Overview of Eurocode 4 part 2

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Contents

1. Introduction to composite bridges in Eurocode 4

EN 1994-1-1 : general rules and rules for <u>buildings</u>

EN 1994-1-2 : structural fire design

EN 1994-2 : general rules and rules for <u>bridges</u>

The general rules valid for bridges from part 1-1 are repeated in part 2 to get a self sufficient document

Scope of EN 1994-2

Composite bridges

- I girders
- Box sections
- Cable stayed bridges not fully covered

Composite members

Filler beam decks

Tension members

Composite plates

Composite bridges

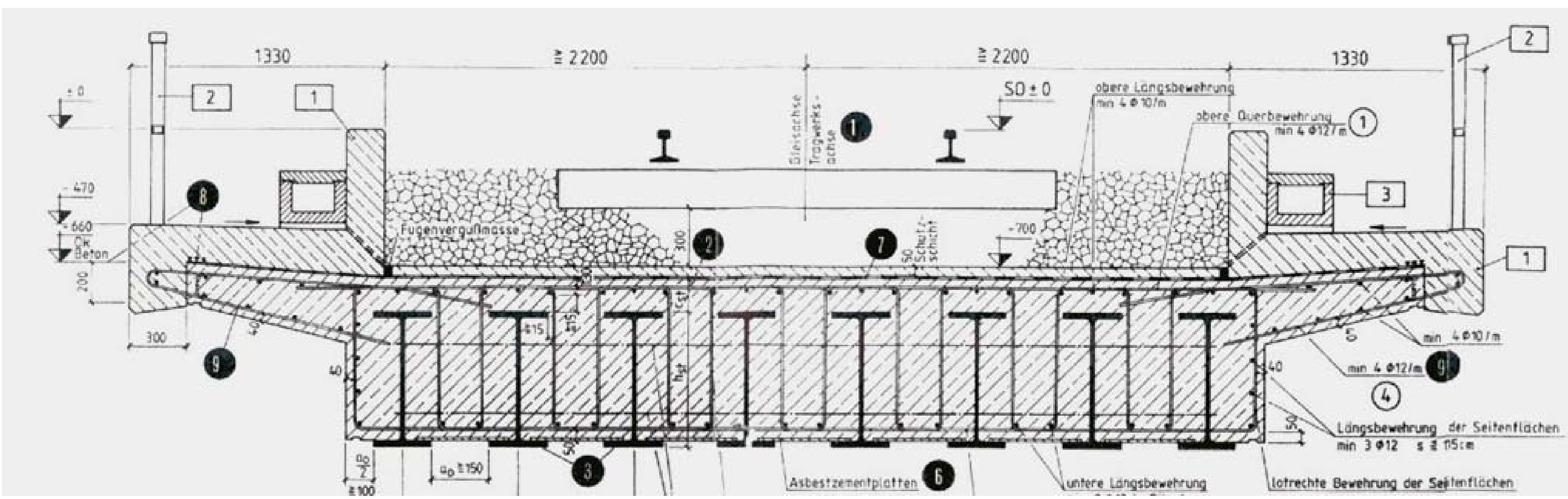
Composite members

Filler beam decks

transversal

(subject to National Annex)

longitudinal



Tension members

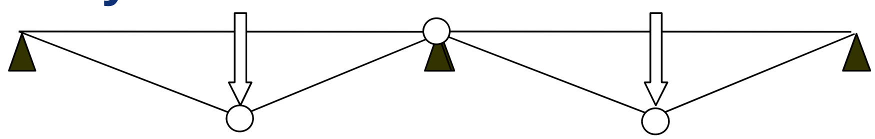
Composite plates

Contents

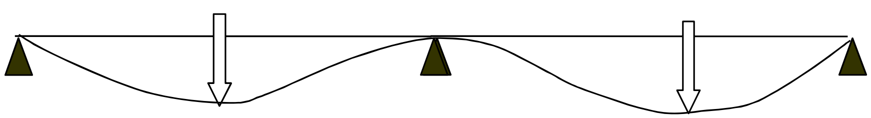
Classification of cross-sections

Not for bridges
Except acc.design situation

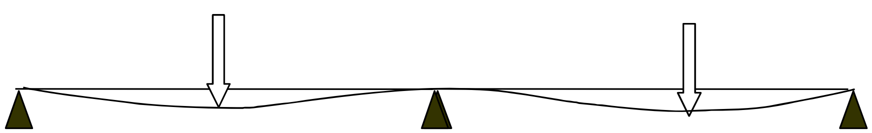
CLASS 1 sections which can form a plastic hinge with the rotation capacity required for a global plastic analysis

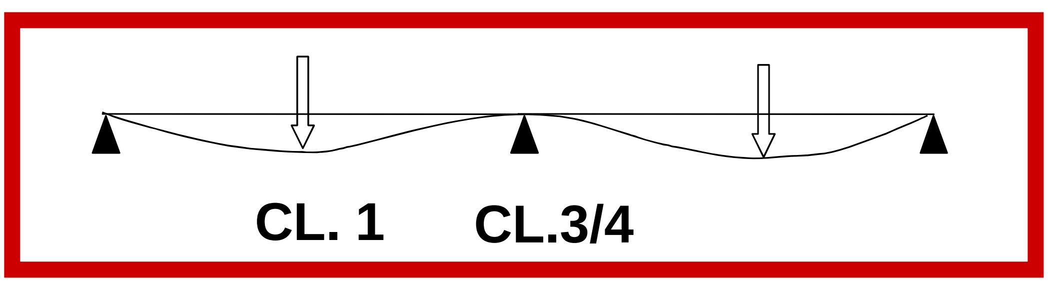


CLASS 2 sections which can develop $M_{pl,Rd}$ with limited rotation capacity



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

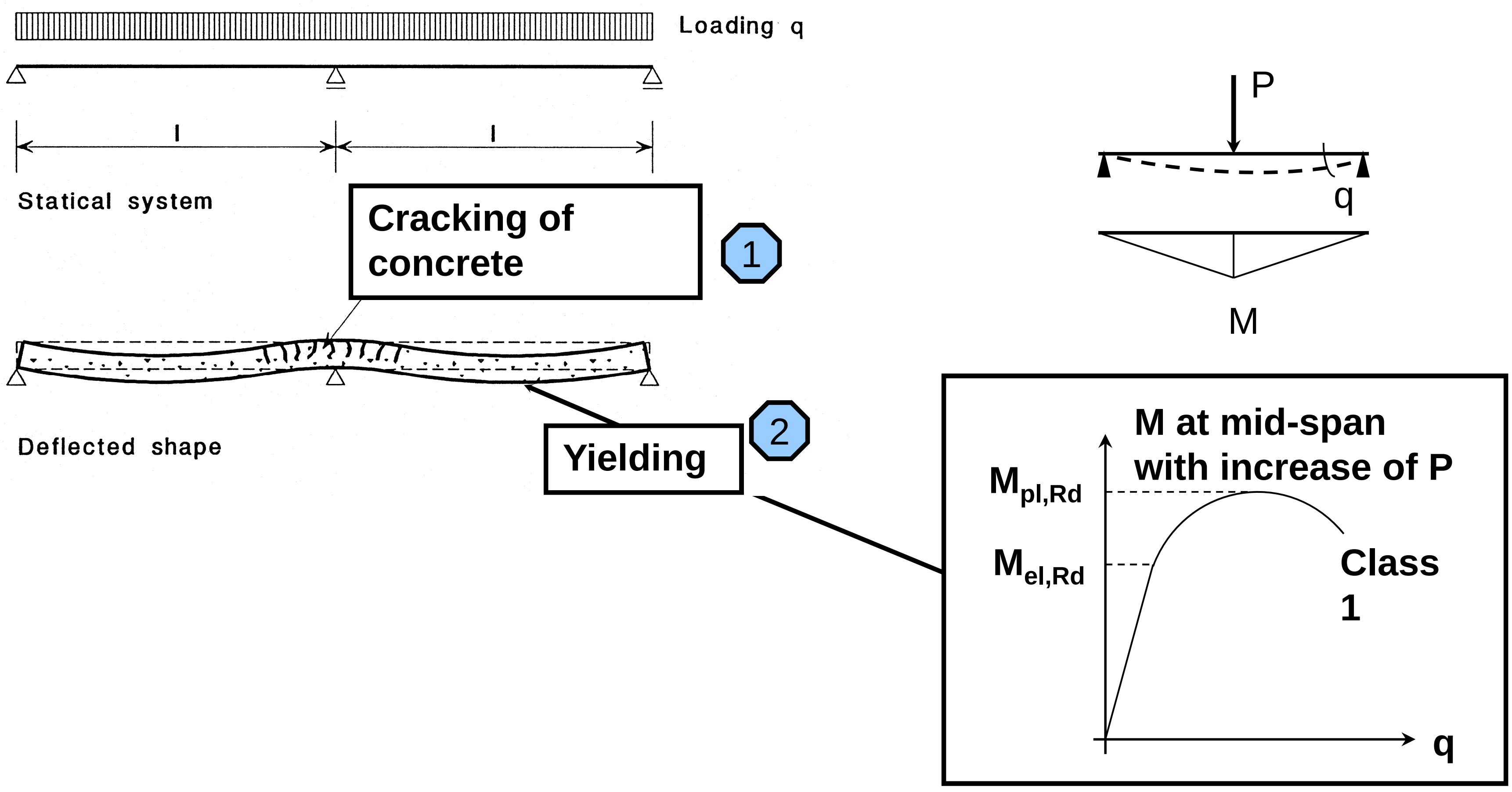




COMPOSITE BRIDGES
Non-uniform section
(except for small spans)

Actual behaviour

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

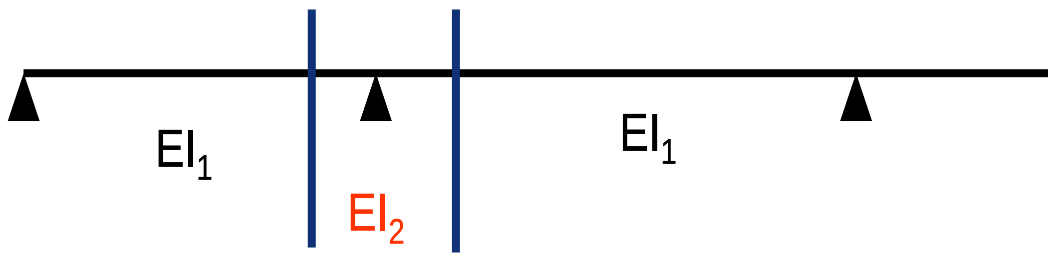


Cracked global analysis

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)

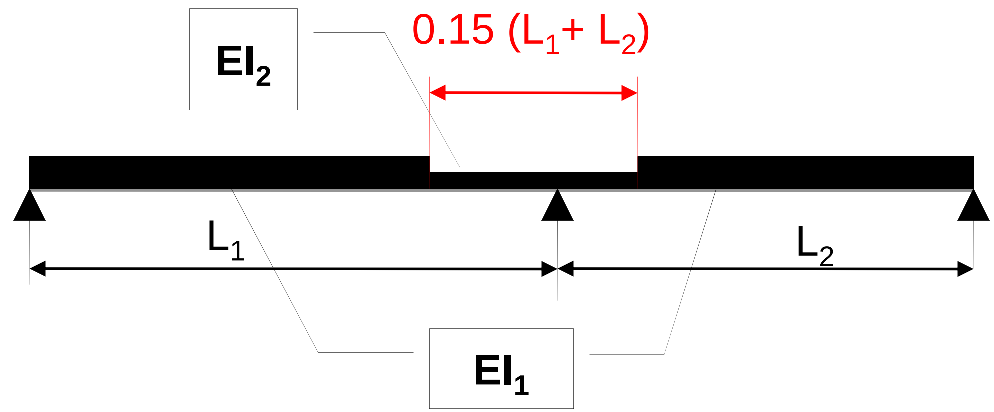
The cracked global analysis does not need an iterative calculation.

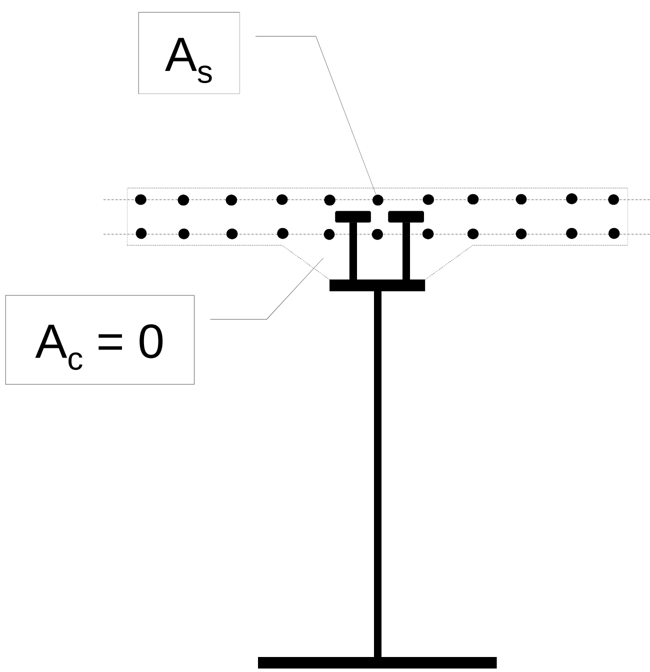
Cracked global analysis

1

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

- no pre-stressing by imposed deformation
- $L_{min}/L_{max}>0.6$





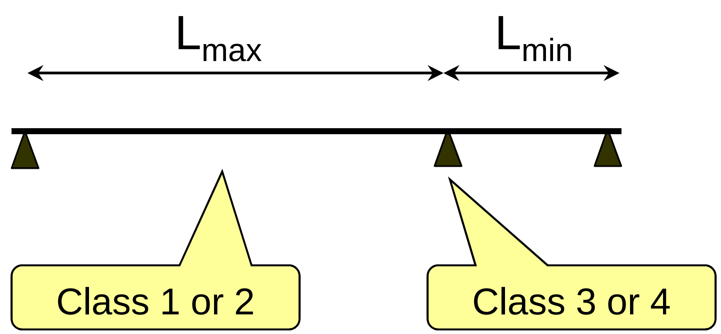
In the cracked zones EI_2 :

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

Yielding

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 (Finite Elements for instance)

Global analysis - Synthesis

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

Global analysis - Synthesis

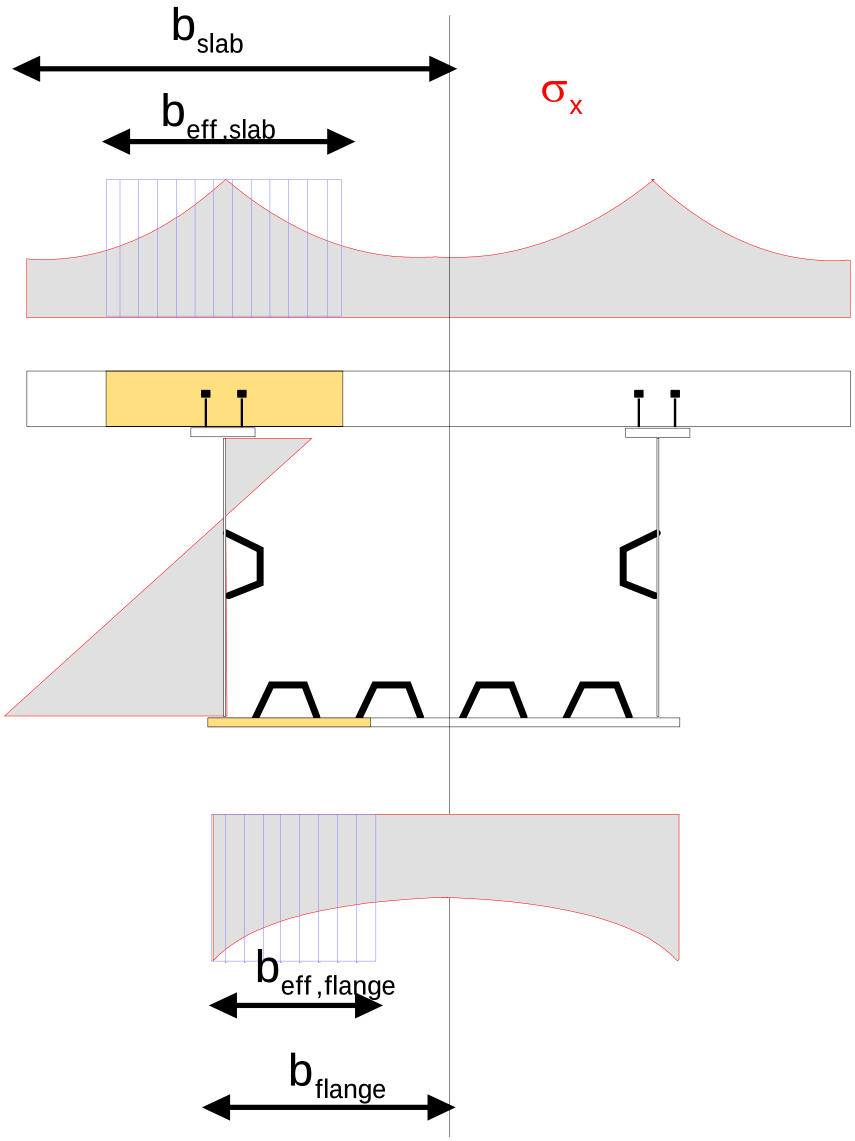
To calculate the internal forces and moments for the SLS combinations of actions

as for ULS

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

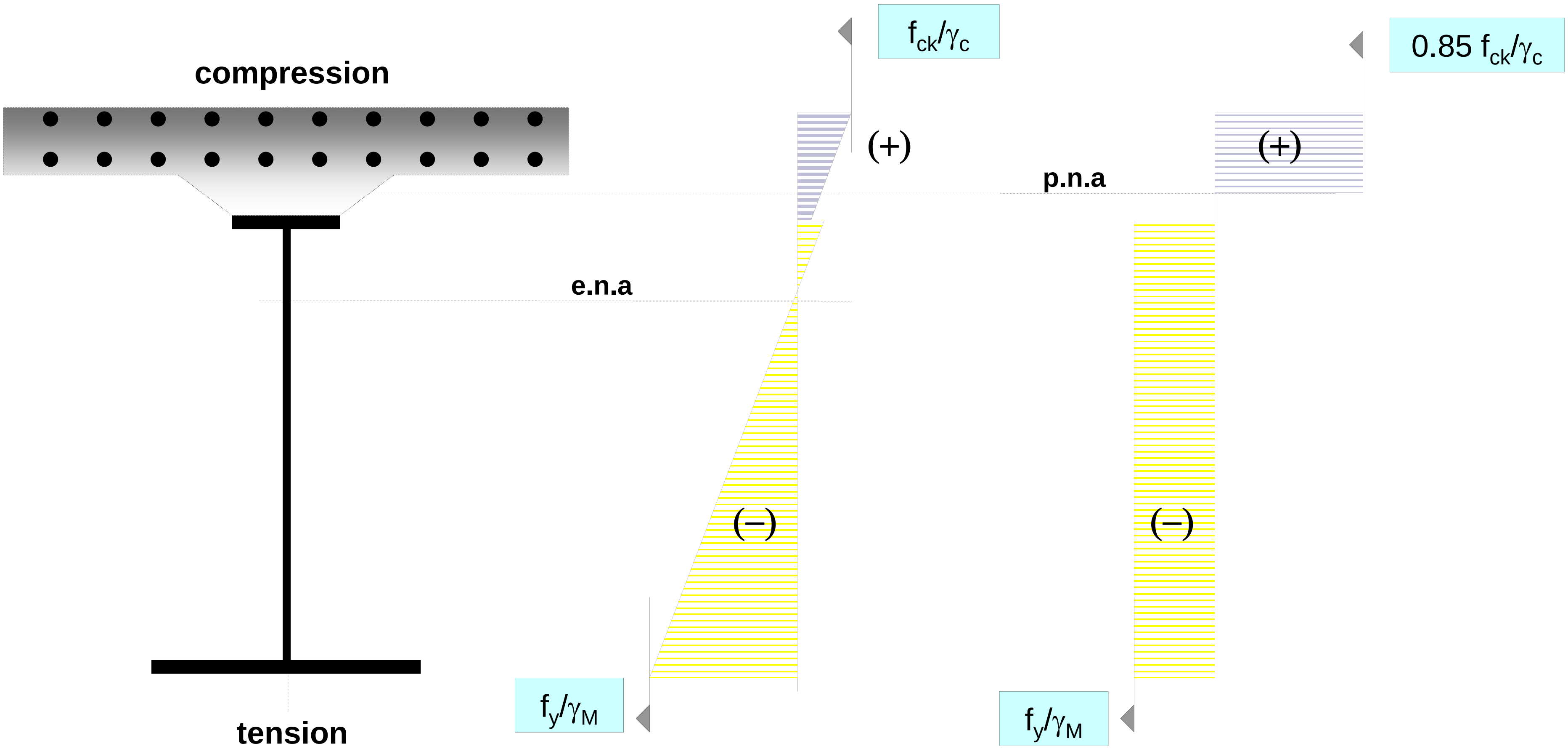
- Used for bottom flange of a box-girder bridge
- Different effective^s width for SLS and ULS combinations of actions
- 3 options at ULS (choice to be performed in the National Annex)

Contents

ULS verifications of a composite bridge

- **resistance of the composite cross-sections**
  - **- bending moment M**
  - **- shear force V**
  - **- interaction M+V**
- ***shear resistance in the concrete slab (EN 1994-2, 6.2.2.5(3))***
- *punching in the concrete slab (EN 1992)*
- **shear connection**
- **fatigue ULS**
- *LTB around intermediate supports*

ULS section resistance under M > 0



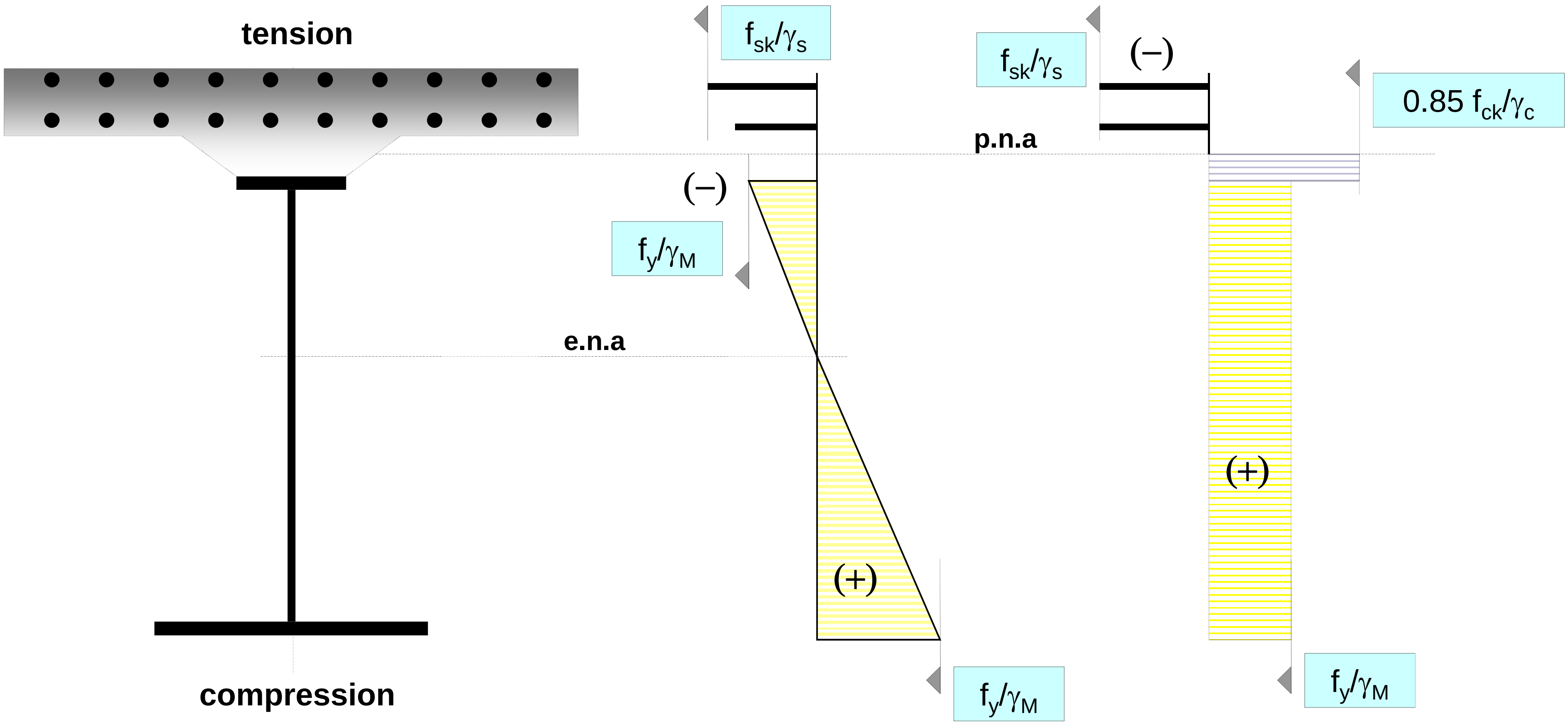
Elastic resistance (for classes 1 to 4)

Plastic resistance (for classes 1 and 2)

e.n.a. = elastic neutral axis

p.n.a. = plastic neutral axis

ULS section resistance under M < 0



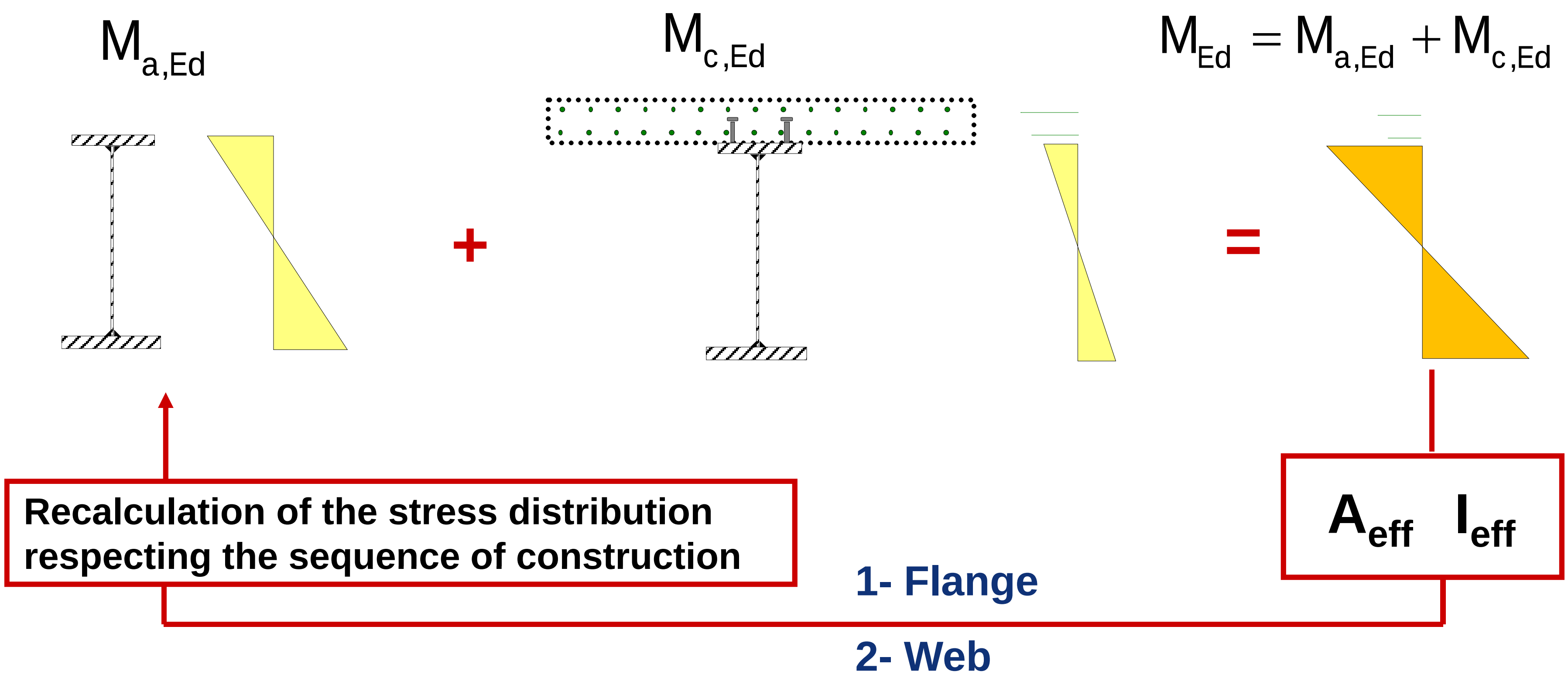
Elastic resistance (for classes 1 to 4)

Plastic resistance (for classes 1 and 2)

