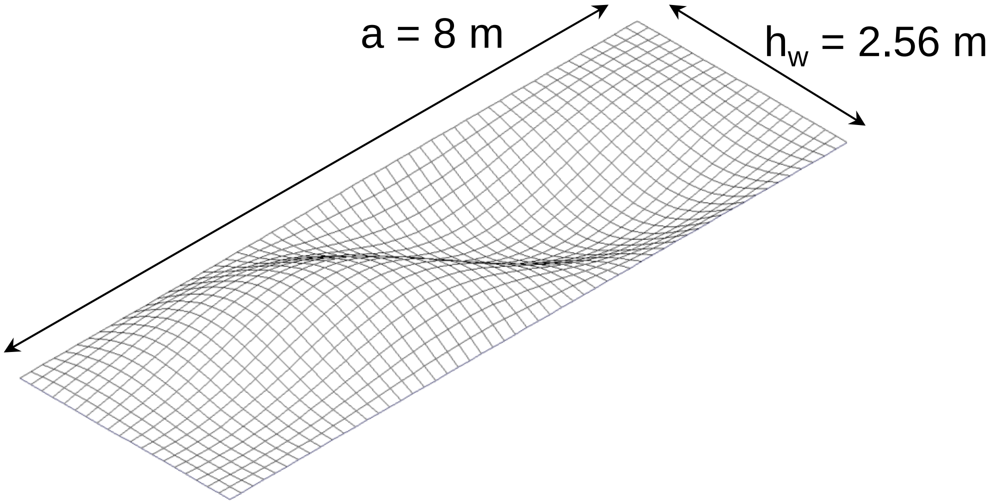
$$|\sigma_s| \le \frac{f_y}{\gamma_{M0}} = 295\,\text{MPa}$$

Example 2 Section Σ_A at internal support P1



- Every cross-frame is designed to act as a rigid vertical posts for the web.
- The shear force is assumed to be uniform (maximum value = 8.12 MN).
- Elastic critical shear stress :

$$\tau_{cr} = k_\tau \sigma_E = 112.6\,\text{MPa}$$



Example 2 Section Σ_A at internal support P1

Design plastic shear resistance

$$V_{pl,Rd} = h_w t_w \cdot \frac{\eta f_{yw}}{\gamma_{M0}\sqrt{3}} = 15.9\,\text{MN}$$

Strain hardening effect up to steel grade S460 : $\eta = 1.2$

Safety factor : $\gamma_{M0} = 1.0$

Design buckling shear resistance

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

$$\leq h_w t_w \cdot \frac{\eta f_{yw}}{\gamma_{M1}\sqrt{3}} = 14.4\,\text{MN}$$

Safety factor : $\gamma_{M1} = 1.1$

Reduced slenderness : $\bar{\lambda}_w = \sqrt{\frac{f_{yw}/\sqrt{3}}{\tau_{cr}}} = 1.33$

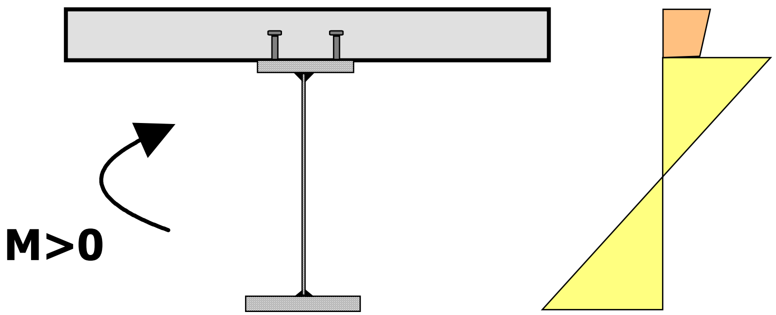
Reduction factor : $\chi_w = \frac{1.37}{0.7 + \bar{\lambda}_w} = 0.675$

$$V_{bw,Rd} = 8.14\,\text{MN}$$

$$V_{Ed} = 8.12\,\text{MN} \leq \min\left(V_{pl,Rd}; V_{bw,Rd}\right) = 8.14\ \text{MN}$$ OK !

SLS stress limitations

1- Concrete in compression

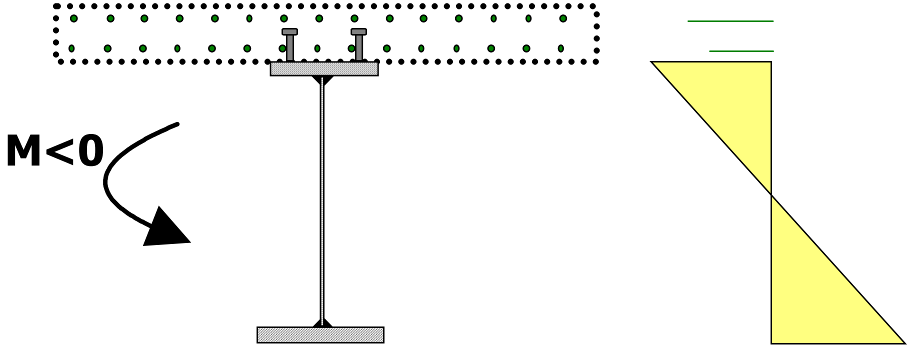


$\sigma_c \leq 0.6 f_{ck}$ To avoid excessive creep and micro-cracking (Characteristic SLS)

$\sigma_c \leq 0.45 f_{ck}$ To insure the linear creep assumption (QP SLS)

$\sigma_a \leq f_{yk}$

2- Concrete in tension



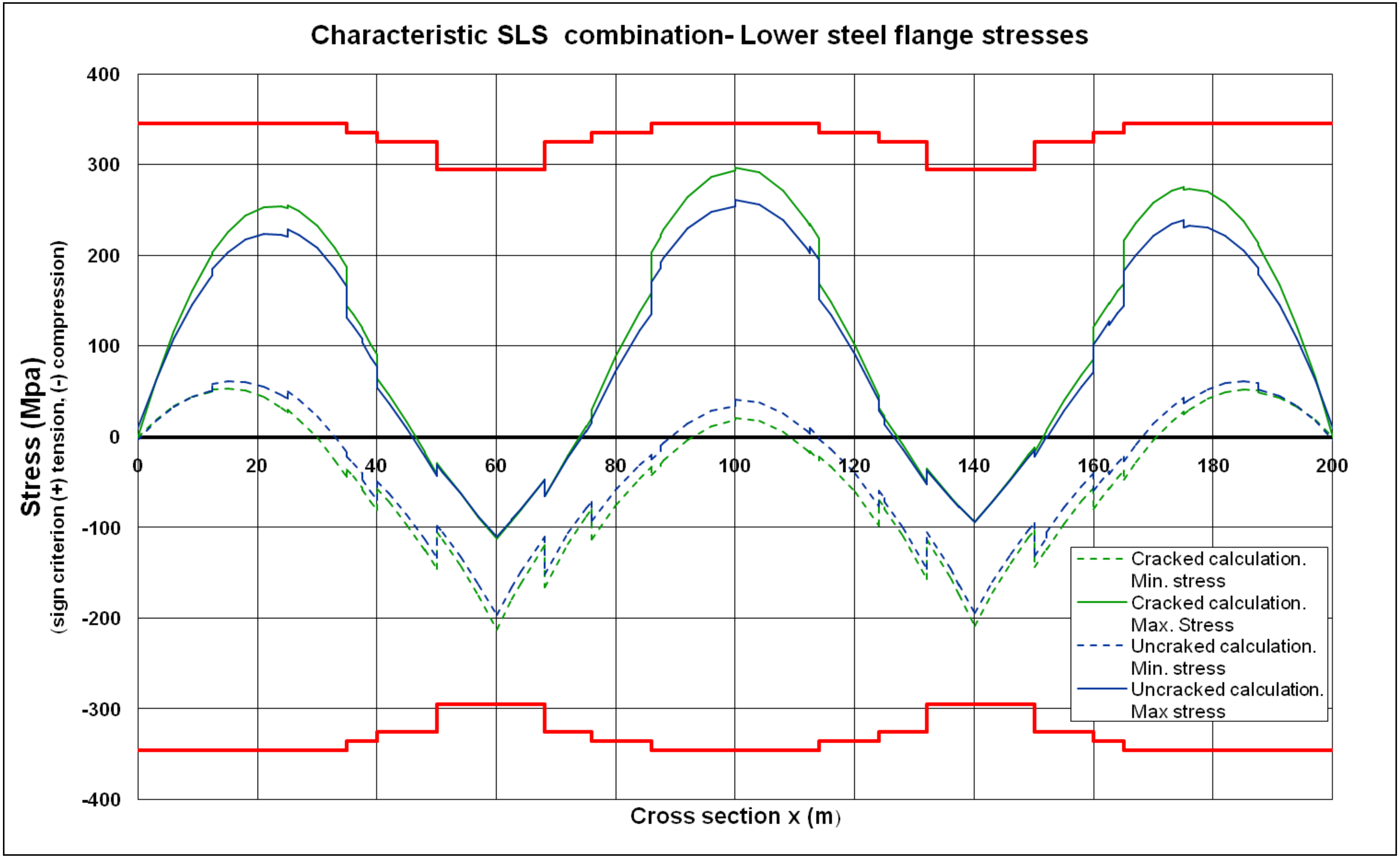
$\sigma_s \leq 0.8.f_{sk}$ To avoid inelastic strain, unacceptable cracking or deformation

$\sigma_a \leq f_{yk}$

The <u>tensile</u> stress σ_s in the reinforcement (calculated without taking the concrete strength into account) should include the tension stiffening term $\Delta\sigma_s$.

