Master Thesis

The elevated metro structure in concrete, UHPC and composite

Appendices design study

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Figure front page: the Bangkok Mass Transit System [i6]

Appendices

Appendix A: Concrete concepts

A.1 Inverted T-beam bridge

Dimensions: ZIP1700 [21]

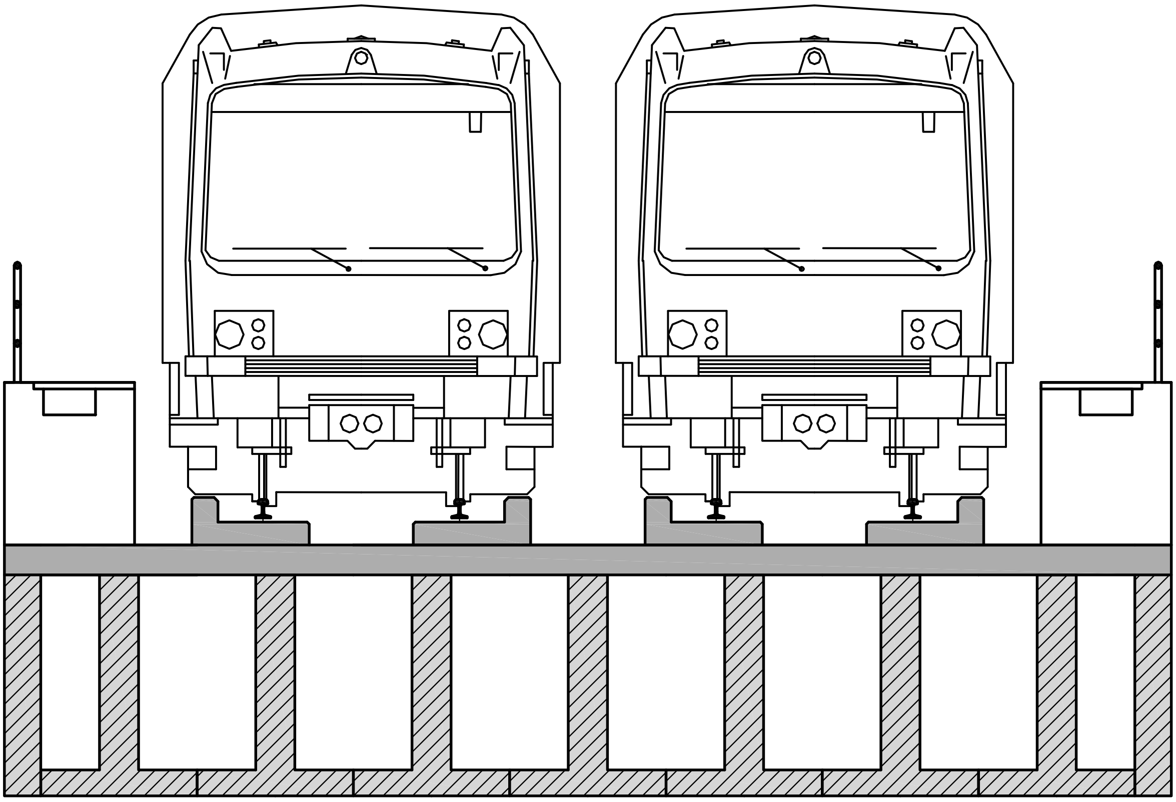

Figure 59: Cross-section superstructure with inverted T-beam bridge

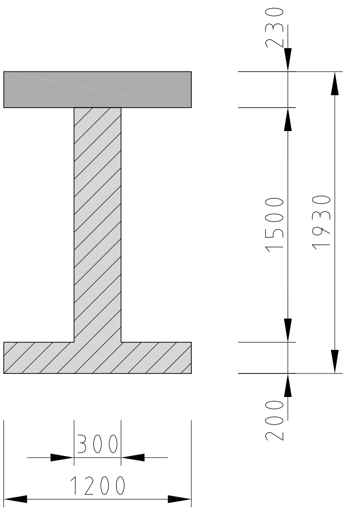

Figure 60: Dimensions inverted T-beam

A.2 Box beam bridge

Dimensions: SKK1600 [21]