Class 4 composite cross-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.**



ULS section resistance under V and interaction M + V

⇨ **Plastic resistance** : steel web only

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

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

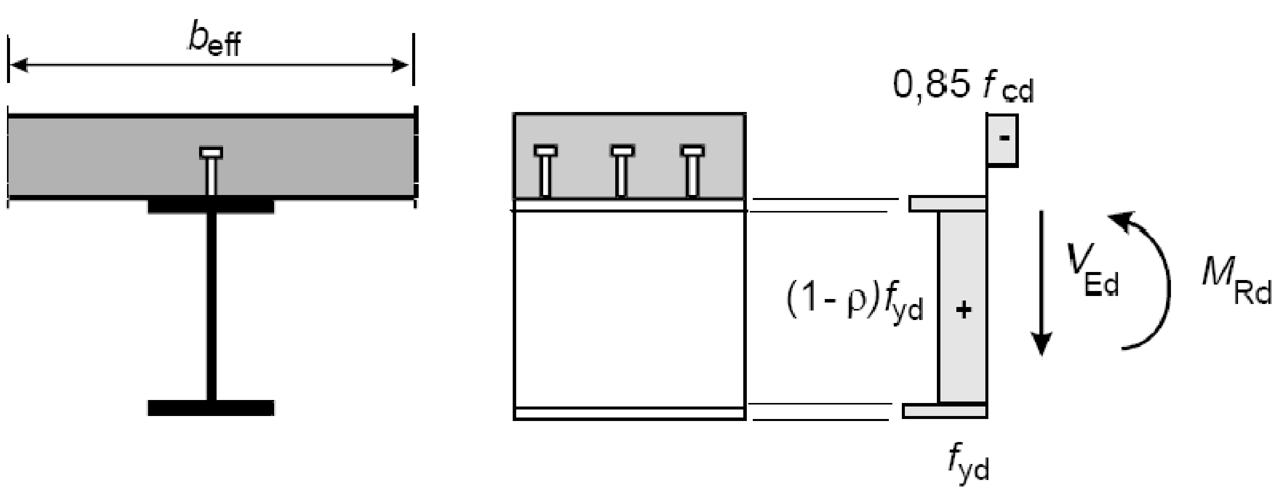
⇨ **Shear buckling resistance** :

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** :



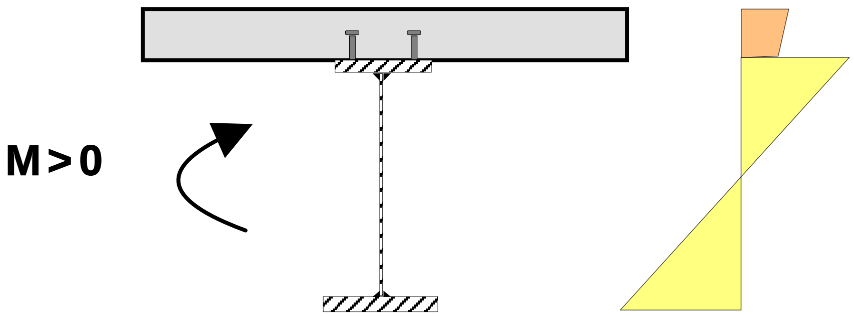
$$\rho = (2V_{Ed} / V_{Rd} - 1)^2$$

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

Contents

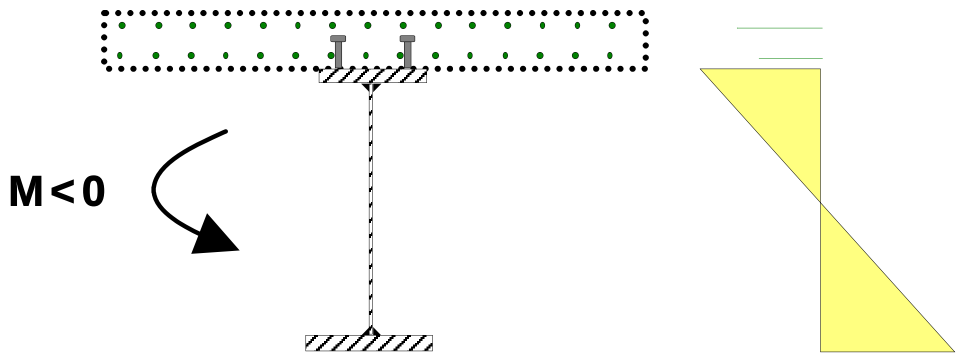
SLS verifications in a composite bridge

• Limitation of stresses in cross-sections at characteristic SLS



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

$$\sigma_a \leq \frac{f_y}{\gamma_{M,ser}} \qquad \gamma_{M,ser} = 1$$



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

$$\sigma_a \leq \frac{f_y}{\gamma_{M,ser}} \qquad \gamma_{M,ser} = 1$$

• Crack width control

• Limitations of deflections (if any)

• Web breathing (web slenderness limit check is in general enough)

Crack width control

1. **Minimum reinforcement required**

   - in cross-sections where tension exists in the concrete slab at characteristic SLS

   - 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 <u>maximum crack width w_{max}</u> by limiting :

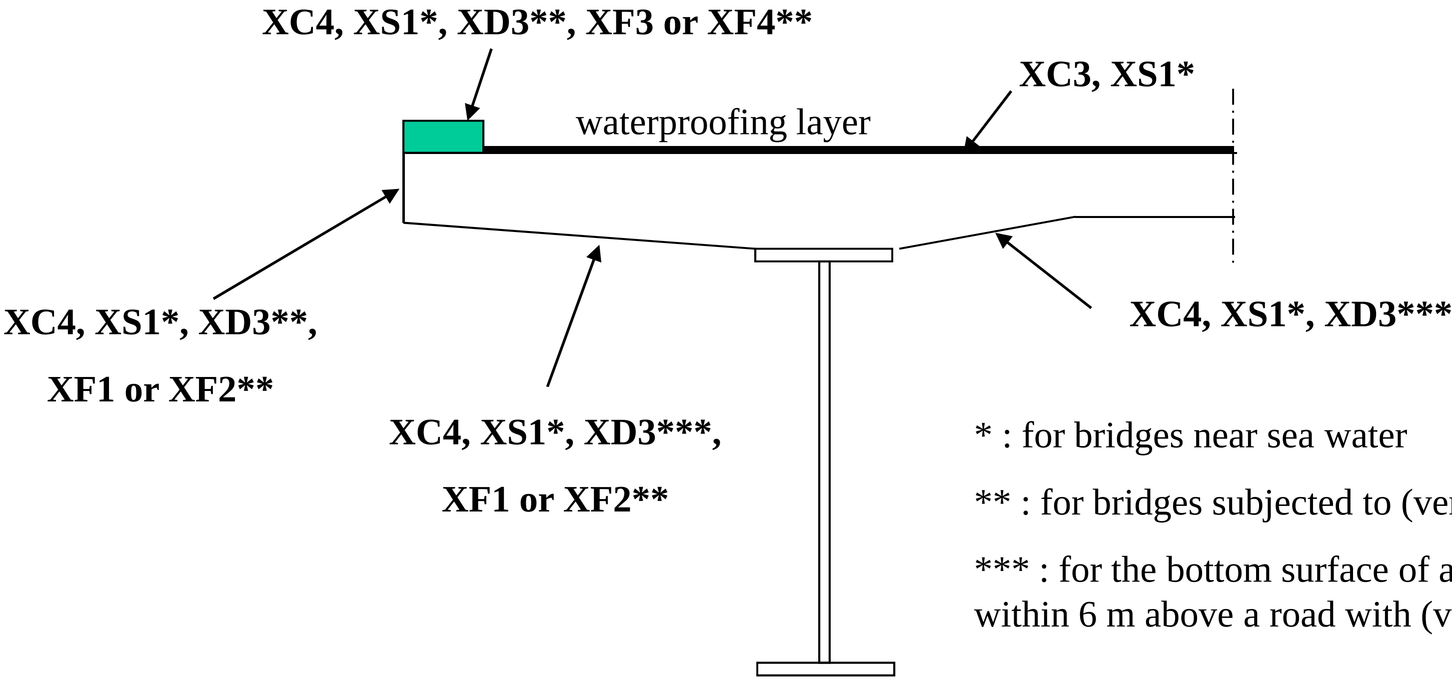
   - bar spacing $d \leq d_{max}$

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

   w_{max} depends on the <u>exposure class</u> of the considered concrete face

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

Exposure classes for composite bridges



* : for bridges near sea water

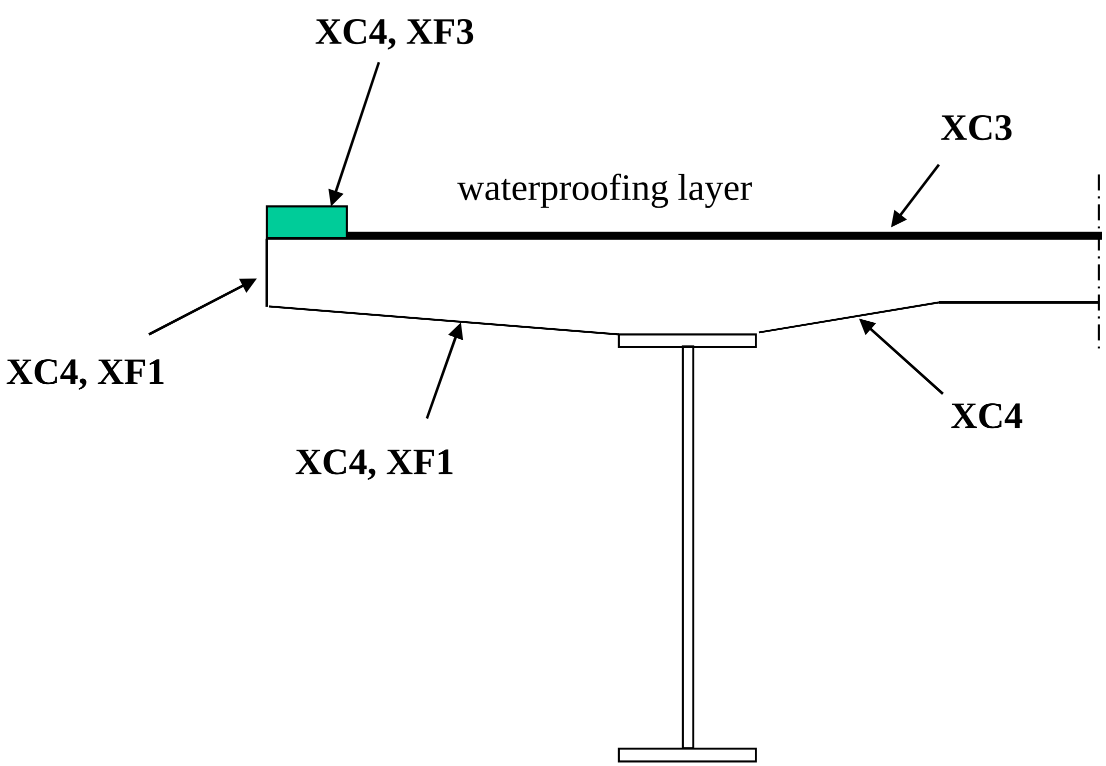
** : for bridges subjected to (very) frequent salting

*** : 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

Hypothesis : Bridge in a low-level frost area



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.

Crack width control

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	-

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

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	-

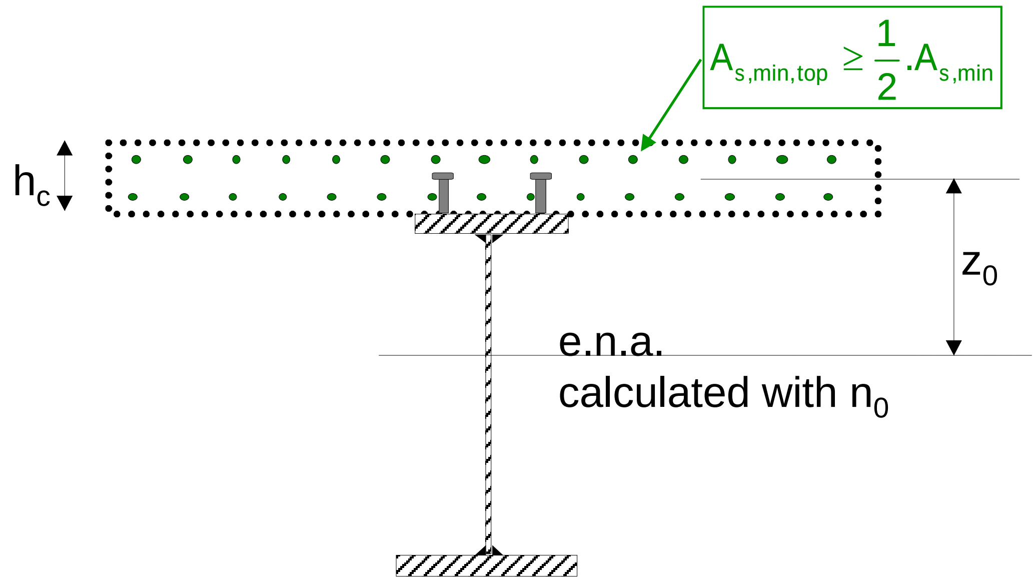
Minimum reinforcement

$$A_s \sigma_s \geq A_{s,min} \sigma_s = k_s k_c k . f_{ct,eff} A_c$$

k_c : stress distribution within the tensile concrete height h_c before cracking

+ change in the location of the neutral axis at cracking time

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



k_s = 0.9 : reduction of the normal force of 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

$f_{ct,eff} = f_{ctm}$

σ_s : maximum stress level allowed in the reinforcement after cracking

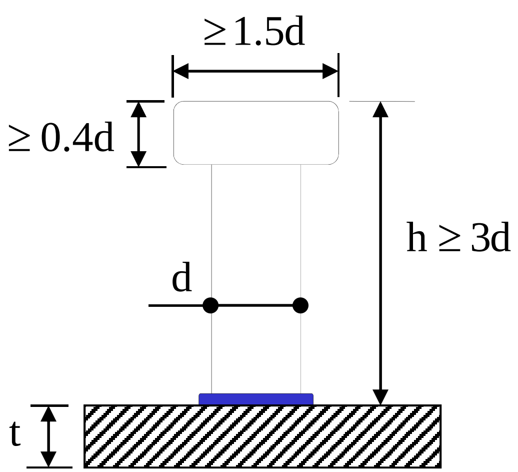
(=f_{sk} at yielding ; or a lower value if required by the control of crack width)

Contents

Steel-concrete connection

- **Full interaction required for bridges**
- **Elastic global analysis at SLS and at ULS**
- **Elasto-plastic global analysis 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**

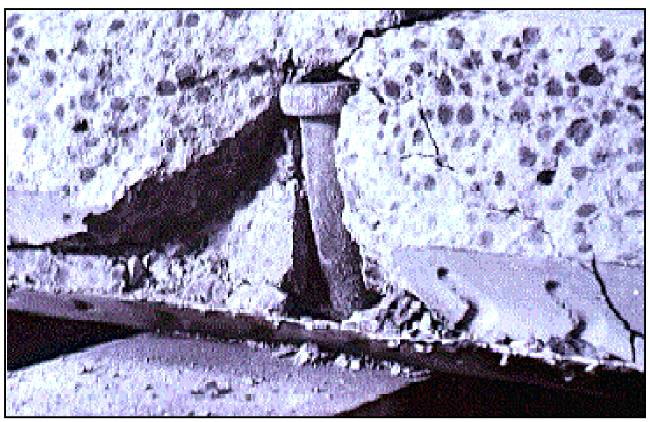
Resistance of the headed stud shear connector



$16 \leq d \leq 25mm$

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

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



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

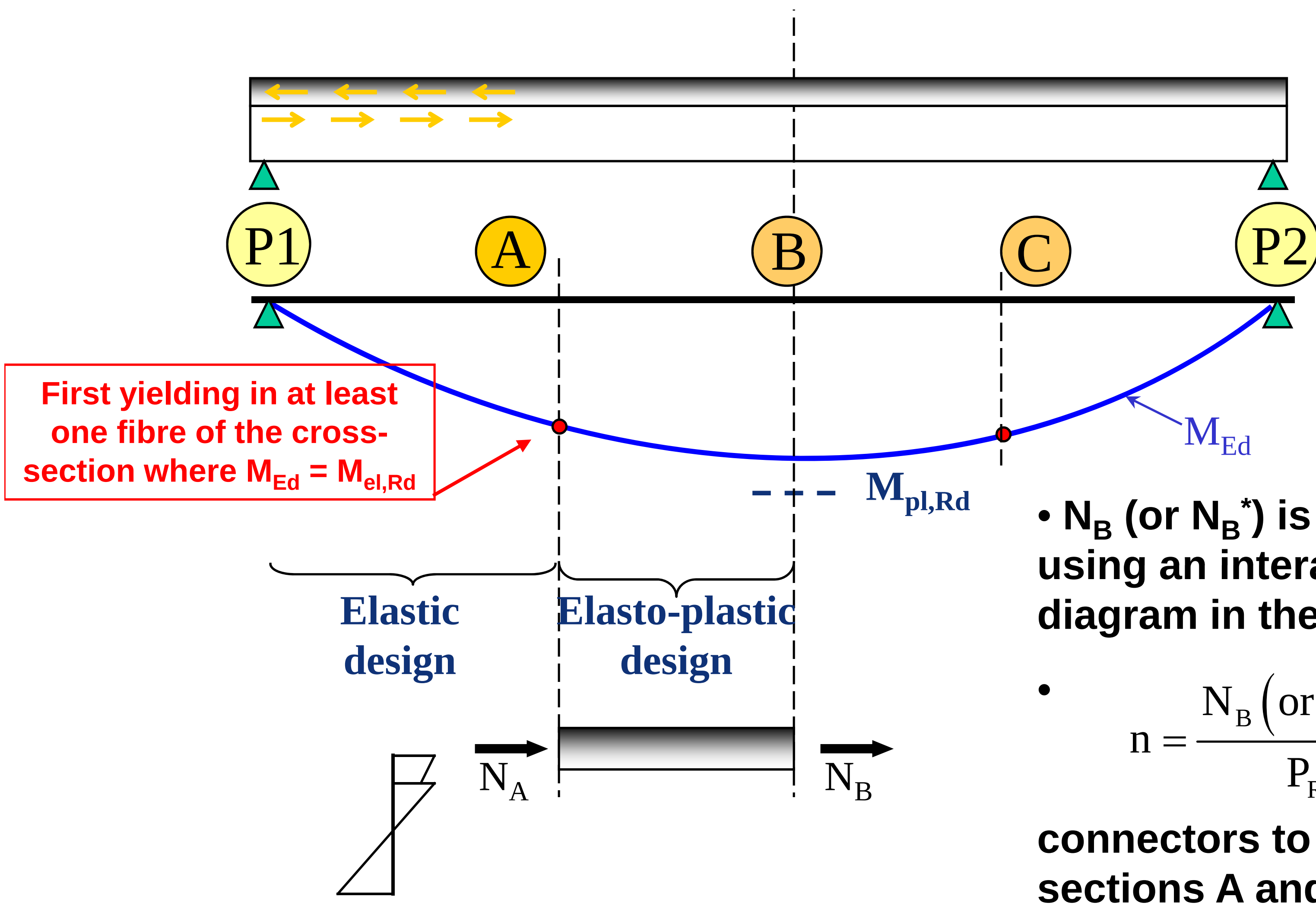
Elastic design of the shear connection

- **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.**

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 :**
$v_{L,Ed}^{SLS}(x) \leq \frac{N_i}{l_i}.\{k_s P_{Rd}\}$ $(0 \leq x \leq l_i)$	$v_{L,Ed}^{ULS}(x) \leq 1.1 \frac{N_i^*}{l_i}.P_{Rd}$ $\int_0^{l_i} v_{L,Ed}^{ULS}(x)\,dx \leq N_i^*.P_{Rd}$

Elasto-plastic design (ULS) of the shear connection

Shear connectors in the elasto-plastic zones where $M_{pl,Rd} > M_{Ed} > M_{el,Rd}$

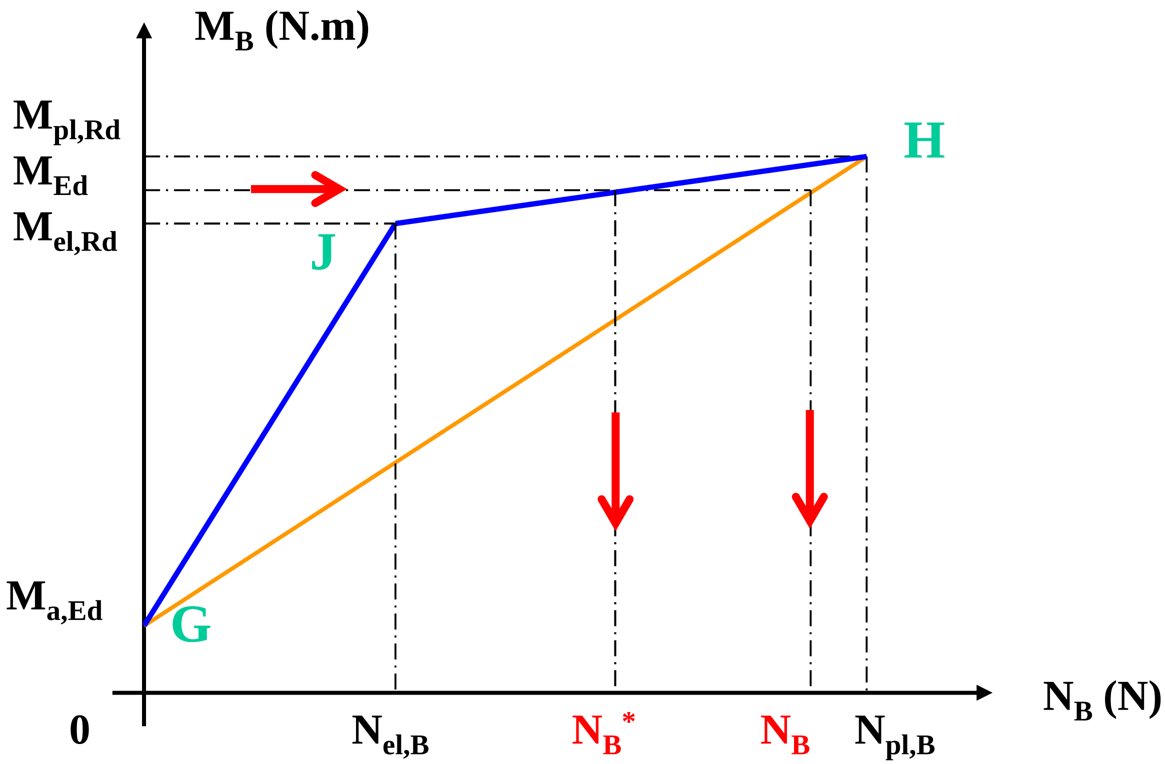


- N_B (or N_B^*) is determined by using an interaction M-N diagram in the section B.
- $$n = \frac{N_B \left(\text{or } N_B^*\right) - N_A}{P_{Rk} / \gamma_V}$$

connectors to put between sections A and B.

Interaction diagram in the cross-section B

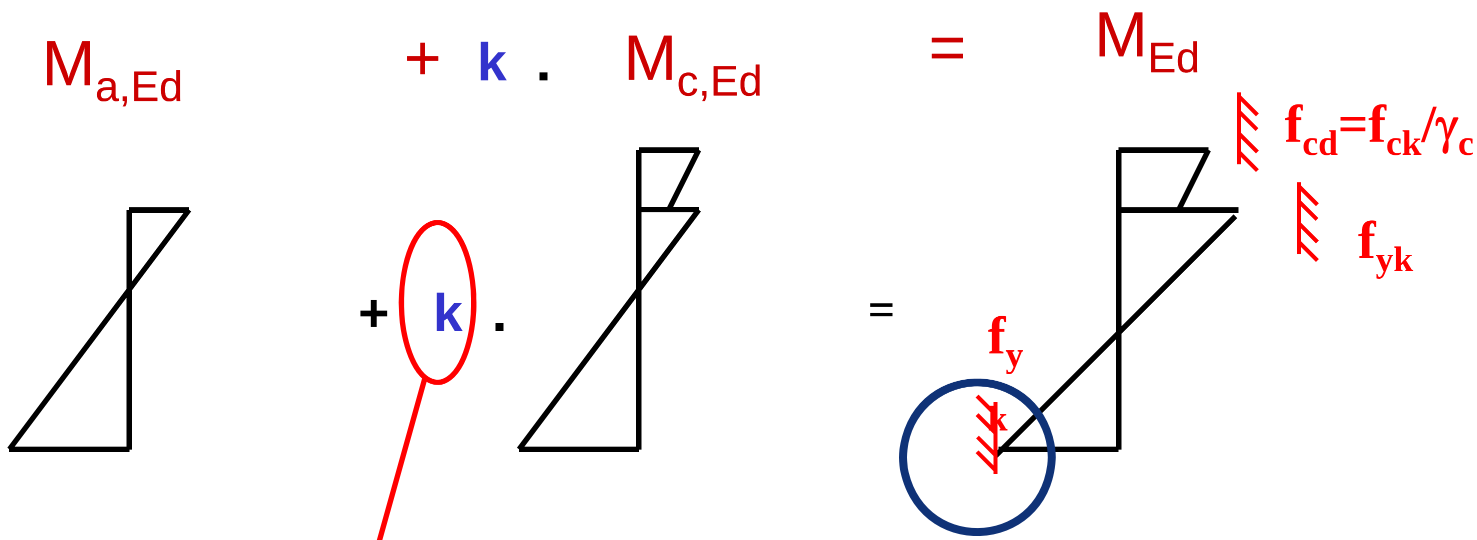
Two options : (straight line GH) / more precise diagram (broken line GJH)



• **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



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

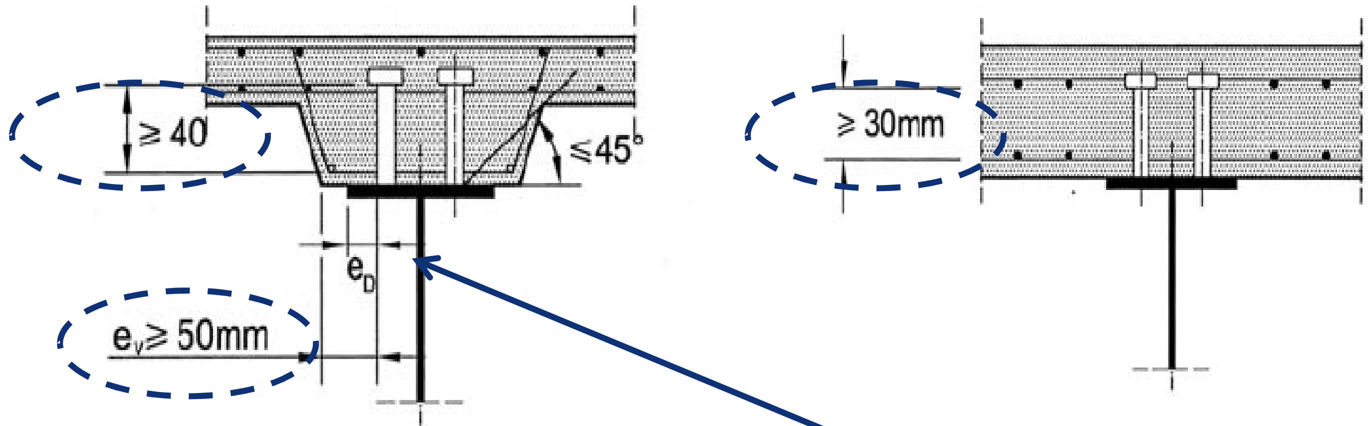
Step 2 : stress diagram for load cases applied to the structure after concreting Section B

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

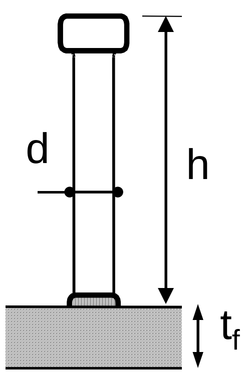
k (< 1) is the maximum value to get the strength limits in step 3. =>

$$M_{el,Rd} = M_{a,Ed} + k.\ M_{c,Ed}$$

Detailing for shear **connectors**



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



- $d \leq 2.5.t_f$
- $d \leq 1.5.t_f$ **for a structural steel flange in tension and subjected to fatigue.**

Detailing for shear **connectors**

Longitudinal spacing between shear connectors :

- **to insure the composite behaviour in all cross-sections :**
  e_{max} = min (800 mm; 4 h) **where h is the concrete slab thickness**
- **to insure Class 1 or 2 for A class 3/4 flange in compression which is connected to the concrete slab :**

$$e_{max} \leq 22 t_f \sqrt{\frac{235}{f_y}}$$

Longitudinal spacing between shear connectors : $5.d \leq e_{max}$

Transversal spacing between shear connectors :

$$e_{trans,max} \geq 2.5.d$$

- **to insure Class 1 or 2 for A class 3/4 flange in compression which is connected to the concrete slab :**

$$e_D \leq 9 t_f \sqrt{\frac{235}{f_y}}$$

Contents

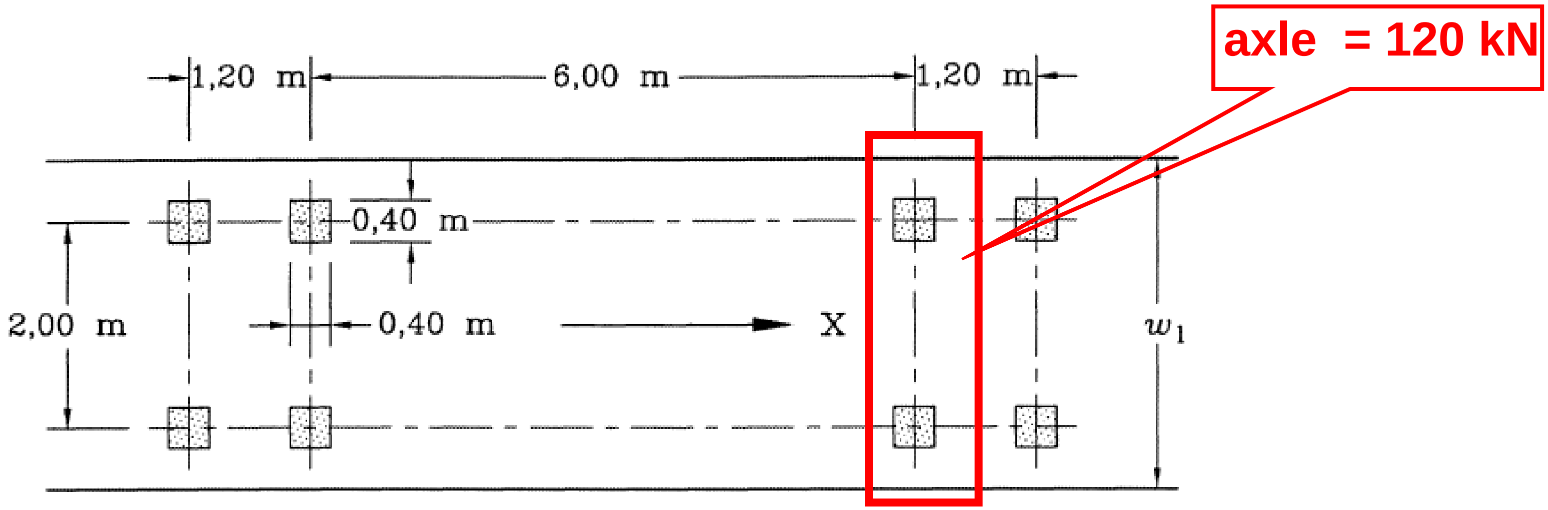
Fatigue ULS in a composite bridge

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)