Example 3 Stress control at SLS

Structural steel (grade S355): bending



Crack width control at SLS

1. **Minimum reinforcement area is required**

2. **Control of cracking due to direct loading**

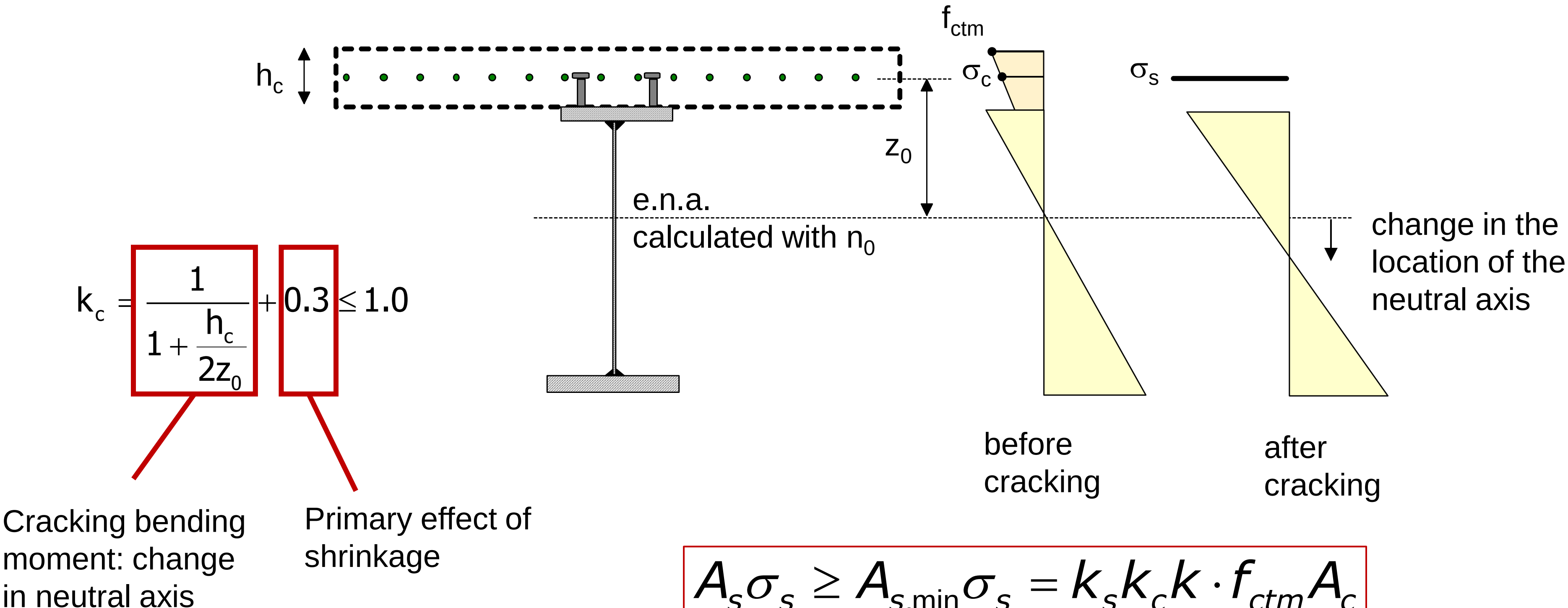
The design crack width w_k should be limited to a maximum crack width w_{max} by limiting :

- bar spacing $d \leq d_{max}$
- or bar diameter $\Phi \leq \Phi_{max}$

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

d_{max} , Φ_{max} depend on the stress level σ_s in the reinforcement and on the design crack width w_k

Minimum reinforcement



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

$$A_s\sigma_s \geq A_{s,min}\sigma_s = k_s k_c k \cdot f_{ctm} A_c$$

$k_s = 0.9$ reduction of the normal force in the concrete slab due to initial cracking and local slip of the shear connection (ductile and flexible)

$k = 0.8$ effect of non-uniform shape in the self-equilibrating stresses within h_c

σ_s maximum stress level allowed in the reinforcement after cracking

(= f_{sk} at yielding or a lower value if required by the control of crack width)

Example 4 Minimum reinforcement

- h_c = 0.307 m : concrete slab thickness
- z_0 is the vertical distance between the centroid of the un-cracked concrete flange, and the un-cracked composite section (calculated using the modular ratio n_0 for short term loading)
- $\sigma_s = f_{sk}$ = 500 Mpa (yield strength)
- A_c = 1.95 m² : area of the concrete slab in tension, due to direct loading and primary effect of shrinkage, immediately prior to cracking

Cross section at support P1	Cross-section at mid-span P1-P2
z_0 = 0.764 m	z_0 = 0.406 m
k_c = 1.13 but should be ≤ 1.0	k_c = 1.02 but should be ≤ 1.0
$A_{s,min}$ = 90 cm² (for ½ slab 6 m wide)	
ϕ = 20 mm every 130 mm (top layer) ϕ = 16 mm every 130 mm (bottom layer) $\rho_s = A_s/A_c$ = 1.22 % A_s = 238 cm² >> $A_{s,min}$ = 90 cm² OK !	ϕ = 16 mm every 130 mm (top layer) ϕ = 16 mm every 130 mm (bottom layer) $\rho_s = A_s/A_c$ = 0.95 % A_s = 186 cm² >> $A_{s,min}$ = 90 cm² OK !

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

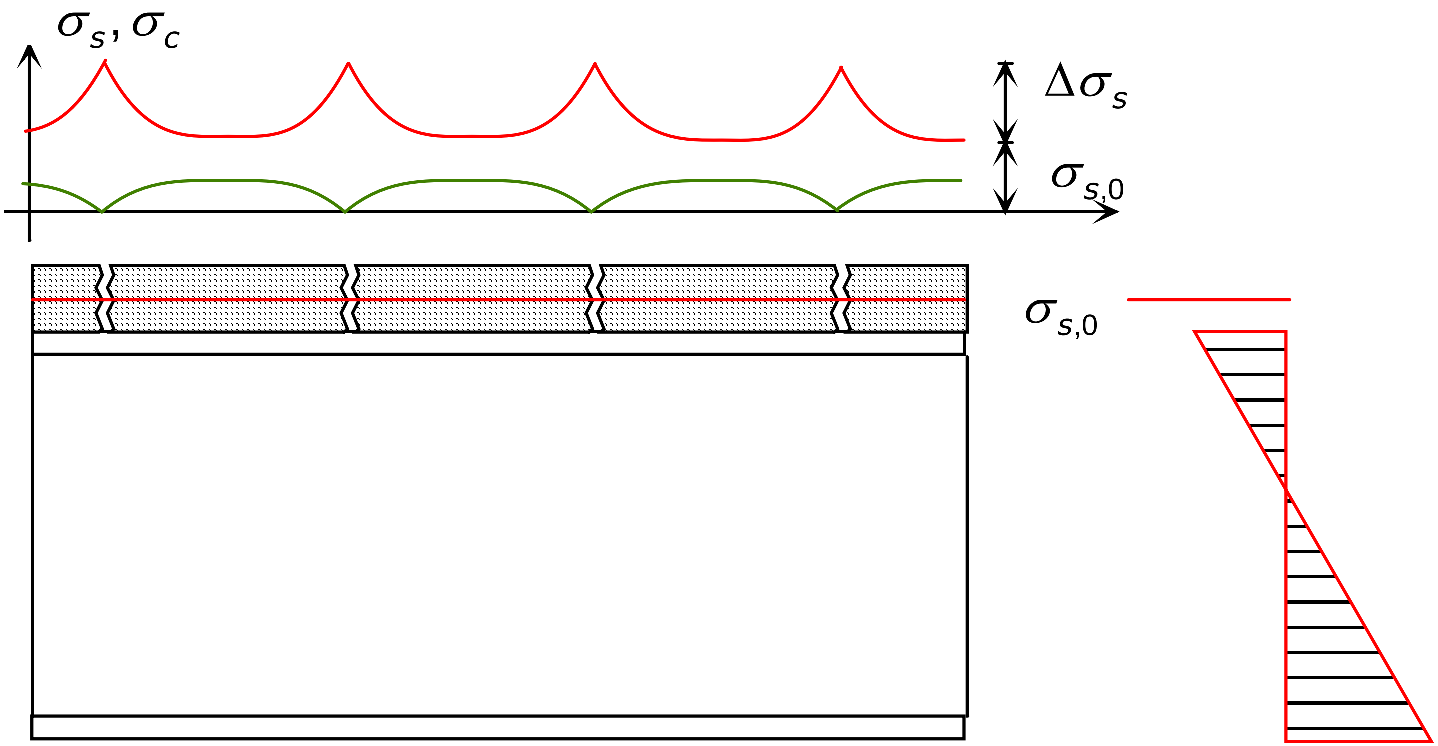
[a] For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed.

[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.