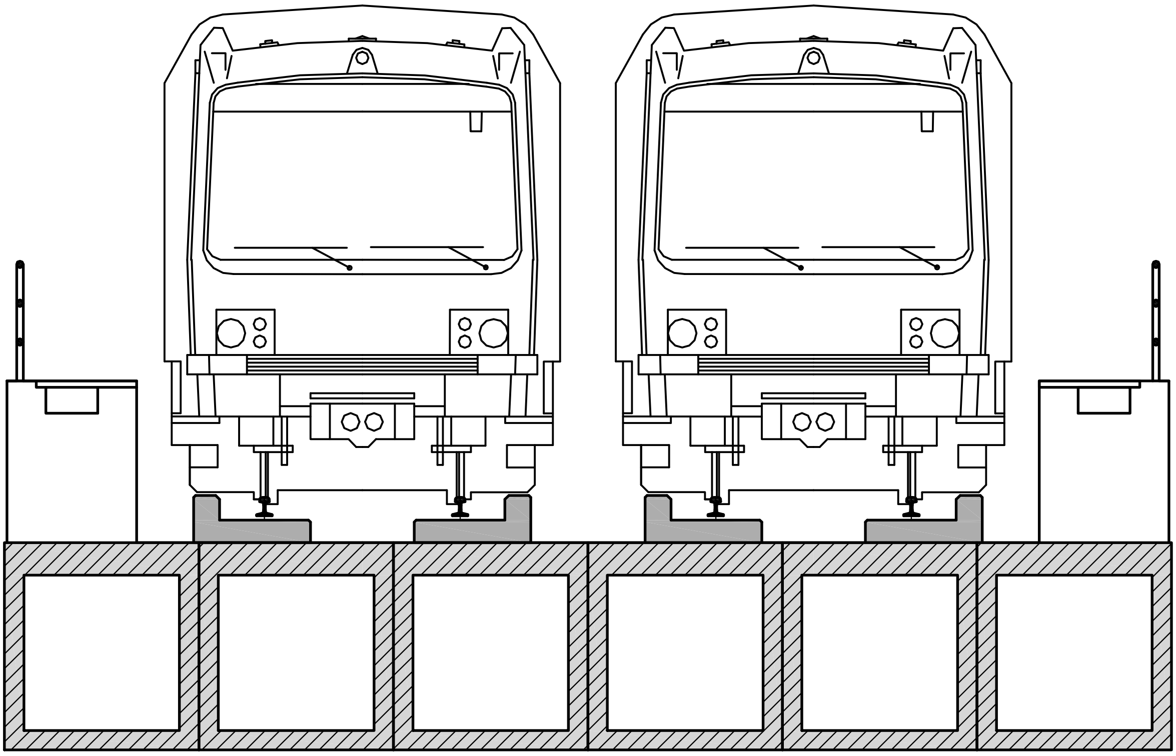

Figure 61: Cross-section superstructure with box beam bridge

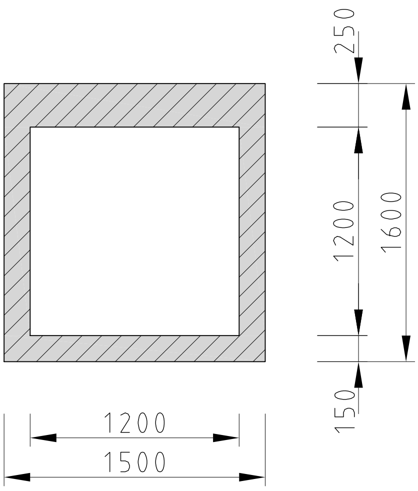

Figure 62: Dimensions box beam

A.3 Cast in-situ box girder bridge (internal prestressing)

Dimensions: according rule of thumbs [15]