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

Fatigue Load Model 3 « equivalent lorry » (FLM3)



- 2.10^6 FLM3 lorries are assumed to cross the bridge per year and per slow lane 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 :

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

□**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):

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

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} (or G_{min}) $+ 1.0$ (or 0.0)$S + 0.6 T_k$		FLM 3
In every section : M_{max} (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 :

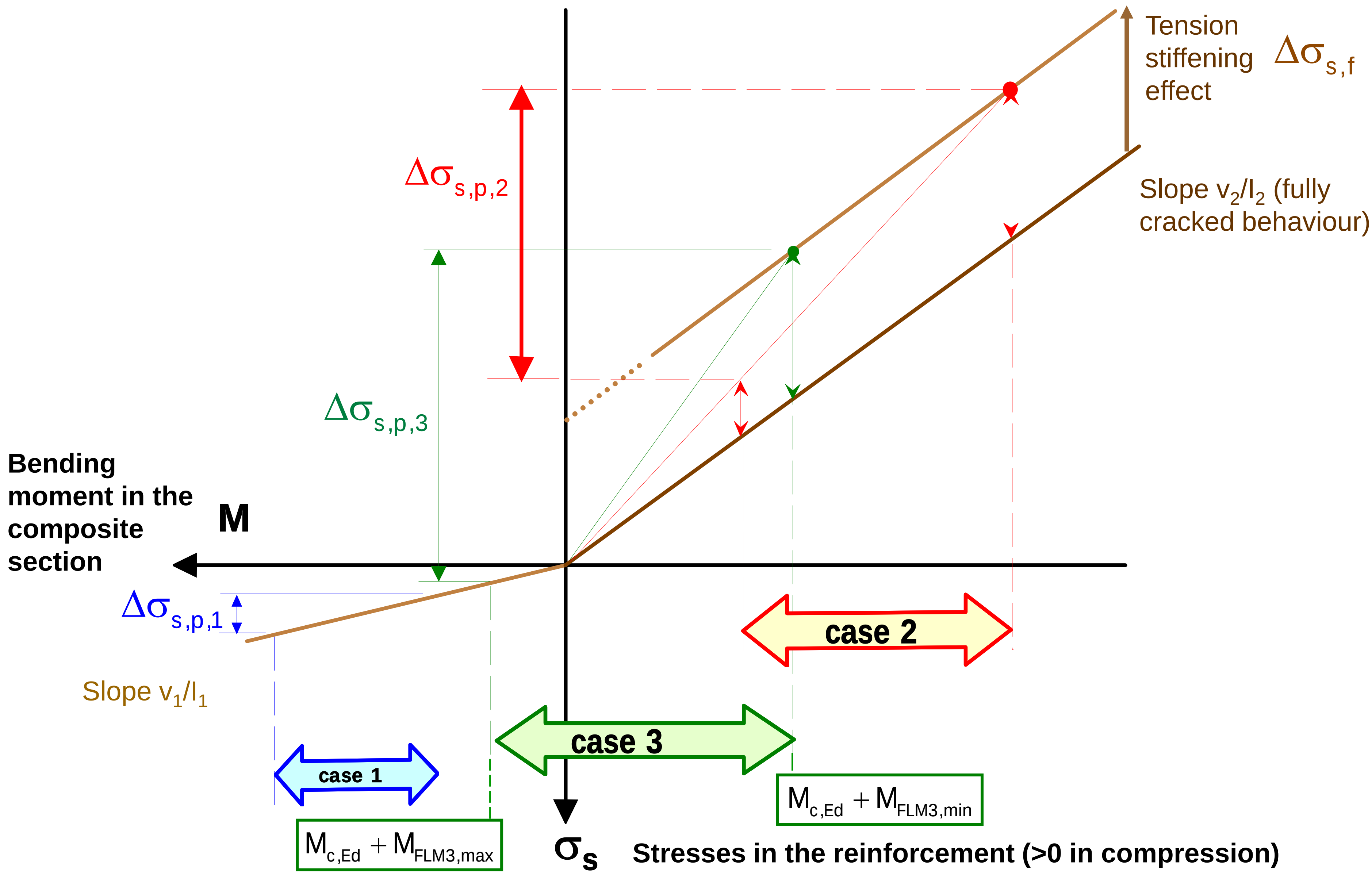
$$M_{Ed,max,f} = M_{a,Ed} + M_{c,Ed} + M_{FLM3,max} \qquad M_{Ed,min,f} = M_{a,Ed} + M_{c,Ed} + M_{FLM3,min}$$

- 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	$\sigma_{c,Ed,max,f} > 0$ $\sigma_{c,Ed,min,f} < 0$	$\Delta\sigma_p = M_{c,Ed}\left(\frac{v_1}{I_1} - \frac{v_2}{I_2}\right) + M_{FLM3,max}\frac{v_1}{I_1} + M_{FLM3,min}\frac{v_2}{I_2}$

Tension stiffening effect



Fatigue verifications

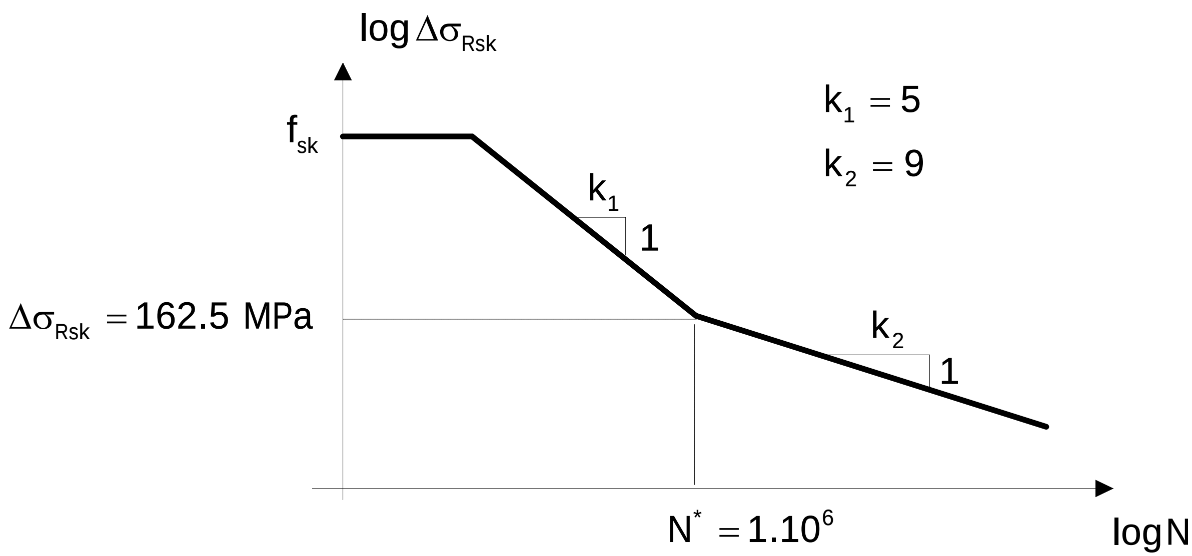
- **In a structural steel detail :**

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

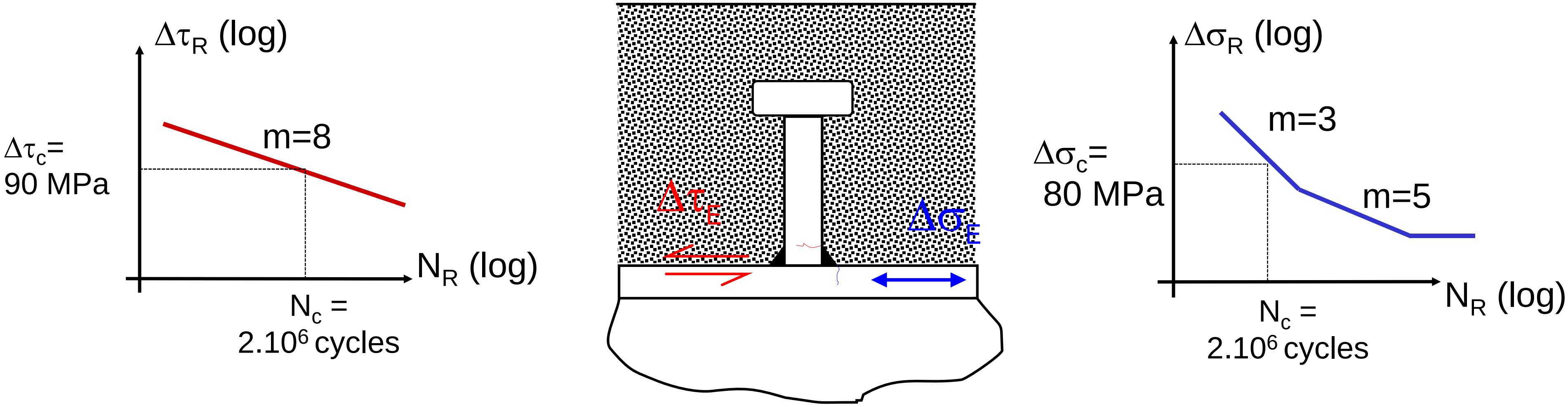
$$\left(\frac{\gamma_{Ff}\Delta\sigma_E}{\Delta\sigma_C/\gamma_{Mf}}\right)^3 + \left(\frac{\gamma_{Ff}\Delta\tau_E}{\Delta\tau_C/\gamma_{Mf}}\right)^5 \leq 1.0$$

- **In the reinforcement :**

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



Fatigue verifications



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 :

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

Composite bridge design (EN1994-2)- illustration of basic element design

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Convenor EN-1994-2

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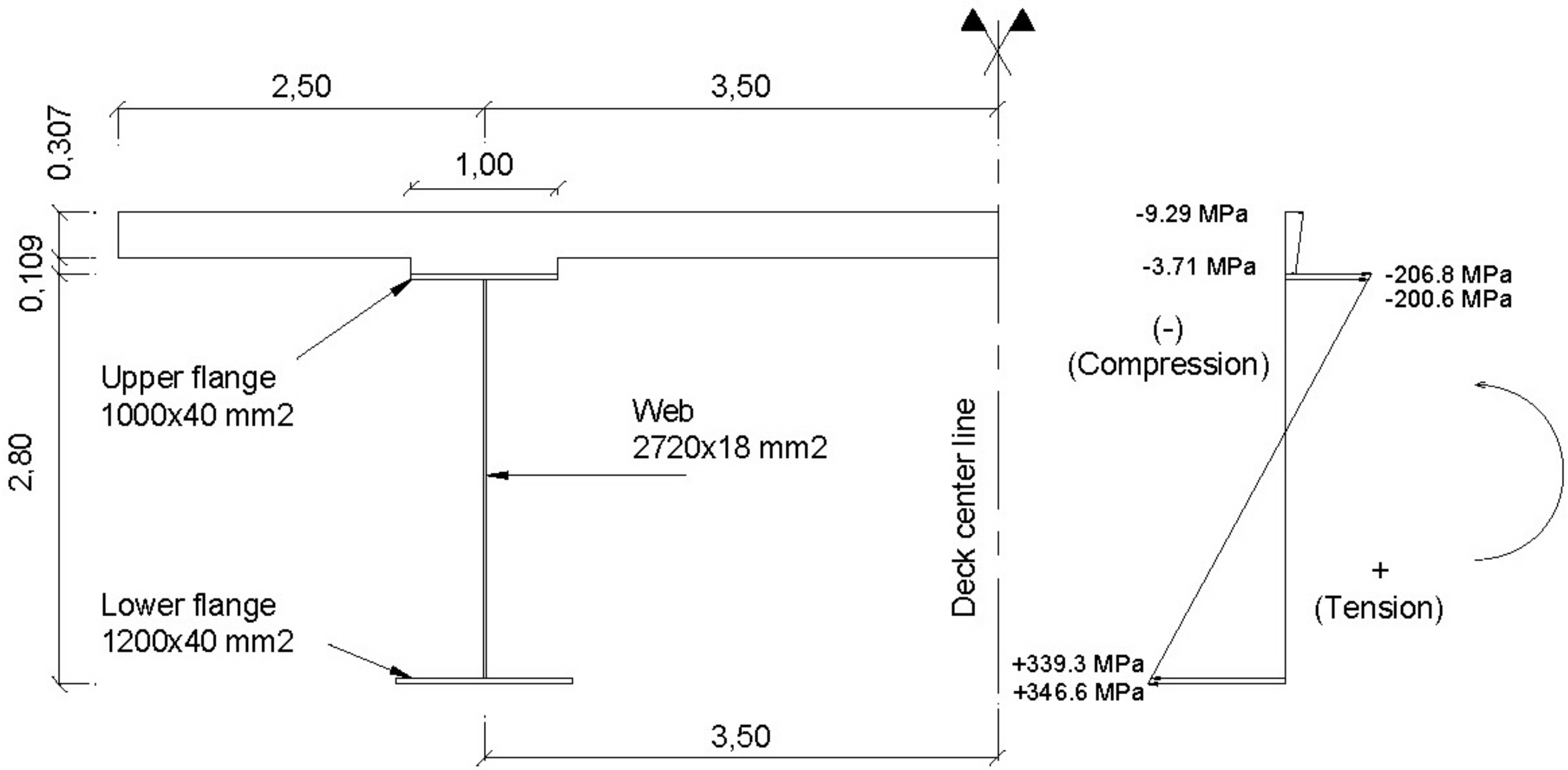
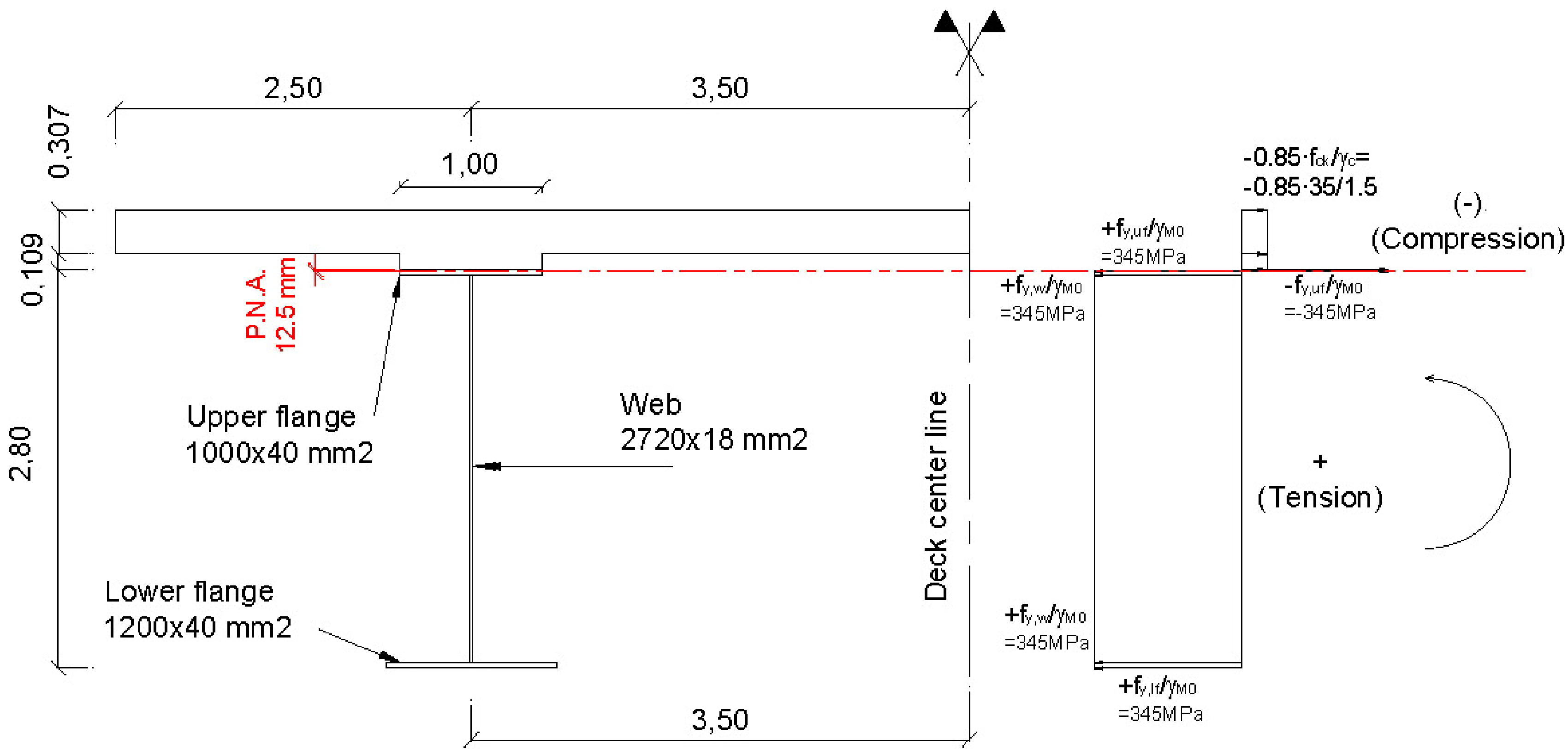
1. Verification of cross-section at mid-span P1-P2
1.1. Geometry and Stresses



- Concrete slab compressed.
- Stresses are calculated with the composite mechanical properties and obtained by summing the various steps whilst respecting the construction phases. (Sign criteria for the example + tension and – compression)
- Internal forces and moments: $\boldsymbol{M_{Ed}}$ = 63.89 MN·m ; $\boldsymbol{V_{Ed}}$ = 1.25 MN

Plastic analysis:

- Lower flange tensioned: Class 1
- The upper flange is composite and connected (following the recommendations of EN1994-2, 6.6): Class 1.
- To classify the steel web, we need to determine the position of the Plastic Neutral Axis (PNA)



1. Verification of cross-section at mid-span P1-P2
1.2. Determining the cross-section Class (EN1994-2, 5.5.2) II

- **Design plastic resistance of the concrete in compression**:
  (force of ½ slab)

$$F_c = A_c \frac{0.85 f_{ck}}{\gamma_c} = 1.9484 . \frac{0.85 \cdot 35}{1.50} = 38.643 \text{ MN}$$

The design compressive strength of concrete is $f_{cd} = \frac{f_{ck}}{\gamma_c}$ (EN-1994-2, 2.4.1.2).

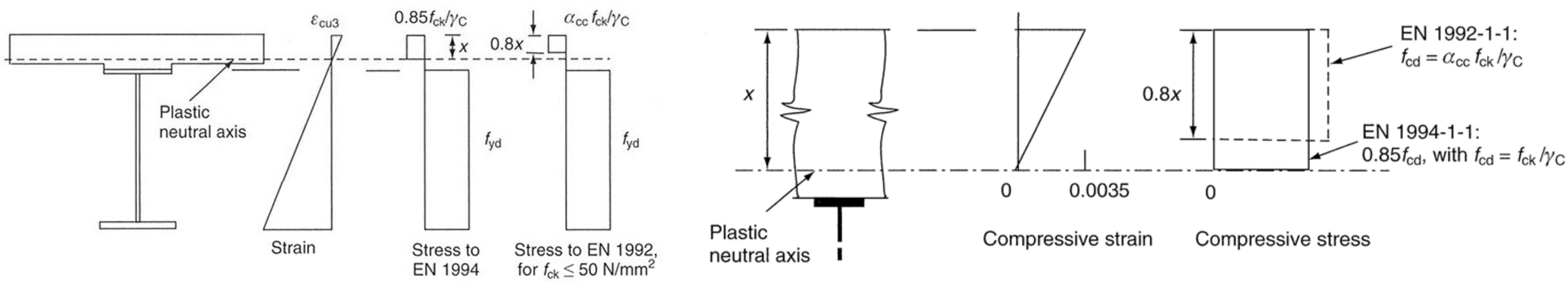
EN-1994 differs from EN-1992-1-1, 3.1.6 (1), in which an additional coefficient α_{cc} is applied: $f_{cd} = \frac{\alpha_{cc} \cdot f_{ck}}{\gamma_c}$

α_{cc} takes into account of the long term effects on the compressive strength and of unfavourable effects resulting from the way the loads are applied.

EN-1994-2 used the value α_{cc}=1.00, without permitting national choice for several reasons

1. Verification of cross-section at mid-span P1-P2
1.2. Determining the cross-section Class (EN1994-2, 5.5.2) III

- The plastic stress block for use in resistance of composite sections, (EN-1994, 6.2.1.2 fig. 6.2) consist of a stress ***0.85 f_{cd}*** extending to the neutral axis.



- Predictions using the stress block of EN-1994 have been verified against the results for composite members conducted independently from the verifications for concrete bridges.

- The EN-1994 block is easier to apply. The Eurocode 2 rule for rectangular block (EN-1992-1-1, 3.1.7 (3)) was not used in Eurocode 4. Because resistance formulae became complex where the neutral axis is close to or within the steel flange adjacent to concrete slab.

- Resistance formulae for composite elements given in EN-1994 are based on calibrations using stress block, with α_{cc}=1.00.

1. Verification of cross-section at mid-span P1-P2
1.2. Determining the cross-section Class (EN1994-2, 5.5.2) IV

- The reinforcing steel bars in compression are neglected.
- Design plastic resistance of the structural steel upper flange (1 flange):

$$F_{s,uf} = A_{s,uf}\frac{f_{y,uf}}{\gamma_{M0}} = (1.0 \cdot 0.04)\cdot\frac{345}{1.0} = 13.80\text{ MN}$$

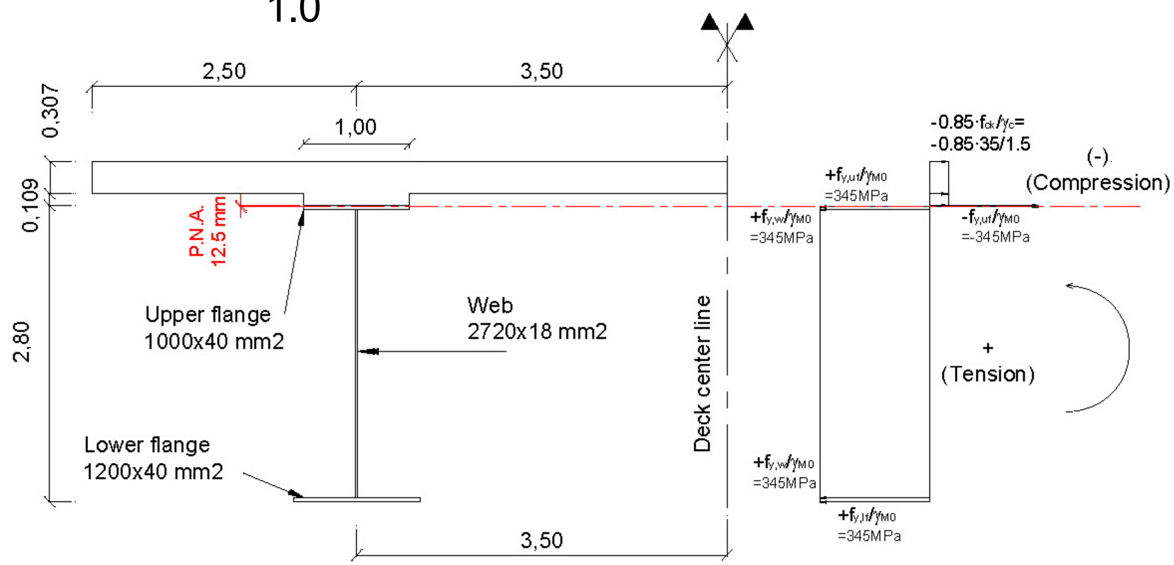
; 16< ***t***≤ 40 mm $\boldsymbol{f_y}$=345 MPa.

- Design plastic resistance of the structural steel web (1 web):

$$F_{s,w} = A_{s,w}\frac{f_{y,w}}{\gamma_{M0}} = (2.72 \cdot 0.018)\cdot\frac{345}{1.0} = 16.891\text{ MN}$$

- Design plastic resistance of the structural steel lower flange (1 flange):

$$F_{s,lf} = A_{s,lf}\frac{f_{y,lf}}{\gamma_{M0}} = (1.20 \cdot 0.04)\cdot\frac{345}{1.0} = 16.56\text{ MN}$$



1. Verification of cross-section at mid-span P1-P2
1.2. Determining the cross-section Class (EN1994-2, 5.5.2) V

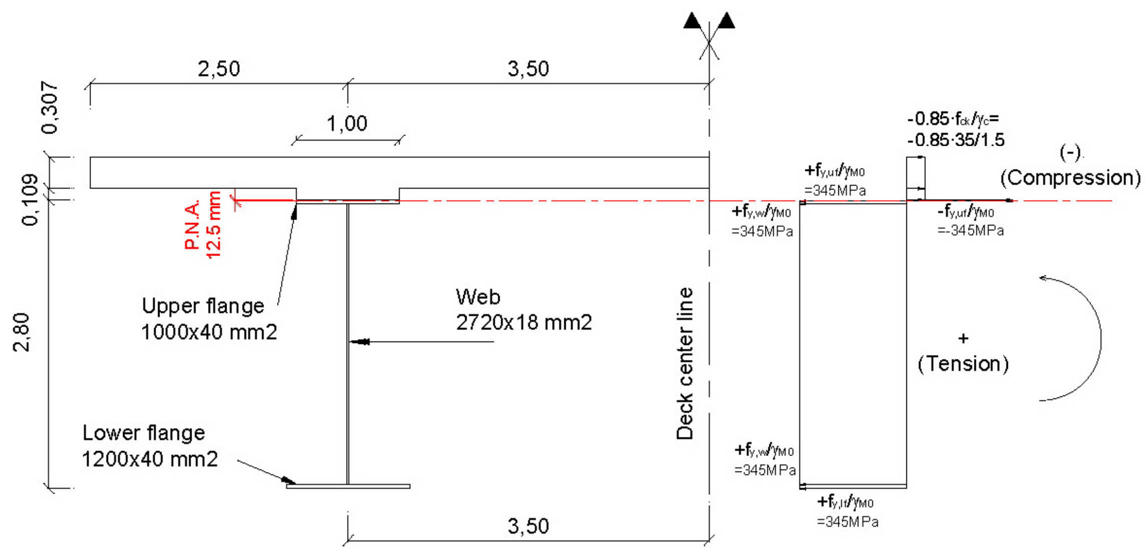
- As $|F_c| \leq |F_{s,uf}| + |F_{s,w}| + |F_{s,lf}|$ and $|F_c| + |F_{s,uf}| \geq |F_{s,w}| + |F_{s,lf}|$ it is concluded that the PNA is located in the steel upper flange at a distance x from the extreme upper fibre

- The internal axial forces equilibrium of the cross section leads to the location of the PNA:

$$-F_c - F_{s,uf} \cdot x + F_{s,uf} \cdot \frac{(0.04 - x)}{0.04} + F_{s,w} + F_{s,lf} = 0$$; $\boldsymbol{x}$=0.0125m

- As the PNA is located in the upper steel flange the whole web and the bottom flange are tensioned and therefore in Class 1

Conclusion: The cross-section at mid-span P1-P2 is in Class 1



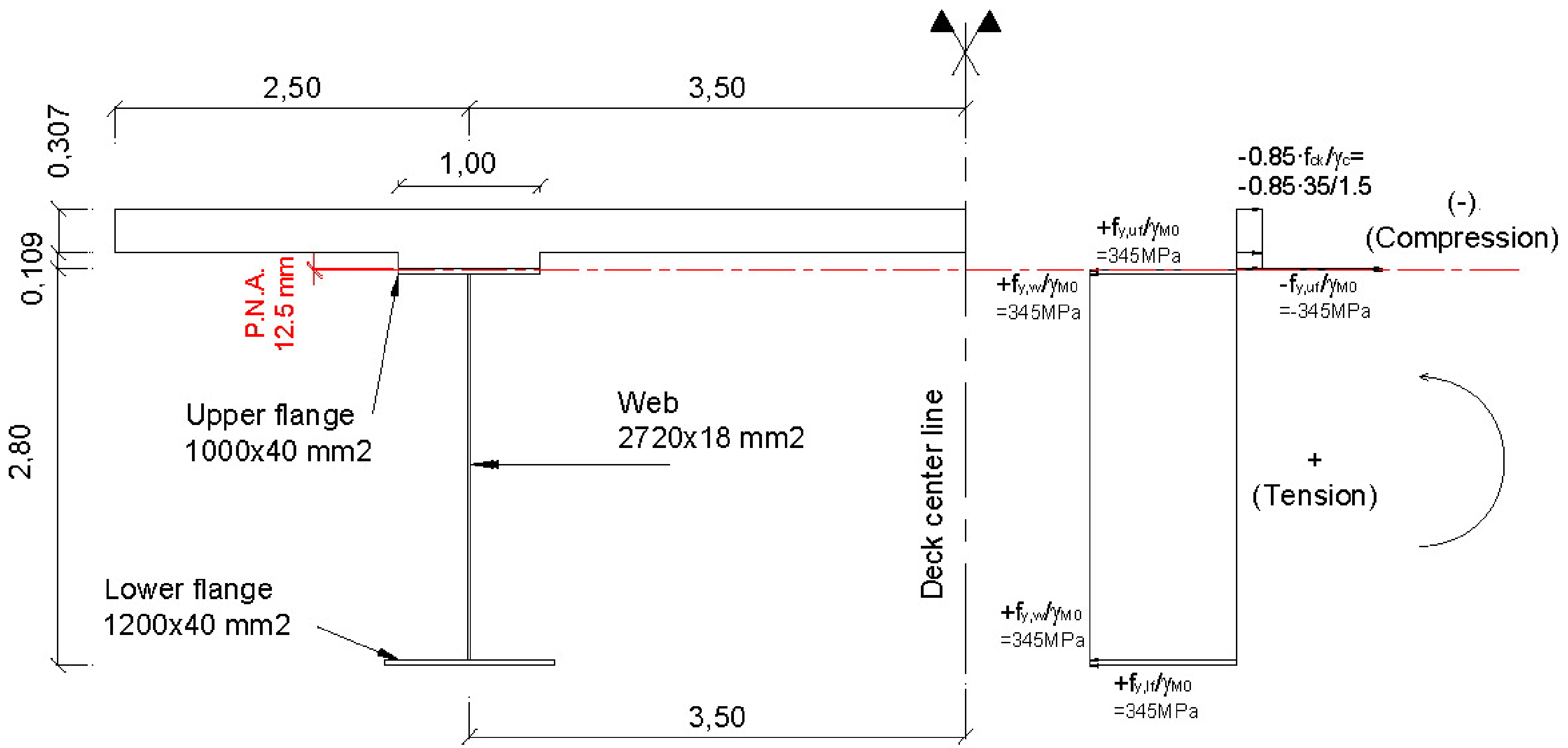
1. Verification of cross-section at mid-span P1-P2
1.3. Plastic section analysis.