Stiffening effect ($\Delta\sigma_s$) of concrete in tension between cracks



$$\sigma_s = \sigma_{s,0} + \Delta\sigma_s$$

$$\Delta\sigma_s = \frac{0.4 \cdot f_{ctm}}{\alpha_{st} \cdot \rho_s}$$

$$\alpha_{st} = \frac{A \cdot I}{A_a \cdot I_a}$$

$$\rho_s = \frac{A_s}{A_{ct}}$$

A, I: area and second moment of area for the effective cracked composite cross-section

A_a, I_a : area and second moment of area for the structural steel cross-section

A_s : area of all layers of longitudinal reinforcement within the effective concrete area A_{ct}

f_{ctm} : mean tensile strength of concrete

Stresses at QP SLS calculated with the cracked composite mechanical properties of the cross-section (including the construction phases)

Example 4 Crack width control

Cross section at support P1	Cross-section at mid-span P1-P2
ϕ = 20 mm every 130 mm (top layer) ϕ = 16 mm every 130 mm (bottom layer) A_s = 238 cm² $\rho_s = A_s/A_{ct}$ = 1.22 %	ϕ = 16 mm every 130 mm (top layer) ϕ = 16 mm every 130 mm (bottom layer) A_s = 186 cm² $\rho_s = A_s/A_{ct}$ = 0.95 %
$\alpha_{st} = AI / A_a I_a = 1.23$	$\alpha_{st} = AI / A_a I_a = 1.42$
$\Delta\sigma_s$ = 85.2 MPa	$\Delta\sigma_s$ = 94.9 MPa
$\sigma_{s,0}$ = 65.9 MPa	$\sigma_{s,0}$ = 27.5 MPa
$\sigma_s = \sigma_{s,0} + \Delta\sigma_s$ = 151.2 Mpa	$\sigma_s = \sigma_{s,0} + \Delta\sigma_s$ = 122.4 MPa

Steel stress σ_s (N/mm²)	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	-

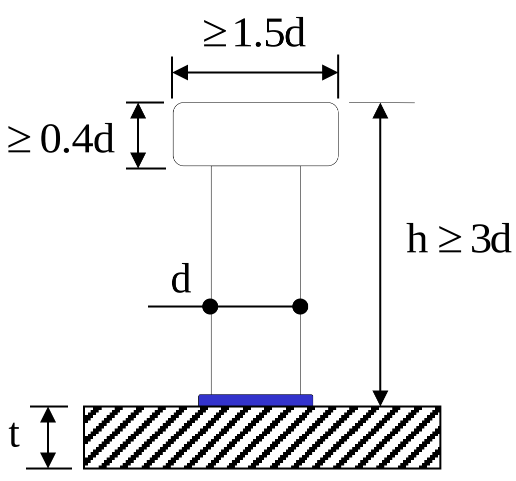
Steel stress σ_s (N/mm²)	Maximum bar spacing (mm) for design crack width w_k		
	w_k=0.4mm	w_k=0.3mm	w_k=0.2mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

Steel-concrete connection

1. Transmit the longitudinal shear force $v_{L,Ed}$ per unit length of the steel-concrete interface
2. Achieved by 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 $M_{el,Rd} \leq M_{Ed} \leq M_{pl,Rd}$
- Uncracked section analysis (even where cracking is assumed in global analysis)
- 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



$16 \leq d \leq 25mm$

$$P_{Rd} = \min(P^{(1)}_{Rd}; P^{(2)}_{Rd})$$

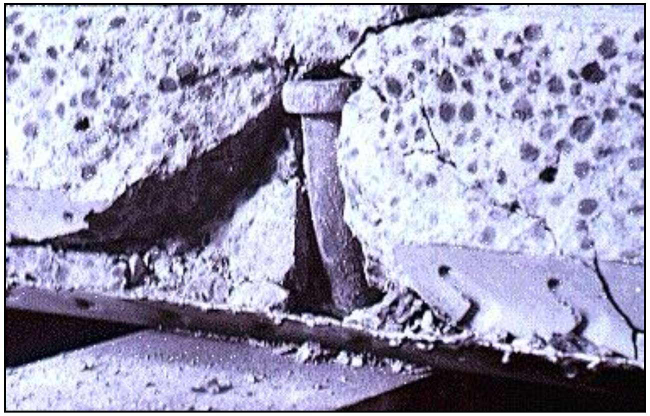
- Shank toe shear resistance : $P^{(1)}_{Rd} = \dfrac{0.8 \cdot f_u \cdot \dfrac{\pi \cdot d^2}{4}}{\gamma_v}$

- Concrete crushing around the shank toe :

$$P^{(2)}_{Rd} = \frac{0.29 \cdot \alpha \cdot d^2 \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_v}$$

$\alpha = 0.2\left(\dfrac{h_{sc}}{d} + 1\right)$ if 3 ≤ h/d ≤ 4

$\alpha = 1.0$ if h/d > 4

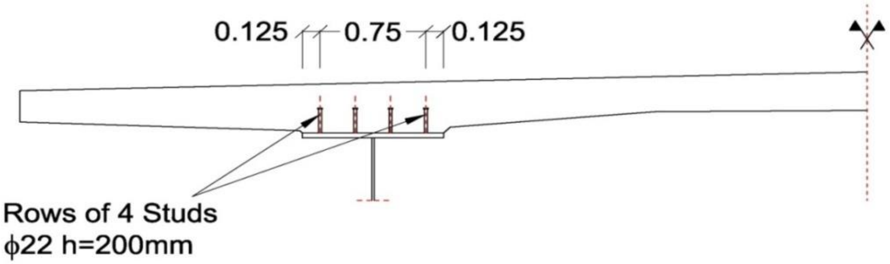


Limit State	Design resistance	Recommended
U.L.S.	$P_{Rd} = \dfrac{P_{Rk}}{\gamma_V}$	$\gamma_V = 1.25$
S.L.S.	$k_s.P_{Rd}$	$k_s = 0.75$

Example 5 Connection at the steel-concrete interface

Resistance of headed studs

d = 22 mm ; h = 200 mm ; f_u = 450 MPa



$$P^{(1)}{}_{Rd} = \frac{0.8 \cdot 450 \cdot \frac{\pi \cdot 22^2}{4}}{1.25} = 0.1095E^6 \text{ N=0.1095 MN}$$

E_{cm} = 34000 MPa (short term secant modulus of a concrete C35/40)

h/d = 200/22 = 9.1 >> 4 => α=1

$$P^{(2)}{}_{Rd} = \frac{0.29 \cdot 1 \cdot 22^2 \cdot \sqrt{35 \cdot 34077.14}}{1.25} = 0.1226 \text{ MN}$$

Each row of 4 headed studs resist at **ULS** : $4 \cdot P_{Rd,1stud} = 0.438\text{MN}$

At **SLS**, the headed stud resistance is reduced to $P_{Rd,SLS} = k_s \cdot P_{Rd,ULS}$ with $k_s = 0.75$

Each row of 4 headed studs resist at SLS: $4 \cdot k_s \cdot P_{Rd} = 0.3064 \text{ MN}$

Detailing for shear connectors

Longitudinal spacing between shear connectors :

- to insure the composite behaviour in all cross-sections :
  s_{max} = min (800 mm; 4 h) where h is the concrete slab thickness

- to insure Class 1 or 2 element for a Class 3 or 4 upper steel flange which is connected to the concrete slab:

  $s_{max} \leq 22 \cdot t_f \cdot \sqrt{235/fy}$ full longitudinal contact between flange and slab

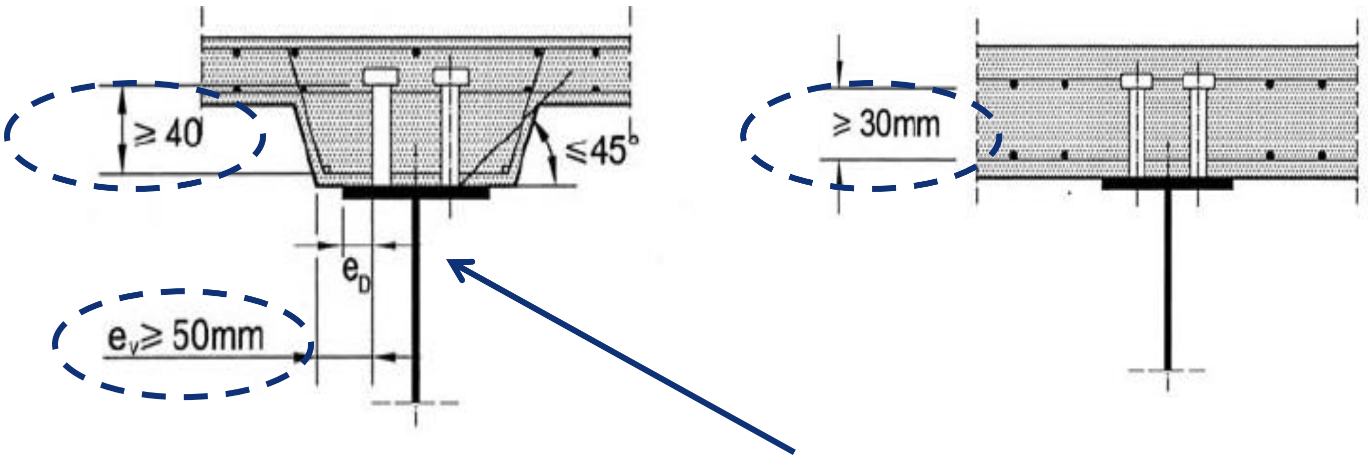
  $s_{max} \leq 15 \cdot t_f \cdot \sqrt{235/fy}$ partial longitudinal contact between flange and slab