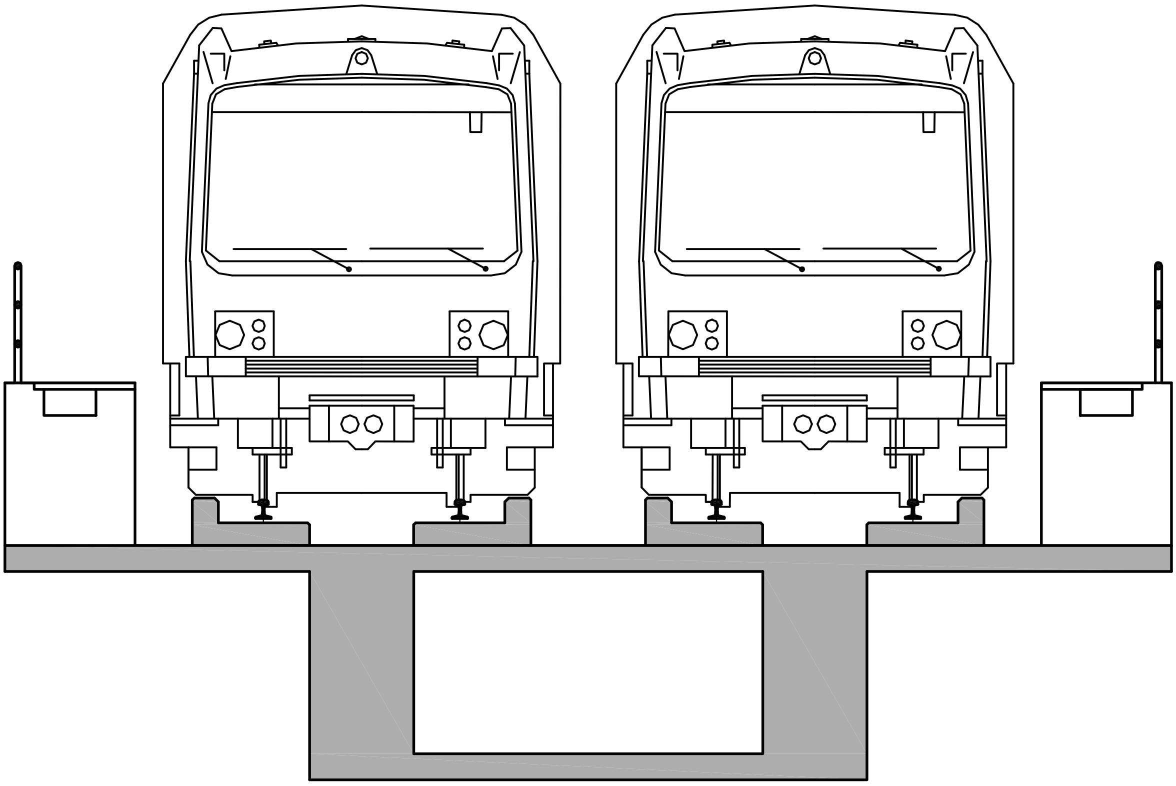

Figure 63: Cross-section superstructure with cast in-situ box girder bridge

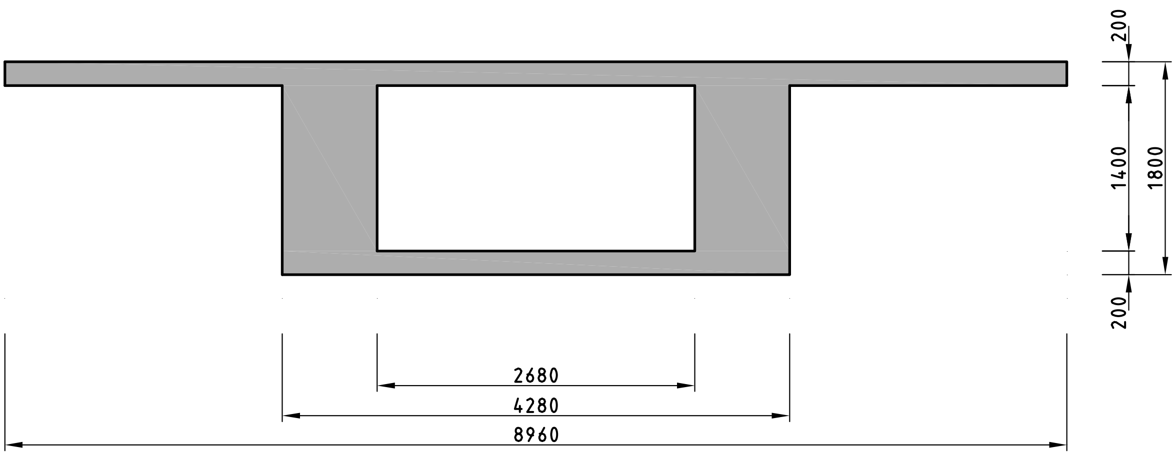

Figure 64: Dimension cast in-situ box girder

A.4 Precast segmental box girder bridge (external prestressing)

Dimensions: deduced from reference projects [6]