1.3.1. Bending resistance check

The design plastic resistance moment is calculated from the position of the PNA

$M_{pl,Rd}$= +79.59 MN·m

As M_{Ed} = 63.89 MN·m ≤ $M_{pl,Rd}$ =79.59 MN·m is then verified.



1. Verification of cross-section at mid-span P1-P2
1.3. Plastic section analysis.

1.3.2. Shear resistance check. Interaction M-V I

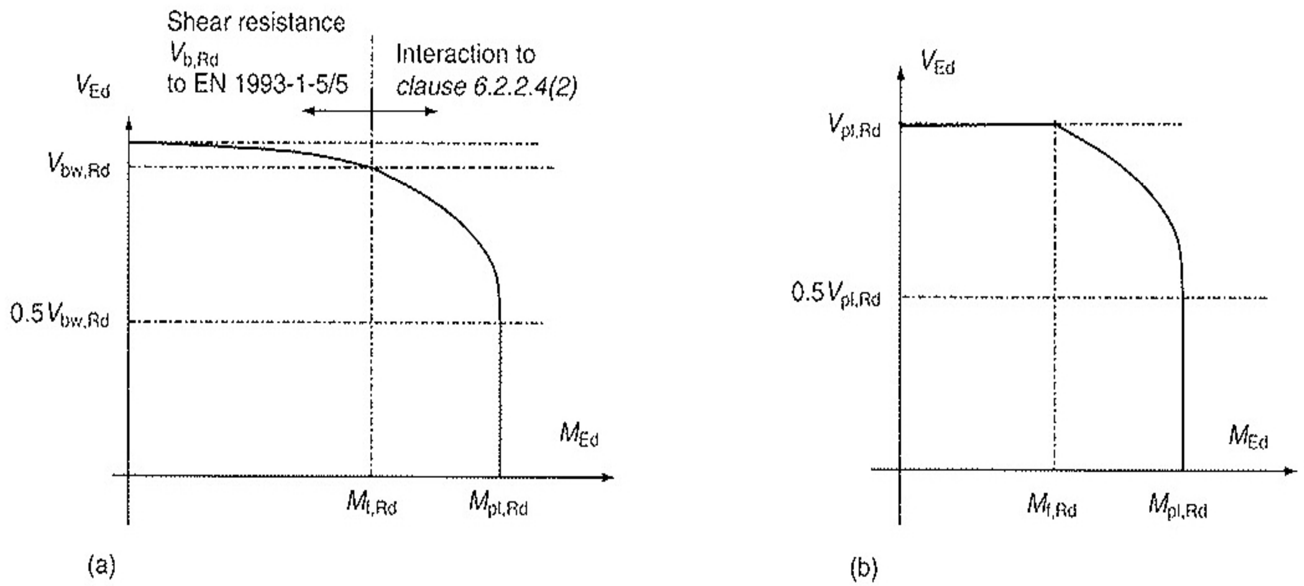
As $\frac{h_w}{t_w} = \frac{2.72}{0.018} = 151.11 \geq \frac{31\varepsilon}{\eta}\sqrt{k_\tau} = 51.36$ the web (stiffened by the vertical stiffeners) should be checked in terms of shear buckling, according to EN-1993-1-5, 5.1

The maximum design shear resistance V_{Rd} is given by

$V_{Rd} = min(V_{bw,Rd}\ ;\ V_{pl,a,Rd})$

Where:

- $V_{bw,Rd}$ is the shear bucking resistance according to EN-1993-1-5, 5
- $V_{pl,a,Rd}$ is the resistance to vertical shear according to EN-1993-1-1, 6.2.6



1. Verification of cross-section at mid-span P1-P2
1.3. Plastic section analysis.

1.3.2. Shear resistance check. Interaction M-V II

$$V_{pl,a,Rd} = \frac{\eta \cdot f_{y,w} \cdot h_w \cdot t}{\sqrt{3} \cdot \gamma_{M0}} = \frac{1.2 \cdot 345 \cdot 2720 \cdot 18}{\sqrt{3} \cdot 1.0} \cdot 10^{-6} = 11.70\ MN$$ (EN 1993-1-1, 6.2.6)

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta \cdot f_{y,w} \cdot h_w \cdot t}{\sqrt{3} \cdot \gamma_{M1}} = \frac{1.2 \cdot 345 \cdot 2720 \cdot 18}{\sqrt{3} \cdot 1.10} \cdot 10^{-6} = 10.63\ MN$$ (EN 1993-1-5, 5.2)

Given the distribution of the transverse bracing frames in the span P1-P2 (spacing ***a***=8 m), a vertical frame post is located in the mid span cross section.

The shear buckling check is therefore performed in the adjacent web panel with the highest shear force.

The maximum shear force registered in this panel is $\boldsymbol{V_{Ed}}$ = 2.21 MN.

The vertical frame posts are assumed to be rigid. This yields:

$$k_\tau = 5.34 + 4 \cdot \left(\frac{h_w}{a}\right)^2 = 5.34 + 4 \cdot \left(\frac{2.72}{8}\right)^2 = 5.802$$

$\boldsymbol{k_\tau}$ is the shear buckling coefficient (EN-1993-1-5 Annex A.3)

1. Verification of cross-section at mid-span P1-P2
1.3. Plastic section analysis.

1.3.2. Shear resistance check. Interaction M-V III

$$\sigma_E = \frac{\pi^2 \cdot E \cdot t_w{}^2}{12 \cdot (1-\nu^2) \cdot h_w{}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 18^2}{12 \cdot (1-0.3^2) \cdot 2720^2} = 8.312\ MPa$$ (EN-1993-1-5 Annex A.1)

$$\tau_{cr} = k_\tau \cdot \sigma_E = 5.802 \cdot 8.312 = 48.22\ MPa$$ (EN 1993-1-5, 5.3)

$$\bar{\lambda}_w = \sqrt{\frac{f_{y,w}}{\tau_{cr} \cdot \sqrt{3}}} = 0.76 \cdot \sqrt{\frac{f_{y,w}}{\tau_{cr}}} = 0.76 \cdot \sqrt{\frac{345}{48.22}} = 2.032$$ slenderness of the panel (EN-1993-1-5, 5.3.)

As $\bar{\lambda}_w \geq 1.08$,then the factor for the contribution of the web to the shear buckling resistance is:

$$\chi_w = \frac{1.37}{(0.7 + \bar{\lambda}_w)} = \frac{1.37}{(0.7 + 2.032)} = 0.501$$ (Table 5.1. of EN-1993-1-5, 5.3)

Finally the contribution of the web to the shear buckling resistance is:

$$V_{bw,Rd} = \frac{\chi_w \cdot f_{y,w} \cdot h_w \cdot t}{\sqrt{3} \cdot \gamma_{M1}} = \frac{0.501 \cdot 345 \cdot 2720 \cdot 18}{\sqrt{3} \cdot 1.10} \cdot 10^{-6} = 4.44\ MN$$

1.3.2. Shear resistance check. Interaction M-V IV

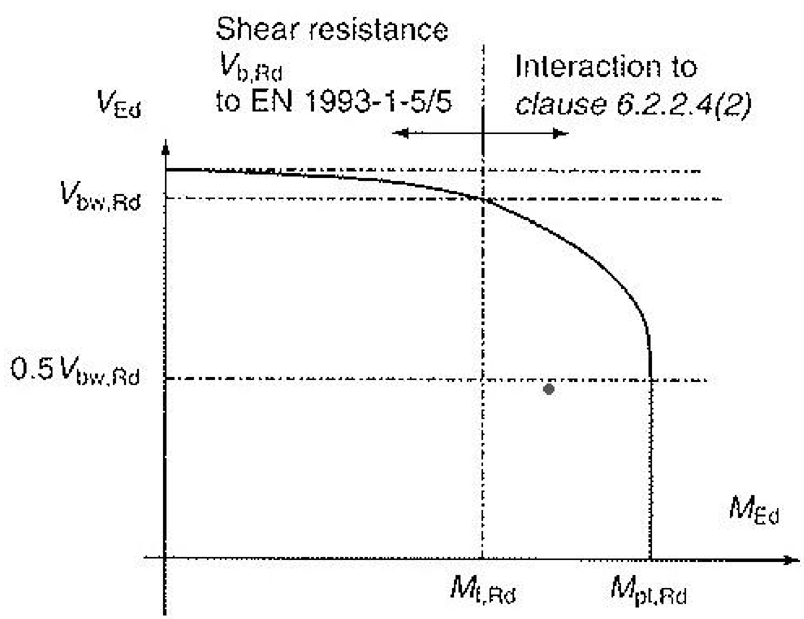
If we neglect the contribution of the flanges to the shear buckling resistance, then:

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} = 4.44 + 0 \leq 10.63\ MN\ ;\ V_{b,Rd} = 4.44\ MN$$

So, as $\boldsymbol{V_{Ed}}$ = 2.21 MN≤ $\boldsymbol{V_{Rd}}$ = $min(\boldsymbol{V_{bw,Rd}}\ ;\ \boldsymbol{V_{pl,a,Rd}})$= min(4.44 ; 11.70)=4.44, then is verified.

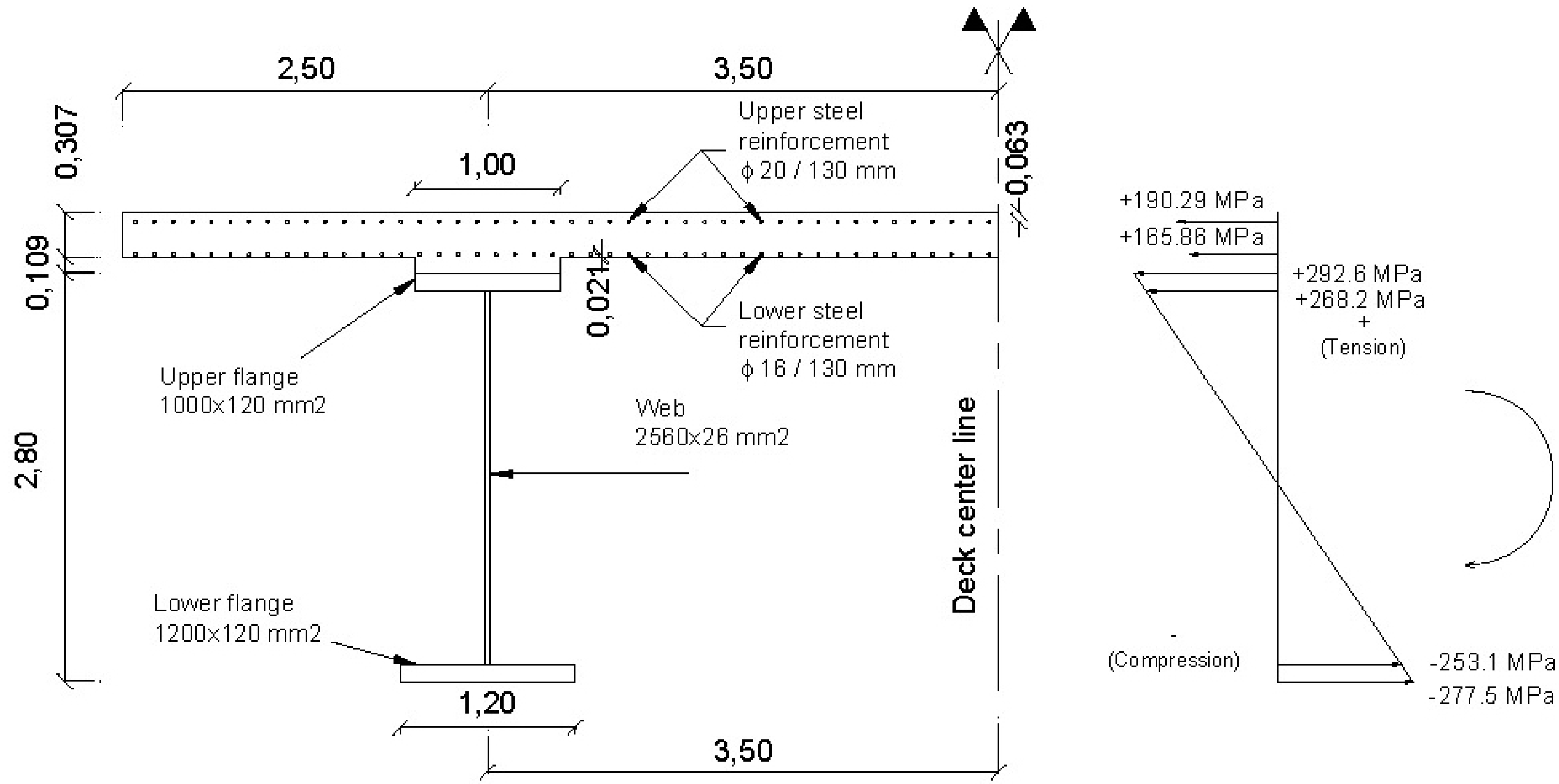
If the vertical shear force $\boldsymbol{V_{Ed}}$ does not exceed half the shear resistance $\boldsymbol{V_{Rd}}$, obtained before, there is no need to check the interaction ***M-V*** (EN-1994-2, 6.2.2.4).

In our case $\boldsymbol{V_{Ed}}$= 2.21<0.5·4.44=2.22 MN then there is no need to check the interaction ***M-V***.



Contents

2. Verification of cross-section at internal support P1
2.1. Geometry and Stresses



- Concrete slab tensioned. It's contribution is therefore neglected in the cross-section resistance.
- Stresses are calculated with the composite mechanical properties and obtained by summing the various steps whilst respecting the construction phases. (Sign criteria for the example + tension and – compression)
- Internal forces and moments: M_{Ed} = - 109.35 MN·m; V_{Ed} = 8.12 MN

2. Verification of cross-section at internal support P1
2.2. Determining the cross-section Class (EN1994-2, 5.5.2) I

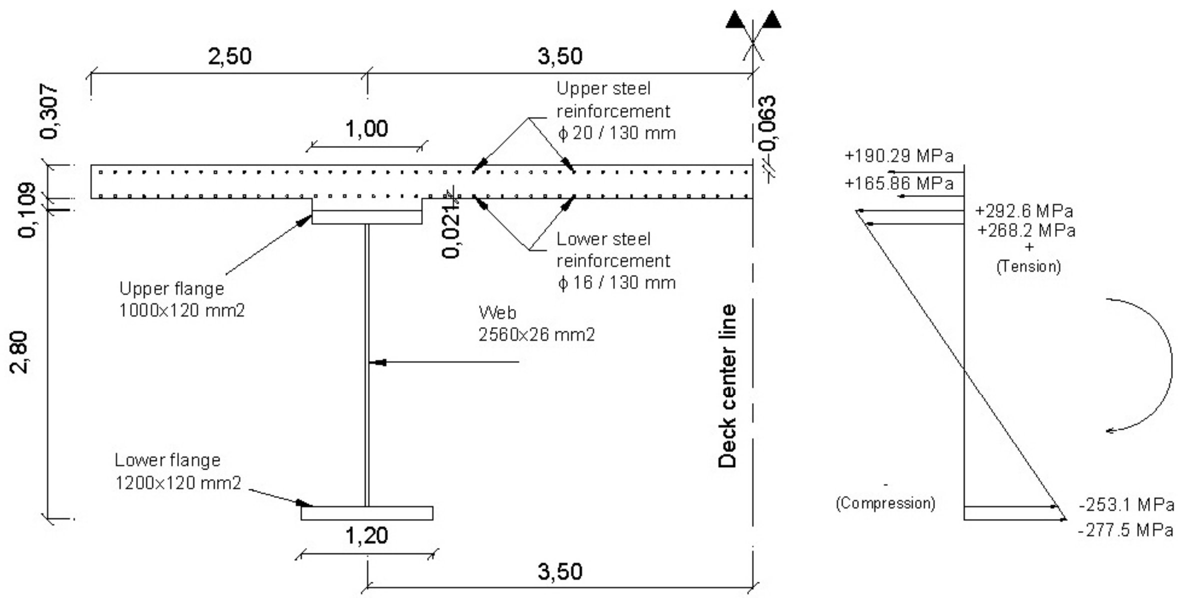
- The upper flange is in tension therefore in Class 1
- The lower flange is in compression, and then must be classified according to (EN 1993-1-1, Table 5.2):

$$c = \frac{b_{lf} - t_w}{2} = \frac{1200 - 26}{2} = 587 \text{ mm} \qquad \frac{c}{t_{lf}} = \frac{587}{120} = 4.891 \leq 9 \cdot \varepsilon = 9 \cdot \sqrt{\frac{235}{295}} = 8.033$$

(Lower flange t_{lf}=120 mm, $f_{y,lf}$=295 MPa)

Then the lower flange is in Class 1

- The web is in tension in its upper part and in compression in its lower part. To classify the steel web, we need to determine the position of the Plastic Neutral Axis (PNA).



2. Verification of cross-section at internal support P1
2.2. Determining the cross-section Class (EN1994-2, 5.5.2) II

The position of the Plastic Neutral Axis (PNA) is determined as follows:

- The tensioned slab is cracked and we neglect its contribution.
- Ultimate force of the tensioned upper reinforcing steel bars (ϕ 20/130 mm):

$$F_{s,1} = A_{s,1}\frac{f_{sk}}{\gamma_s} = 144.996 \cdot 10^{-4}\, m^2 \cdot \frac{500}{1.15} = 6.304 \text{ MN}$$

- Ultimate force of the tensioned lower reinforcing steel bars (ϕ 16/130 mm):

$$F_{s,2} = A_{s,2}\frac{f_{sk}}{\gamma_s} = 92.797 \cdot 10^{-4}\, m^2 \cdot \frac{500}{1.15} = 4.034 \text{ MN}$$

- Design plastic resistance of the structural steel upper flange (1 flange):

$$F_{s,uf} = A_{s,uf}\frac{f_{y.uf}}{\gamma_{M0}} = (1.2 \cdot 0.12) \cdot \frac{295}{1.0} = 35.4 \text{ MN}$$

- Design plastic resistance of the total structural steel web (1 web):

$$F_{s,w} = A_{s,w}\frac{f_{y,w}}{\gamma_{M0}} = (2.56 \cdot 0.026) \cdot \frac{345}{1.0} = 22.963 \text{ MN}$$

- Design plastic resistance of the structural steel lower flange (1 flange):

$$F_{s,lf} = A_{s.lf}\frac{f_{y,lf}}{\gamma_{M0}} = (1.20 \cdot 0.12) \cdot \frac{295}{1.0} = 42.48 \text{ MN}$$

- As $|F_{s,1}|+|F_{s,2}|+|F_{s,uf}|\leq|F_{s,w}|+|F_{s,lf}|$ and $|F_{s,1}|+|F_{s,2}|+|F_{s,uf}|+|F_{s,w}|\geq|F_{s,lf}|$ the PNA is deduced to be located in the steel web.
- If we consider that the P.N.A. is located at a distance ***x*** from upper the extreme fibre of the web, then the internal axial forces equilibrium of the cross section leads to the location of the PNA:

$$F_{s,1}+F_{s,2}+F_{s,uf}+F_{s,w}\cdot\frac{x}{2.56}-F_{s,w}\cdot\frac{(2.56-x)}{2.56}F_{s,w}-F_{s,lf}=0$$; ***x***=1.098 m

- Over half of the steel web is in compression (the lower part): 2.56-1.098=1.462 m.

$$\alpha=\frac{h_w-x}{h_w}=\frac{2.56-1.098}{2.56}=0.571>0.50$$ if α >0.50 (EN-1993-1-1, 5.5 and table 5.2 sheet 1 of 3), then the limiting slenderness between Class 2 and Class 3 is given by:

$$\frac{c}{t}=\frac{2.56}{0.026}=98.46>>\frac{456\varepsilon}{13\alpha-1}=\frac{456\cdot\sqrt{\frac{235}{345}}}{13\cdot 0.571-1}=58.59$$

Then, the steel web is at least in Class 3

2. Verification of cross-section at internal support P1

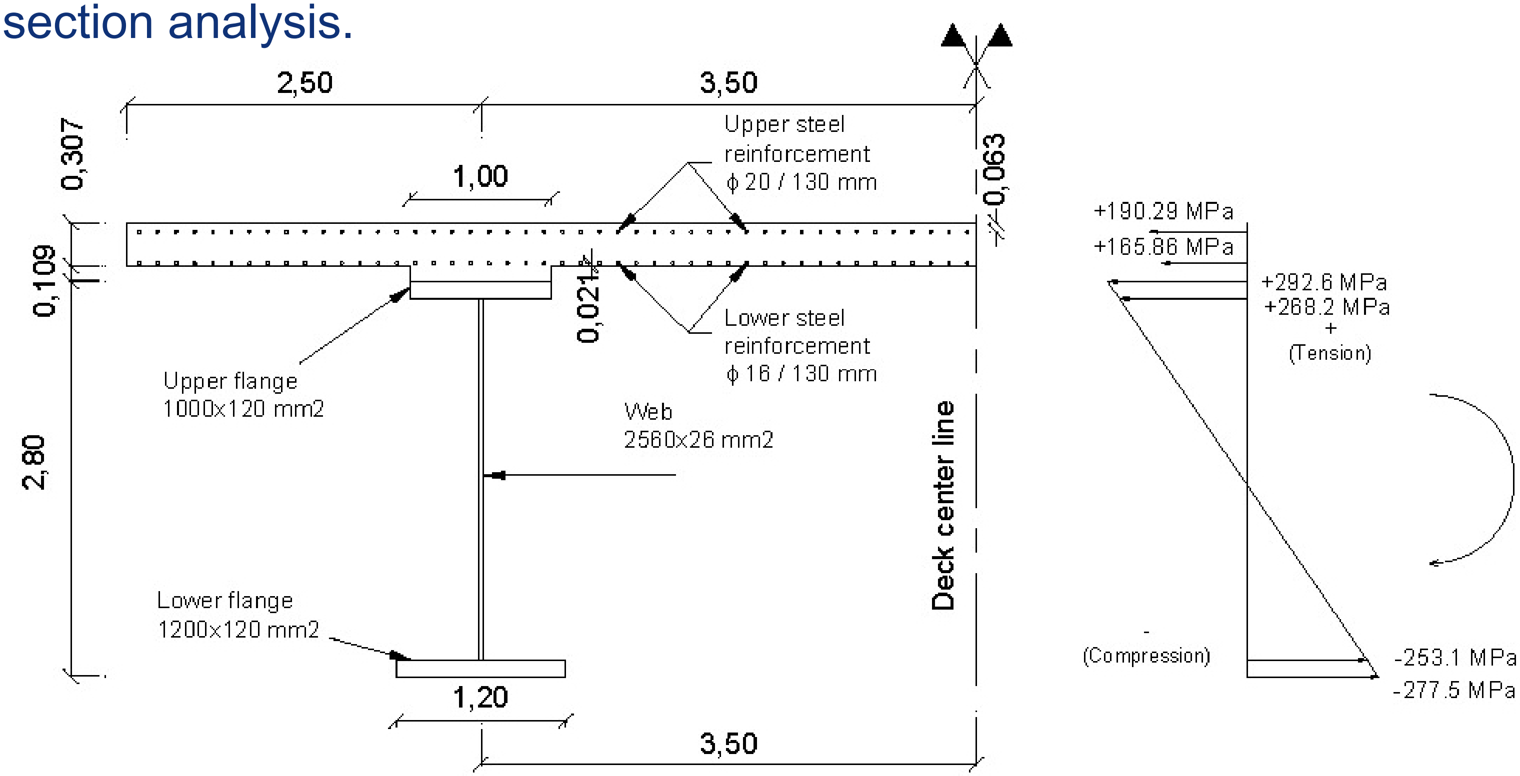
2.2. Determining the cross-section Class (EN1994-2, 5.5.2) IV

- Based on the elastic stress distribution at ULS: ψ= -(268.2 / 253.1) = -1.059≤ -1 therefore the limiting slenderness between Class 3 and Class 4 is given by:

$$\frac{c}{t} = \frac{2.56}{0.026} = 98.46 \leq 62 \cdot \varepsilon \cdot (1-\psi) \cdot \sqrt{(-\psi)} = 62 \cdot \sqrt{\frac{235}{345}} \cdot (1+1.059) \cdot \sqrt{1.059} = 108.49$$

It is concluded that the steel web is in Class 3.

Conclusion: The cross-section at support P1 is in Class 3 and is checked by an elastic section analysis.



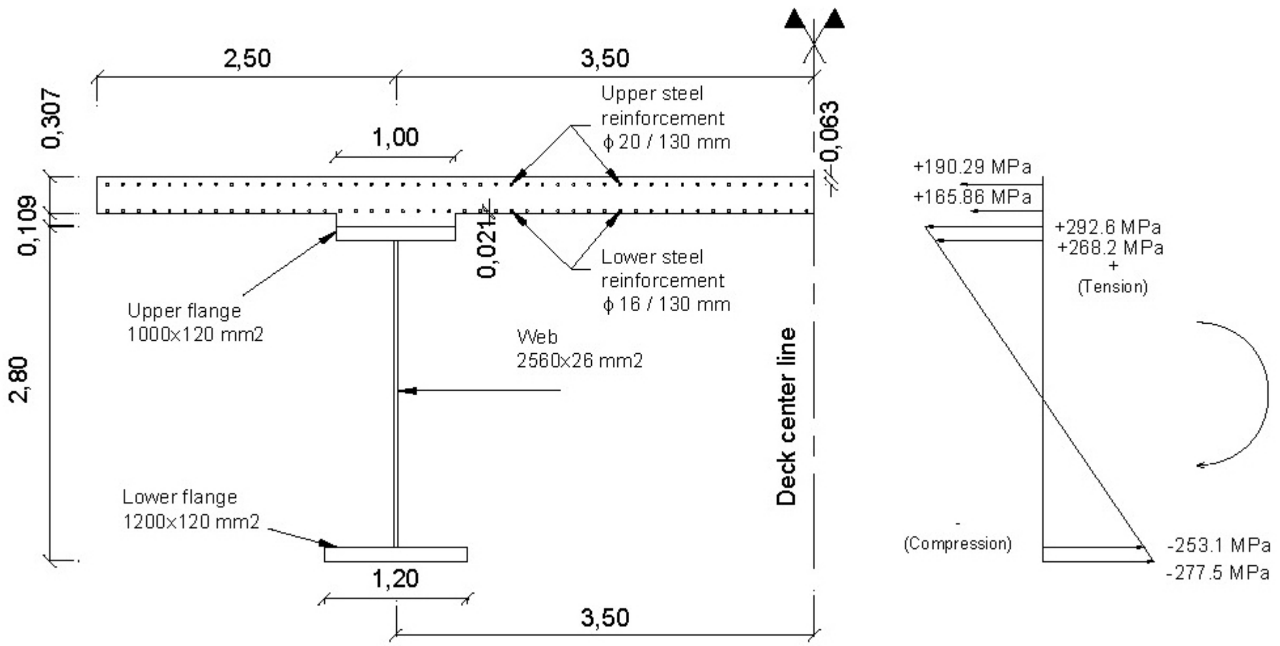
2. Verification of cross-section at internal support P1
2.3. Section analysis

2.3.1. Elastic bending verification

In the elastic bending verification (class 3), the maximum stresses in the structural steel must be below the yield strength: $\left|\sigma_s\right| \leq \frac{f_y}{\gamma_{M0}}$

As we have 292.63 MPa in the upper steel flange and -277.54 MPa in the lower steel flange, which are below the limit of f_y/γ_{M0}=295 MPa admitted in an elastic analysis for t_f=120 mm, the bending resistance is verified.