Transversal spacing between shear connectors :

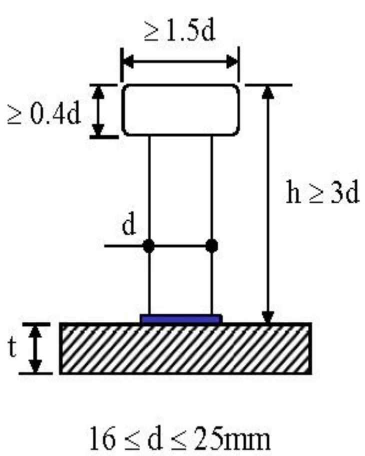
- Maximum distance of shear connectors closest to the free edge of the upper flange in compression (Class 1 element for the outstand not connected flange element):

$$e_D \leq 9 \cdot t_f \cdot \sqrt{235/fy}$$

Specific detailing for headed studs



- to insure a correct headed stud welding: $25\text{ mm} \le e_D$



- $d \le 2.5 \cdot t_f$
- $d \le 1.5 \cdot t_f$ for a structural steel flange in tension and subjected to fatigue
- minimum longitudinal spacing : $5\,d \le s_{min}$
- minimum transversal spacing : $e \ge 2.5\,d$ in solid slab

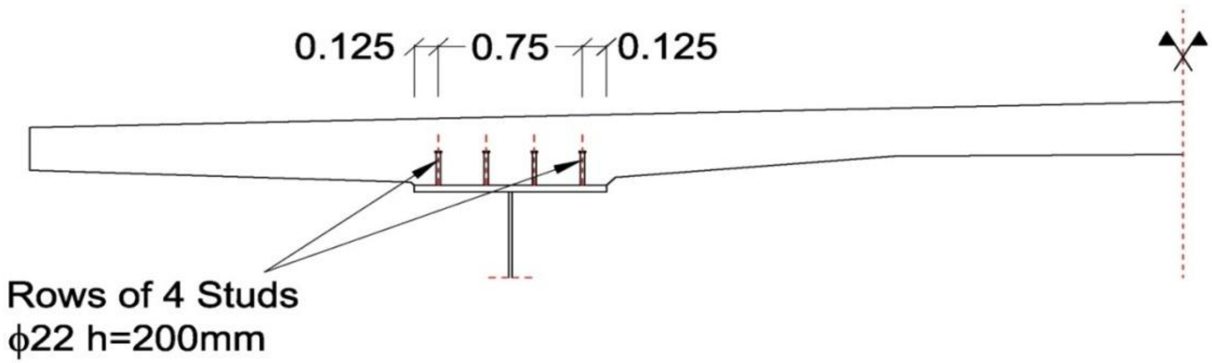
  $e \ge 4\,d$ otherwise

Example 5 Connection at the steel-concrete interface

Detailing of headed studs

Upper Steel flange t_f (mm)	f_y (N/mm²)	s_{max}	e_D
40	345	**726**	297
55	335	**800**	414
80	325	**800**	*
120	295	**800**	*

Longitudinal spacing : $s_{min} \geq 5d = 110$ mm



$$e_D = \frac{b_f - b_0}{2} - \frac{d}{2} = \frac{1000 - 750}{2} - \frac{22}{2} = 114 > 25$$

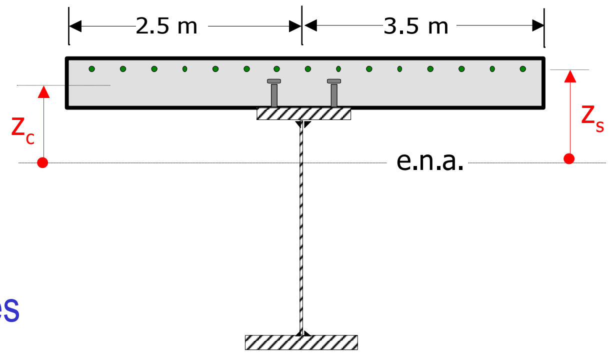
Elastic design of the shear connection

- 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

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

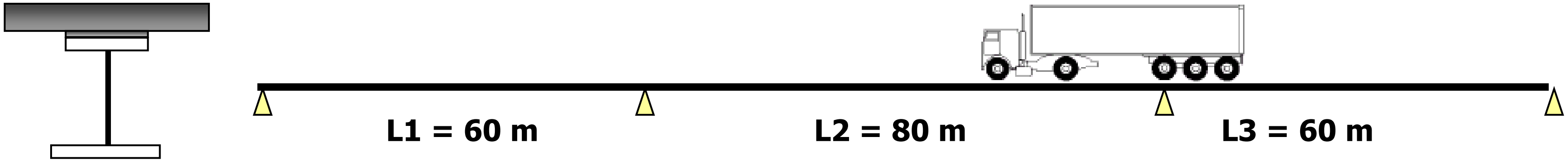
Uncracked mechanical properties

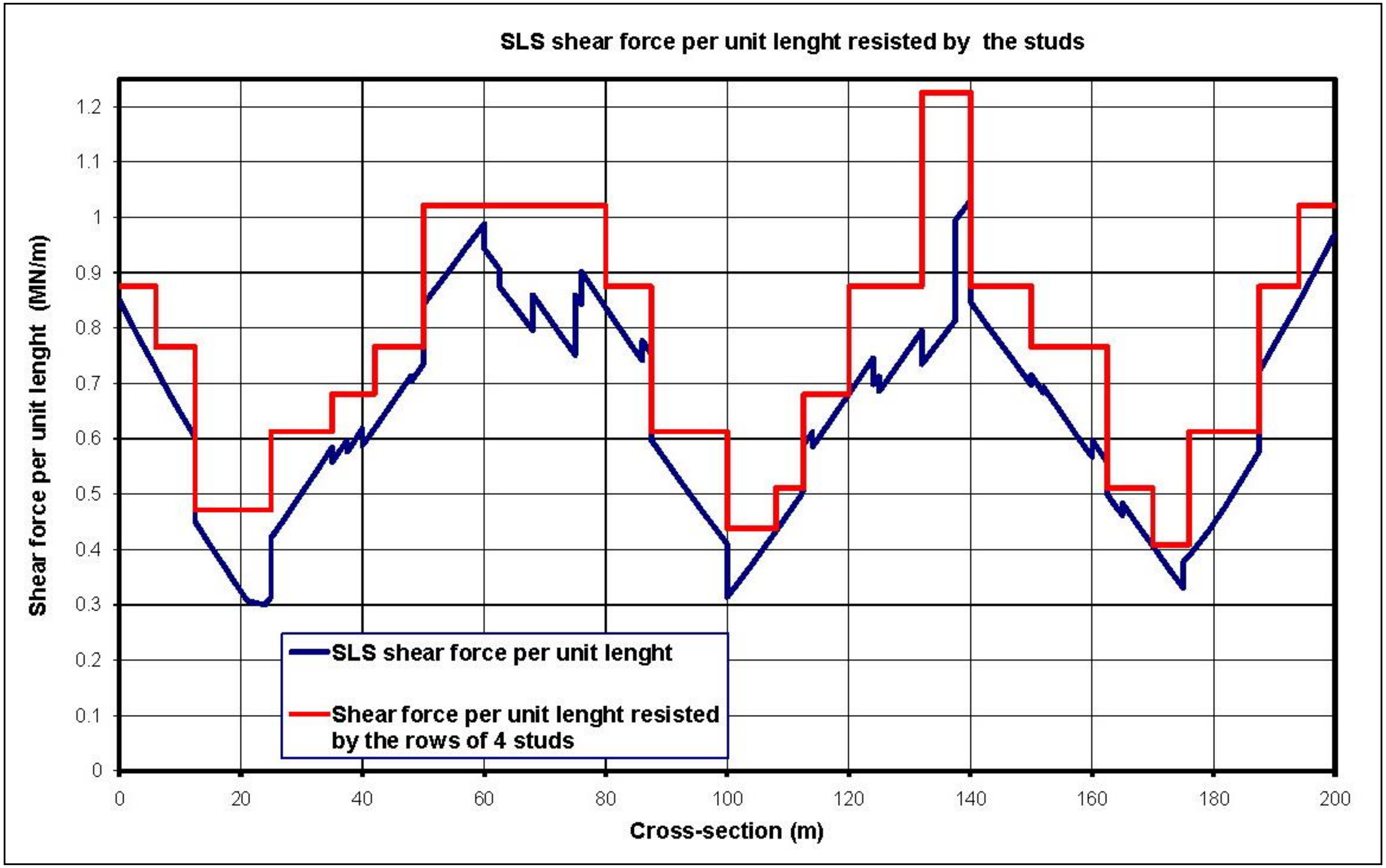


SLS	ULS
For a given length l_i of the girder (to be chosen by the designer), the N_i shear connectors are uniformly distributed and satisfy :	For a given length l_i of the girder (to be chosen by the designer), the N_i^* shear connectors are uniformly distributed and satisfy :
$v_{L,Ed}{}^{SLS}(x) \leq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