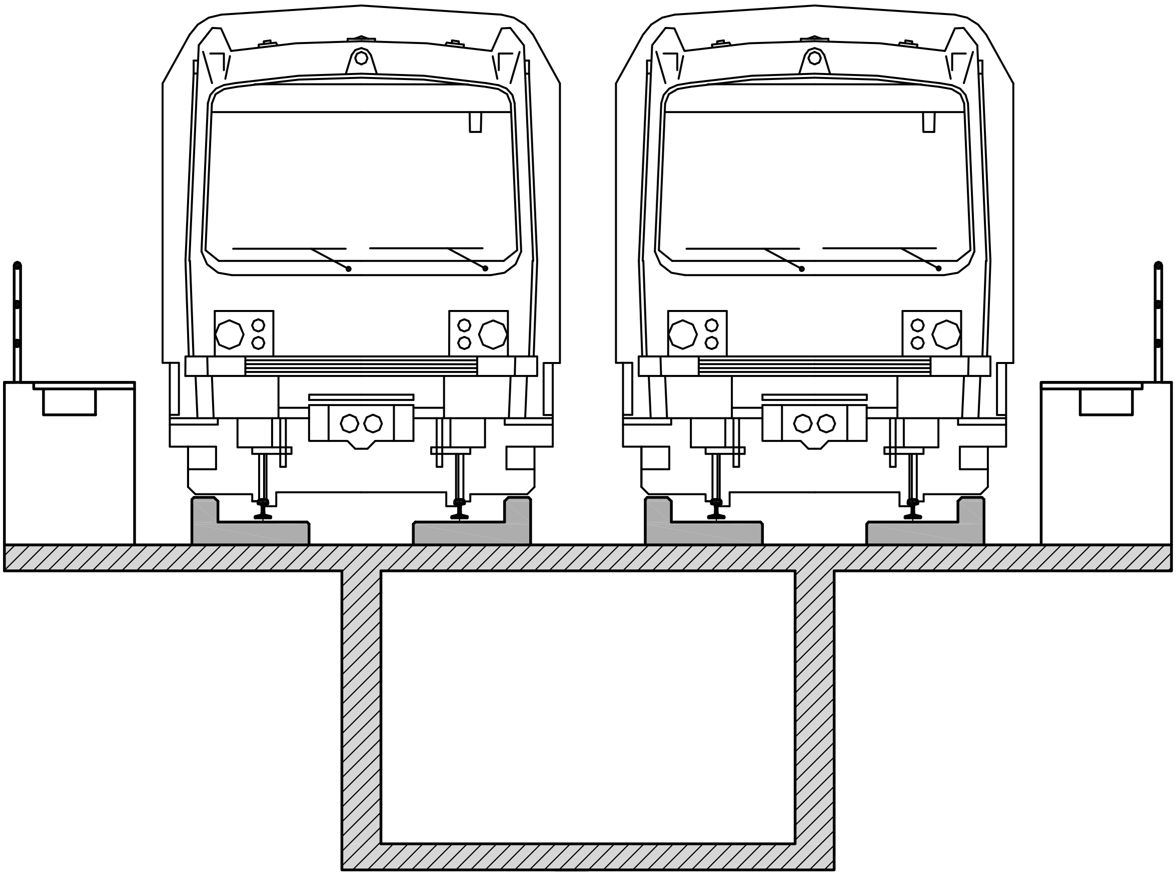

Figure 65: Cross-section superstructure with precast segmental box girder bridge

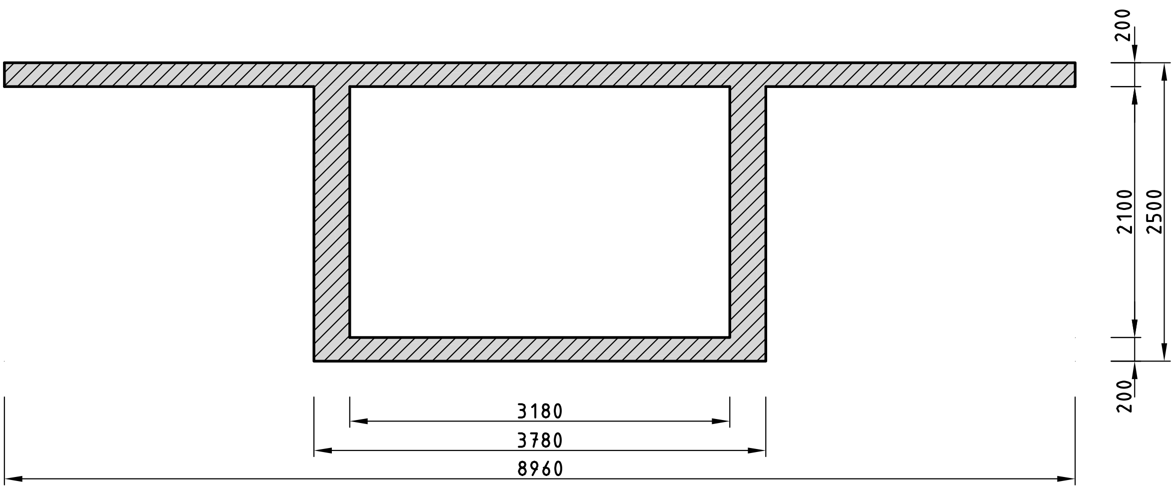

Figure 66: Dimensions precast segmental box girder

A.5 Trough bridge

Dimensions: deduced from reference projects [5]