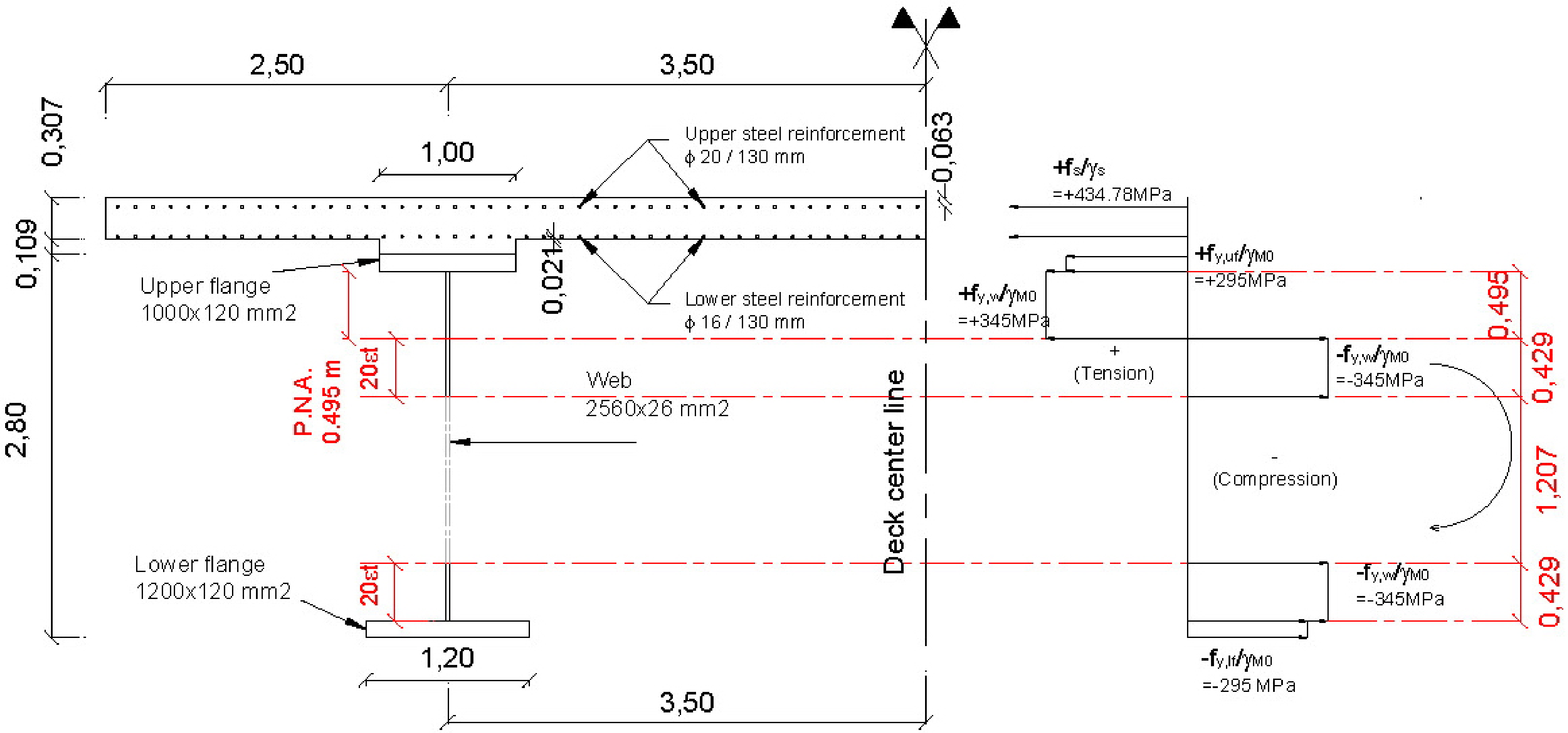
This verification could be made, not with the extreme fibre stresses, but with the stresses of the center of gravity of the flanges (EN-1993-1-1, 6.2.1(9)).



2. Verification of cross-section at internal support P1
2.3. Section analysis

2.3.2. Alternative: Plastic verification (Effective class 2 cross-section) I

Eurocode EN-1994-2, 5.5.2(3) establishes that a cross-section with webs in Class 3 and flanges in Classes 1 or 2 may be treated as an effective cross-section in Class 2 with an effective web in accordance to EN-1993-1-1, 6.2.2.4.



2. Verification of cross-section at internal support P1
2.3. Section analysis

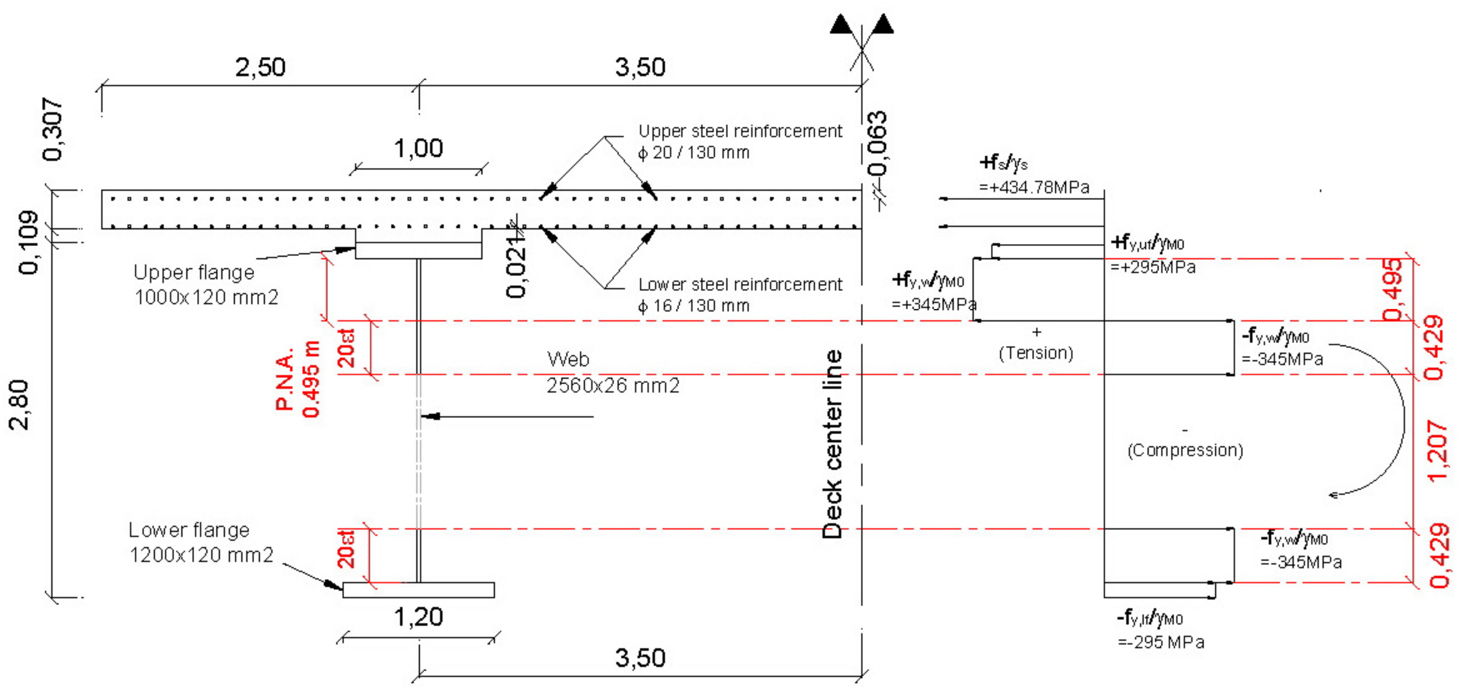
2.3.2. Alternative: Plastic verification (Effective class 2 cross-section) II

If we consider that the PNA is located at a distance **x** from the extreme upper fibre of the upper part of the web, then the internal axial forces equilibrium of the cross section deduces the location of the PNA:

$$F_{s,1} + F_{s,2} + F_{s,uf} + x \cdot t_w \cdot \frac{f_{y,w}}{\gamma_{M0}} - 2 \cdot \left(20 \cdot \varepsilon \cdot t_w \cdot \frac{f_{y,w}}{\gamma_{M0}} \right) - F_{s,lf} = 0$$

x=0.495 m (tensioned zone of the web)

And the hogging bending moment resistance of the effective class 2 cross-section is: $\boldsymbol{M_{pl,Rd}}$=-122.97 MN·m. As $\boldsymbol{M_{Ed}}$=-109.35 then the bending resistance is verified.



2. Verification of cross-section at internal support P1
2.3. Section analysis

2.3.3. Shear resistance check

As $\frac{h_w}{t_w} = \frac{2.56}{0.026} = 98.46 \geq \frac{31\varepsilon}{\eta}\sqrt{k_\tau} = 51.36$ the web (stiffened by the vertical stiffeners) should be checked in terms of shear buckling, according to EN-1993-1-5, 5.1

The maximum design shear resistance $\boldsymbol{V_{Rd}}$ is given by

$\boldsymbol{V_{Rd}} = min(\boldsymbol{V_{bw,Rd}}\ ;\ \boldsymbol{V_{pl,a,Rd}})$

Where:

- $\boldsymbol{V_{bw,Rd}}$ is the shear bucking resistance according to EN-1993-1-5, 5
- $\boldsymbol{V_{pl,a,Rd}}$ is the resistance to vertical shear according to EN-1993-1-1, 6.2.6

$$V_{pl,a,Rd} = \frac{\eta \cdot f_{y,w} \cdot h_w \cdot t}{\sqrt{3} \cdot \gamma_{M0}} = \frac{1.2 \cdot 345 \cdot 2560 \cdot 26}{\sqrt{3} \cdot 1.0} \cdot 10^{-6} = 15.91\ MN \qquad \text{(EN 1993-1-1, 6.2.6)}$$

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta \cdot f_{y,w} \cdot h_w \cdot t}{\sqrt{3} \cdot \gamma_{M1}} = \frac{1.2 \cdot 345 \cdot 2560 \cdot 26}{\sqrt{3} \cdot 1.10} \cdot 10^{-6} = 14.46\ MN \qquad \text{(EN 1993-1-5, 5.2)}$$

2. Verification of cross-section at internal support P1
2.3. Section analysis

2.3.3. Shear resistance check II

Given the distribution of the transverse bracing frames in the span P1-P2 (spacing a=8 m), a vertical frame post is located in the cross section at P-1.

The shear buckling check is therefore performed in the adjacent web panel with the highest shear force.

The maximum shear force registered in this panel is $\boldsymbol{V_{Ed}}$ = 8.12 MN.

The vertical frame posts are assumed to be rigid. This yields:

$$k_\tau = 5.34 + 4 \cdot \left(\frac{h_w}{a}\right)^2 = 5.34 + 4 \cdot \left(\frac{2.56}{8}\right)^2 = 5.75$$

is the shear buckling coefficient (EN-1993-1-5 Annex A.3)

2. Verification of cross-section at internal support P1
2.3. Section analysis

2.3.3. Shear resistance check III

$$\sigma_E = \frac{\pi^2 \cdot E \cdot t_w{}^2}{12\left(1-\nu^2\right) \cdot h_w{}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 26^2}{12\left(1-0.3^2\right) \cdot 2560^2} = 19.58\ MPa$$

(EN-1993-1-5 Annex A.1)

$$\tau_{cr} = k_\tau \cdot \sigma_E = 5.75 \cdot 19.58 = 112.58\ MPa$$

(EN 1993-1-5, 5.3)

$$\bar{\lambda}_w = \sqrt{\frac{f_{y,w}}{\tau_{cr} \cdot \sqrt{3}}} = 0.76 \cdot \sqrt{\frac{f_{y,w}}{\tau_{cr}}} = 0.76 \cdot \sqrt{\frac{345}{112.58}} = 1.33$$

slenderness of the panel (EN-1993-1-5, 5.3.)

As $\bar{\lambda}_w \geq 1.08$,then the factor for the contribution of the web to the shear buckling resistance is:

$$\chi_w = \frac{1.37}{\left(0.7 + \bar{\lambda}_w\right)} = \frac{1.37}{\left(0.7 + 1.33\right)} = 0.675$$

(Table 5.1. of EN-1993-1-5, 5.3)

Finally the contribution of the web to the shear buckling resistance is:

$$V_{bw,Rd} = \frac{\chi_w \cdot f_{y,w} \cdot h_w \cdot t}{\sqrt{3} \cdot \gamma_{M1}} = \frac{0.675 \cdot 345 \cdot 2560 \cdot 26}{\sqrt{3} \cdot 1.10} \cdot 10^{-6} = 8.14\ MN$$

2.3.3. Shear resistance check IV

If we neglect the contribution of the flanges to the shear buckling resistance, then:

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} = 8.14 + 0 \leq 14.46\ MN\ ;\ V_{b,Rd} = 8.14\ MN$$

So, as $\boldsymbol{V_{Ed}}$ = 8.12 MN≤ $\boldsymbol{V_{Rd}}$ = ***min($V_{bw,Rd}$; $V_{pl,a,Rd}$)=*** min(8.14 ; 15.91)=8.14, then the shear resistance is verified without considering the interaction M-V.

When the flange resistance is not fully used to resist the design bending moment, and therefore $\boldsymbol{M_{Ed}<M_{f,Rd}}$ the contribution from the flanges to the shear buckling resistance could be evaluated according to EN-1993-1-5, 5.4.

$$V_{bf,Rd} = \frac{b_f \cdot t_f^{\,2} \cdot f_{yf}}{c \cdot \gamma_{M1}} \cdot \left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}}\right)^2\right)$$

(Usually this term has a very low influence, and can be neglected)

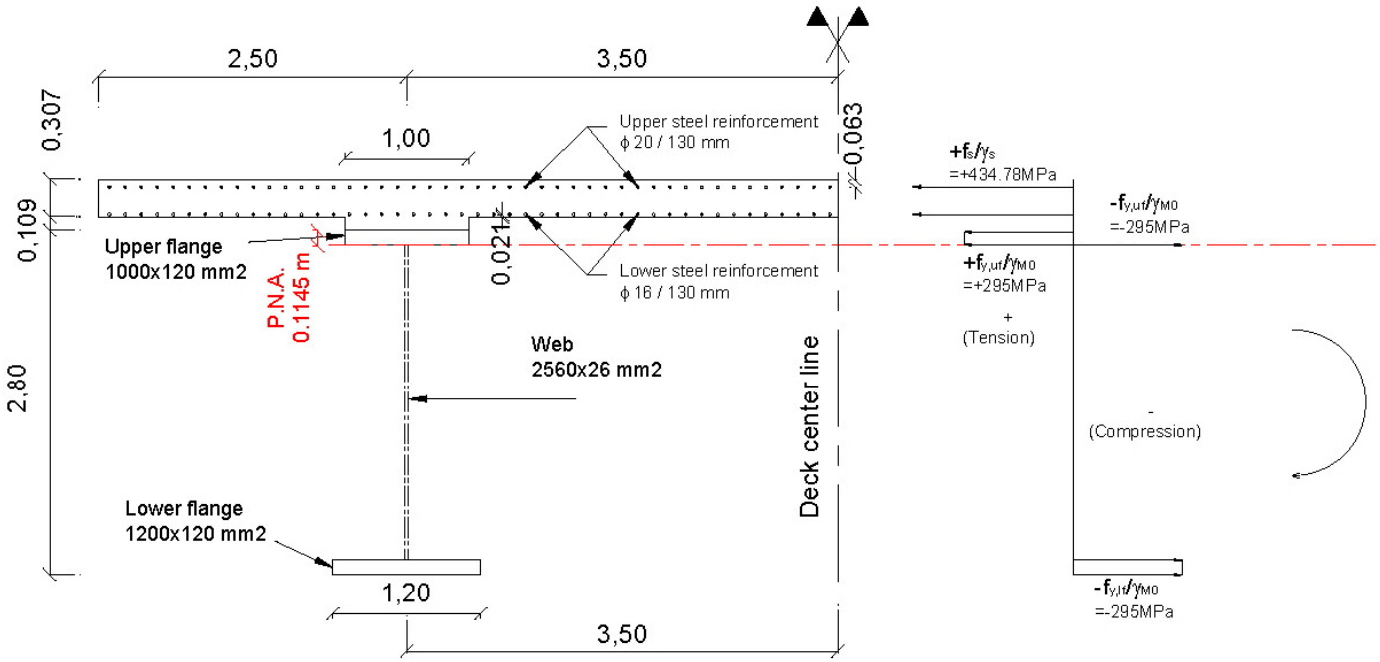
In our case: $V_{bf,Rd} = 0.197$ MN (For calculation details see the paper)

$\boldsymbol{M_{f,Rd}}$ is the bending resistance of the cross-section without the web, only considering the flanges

2. Verification of cross-section at internal support P1
2.3. Section analysis

2.3.3. Shear resistance check **V**

The hogging bending moment resistance of the effective cross-section neglecting the web area is $\boldsymbol{M_{f,Rd}}$=-117.40 MN·m.



As $|M_{Ed}| = 109.35 < |M_{f,Rd}| = 117.40$, the bending resistance is verified without considering the influence of the web, and the shear resistance is already verified neglecting the contribution of the flanges, there's no need no verify the interaction ***M-V***.

However we will check the interaction ***M-V*** for the example

2.3.4. Interaction M-V

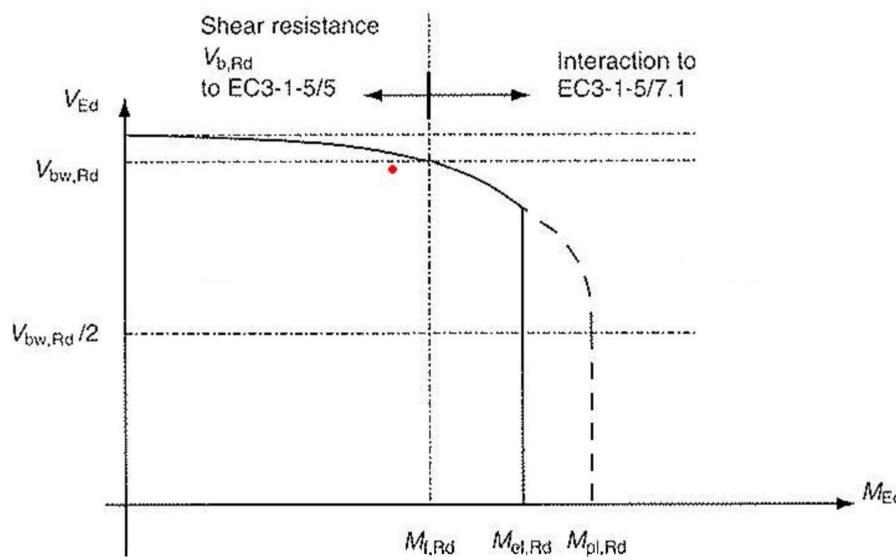
The interaction ***M-V*** should be considered according to EN-1993-1-5, 7.1 (1).

In our case, as the design shear force is higher than 50% of the shear buckling resistance then is has to be verified:

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

Where:

$$\bar{\eta}_1 = \frac{M_{Ed}}{M_{pl,Rd}} \quad \text{and} \quad \bar{\eta}_3 = \frac{V_{Ed}}{V_{bw,Rd}}$$



This criterion should be verified, according to EN-1993-1-5. 7.1 (2) at all sections other than those located at a distance less than ***h_w/2*** from a support with vertical stiffness. If we have: ***V_{Ed}***=7.885 MN, and ***M_{Ed}***=-100.605 mMN at that point, then:

$$\bar{\eta}_1 = \frac{100.605}{122.97} = 0.818 \qquad \bar{\eta}_3 = \frac{7.885}{8.14} = 0.9686 \qquad \bar{\eta}_1 + \left[1 - \frac{M_{f,Rd}}{M_{pl,Rd}}\right]\left[2\bar{\eta}_3 - 1\right]^2 = 0.818 + \left[1 - \frac{117.40}{122.97}\right]\left[2 \cdot 0.9686 - 1\right]^2 = 0.858 \leq 1$$

Contents

3. Alternative double composite cross-section at internal support P-1

- Double composite cross section alternative at hogging bending moments, with inferior concrete located between the two steel girders, connected to them.
- Economical alternative to reduce steel weight of the compressed bottom flange.
- Compression stresses from negative bending usually keep the bottom slab uncracked, so bending and torsional stiffness are noticeably higher than those classically obtained with steel sections.
- Double composite action greatly improves the deformational and dynamic response both to bending and torsion.
- The cross sections along the whole bridge are in Class 1 or Class 2, also in hogging areas. Thus instability problems at ultimate limit state are avoided, not only at the bottom flanges because of their connection to the concrete, but also in webs, due to the low position of the neutral axis in an ultimate limit state.

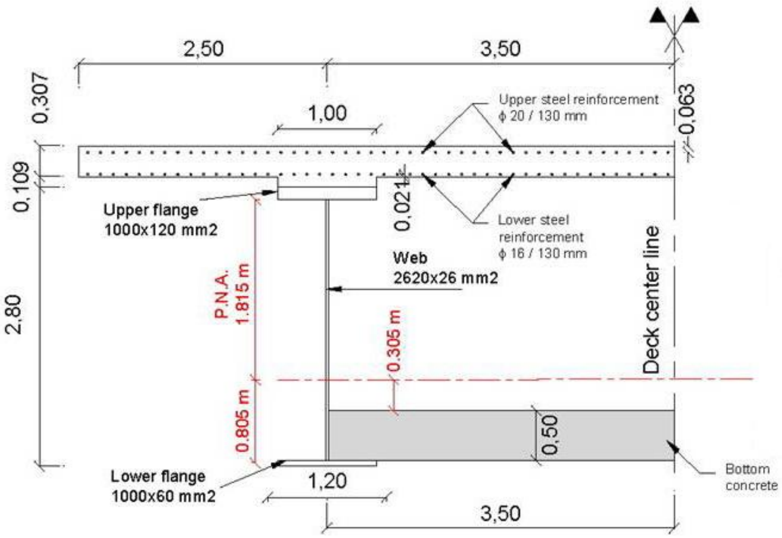
3. Alternative double composite cross-section at internal support P-1 II

Two examples of Spanish road composite bridges with double composite action in hogging areas.

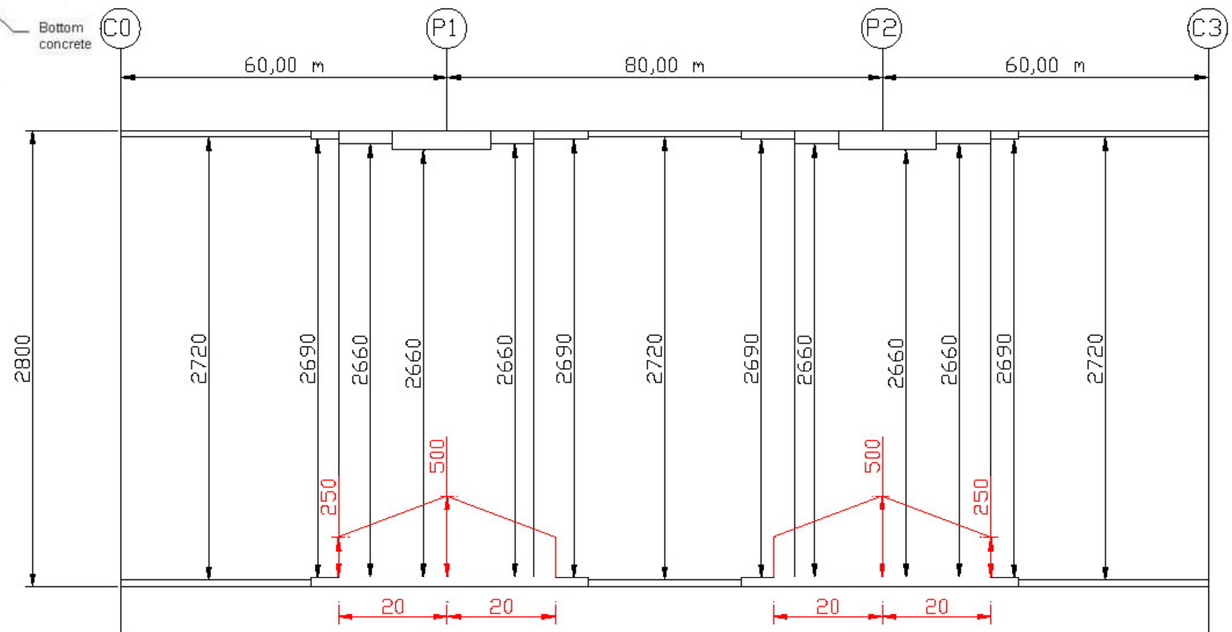
3. Alternative double composite cross-section at internal support P-1 III

Example of the first Spanish High Speed Railway Viaduct with double composite action

3. Alternative double composite cross-section at internal support P-1 IV



If we change the lower steel flange from 1200x120 mm[2] to a smaller one, of 1000x60 mm[2] plus a 0.50 m thick bottom slab of concrete C35/45, we could verify the bending resistance check to compare both cross sections



UPPER FLANGE Constant width 1000	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 x48000	26 x 36000	18 x 40000								
LOWER FLANGE Constant width 1200	35000 x 40	5000 x 55	40000 x (1000x60)	6000 x 55	28000 x 40	6000 x 55	40000 x (1000x60)	5000 x 55	35000 x 40				

3. Alternative double composite cross-section at internal support P-1
3.1. Determining the cross-section Class (EN1994-2, 5.5.2)

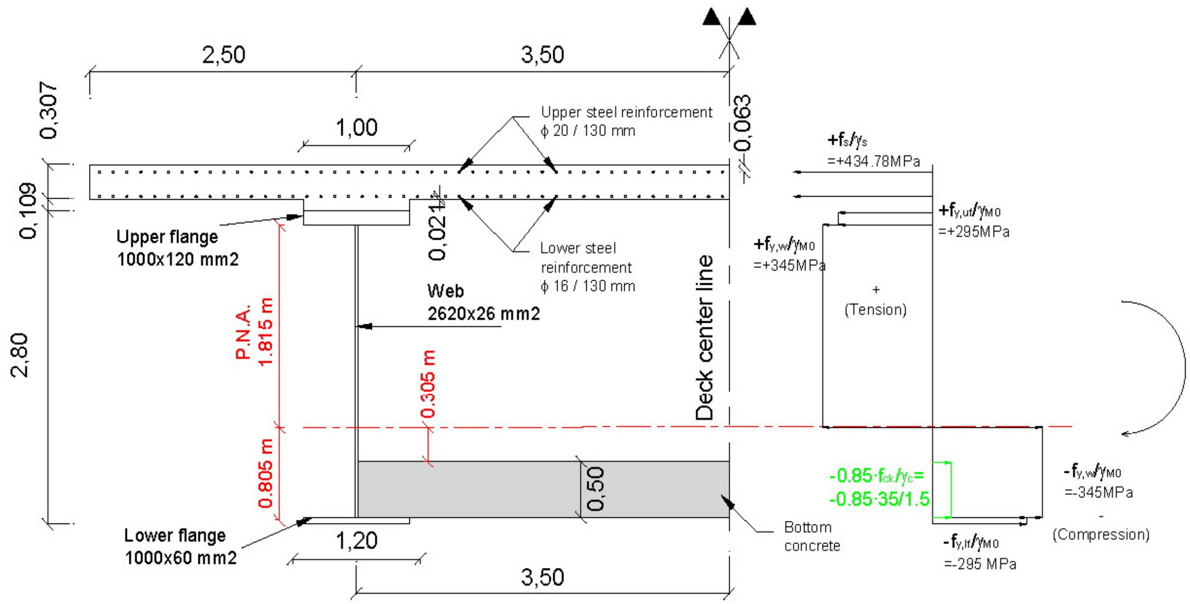
- The upper flange is in tension therefore it is in Class 1
- The lower flange is in compression， and then must be classified according to (EN 1993-1-1, Table 5.2):

$$c = \frac{b_{lf} - t_w}{2} = \frac{1000 - 26}{2} = 487\ \text{mm} \qquad \frac{c}{t_{lf}} = \frac{487}{60} = 8.116 \le 10 \cdot \varepsilon = 10 \cdot \sqrt{\frac{235}{335}} = 8.375$$

(Lower flange $\boldsymbol{t_{lf}}$=60 mm, $\boldsymbol{f_{y,lf}}$=355 MPa)

Then the lower flange is in Class 2

- The upper part of the web is in tension and lower part is in compression. To classify the steel web, we need to determine the position of the Plastic Neutral Axis (PNA).