Example 5 Connection at the steel-concrete interface

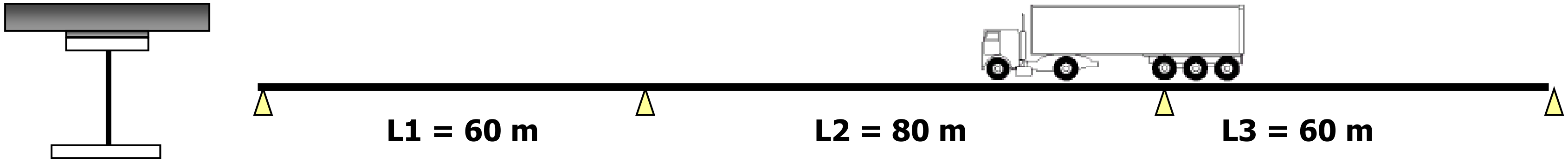
Design at characteristic SLS

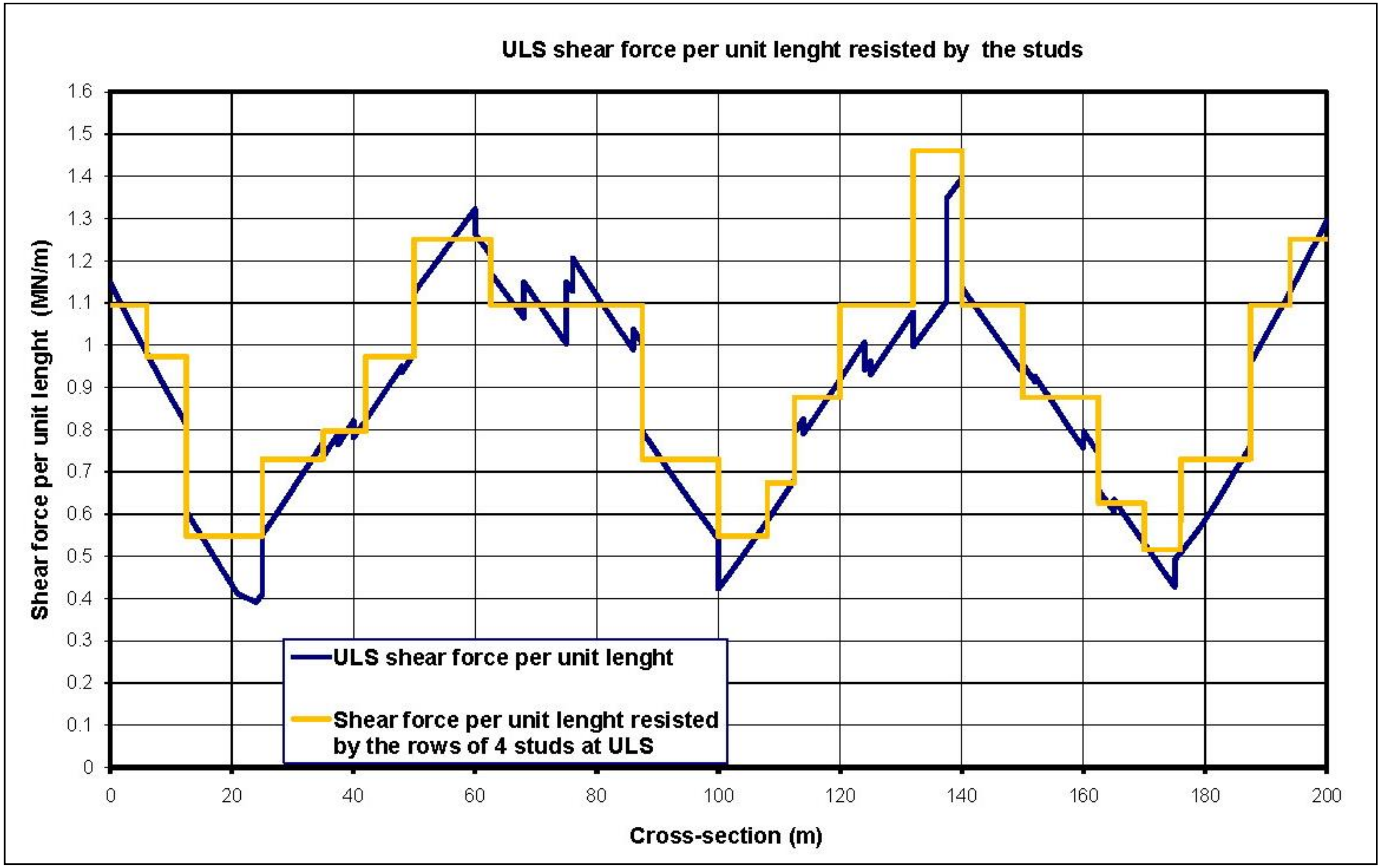




Example 5 Connection at the steel-concrete interface

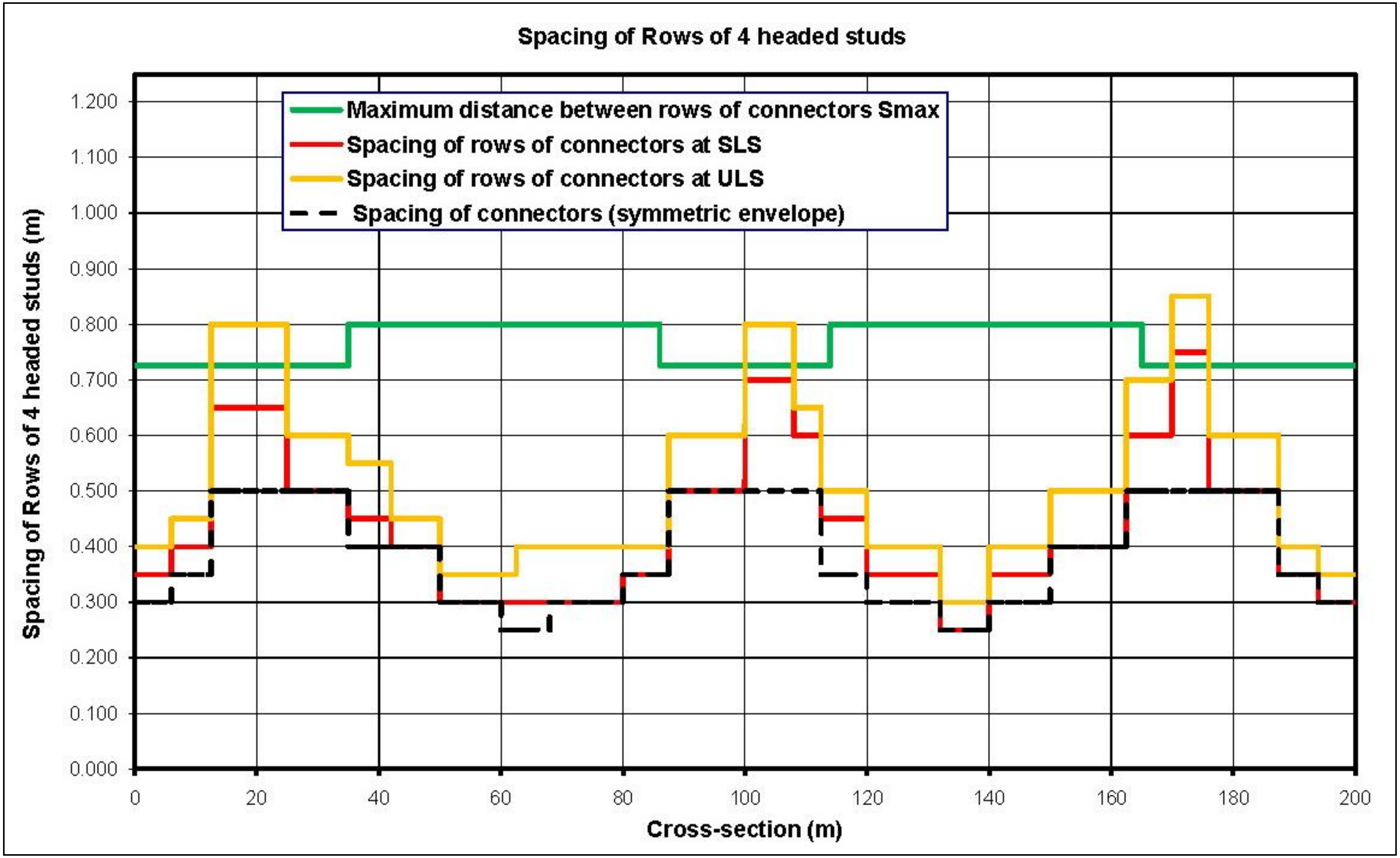
Design at ULS





Example 5 Connection at the steel-concrete interface

Synopsis for the worked example



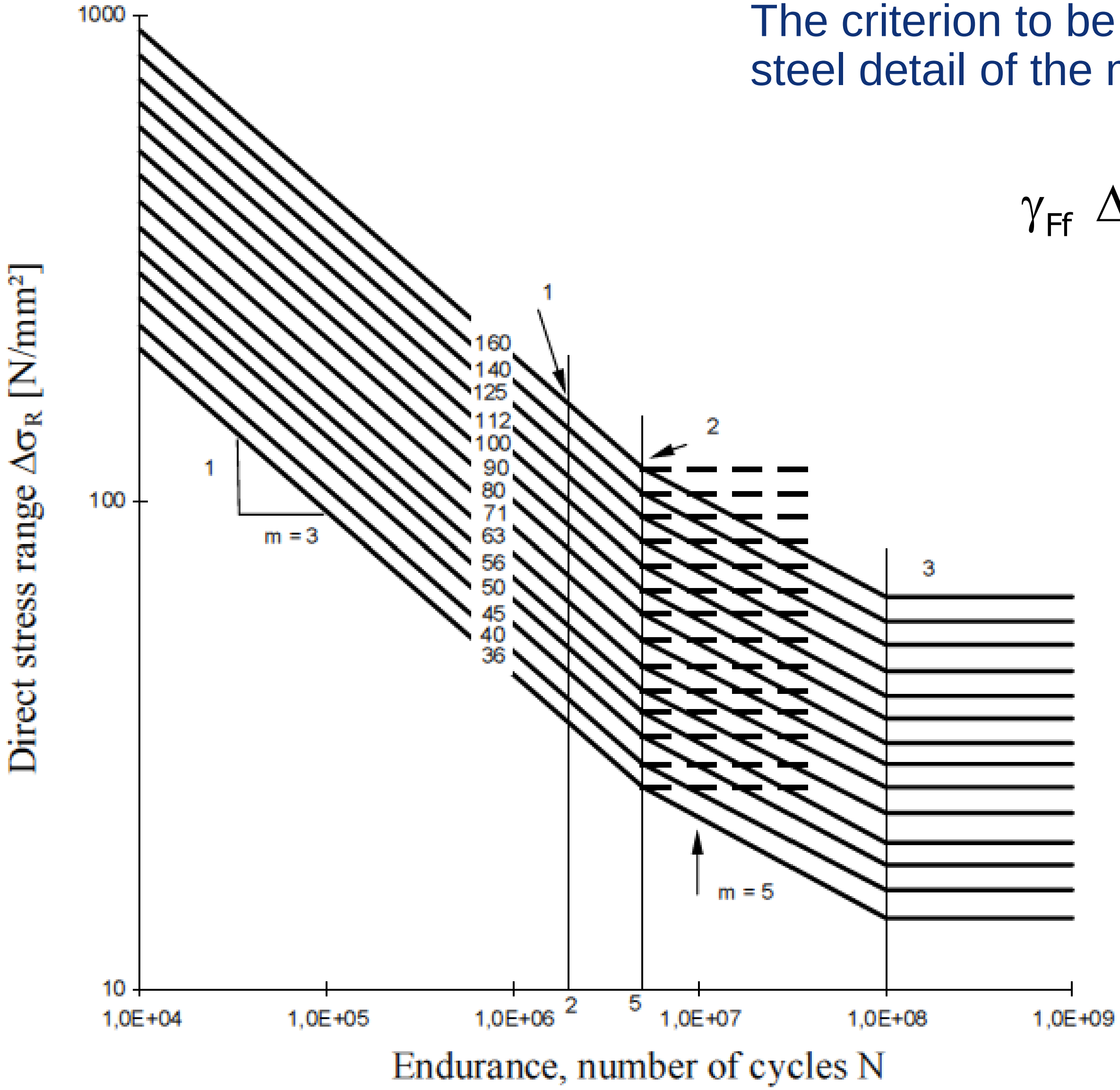
Fatigue ULS in a composite bridge

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

- the ***structural steel*** details of the main girder