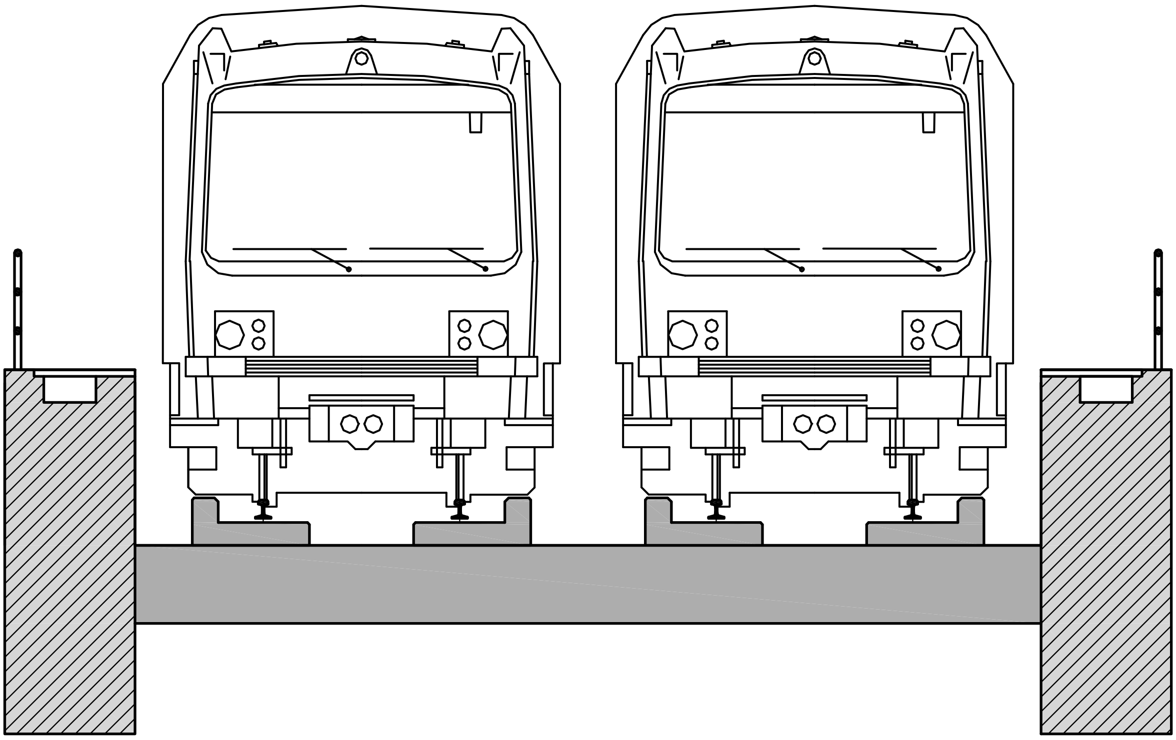

Figure 67: Cross-section superstructure with trough bridge

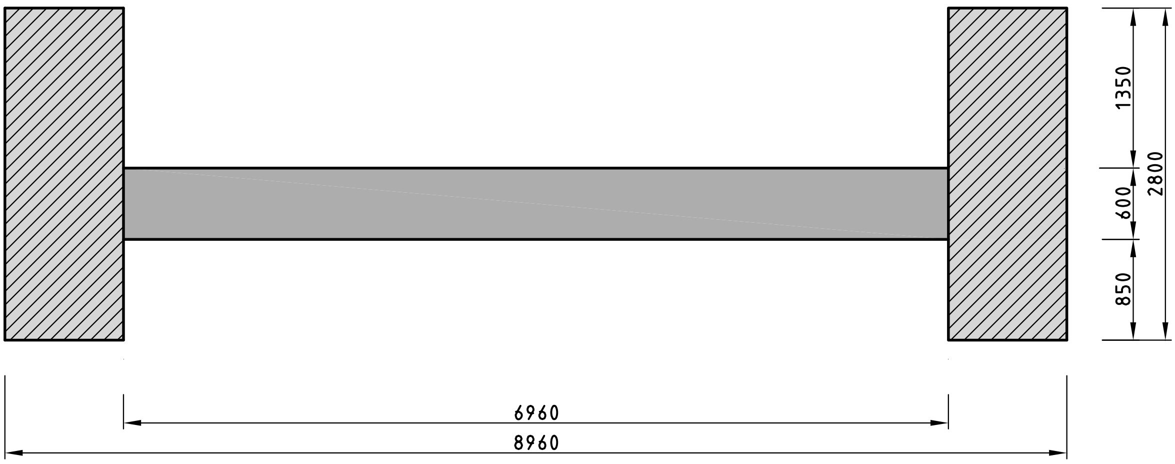

Figure 68: Dimensions trough bridge

Appendix B: Calculations concrete box girder C50/60

B.1 Introduction

This Appendix presents the calculations of the optimal box girder in concrete C50/60. First the material characteristics of the concrete and steel are described. Paragraph 3 deals with the geometry and the structural schematisation of the box girder and its characteristics. The loads to which the box girder is subjected are treated in paragraph 4. In the next paragraph the layout of the external prestressing tendons is shown and the stresses in the box girder due to loading and prestressing are calculated. It also contains the calculations of the prestressing losses. Furthermore this Appendix describes the calculations on deflection, shear + torsion and the ultimate resistance moment of the box girder in respectively the paragraphs 6, 7 and 8. The calculations on the deck thickness are treated in paragraph 9. Finally this Appendix deals with the calculations on fatigue and vibration of the box girder and buckling of the webs.