3. Alternative double composite cross-section at internal support P-1
3.1. Determining the cross-section Class (EN1994-2, 5.5.2) II

The position of the Plastic Neutral Axis (PNA) is determined as follows:

- The tensioned upper slab is cracked and we neglect its contribution.
- Ultimate force of the tensioned upper reinforcing steel bars (ϕ 20/130 mm):

$$F_{s,1} = A_{s,1}\frac{f_{sk}}{\gamma_s} = 144.996{\cdot}10^{-4}\,m^2{\cdot}\frac{500}{1.15} = 6.304\ \text{MN}$$

- Ultimate force of the tensioned lower reinforcing steel bars (ϕ 16/130 mm):

$$F_{s,2} = A_{s,2}\frac{f_{sk}}{\gamma_s} = 92.797{\cdot}10^{-4}\,m^2{\cdot}\frac{500}{1.15} = 4.034\ \text{MN}$$

- Design plastic resistance of the structural steel upper flange (1 flange):

$$F_{s,uf} = A_{s,uf}\frac{f_{y.uf}}{\gamma_{M0}} = (1.2\cdot 0.12){\cdot}\frac{295}{1.0} = 35.4\ \text{MN}$$

- Design plastic resistance of the total structural steel web (1 web):

$$F_{s,w} = A_{s,w}\frac{f_{y,w}}{\gamma_{M0}} = (2.62\cdot 0.026){\cdot}\frac{345}{1.0} = 23.50\ \text{MN}$$

- Design plastic resistance of the structural steel lower flange (1 lower flange):

$$F_{s,lf} = A_{s.lf}\frac{f_{y,lf}}{\gamma_{M0}} = (1.00\cdot 0.06){\cdot}\frac{335}{1.0} = 20.1\ \text{MN}$$

- Design plastic resistance of the bottom concrete slab in compression:

$$F_{c,\inf} = A_c\frac{0.85 f_{ck}}{\gamma_c} = 3.5{\cdot}0.5.\frac{0.85{\cdot}35}{1.50} = 34.7\ \text{MN}$$

3. Alternative double composite cross-section at internal support P-1
3.1. Determining the cross-section Class (EN1994-2, 5.5.2) III

- As $|F_{s,1}|+|F_{s,2}|+|F_{s,uf}|\leq|F_{s,w}|+|F_{s,lf}|+|F_{c,\inf}|$ and $|F_{s,1}|+|F_{s,2}|+|F_{s,uf}|+|F_{s,w}|\geq|F_{s,lf}|+|F_{c,\inf}|$ the PNA is deduced to be located in the steel web.
- If we consider that the P.N.A. is located at a distance ***x*** from the upper extreme fibre of the web, then the internal axial forces equilibrium of the cross section gives the location of the PNA:

$$F_{s,1}+F_{s,2}+F_{s,uf}+F_{s,w}\cdot\frac{x}{2.62}-F_{s,w}\cdot\frac{(2.62-x)}{2.62}F_{s,w}-F_{s,lf}-F_{c,\inf}=0$$

; ***x***=1.815 m

- Only around 30% of the steel web is in compression (the lower part)

$$\alpha=\frac{h_w-x}{h_w}=\frac{2.62-1.815}{2.62}=0.307\leq0.50$$

if α >0.50 (EN-1993-1-1, 5.5 and table 5.2 sheet 1 of 3), then the limiting slenderness between Class 2 and Class 3 is given by:

$$\frac{c}{t}=\frac{2.62}{0.026}=100.76<\frac{41.5\varepsilon}{\alpha}=\frac{41.5\cdot\sqrt{\frac{235}{345}}}{0.307}=111.56$$

Then, the steel web is at least in Class 2

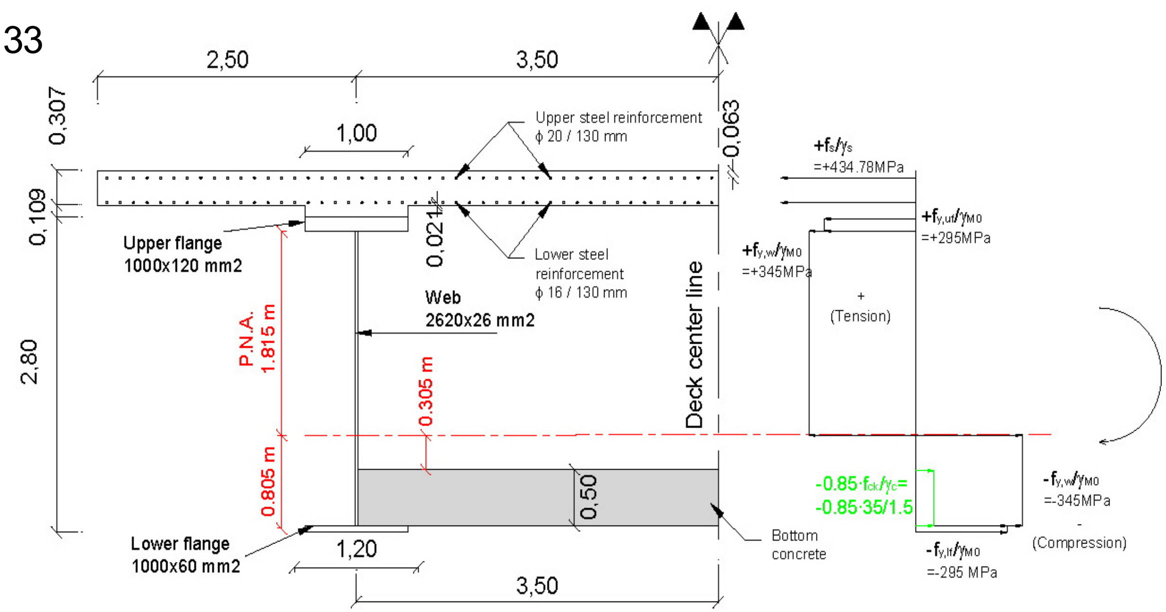
3. Alternative double composite cross-section at internal support P-1
3.1. Determining the cross-section Class (EN1994-2, 5.5.2) IV

- However, the part of the web in touch with the bottom concrete slab, is laterally connected to it, so only 0.305 m of the total length under compression (0.805 m) could have buckling problems. If we take this into consideration, the actual depth of the web considered for the classification of the compressed panel is 1.815+0.305=2.12m instead of 2.62 m, considered before.
- With these new values: $\alpha = \frac{h^*_w - x}{h^*_w} = \frac{2.12 - 1.815}{2.12} = 0.144 \leq 0.50$
- According to EN-1993-1-1, 5.5 and table 5.2 (sheet 1 of 3), if α <0.50 then the limiting slenderness between Class 1 and Class 2 is given by:

$$\frac{c}{t} = \frac{2.12}{0.026} = 81.54 < \frac{36\varepsilon}{\alpha} = \frac{36 \cdot \sqrt{\frac{235}{345}}}{0.144} = 206.33$$

The steel web could be classified as Class 1.

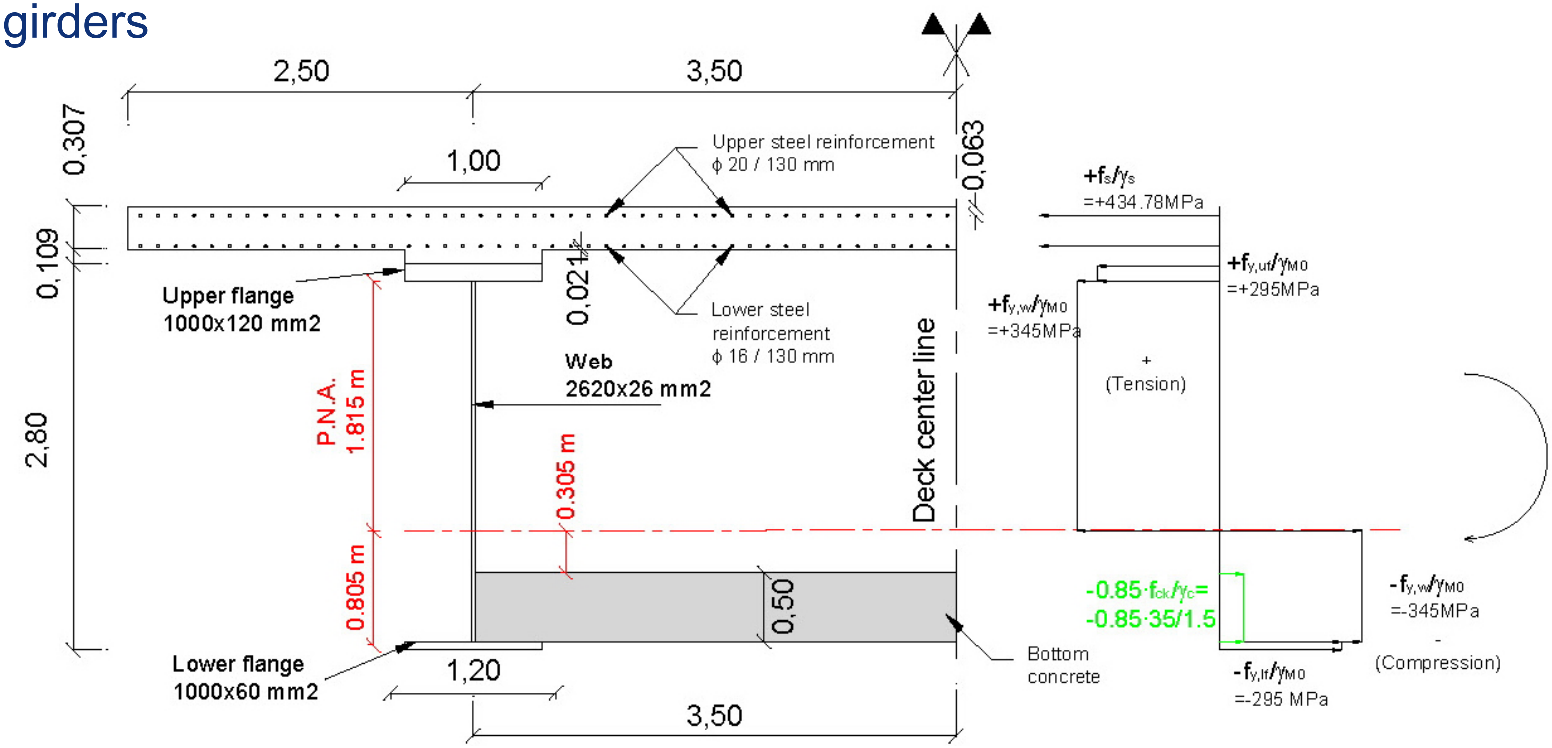
Conclusion: The cross-section at support P1 with double composite action is in Class 2 (due to the lower steel flange)



3. Alternative double composite cross-section at internal support P-1
3.2. Plastic section analysis. Bending resistance check

If we consider that the PNA is located at a distance ***x***=1.815 m from the upper extreme fibre of the web, then the hogging bending moment resistance of the Class 2 cross-section is: $M_{pl,Rd}$=-142.85 MN·m

In comparison with the ultimate resistance of the simple composite action cross-section we have significantly increased the bending resistance, locally reducing the amount of structural steel just by adding the bottom concrete slab connected to the steel girders



Contents

4. Justification of the Serviceability Limit States (SLS)

EN-1994-2, 7.1 (1) establishes that a composite bridge shall be designed such that all the relevant SLS are satisfied according to the principles of EN-1990, 3.4. The limit states that concern are:

- The functioning of the structure or structural members under normal use.
- The comfort of people.
- The “appearance” of the construction work. (Related with such criteria as high deflections and extensive cracking, rather than aesthetics)

At SLS under global longitudinal bending the following should be verified:

- Stress limitation and web breathing, according to EN-1994, 7.2.
- Deformations: deflections and vibrations, according to EN-1994, 7.3.
- Cracking of concrete, according to EN-1994, 7.4

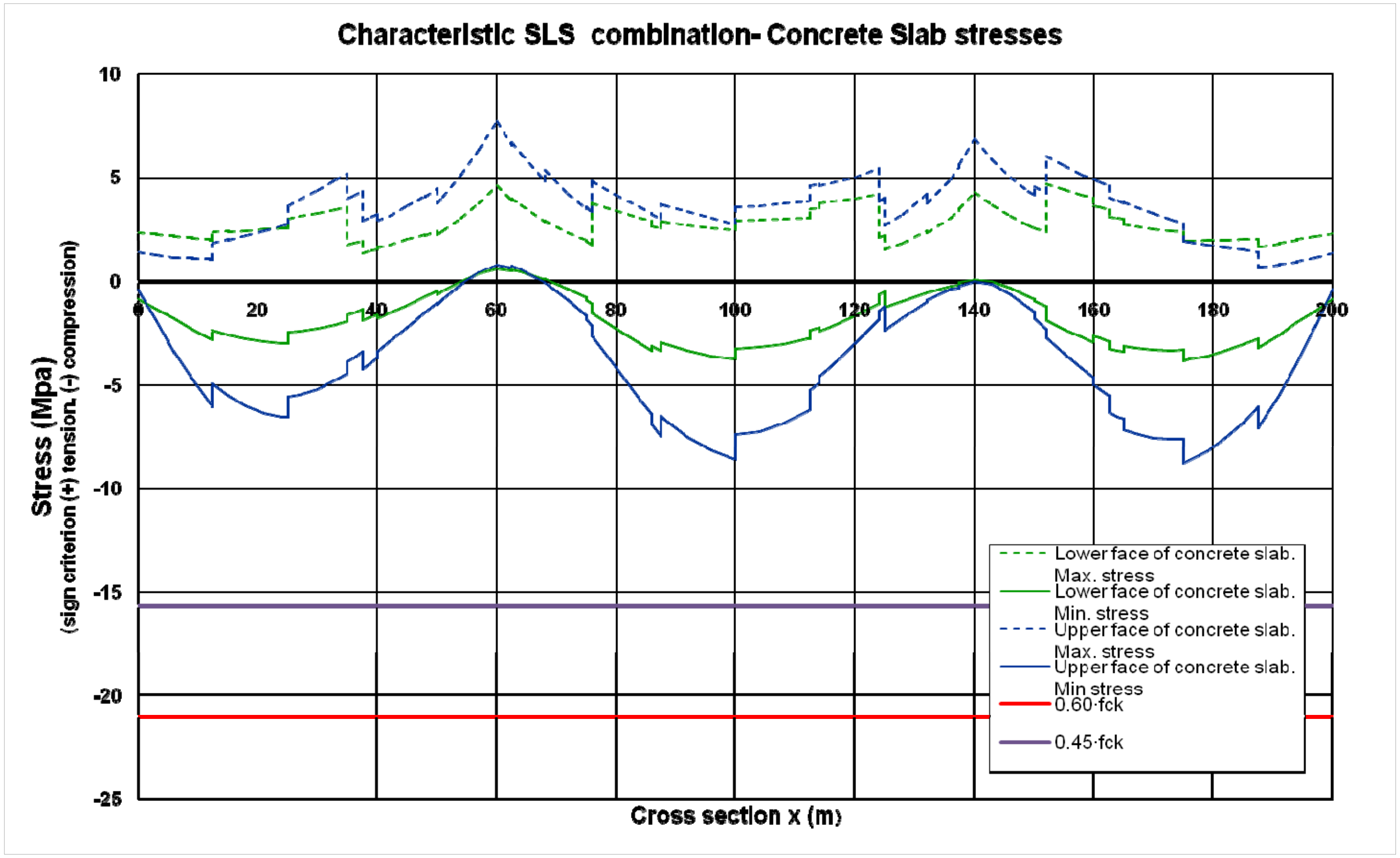
Deflection or vibration control, should be done according to EN-1994, 7.3.

Contents

5. Stresses control at Serviceability Limit States
5.1. Control of compressive stress in concrete

EN-1994-2, 7.2.2 (1) establishes that the excessive creep and microcraking of concrete shall be avoided by limiting the compressive stress in concrete.

EN-1992-1-1, 7.2 (2) recommends to limit the compressive stress under the characteristic combination to a value of $k_1 \cdot f_{ck}$ (k_1=0.60), and also recommends to limit compressive under the quasi-permanent loads to $k_2 \cdot f_{ck}$ (k_2=0.45) in order to admit linear creep assumption.

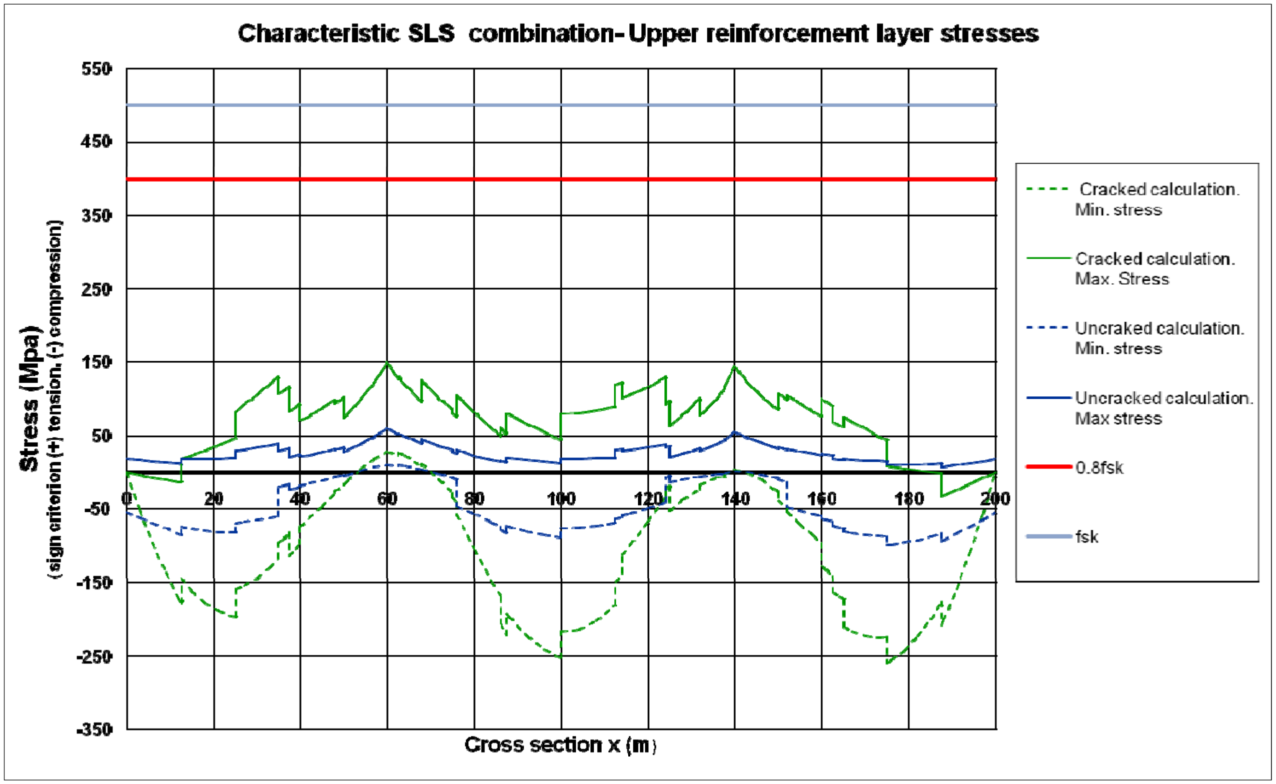


5. Stresses control at Serviceability Limit States
5.2. Control of stress in reinforcement steel bars

Tensile stresses in the reinforcement shall be limited in order to avoid inelastic strain, unacceptable cracking or deformation according to EN-1992-1-1, 7.2(4).

This may be assumed to be avoided if, under the characteristic combination, the tensile stress in the reinforcement does not exceed $k_3 \cdot f_{sk}$ (k_3=0.8) and where the stress is caused by an imposed deformation, the tensile stress should not exceed $k_4 \cdot f_{sk}$ (k_4=1.0)

When $M_{c,Ed}$ is negative, the tension stiffening term $\Delta\sigma_s$ should be added to the stress values calculated without taking the concrete strength into account.



5. Stresses control at Serviceability Limit States
5.3. Stress limitation in structural steel

For the characteristic SLS combination of actions the following criteria for the normal and shear stresses in the structural steel should be verified (EN-1993-2, 7.3):

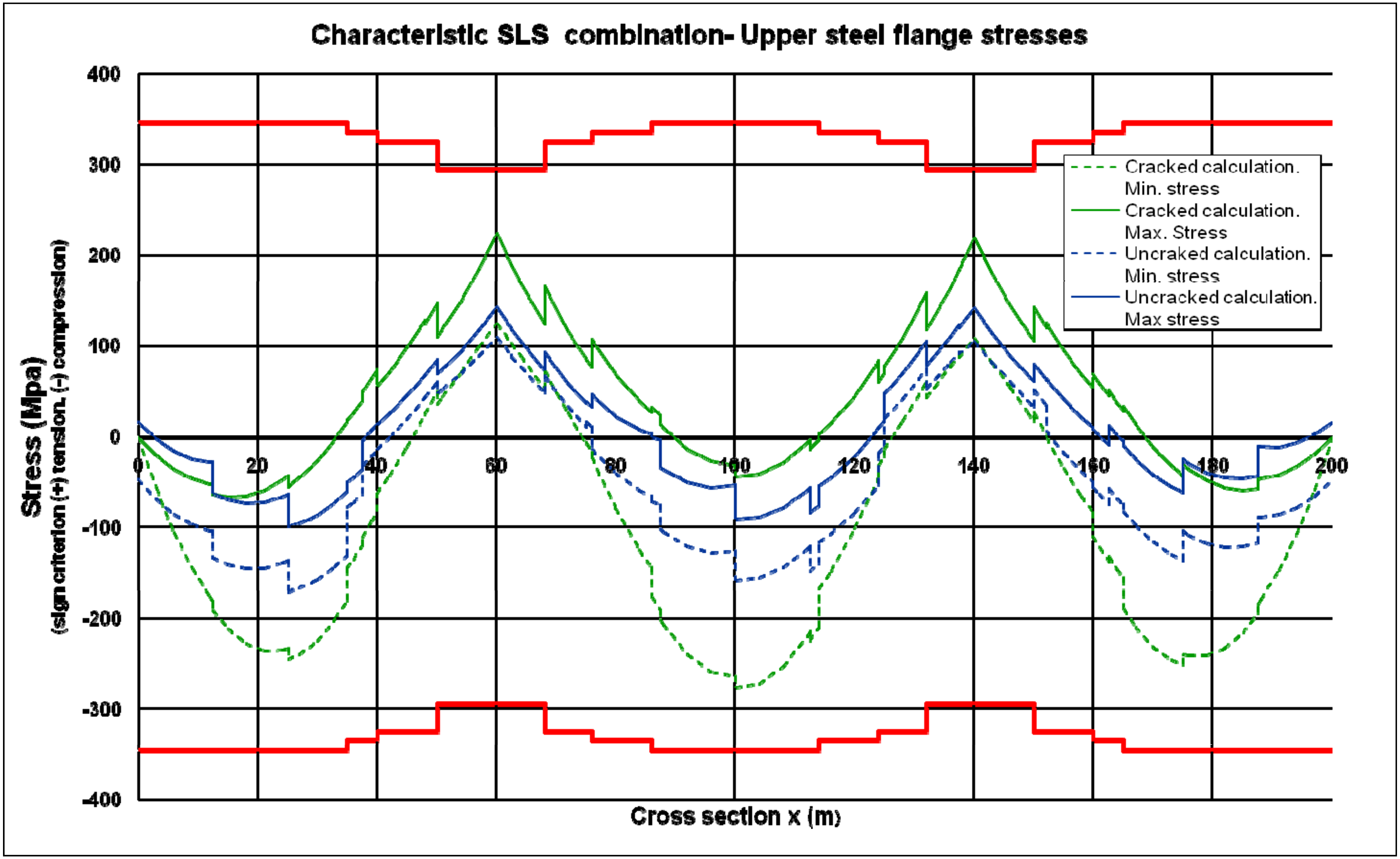
$$\sigma_{Ed,ser} \leq \frac{f_y}{\gamma_{M,ser}}$$

$$\tau_{Ed,ser} \leq \frac{f_y}{\sqrt{3} \cdot \gamma_{M,ser}}$$

$$\sqrt{\sigma^2_{Ed,ser} + 3 \cdot \tau^2_{Ed,ser}} \leq \frac{f_y}{\gamma_{M,ser}}$$

The partial factor $\gamma_{M,ser}$ is a national parameter, and the recommended value is 1.0 (EN-1993-2, 7.2 note 2)

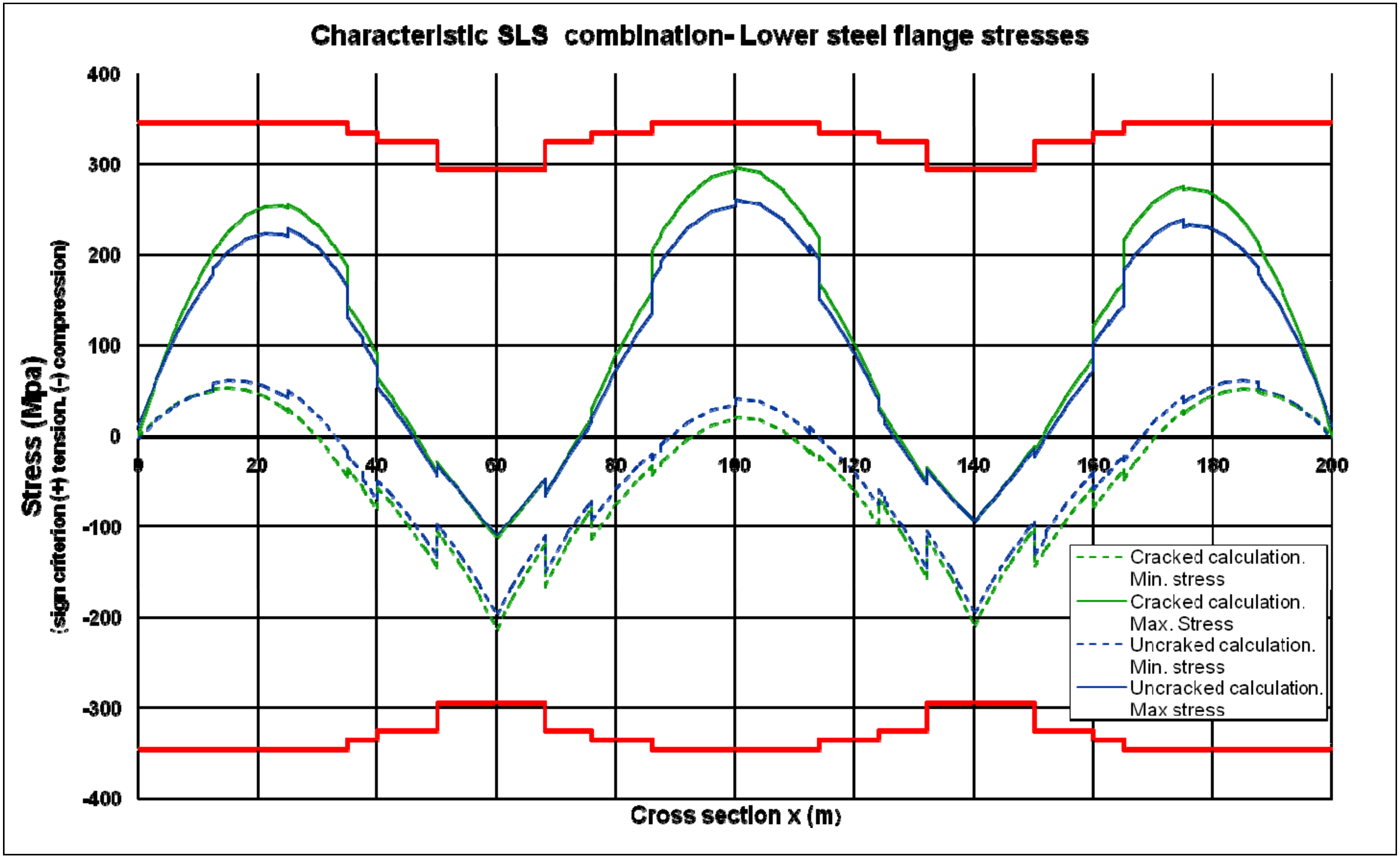
For the verification of the stresses control at SLS, the stresses should be considered on the external faces of the steel flanges, and not in the flange midplane (EN-1993-1-1, 6.2.1 (9))



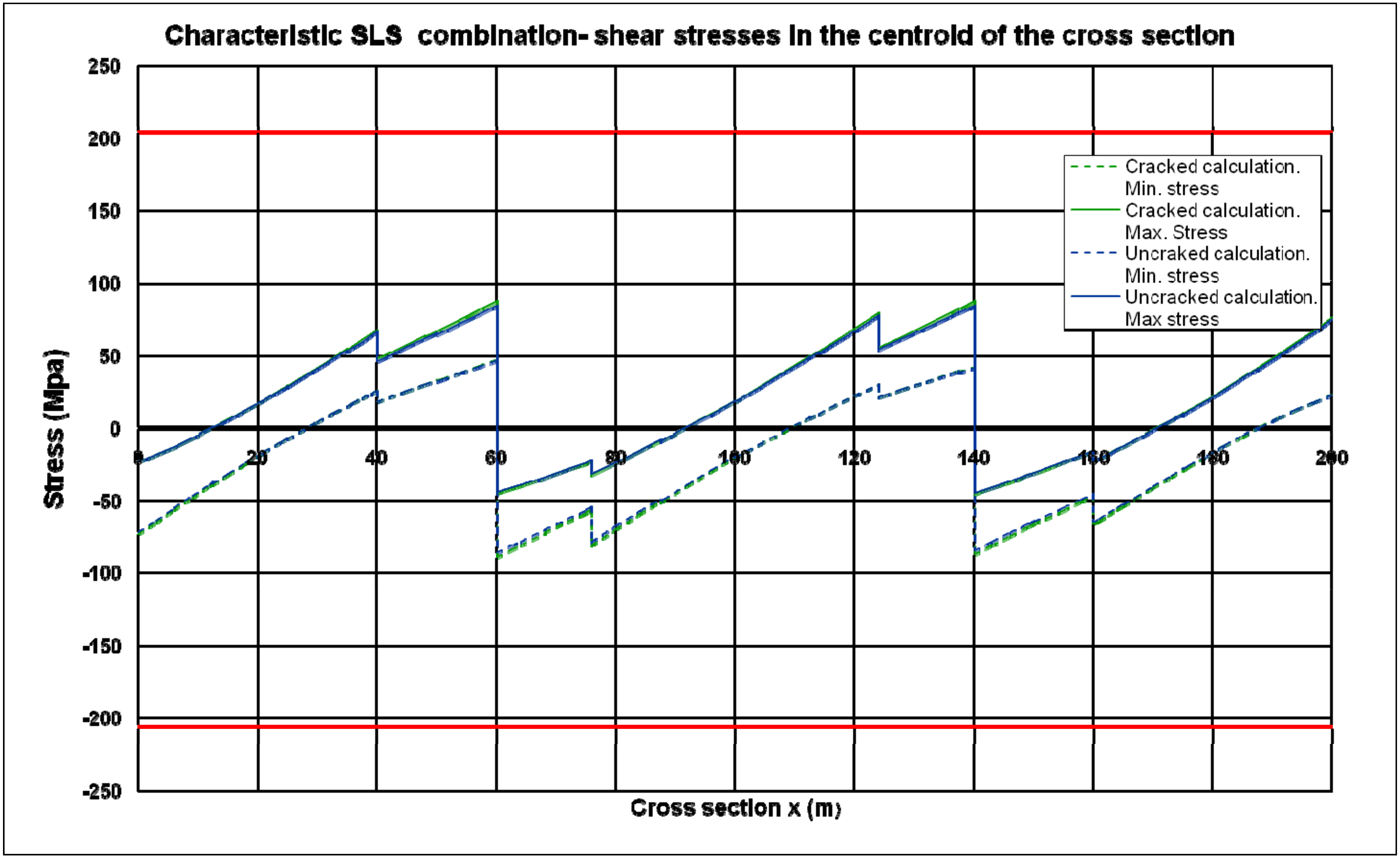
Characteristic SLS combination- Upper steel flange stresses
Stress (Mpa)
(sign criterion (+) tension, (-) compression)
400
300
200
100
0
-100
-200
-300
-400
0
20
40
60
80
100
120
140
160
180
200
Cross section x (m)
Cracked calculation. Min. stress
Cracked calculation. Max. Stress
Uncraked calculation. Min. stress
Uncracked calculation. Max stress