- the slab ***concrete***
- the slab ***reinforcement***
- the shear ***connection***

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**	$\gamma_{Mf} = 1.15$	$\gamma_{Mf} = 1.35$

The S-N curves in Eurocode 3 for structural steel details



The criterion to be checked in a structural steel detail of the main girder is :

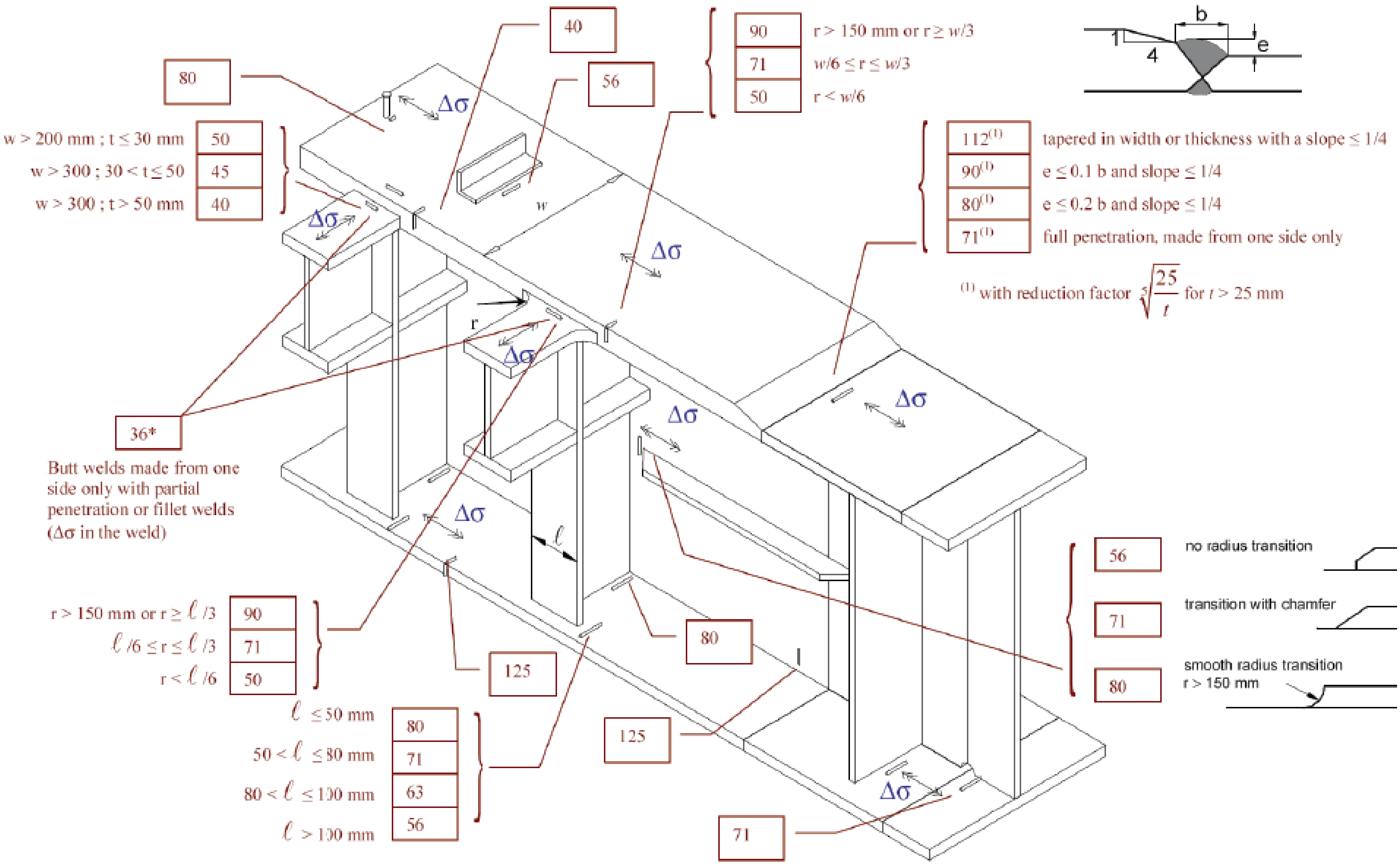
$$\gamma_{Ff}\ \Delta\sigma_E \leq \frac{\Delta\sigma_c}{\gamma_{Mf}}$$

1 Detail category $\Delta\sigma_C$

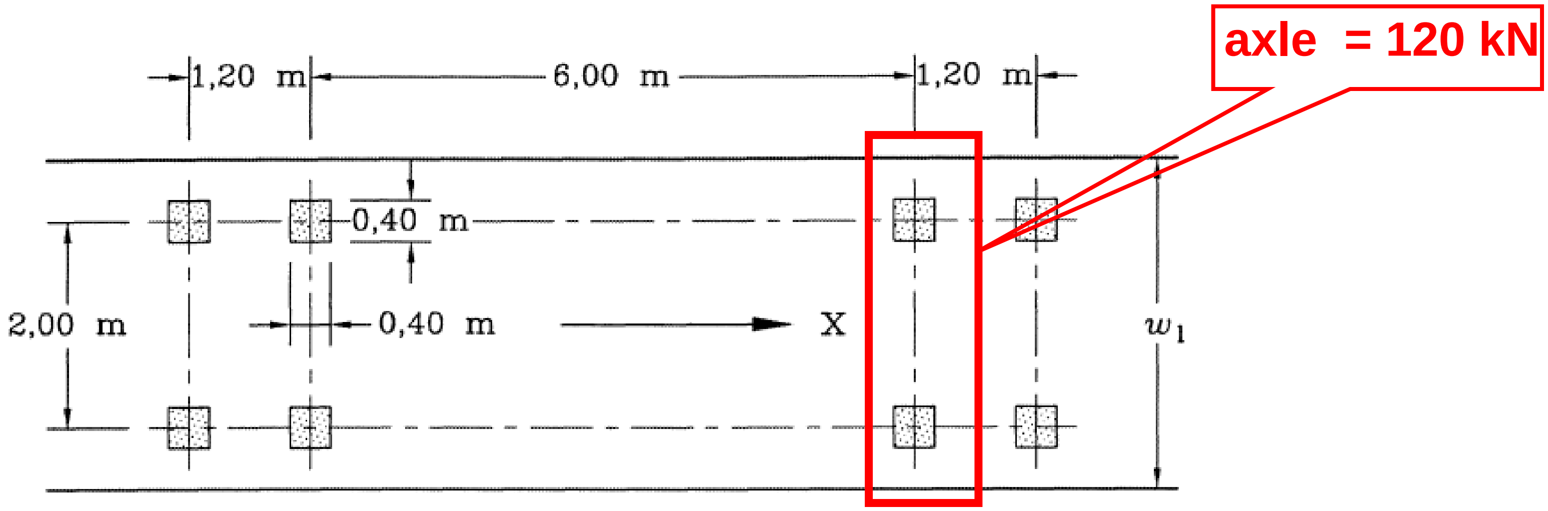
2 Constant amplitude fatigue limit $\Delta\sigma_D$