The formulas and values used in the calculations are taken from [11] and other references, which are then stated in the text.

B.2 Material characteristics

B.2.1 Concrete C50/60

Density of concrete [8]	ρ_c	2500 kg/m^3
Partial factor for concrete	γ_c	1.5
Coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied [12]	α_{cc}	0.85
Coefficient taking account of long term effects on the tensile strength and of unfavourable effects resulting from the way the load is applied.	α_{ct}	1.0
Characteristic compressive cylinder strength of concrete at 28 days	f_{ck}	50 N/mm^2
Mean value of concrete cylinder compressive strength	$f_{cm} = f_{ck} + 8$	58 N/mm^2
Mean value of axial tensile strength of concrete	$f_{ctm} = 0.30 * f_{ck}^{(2/3)}$	4.07 N/mm^2
5% fractile characteristic axial tensile strength of concrete	$f_{ctk,0.05} = 0.7 * f_{ctm}$	2.85 N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22[(f_{cm})/10]^{0.3}$	37277.87 N/mm^2
Compressive strain in the concrete at the end of the linear part	ε_{c3}	1.75 ‰
Ultimate compressive strain in the concrete	ε_{cu3}	3.5 ‰
Design value of concrete compressive strength	$f_{cd} = \alpha_{cc} f_{ck} / \gamma_c$	28.33 N/mm^2
Design value of concrete tensile strength	$f_{ctd} = \alpha_{ct} f_{ctk,0.05} / \gamma_c$	1.90 N/mm^2

B.2.2 Reinforcing steel FeB 500

Density of reinforcing steel	ρ_s	7850 kg/m^3
Characteristic yield strength of reinforcement	f_{yk}	500 N/mm^2
Partial factor for reinforcing steel	γ_s	1.15
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_s$	435 N/mm^2
Design value of modulus of elasticity of reinforcing steel	E_s	200,000 N/mm^2

B.2.3 Prestressing steel FeP 1860

Density of prestressing steel	ρ_p	7850 kg/m^3
Characteristic tensile strength of prestressing steel [24]	f_{pk}	1860 N/mm^2
Characteristic 0.1% proof-stress of prestressing steel [24]	$f_{p0.1k}$	1600 N/mm^2
Partial factor for prestressing steel	γ_s	1.15
Design tensile strength of prestressing steel	$f_{pd} = f_{p0.1k} / \gamma_s$	1391 N/mm^2
Ultimate tensile strength of prestressing steel	f_{pk} / γ_s	1617 N/mm^2
Design value of modulus of elasticity of prestressing steel	E_p	200,000 N/mm^2
Factor	k_1	0.8
Factor	k_2	0.9
Factor	k_7	0.75
Factor	k_8	0.85
Maximum tensile stress in the tendon (during tensioning)	$\sigma_{p,\max} = \min\{k_1 * f_{pk}; k_2 * f_{p0.1k}\}$	1440 N/mm^2
Maximum tensile stress in the tendon (after tensioning, initial stress at t=0)	$\sigma_{pm0} = \min\{k_7 * f_{pk}; k_8 * f_{p0.1k}\}$	1360 N/mm^2