5. Stresses control at Serviceability Limit States
5.3. Stress limitation in structural steel

III

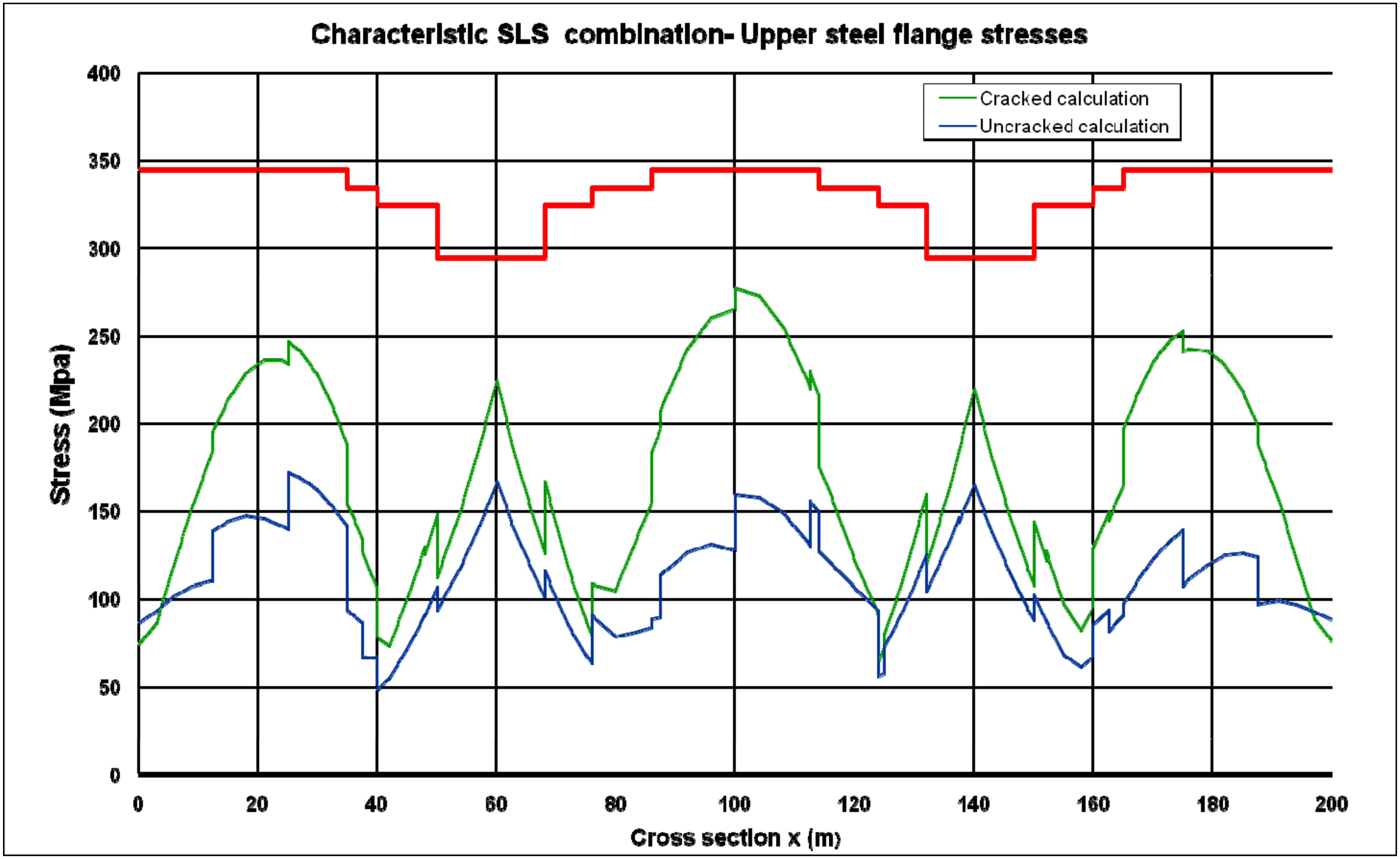


5. Stresses control at Serviceability Limit States
5.3. Stress limitation in structural steel

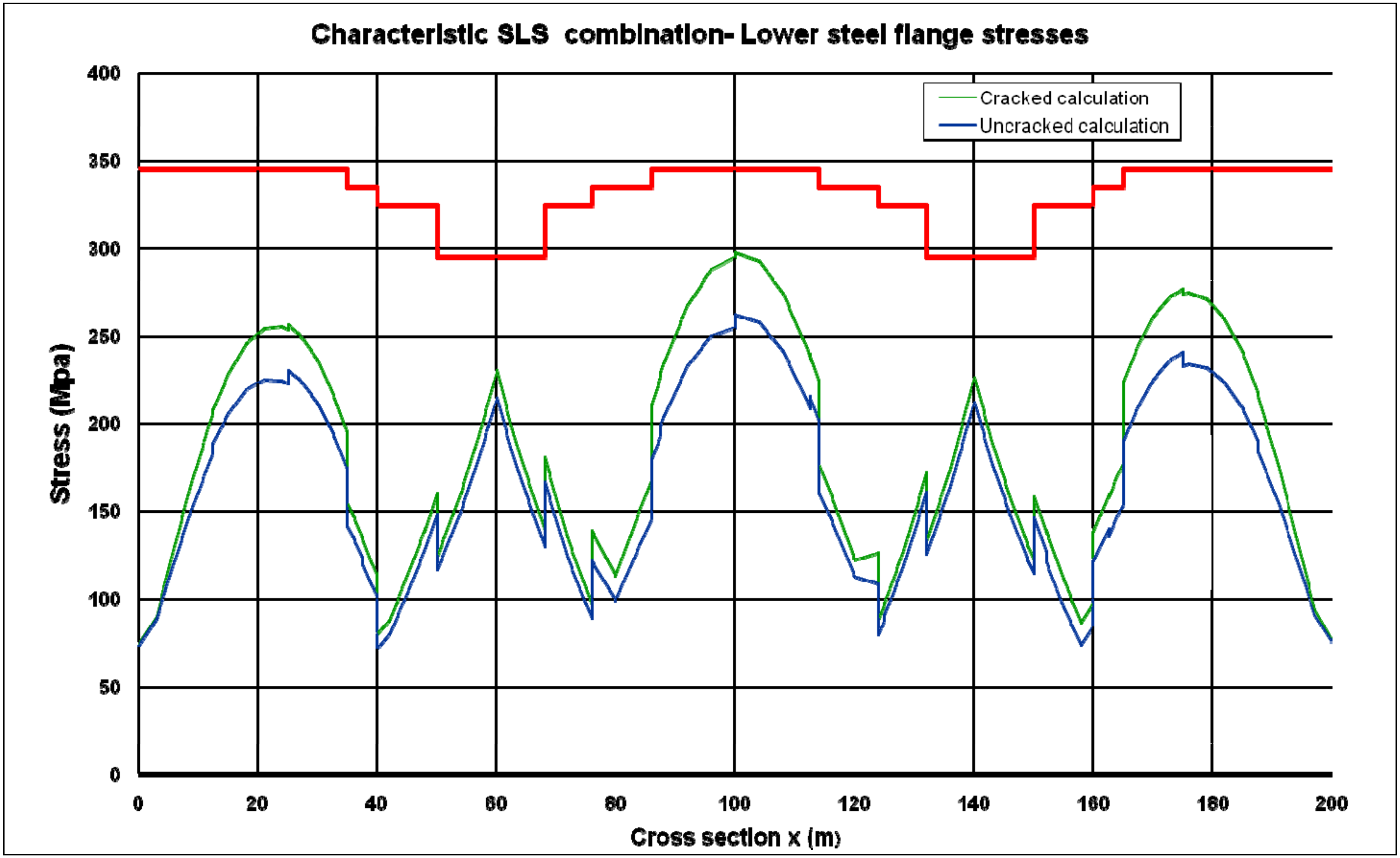


5. Stresses control at Serviceability Limit States
5.3. Stress limitation in structural steel

V



5. Stresses control at Serviceability Limit States
5.3. Stress limitation in structural steel



5. Stresses control at Serviceability Limit States
5.4. Additional verification of fatigue under a low number of cycles

According to EN-1993-2, 7.3 (2), it is assumed that the nominal stress range in the structural steel framework due to the SLS frequent load combination is limited to:

$$\Delta\sigma_{fre} \leq \frac{1{,}5 \cdot f_y}{\gamma_{M,ser}}$$

This criterion is used to ensure that the "frequent" variations remain confined in the strictly linear part (+/- 0.75 $\boldsymbol{f_y}$) of the structural steel stress-strain relationship. With this, any fatigue problems for a low number of cycles are avoided.

5. Stresses control at Serviceability Limit States
5.5. Limitation of web breathing

Every time a vehicle crosses the bridge, the web gets slightly deformed out of its plane according to the deformed shape of the first buckling mode and then returns to its initial shape. This repeated deformation called web breathing is likely to generate fatigue cracks at the weld joint between web and flange or between web and vertical stiffener.

According to EN-1993-2, 7.4 (2), for webs without longitudinal stiffeners (or for a sub-panel in a stiffened web), the web breathing occurrence can be avoided for road bridges if:

$$\frac{h_w}{t_w} \leq 30 + 4{,}0 \cdot L \leq 300$$

Where L is the span length in m, but not less than 20 m.

For the design example:

- in end-span: h_w/t_w = **151.1 ≤** 30+4·60=**270**
- in central span: h_w/t_w = **151.1 ≤ 300**

Generally speaking this criterion is widely verified for road bridges.

Contents

6. Control of cracking for longitudinal global bending
6.1. Maximum value of crack width

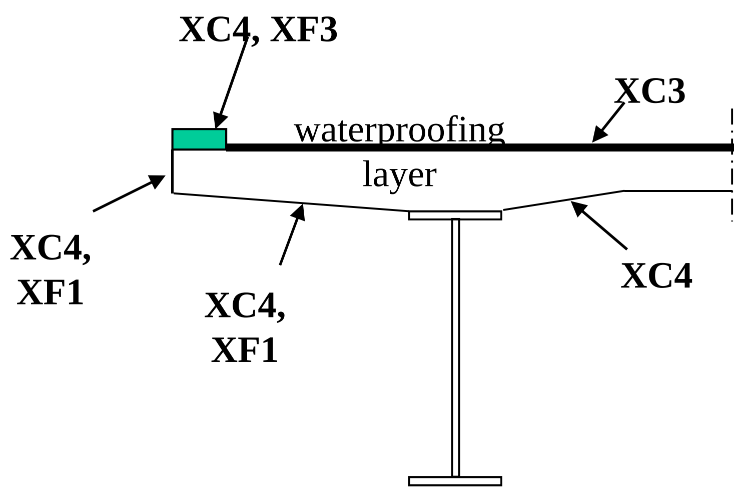
Table 7.101N — Recommended values of w_{max} and relevant combination rules

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.

Recommended values defined in EN1992-2, 7.3.1 (concrete bridges)



6. Control of cracking for longitudinal global bending
6.2. Cracking of concrete. Minimum reinforcement area

I

The simplified procedure of EN-1994-2, 7.4.2 (1) requires a minimum reinforcement area for the composite beams:

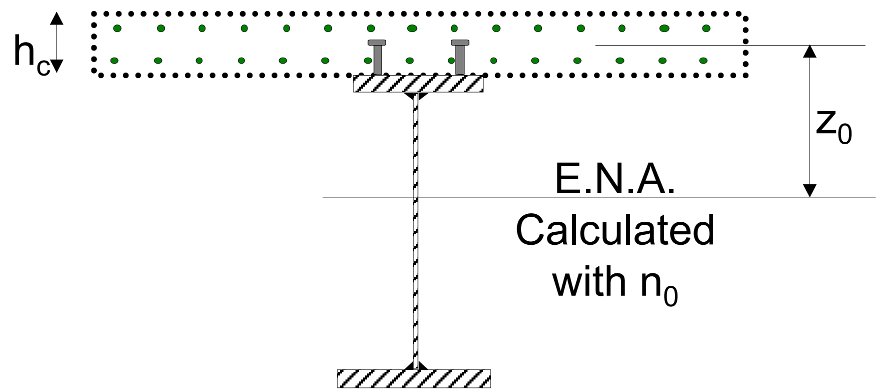
$$A_s = k_s k_c k f_{ct,eff} A_{ct} / \sigma_s$$

Where:

- $f_{ct,eff}$ is the mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur. $f_{ct,eff}$=3.2 MPa for a concrete C35/45 (EN-1992-1-1 table 3.1).
- k is a coefficient which allows for the effect of non-uniform self balanced stresses . k=0.80 (EN-1994-2, 7.4.2 (1).
- k_s is a coefficient which accounts for the effect of the reduction of the normal force of the concrete slab due to initial cracking and local slip of the shear connection. k_s=0.90 (EN-1994-2, 7.4.2 (1)).
- k_c is a coefficient which takes into account the stress distribution within the section immediately prior to cracking, and is given by:

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

- $\boldsymbol{h_c}$ is the thickness of the concrete slab, excluding any haunch or ribs. In our case $\boldsymbol{h_c}$=0.307 m
- $\boldsymbol{z_0}$ is the vertical distance between the centroid of the uncracked concrete flange, and the uncracked composite section, calculated using the modular ratio $\boldsymbol{n_0}$ for short term loading.



In our case the support P-1 cross section $\boldsymbol{z_0}$=1.027-(0.109+0.307/2)=0.764m, and at the mid span P-1/P-2 cross section , $\boldsymbol{z_0}$=0.669-(0.109+0.307/2)=0.406m

- $\boldsymbol{\sigma_s}$ is the maximum stress permitted in the reinforcement immediately after cracking. This may be taken as its characteristic yield strength $\boldsymbol{f_{sk}}$=500 MPa (according to EN-1994-2, 7.4.2).
- $\boldsymbol{A_{ct}}$ is the area of the tensile zone, caused by direct loading and primary effect of shrinkage, immediately prior to cracking of the cross section. For simplicity the area of the concrete section within the effective width may be used: 1.95 m^2.

Then:

$$k_c = \frac{1}{1+0.307/(2 \cdot 0.764)} + 0.3 = 1.13 \leq 1.0$$ for the support P-1 cross section, hence $\boldsymbol{k_c}$=1.0

$$k_c = \frac{1}{1+0.307/(2 \cdot 0.406)} + 0.3 = 1.02 \leq 1.0$$ for the mid span P-1-P-2 cross section, hence $\boldsymbol{k_c}$=1.0

$$A_{s,min} = 0.9 \cdot 1.0 \cdot 0.8 \cdot 3.2 \cdot 1.950 \cdot 10^6 / 500 = 8985.6 mm^2 = 89.85 cm^2$$

for half of slab (6 m).

As we have ϕ20/130 in the upper reinforcement level and ϕ16/130 in the lower reinforcement level: (24.166+15.466)cm^2/m·6.0m=**237.79**cm^2>>$\boldsymbol{A_{smin}}$=**89.85**cm^2, so the minimum reinforcement of the slab is verified

According to EN-1994-2, 7.4.3 (1), when the minimum reinforcement is provided, the limitation of crack widths may generally be achieved by limiting the maximum bar diameter (EN-1994-2 table 7.1), and limiting the maximum bar spacing of bar diameter (EN-1994-2 table 7.2). Both limits depend on the stress in the reinforcement σ_s and the crack width w_k.

The maximum bar diameter ϕ for the minimum reinforcement may be obtained according to EN-1994-2, 7.4.2 (2):

$$\phi = \phi^* \frac{f_{ct,eff}}{(f_{ct,0} = 2.9\ \text{MPa})}$$

EN-1994-2 table 7.1

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	-

EN-1994-2 table 7.2

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	-

The stresses in the reinforcement should be determined taking into account the effect of tension stiffening of concrete between cracks. In EN-1994-2, 7.4.3 (3) there is a simplified procedure for calculating this.

In a composite beam where the concrete slab is assumed to be cracked, stresses in reinforcement increase due to the effect of tension stiffening of concrete between cracks compared with the stresses based on a composite section neglecting concrete.

The direct tensile stress in the reinforcement σ_s due to direct loading may be calculated according to EN-1994-2, 7.4.3 (3):

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

$$\Delta\sigma_s = \frac{0.4 \cdot f_{ctm}}{\alpha_{st} \cdot \rho_s}$$ is the effect of tension stiffening of concrete between cracks, with:

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

$$\sigma_s = \sigma_{s,0} + \Delta\sigma_s \qquad \Delta\sigma_s = \frac{0.4 \cdot f_{ctm}}{\alpha_{st} \cdot \rho_s} \qquad \alpha_{st} = \frac{A \cdot I}{A_a \cdot I_a}$$

Where:

$\sigma_{s,0}$ is the stress in the reinforcement caused by the internal force acting on the composite section, calculated neglecting concrete in tension.

f_{ctm} is the mean tensile strength of the concrete. For a concrete C35/45 (EN-1992-1-1 table 3.1) f_{ctm}=3.2 MPa.

ρ_s is the reinforcement ratio, given by: $\rho_s = \frac{A_s}{A_{ct}}$

A_{ct} is the area of the tensile zone. For simplicity the area of the concrete section within the effective width may be used. In our case A_{ct}=1.95 m2.

A_s is the area of all layers of longitudinal reinforcement within the effective concrete area.

A, I are area and second moment of area, respectively, of the effective composite section neglecting concrete in tension.

A_a, I_a are area and second moment of area, respectively ,of the structural steel section.

In our case:

For the support P-1 cross-section:

$\boldsymbol{A_s}$=237.79 cm² (ϕ 20/130 + ϕ 16/130 in 6 m), hence $\rho_s = \frac{237.79 \cdot 10^{-4}}{1.95} = 0.01219 = 12.19\%$

$$\alpha_{st} = \frac{0.3543 \cdot 0.5832}{0.3305 \cdot 0.5076} = 1.232$$

Then, the effect of tension stiffening in the support P-1 cross section is

$$\Delta\sigma_s = \frac{0.4 \cdot 3.2}{1.232 \cdot 0.01219} = 85.23 Mpa$$

For the mid span cross-section P-1/P-2:

$\boldsymbol{A_s}$=185.59 cm² (ϕ 16/130 + ϕ 16/130 in 6 m), hence $\rho_s = \frac{185.59 \cdot 10^{-4}}{1.95} = 0.00952 = 0.952\%$

$$\alpha_{st} = \frac{0.1555 \cdot 0.2456}{0.1369 \cdot 0.1969} = 1.416$$

Then, the effect of tension stiffening in mid span P-1/P-2 cross section is

$$\Delta\sigma_s = \frac{0.4 \cdot 3.2}{1.416 \cdot 0.0} = 94.95 Mpa$$

6. Control of cracking for longitudinal global bending
6.3. Control of cracking under direct loading

V

As the tensile stresses in the reinforcement caused by the internal forces acting on the composite section(under the quasi-permanent combination of loads), calculated neglecting concrete in tension are:

- Support P-1 cross-section: $\sigma_{s,0}$ = 65.94 MPa
- Mid span P-1/P-2 cross-section: $\sigma_{s,0}$ = 27.45 MPa

Then the direct tensile stress in reinforcement σ_s due to direct loading (EN-1994-2, 7.4.3) are:

- Support P-1 cross-section: $\sigma_s = \sigma_{s,0} + \Delta\sigma_s$ =65.94+85.23= 151.17 Mpa
- Mid span P-1/P-2 cross-section: $\sigma_s = \sigma_{s,0} + \Delta\sigma_s$ =27.45+94.95= 122.4 MPa

6. Control of cracking for longitudinal global bending

6.3. Control of cracking under direct loading

- As both values are below 160MPa, the maximum bar spacing for the for design crack width w_k=0.3mm is 300 mm. As we have 130 mm, the maximum bar spacing is verified.
- The maximum bar diameter ϕ^* for the minimum reinforcement should be 32 mm, and $\phi = \phi^* \dfrac{f_{ct,eff}}{(f_{ct,0} = 2.9 \text{ MPa})} = 32mm \dfrac{3.2}{2.9} = 35.31mm$
- As the example verifies the minimum reinforcement, the actual maximum diameter used on the longitudinal steel reinforcement is ϕ 20, lower than the limit established by EN-1994-2 table 7.1, and the bar spacing also verifies the limits established by EN-1994-2 table 7.2, then **the crack width is controlled**.

EN-1994-2 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	-

EN-1994-2 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	-

6. Control of cracking for longitudinal global bending
6.4. Control of cracking under indirect loading

- It has to be verified that the crack widths remain below 0.3 mm using the indirect method in the tensile zones of the slab for characteristic SLS combination of actions.
- This method assumes that the stress in the reinforcement is known. But that is not true under the effect of shrinkages (drying, endogenous and thermal shrinkage).
- The following conventional calculation is then suggested:
  - We could invert the minimum reinforcement area for the composite beams given by EN-1994-2, 7.4.2 (1) $A_s = k_s k_c k f_{ct,eff} A_{ct} / \sigma_s$ to: $\sigma_s = k_s k_c k f_{ct,eff} A_{ct} / A_s$
  - Let's consider that this is the stress in the reinforcement due to shrinkage at the cracking instant.
  - In our case, for the support P-1 cross-section: $\boldsymbol{A_s}$=237.79 cm^2, and the mid-span P-1/P-2 cross-section $\boldsymbol{A_s}$=185.59 cm^2.

  This gives:

  Support P-1 cross-section:

  $\boldsymbol{\sigma_s}$=0.9x1.0x0.8x3.2x1.95/(237.79E-4)=**188.94** MPa

  Mid-span P-1/P-2 cross-section:

  $\boldsymbol{\sigma_s}$=0.9x1.0x0.8x3.2x1.95/(185.59E-4)=**242.08** MPa

6. Control of cracking for longitudinal global bending

6.4. Control of cracking under indirect loading II

High bond bars with diameter ϕ=20 mm have been chosen in the upper reinforcement layer of the slab at the support cross-section, and ϕ=16 mm at the mid- span P-1/P-2 cross section. This gives:

- Support P-1 cross-section: $\phi^{*}= \phi \cdot 2.9/3.2=18.125$ mm
- Mid-span P-1/P-2 cross-section: $\phi^{*}= \phi \cdot 2.9/3.2=14.5$ mm

The maximum reinforcement stress is obtained by linear interpolation in EN1994-2 Table 7.1:

- Support P-1 cross-section: 230.18 MPa $>\sigma_s$=188.94 MPa
- Mid-span P-1/P-2 cross-section: 255.00 MPa$>\sigma_s$=242.08 MPa

Hence both sections are verified.

EN-1994-2 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	-

Contents

7. Connection at the steel–concrete interface
7.1. Resistance of headed studs

Design shear resistance of a headed stud:

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

with:

$$P^{(1)}{}_{Rd} = \frac{0.8 \cdot f_u \cdot \dfrac{\pi \cdot d^2}{4}}{\gamma_v}$$

Design resistance when failure is due to the shear of the steel shank toe of the stud

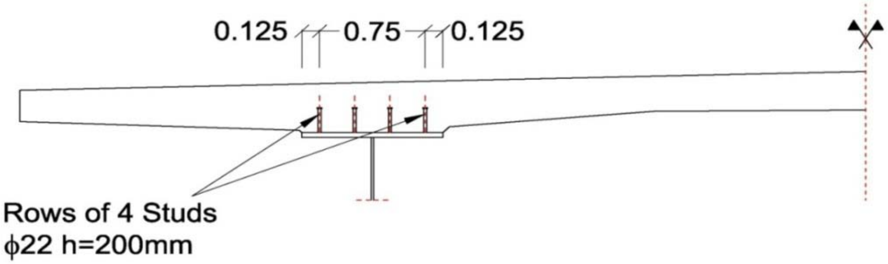
$$P^{(2)}{}_{Rd} = \frac{0.29 \cdot \alpha \cdot d^2 \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_v}$$

Design resistance when the failure is due to the concrete crushing around the shank of the stud

With: $\alpha = 0.2\left(\dfrac{h_{sc}}{d} + 1\right)$ for 3≤ h_{sc}/d ≤4 ; $\alpha = 1.0$ for h_{sc}/d >4

Where:

- γ_v partial factor. The recommended value is γ_v =1.25
- ***d*** diameter of the shank (16≤d≤25mm)
- f_u ultimate tensile strength of the material of the stud (fu≤500MPa)
- f_{ck} characteristic cylinder compressive strength of the concrete
- E_{cm} secant modulus of elasticity of concrete
- h_{sc} height of the stud.



In our case, with:

Headed stud of steel S-235-J2G3 of $\boldsymbol{d}$=22 mm, $\boldsymbol{h_{sc}}$=200 mm, and f_u=450 MPa, then:

$\boldsymbol{E_{cm}}$=22000($\boldsymbol{f_{cm}}$/10)$^{0.3}$=**34077.14** MPa (concrete C35/40)

$$P^{(1)}_{Rd} = \frac{0.8 \cdot 450 \cdot \frac{\pi \cdot 22^2}{4}}{1.25} = 0.1095 \cdot 10^6 \text{ N} = 0.1095 \text{ MN}$$

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

Note that α=1 $\frac{h_{sc}}{d} = \frac{200}{22} = 9.09 >> 4$

Then $P_{Rd} = 0.109\text{MN}$, and each row of 4 headed studs resist (ULS): $4 \cdot P_{Rd,1stud} = 0.438\text{MN}$

For <u>Serviceability State Limit</u>, the maximum longitudinal shear force per connector should not exceed $\boldsymbol{k_s \cdot P_{Rd}}$ (the recommended value for $\boldsymbol{k_s}$=0.75).

Then: $k_s \cdot P_{Rd} = 0.75 \cdot 0.1095$ MN=0.0766 MN

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

Maximum longitudinal spacing between connectors (EN-1994-2, 6.6.5.5 (3))

- To ensure a composite behaviour of the main girder

$\boldsymbol{s_{max}} \leq$ min (800 mm ; 4 $\boldsymbol{h_c}$) , with $\boldsymbol{h_c}$ the concrete slab thickness

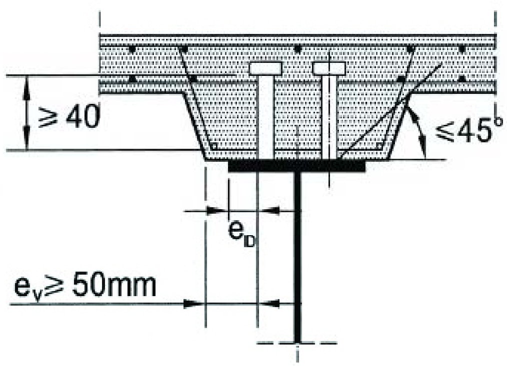
- In order to classify a compressed upper flange connected to the slab as a Class 1 / 2

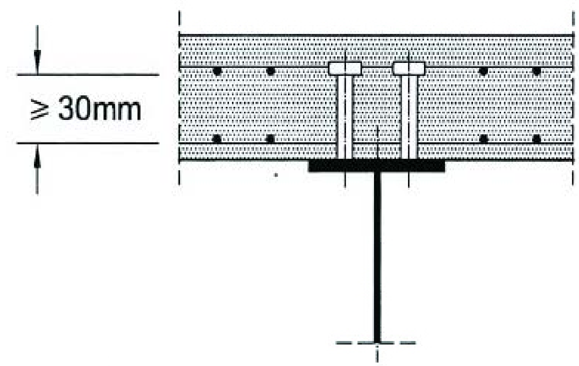
$s_{max} \leq 22 \cdot t_f \cdot \sqrt{235/fy}$ solid concrete slab and there is contact over the full length

$s_{max} \leq 15 \cdot t_f \cdot \sqrt{235/fy}$ concrete slab is not in contact over the full length

Maximum distance of shear connectors closest to the free edge of the upper flange in compression

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





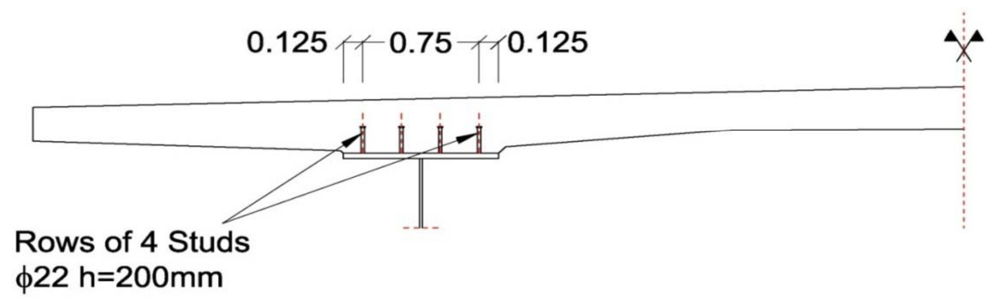
Results of applying both conditions to our case

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**	*

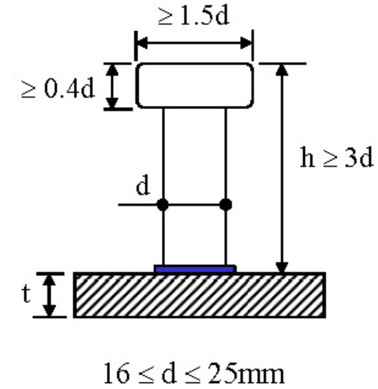
Minimum distance between the edge of a connector and the edge of a plate

$e_D \geq 25$ mm in order to ensure the correct stud welding (EN-1994-2, 6-6-5-6 (2))

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



Minimum dimensions of the headed studs (EN-1994-2 6.6.5.7 (1) and (2))



- Height of a stud $h_{sc} \geq 3 \cdot d$
- The head of the stud should have:
  Diameter $\geq 1.5 \cdot d$
  Depth $\geq 0.4 \cdot d$
- For studs welded to steel tensioned plates subjected to fatigue loading, the diameter of the stud should be (EN-1994-2, 6.6.5.7 (3))
  $$d \leq 1.5 \cdot t_f$$
- EN-1994-2, 6.6.5.7 (5) establishes that the limit for other elements than tensioned plates or webs is:
  $$d \leq 2.5 \cdot t_f$$

Minimum spacing between rows of connectors (EN-1994-2 6.6.5.6 (4))

- Longitudinal spacing $>5 \cdot d$=110 mm in our case
- Spacing in the transverse direction $>2.5 \cdot d$ in solid slabs
  $>4 \cdot d$ in other cases

With elastic behaviour, each load case produces a longitudinal shear force per unit length $\nu_{L,k}$ at the interface between concrete slab and steel.

$$\nu_{L,k} = \frac{S_c \cdot V_k}{I}$$

Where:

- $\nu_{L,k}$ is the longitudinal shear force per unit length at the interface concrete-steel
- S_c is the moment of area of the concrete slab with respect to the centre of gravity of the composite cross-section
- I is the second moment of area of the composite cross-section
- V_k is the shear force for each load case from the elastic global cracked analysis

To calculate the shear force per unit length at the interface, the cross-section properties are calculated by taking the concrete strength into account (**uncracked composite behaviour of the cross-section**), even if $M_{c,Ed}$ is negative.