3 Cut-off limit $\Delta\sigma_L$

Classification of typical bridge details



Fatigue Load Model 3 « equivalent lorry » (FLM3)



- 2.10^6 FLM3 “equivalent lorries” are assumed to cross the bridge per slow lane over the lifetime (100 years)
- every crossing induces a stress range $\Delta\sigma_p = |\sigma_{max,f} - \sigma_{min,f}|$ in a given structural detail
- the equivalent stress range $\Delta\sigma_E$ at 2. 10^6 cycles in this detail is obtained as follows :

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

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)

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

Basic combination of non-cyclic actions	+	Fatigue loads
$G_{max}\ (\text{or } G_{min}) + 1.0\ (\text{or } 0.0) S + 0.6 T_k$		FLM 3
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 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 detail (cross-section where the concrete participates):

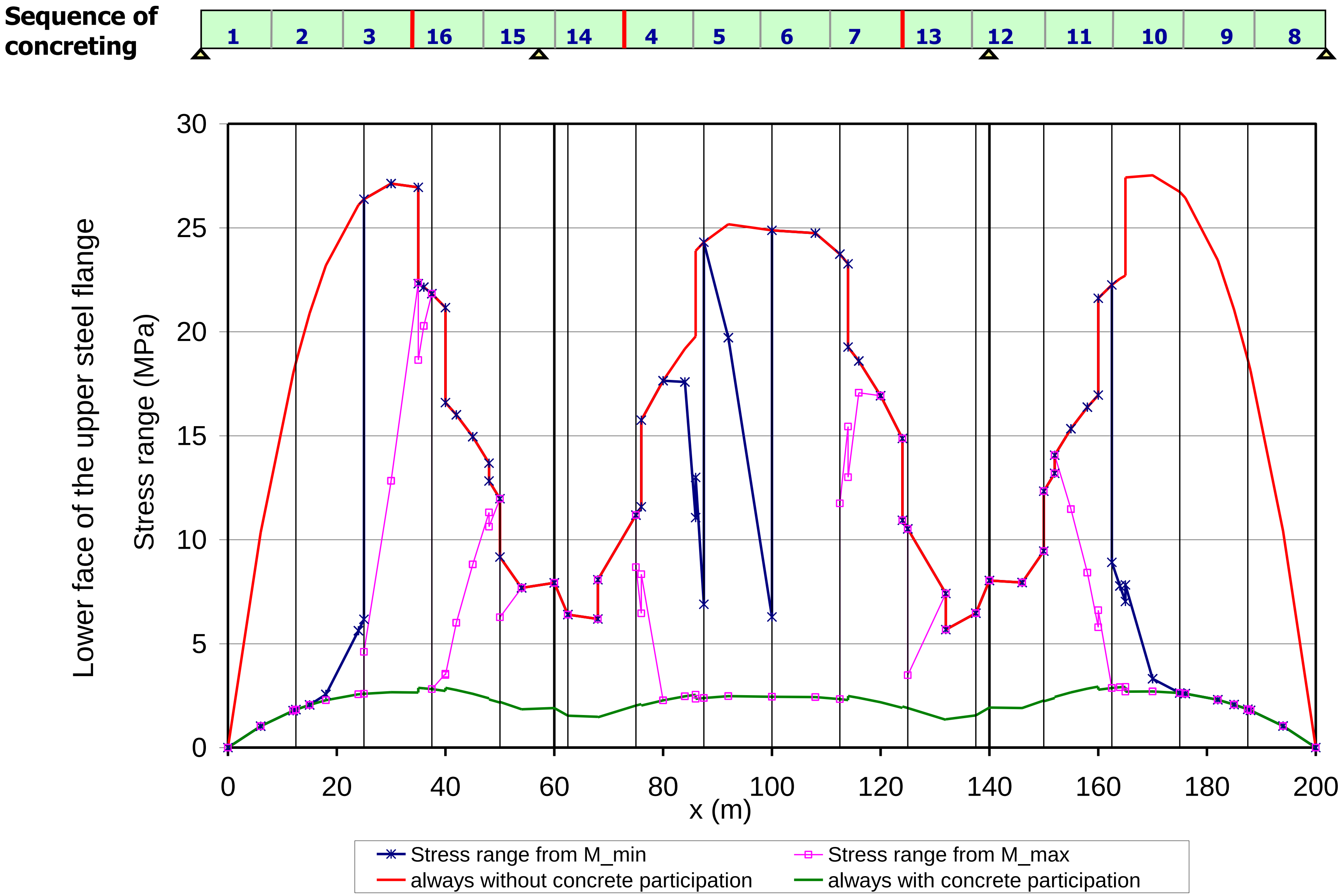
$$\sigma_{Ed,max,f} = M_{a,Ed}\left(\frac{v_a}{I_a}\right) + M_{c,Ed}\left(\frac{v_1}{I_1}\right)_{n_L} + M_{FLM3,max}\left(\frac{v_1}{I_1}\right)_{n_0}$$

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

- Stress range in the detail : $\Delta\sigma_p = \left|\sigma_{Ed,max,f} - \sigma_{Ed,min,f}\right|$

Example 6 Fatigue in the structural steel main girders

Stress range $\Delta\sigma_p$



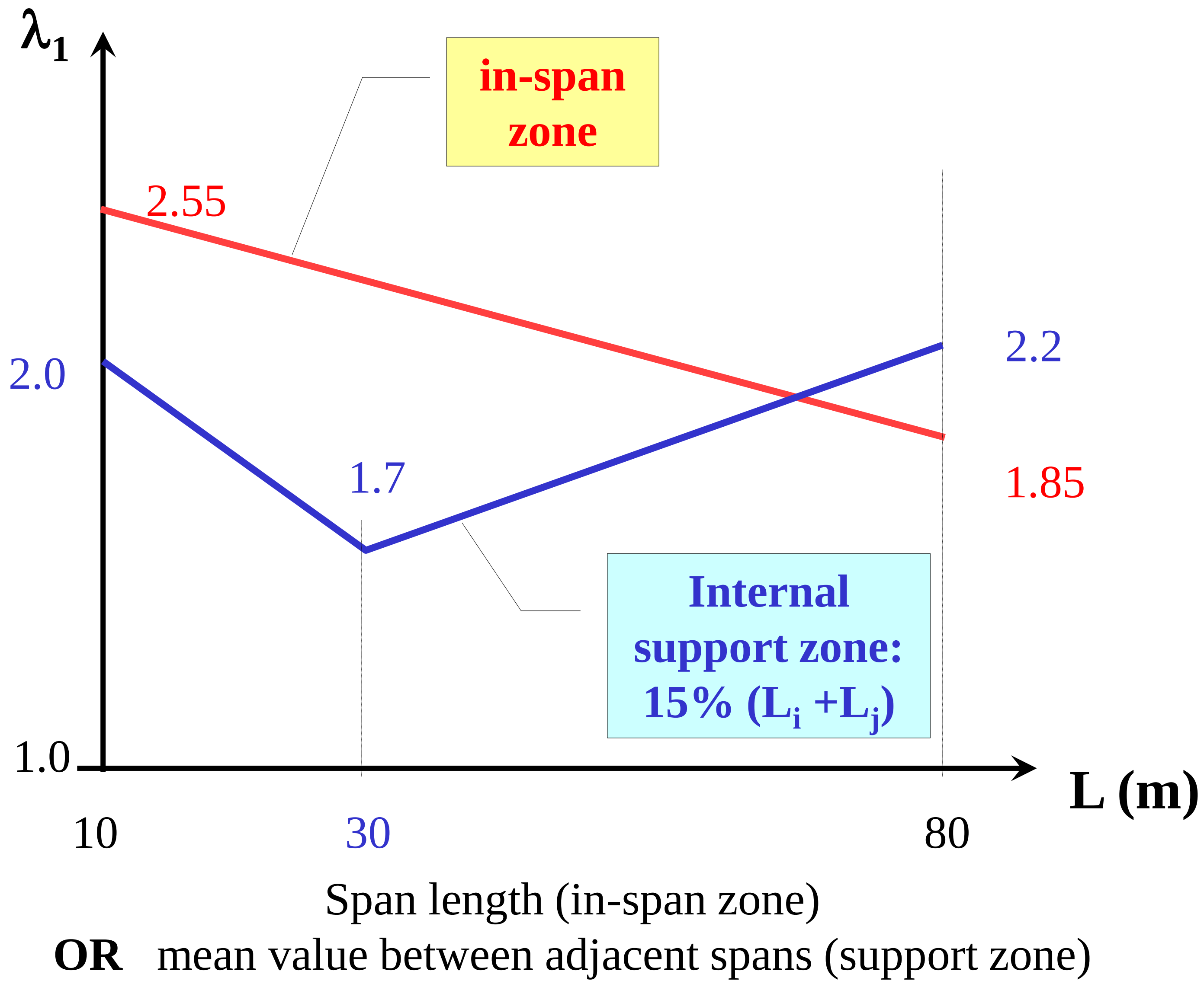
Damage equivalence factor λ

For a structural steel detail :