7. Connection at the steel–concrete interface
7.3. Connection design for the characteristic SLS II

- The final shear force per unit length is obtained by adding algebraically the contributions of each single load case and considering the construction phases.
- The modular ratio used in $\boldsymbol{S_c}$ and ***I*** is the same as the one used to calculate the corresponding shear force contribution for each single load case.
- For SLS combination, the structure behaviour remains entirely elastic and the longitudinal global bending calculation is performed as an envelope.

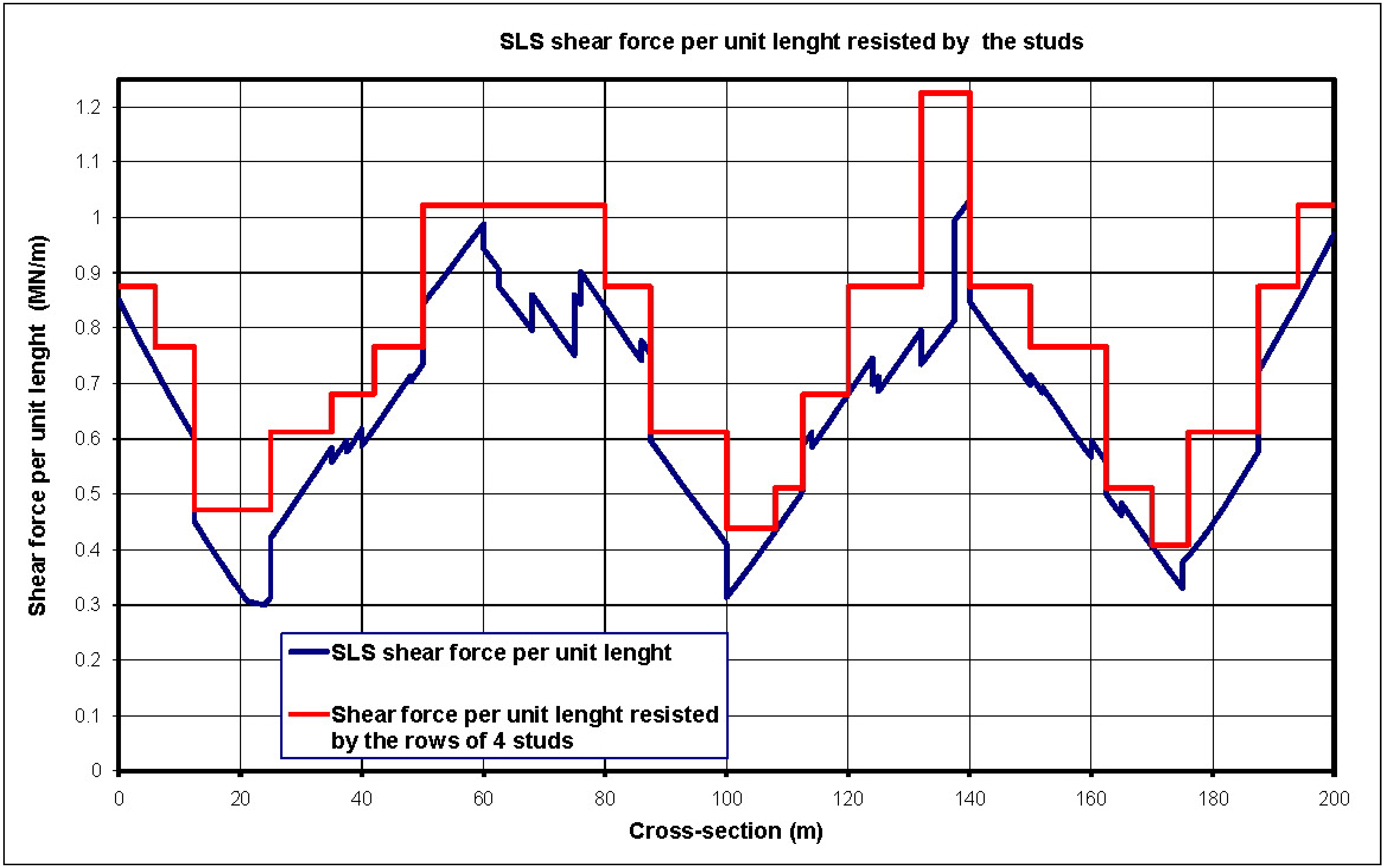
  The value of the shear force per unit length is determined in each cross-section at abscissa ***x*** by:

$$v_{L,k}(x) = \max\left[\left|v_{\min,k}(x)\right|;\left|v_{\max,k}(x)\right|\right]$$

In each cross-section there should be enough studs to resist all the shear force per unit length:

$$v_{L,k}(x) \leq \frac{N_i}{L_i} \cdot (k_s \cdot \mathrm{P}_{Rd})$$

with $\boldsymbol{k_s \cdot P_{Rd} = 4 \cdot k_s \cdot P_{Rd,\ of\ 1\ stud} = 0.3064\ MN}$ is the SLS resistance of a row of 4 headed studs.



SLS shear force per unit lenght resisted by the studs
Shear force per unit lenght (MN/m)
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
20
40
60
80
100
120
140
160
180
200
Cross-section (m)
SLS shear force per unit lenght
Shear force per unit lenght resisted by the rows of 4 studs

The design of the connection at ULS starts by:

- Elastic calculation of the shear force per unit length at the steel-concrete interface
- Elastic analysis with the cross sections properties of the uncracked section taking into account the effects of construction, as made for SLS

The shear force per unit length at ULS is therefore given by:

$$\nu_{L,Ed}(x) = \max\left[\left|\nu_{\min,Ed}(x)\right|;\left|\nu_{\max,Ed}(x)\right|\right] \quad \text{with} \quad \nu_{L,Ed} = \frac{S_c \cdot V_{Ed}}{I}$$

The number of shear connectors per unit length, constant per segment, should verify the following two criteria (EN-1994-2, 6.6.1.2(1)):

- Locally in each segment ***"i"***, the shear force per unit length should not exceed by more than 10% what the number of shear connectors per unit length can resist:

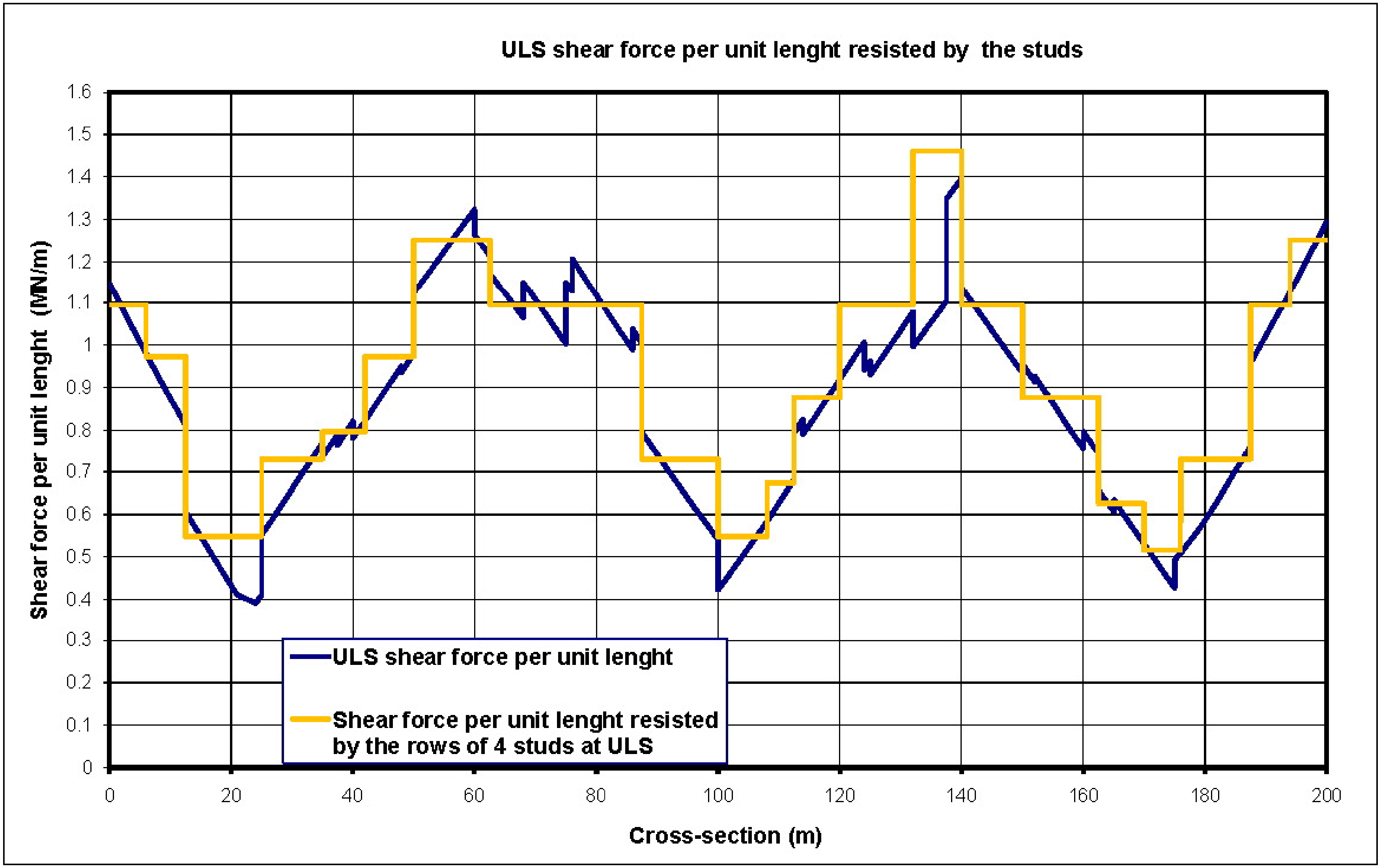
$$\nu_{L,Ed}(x) \leq 1.1 \cdot \frac{N_i}{L_i} \cdot P_{RD} \quad \text{with} \quad P_{RD} = 4 \cdot P_{RD,\text{ of } 1\,stud}$$

- Over every segment length (***L_i***), the number of shear connectors should be sufficient so that the total design shear force does not exceed the total design shear resistance:

$$\int_{x_i}^{x_{i+1}} \nu_{L,Ed}(x)\,dx \leq N_i(P_{RD}) \quad \text{with} \quad P_{RD} = 4 \cdot P_{RD,\text{ of } 1\,stud}$$

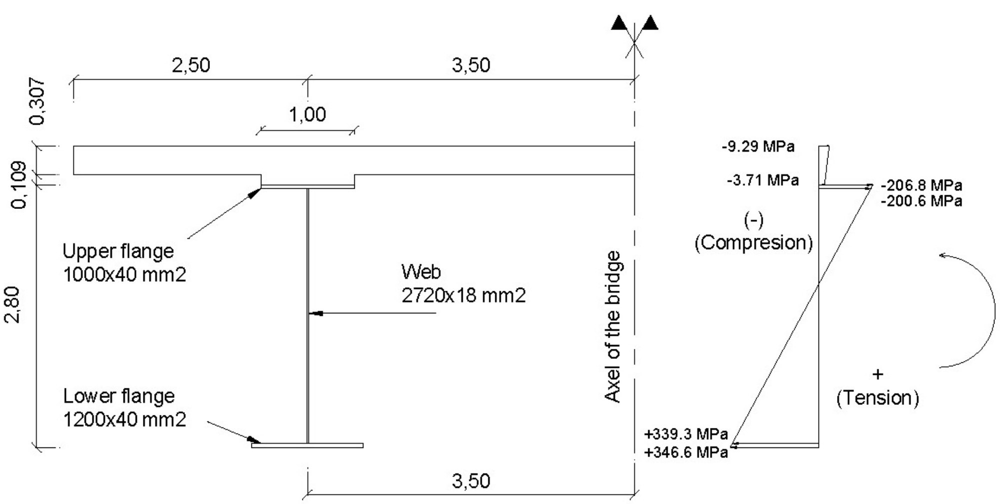
7. Connection at the steel–concrete interface
7.4. Connection design for the ULS combination

II



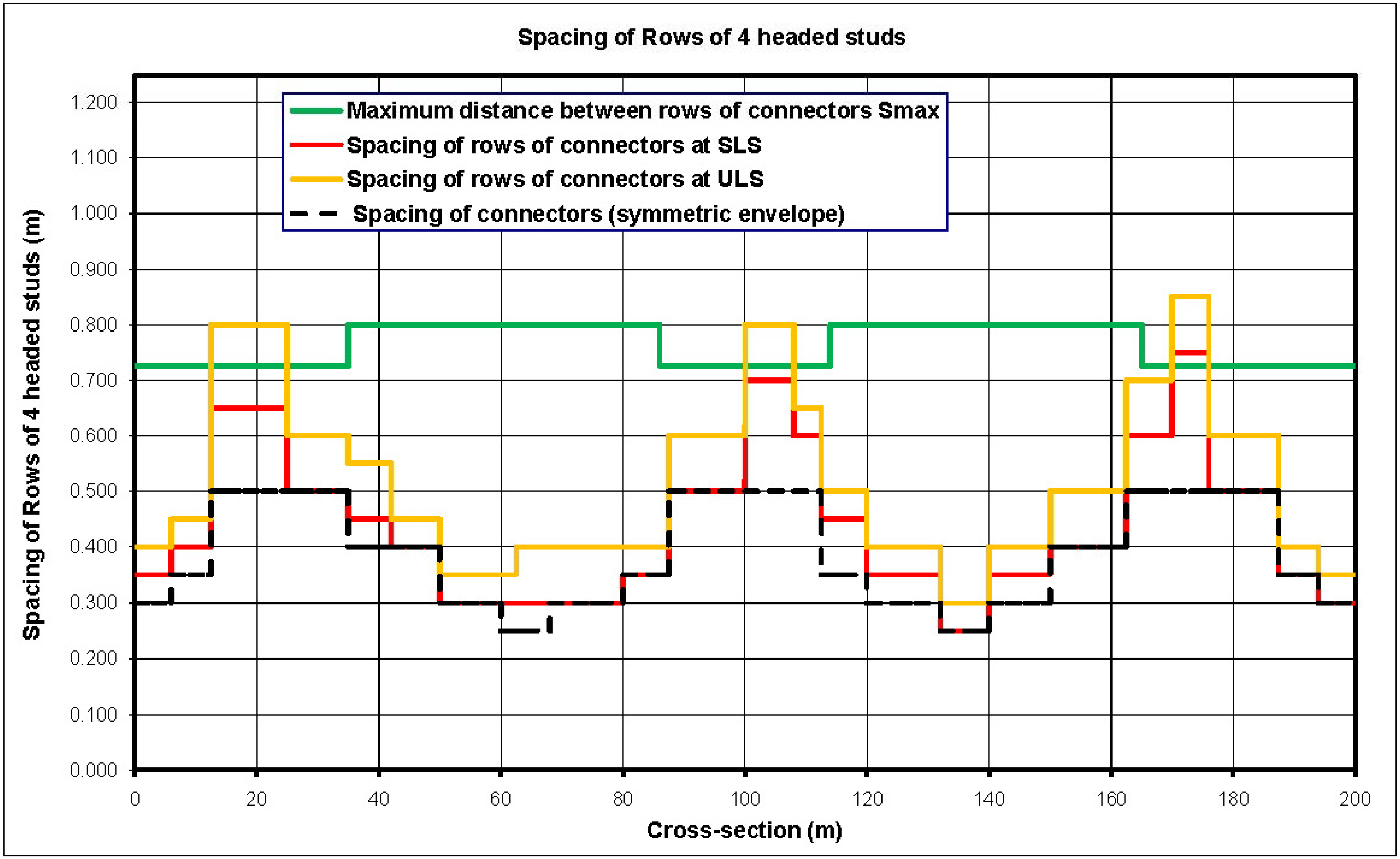
Design with plastic zones in sagging bending areas: EN-1994-2, 6.6.2.2

- If a cross-section with a positive bending moment at ULS, has partially yielded the elastic calculation for the ULS combination of actions, could be not secure.
- As far as the structure behaviour is no longer elastic, the relationship between the shear force per unit length and the global internal forces and moments is no longer linear. In a plastic zone, the shear connection is normally heavy loaded and a significant bending moment redistribution occurs between close cross-sections.



- In our case study, the mid span cross-sections are Class 1 sections, but no yielding occurs, with a medium tensile value for the lower flange of 342.9 MPa < f_y=345MPa.
- There is no need to perform the more complex calculations

7. Connection at the steel–concrete interface
7.5. Synopsis of the design example



Fatigue load model FLM3 crossing induces the stress ranges:

- $\Delta\tau$ shear stress range in the stud shank, calculated at the level of its weld on the upper structural steel flange, calculated using the uncracked cross-section mechanical properties.

  $\Delta\tau$ is thus deduced from variations in the shear force per unit length under the FLM3 crossing using the short term modular ratio n_0.

$$\Delta\tau = \frac{\Delta v_{L,FLM3}}{\left(\frac{\pi d^2}{4}\right) \cdot \frac{N_i}{L_i}}$$

- $\Delta\sigma_p$ normal stress range in the upper steel flange to which the studs are welded

Fatigue verifications for the connection in a steel flange in compression

$$\gamma_{Ff}\ \Delta\tau_E \leq \frac{\Delta\tau_c}{\gamma_{Mf,s}}$$ (EN-1994-2, 6.8.7.2 (1))

- $\Delta\tau_{E,2}$ equivalent constant range of shear stress for $2 \cdot 10^6$ cycles

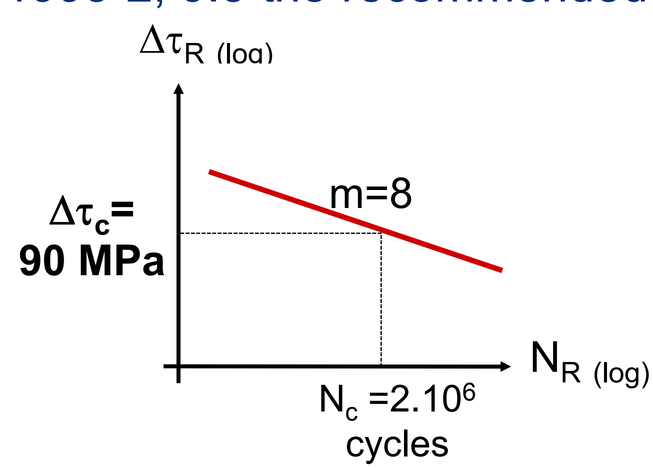
  $\Delta\tau_{E,2} = \lambda_v \cdot \Delta\tau$

- λ_v is the damage equivalent factor. For bridges $\lambda_v = \lambda_{v1} \cdot \lambda_{v2} \cdot \lambda_{v3} \cdot \lambda_{v4}$

  λ_{v1}= 1.55 for road bridges up to 100 m span.

  λ_{v2} to λ_{v4} determined according to EN-1993-2, 9,5,2 (3) to (6), with ***m=8***

- $\Delta\tau_c$ is the reference value of fatigue strength at 2 million cycles. $\Delta\tau_c$ **=90 MPa**
- γ_{Ff} is the fatigue partial factor. According to EN-1993-2, 9.3 the recommended value is γ_{Ff}=1.0
- $\gamma_{Mf,s}$ is the partial factor for verification of headed studs in bridges. Recommended value $\gamma_{Mf,s}$**=1.0** (EN-1994-2, 2.4.1.2 (6))



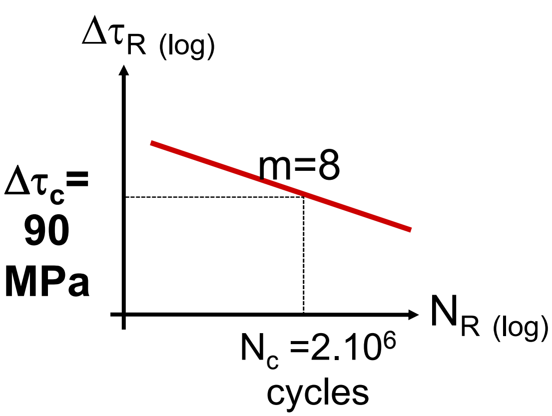
Fatigue verifications for the connection in a steel flange in tension

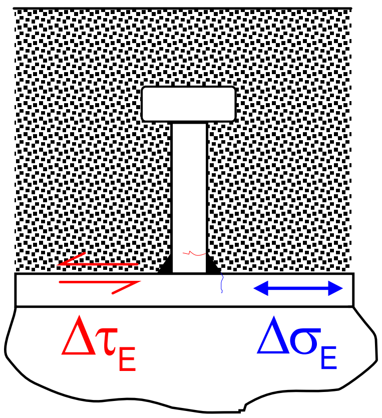
$$\frac{\gamma_{Ff}\,\Delta\sigma_E}{\Delta\sigma_c / \gamma_{Mf}} \le 1.0 \qquad \frac{\gamma_{Ff}\,\Delta\tau_E}{\Delta\tau_C / \gamma_{Mf,s}} \le 1.0 \qquad \frac{\gamma_{Ff}\,\Delta\sigma_E}{\Delta\sigma_C / \gamma_{Mf}} + \frac{\gamma_{Ff}\,\Delta\tau_E}{\Delta\tau_C / \gamma_{Mf,s}} \le 1.3$$

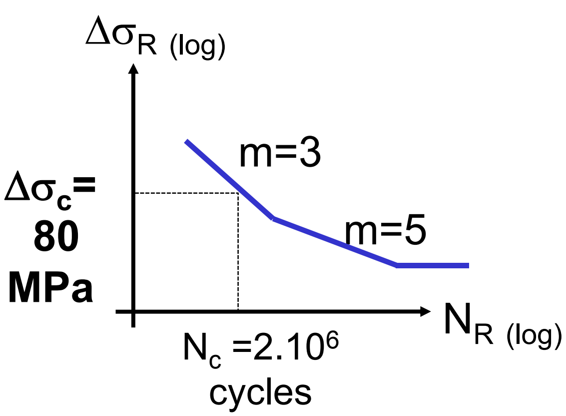
(EN-1994-2, 6.8.7.2 (2))

- $\Delta\sigma_E$ stress range for the upper steel plate (EN-1994-2, 6.8.6.1 (2))

$$\Delta\sigma_E = \lambda\phi\sigma_p = \lambda\phi\left|\sigma_{max,f} - \sigma_{min,f}\right|$$







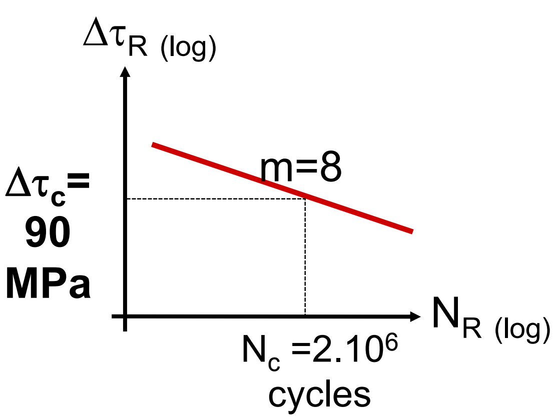
7. Connection at the steel–concrete interface
7.6. Design of the shear connection for fatigue ULS

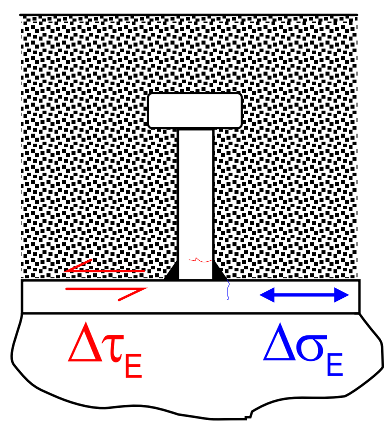
$$\frac{\gamma_{Ff}\,\Delta\sigma_E}{\Delta\sigma_c\,/\,\gamma_{Mf}} \le 1.0 \qquad \frac{\gamma_{Ff}\Delta\tau_E}{\Delta\tau_C/\gamma_{Mf,s}} \le 1.0 \qquad \frac{\gamma_{Ff}\Delta\sigma_E}{\Delta\sigma_C/\gamma_{Mf}} + \frac{\gamma_{Ff}\Delta\tau_E}{\Delta\tau_C/\gamma_{Mf,s}} \le 1.3 \qquad \Delta\sigma_E = \lambda\phi\left|\sigma_{max,f} - \sigma_{min,f}\right|$$

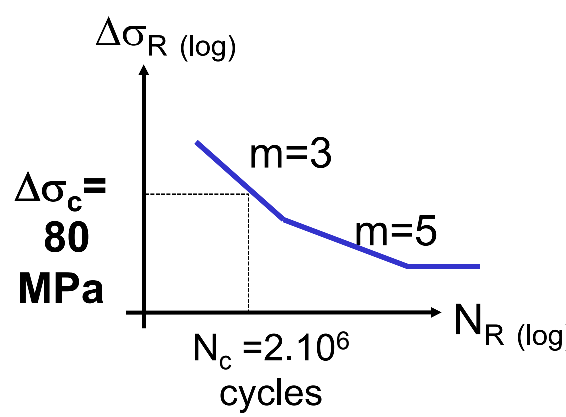
- λ is the damage equivalent factor. $\lambda=\lambda_1\cdot\lambda_2\cdot\lambda_3\cdot\lambda_4$ with ***m*=5** (EN-1993-2, 9,5,2)
- ϕ is the damage equivalent impact factor. For road bridges ϕ=1.0 (EN-1994-2, 6.8.6.1 (7)). ϕ is increased when crossing an expansion joint, (EN-1991-2. 4.6.1(6)), ϕ=1.3[1-***D***/26]≥1.0, with ***D***≤6m.
- $\sigma_{max,f}$ and $\sigma_{min,f}$: max & min stresses due to the max & min internal bending moments resulting from the combination of actions (EN-1992-1-1, 6.8.3 (3)):

$$\left(\sum_{j\ge 1} G_{k,j}\,"+"\,P\,"+"\,\psi_{1,1}Q_{k,1}\,"+"\sum_{i>1}\psi_{2,i}Q_{k,i}\right)"+"\,Q_{fat}$$

- $\Delta\sigma_c$ is the reference value of fatigue strength at 2 million cycles. $\Delta\sigma_c$ **=80 MPa**
- γ_{Mf} is the partial factor for fatigue resistance according to EN-1993-1-9(table 3.1)







7. Connection at the steel–concrete interface
7.7. Influence of shrinkage and thermal action on the connection

It is necessary to anchor the shear force per unit length coming from the isostatic (or primary) effects of shrinkage and thermal actions (EN-1994-2 6.6.2.4 (1)) at both bridge ends.

- Step 1: obtaining, in the cross-section at a distance $L_v = b_{eff}$=6 m from the free deck end (anchorage length), the normal stresses due to the isostatic effects of the shrinkage (envelope of short-term and long-term calculations) and thermal actions.
- Step 2: determining the maximum longitudinal spacing between stud rows over the length $L_v = b_{eff}$ which is necessary to resist the corresponding shear force per unit length.

In our example, the maximum longitudinal shear force at the steel/concrete interface:

- 2.15 MN under shrinkage action (long-term calculation)
- 1.14 MN under thermal actions.

This gives $V_{L,Ed}$=1.0·2.15 + 1.5·1.14 = **3.86 MN** for ULS combination of actions.

$$v_{L,Ed} = \frac{V_{L,Ed}}{b_{eff}} = 0.643 \text{ MN/m} \qquad s_{max} = \frac{4P_{Rd,\,1\,stud}}{v_{L,Ed}} = \frac{0.438}{0.643} = 0.681 \text{ m}$$

This spacing is higher than the one already obtained. Generally, the anchorage of the shrinkage and thermal actions at the free deck ends doesn’t govern the connection design

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Thank you very much for your kind attention