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

- λ_1 : influence of the bridges span length and of the shape of the influence line for the internal forces and moments in the structural steel detail
- 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 slow lanes
- λ_{max} : influence of the constant amplitude fatigue limit $\Delta\sigma_D$ at 5.10^6 cycles (threshold effect)

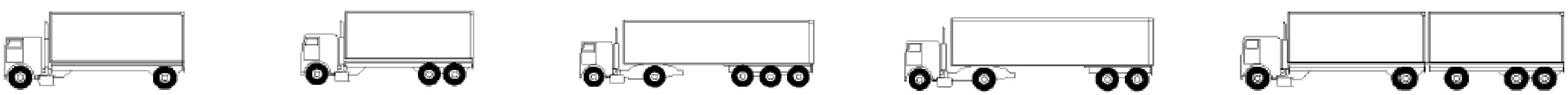
Damage equivalence factor λ



Damage equivalence factor λ

Assumptions are needed for the traffic volume (based on the expected traffic on the bridge):

$N_{obs} = 0.5.10^6$ 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 :

$$Q_{ml} = \left(\frac{\sum n_i Q_i^5}{\sum n_i} \right)^{1/5} = 407 \text{ kN}$$

$$\lambda_2 = \frac{Q_{ml}}{480} \left(\frac{N_{obs}}{0.5.10^6} \right)^{(1/5)} = \frac{407}{480} = 0.848$$

Damage equivalence factor λ

$$\lambda_3 = \left(\frac{t_{Ld}}{100}\right)^{1/5}$$

where t_{Ld} is the bridge lifetime (in years)

Design life in years	50	60	70	80	90	100	120
Factor λ_3	0,871	0,903	0,931	0,956	0,979	1,00	1,037

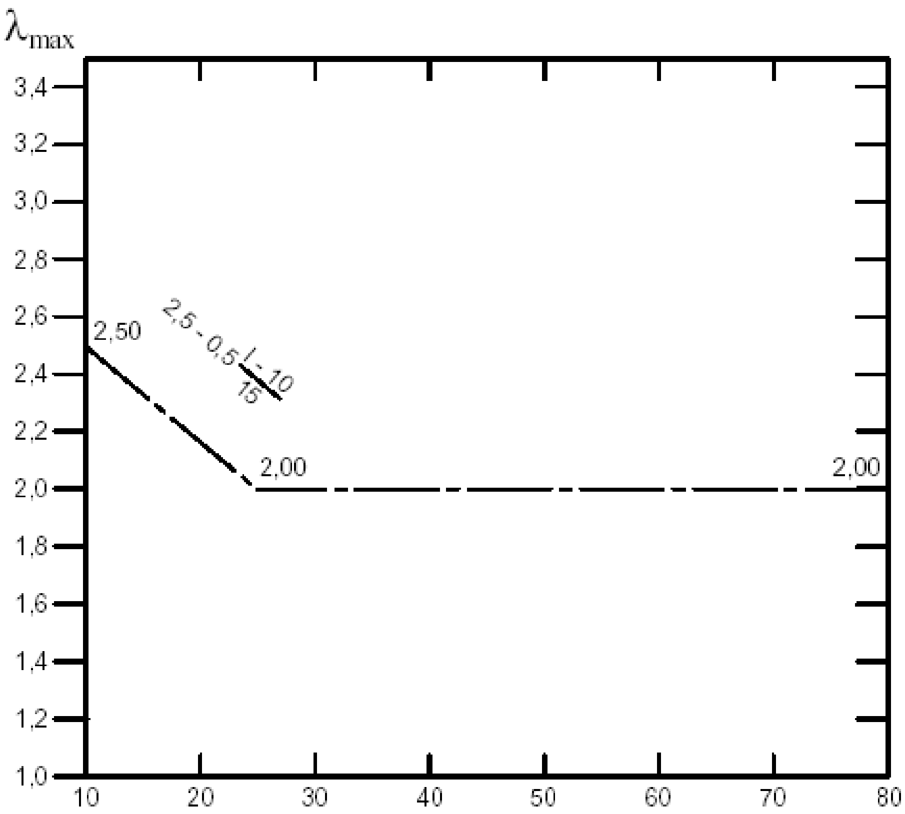
$$\lambda_4 = \left[1 + \frac{N_2}{N_1}\left(\frac{\eta_2 Q_{m2}}{\eta_1 Q_{m1}}\right)^5 + \frac{N_3}{N_1}\left(\frac{\eta_3 Q_{m3}}{\eta_1 Q_{m1}}\right)^5 + \ldots\ldots + \frac{N_k}{N_1}\left(\frac{\eta_k Q_{mk}}{\eta_1 Q_{m1}}\right)^5\right]^{1/5}$$

where k is the number of slow lanes on the bridge

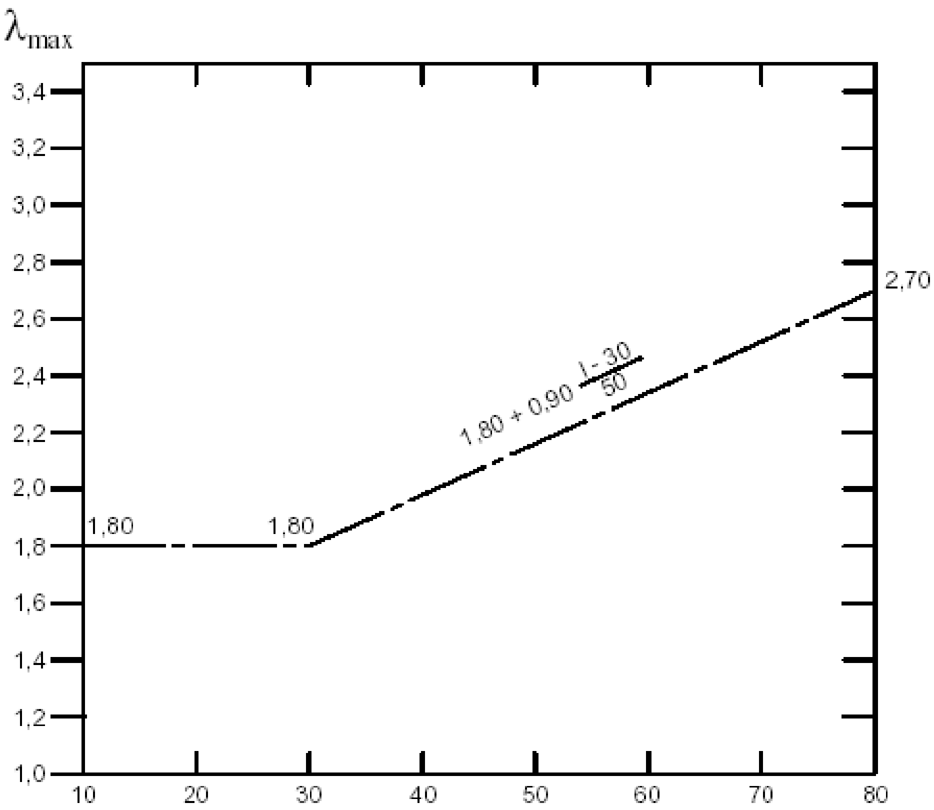
For the worked example :

- 2 slow lanes (one per direction)
- Same road traffic (N_1=N_2 and Q_{m1} = Q_{m2})
- 75% (=η_1) of the traffic loads in the slow lane 1 and 25% (= η_2) from the slow lane 2, are transmit to the modelled main girder
- λ_4 = 1.0

Damage equivalence factor λ



at midspan



at support

L = 60 or 80 m

λ_{max} = 2.00

L = (60+80)/2 = 70 m

λ_{max} = 2.52

Damage equivalence factor λ

initial	final	L (m)	λ_1	λ_2	λ_3	λ_4	λ_{max}	λ
x = 0 m	x = 51 m	60	2.05	0.848	1	1	2.0	1.738
x = 51 m	x = 72 m	(60+80)/2 = 70	2.1	0.848	1	1	2.52	1.780
x = 72 m	x = 100 m	80	1.85	0.848	1	1	2.0	1.568

- Fatigue detail = flange of a vertical T-section stiffener welded to the lower face of the upper main steel flange :
  $\Delta\sigma_C$ = 56 MPa (detail category)
  γ_{Mf} = 1.35 (crack in the upper main steel flange)

- Location of the detail : x = 100 m at mid-central span
  $\Delta\sigma_p$ = 25 MPa
  λ = 1.568
  Φ = 1.0 (damage impact factor)
  $\Delta\sigma_E = \lambda \, \Phi \, \Delta\sigma_p$ = 39.2 MPa

- Check : $\Delta\sigma_E < \Delta\sigma_C / \gamma_{Mf}$ = 41.5 MPa OK !

SOME INNOVATIONS / ECONOMY ISSUES

- Enormous scientific work
- Simplicity of calculations
- Robustness (fatigue + brittle fracture)
- Full exploitation of the materials (post-critical range)
- Steels up to S690
- Hybrid girders
- Harmonization of the format and the reliability of all the instability formulae
- Treatment of stiffened plates
- Design of orthotropic decks