B.3 Geometry box girder

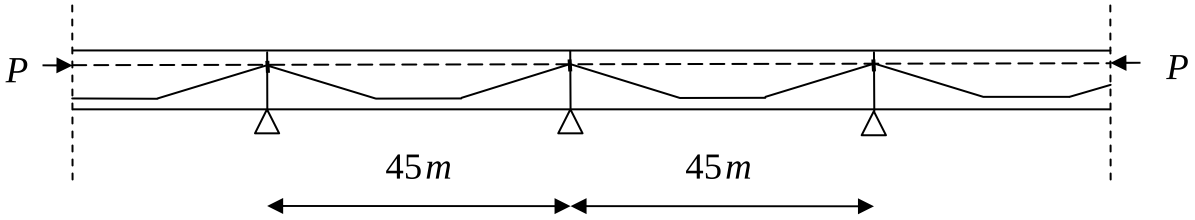

Figure 69: Statically determinate box girders supported by columns

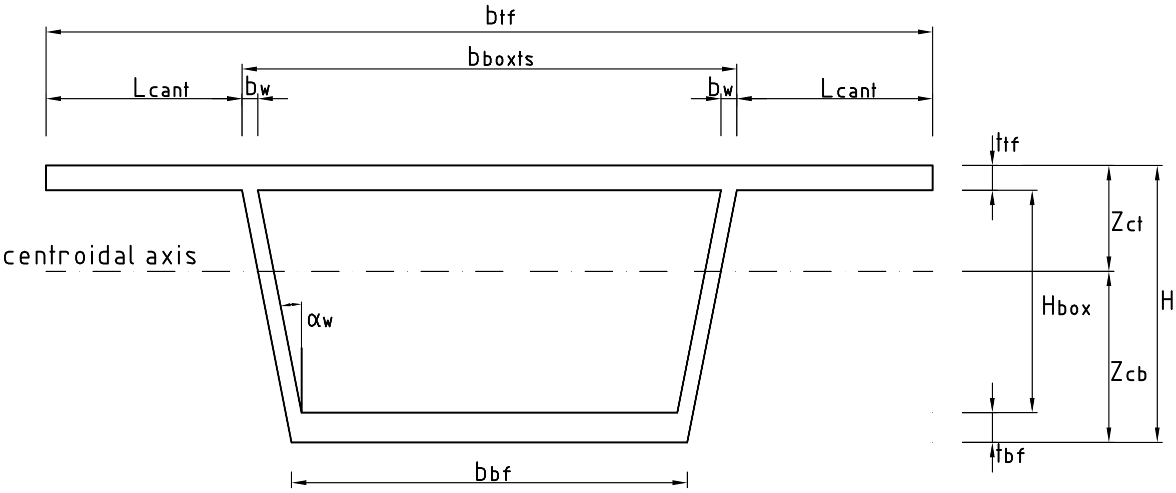

Figure 70: Cross-section of the box girder

B.3.1 General

Length span	L	45	m
Depth box girder	H	2.8	m
Width top flange	b_{tf}	8.96	m
Thickness top flange	t_{tf}	0.25	m
Width web	b_w	0.16	m
Width bottom flange	b_{bf}	4	m
Thickness bottom flange	t_{bf}	0.3	m
Width box top side	b_{boxts}	5	m
Cantilever length top flange	L_{cant}	1.98	m
Depth webs	H_{box}	2.25	m

Angle of webs with vertical axis $\alpha_w = \tan^{-1}\left(\frac{(b_{boxts} - b_{bf})/2}{H_{box} + t_{bf}}\right) = 11.1°$

B.3.2 Determination of cantilever length

Rule of thumbs to determine the top flange thickness:

$$t_{tf} = \frac{1}{10} L_{cant}$$

$$t_{tf} = \frac{1}{30} * (b_{tf} - 2(b_w + L_{cant}))$$

This gives:

$$\rightarrow \frac{1}{10} L_{cant} = \frac{1}{30} * (b_{tf} - 2(b_w + L_{cant}))$$

$$\rightarrow (\frac{1}{10} + \frac{2}{30}) * L_{cant} = \frac{1}{30} * (b_{tf} - 2 * b_w)$$

$$\rightarrow L_{cant} = \frac{6}{30} * (b_{tf} - 2 * b_w) = 1.728m$$

As the main forces act in the middle of the deck (metros) it is chosen to bring the webs more underneath the metros, see Figure 73. Therefore the following dimensions are chosen:

b_{boxts}	5	m
L_{cant}	1.98	m
b_{bf}	4	m

The width of the bottom flange b_{bf} is chosen smaller than the width of the box top side b_{boxts}. This way the railway girder requires a smaller support and the angle α_w is still small enough for the webs to transfer the vertical loads mainly by normal forces than by bending. The dimensions b_{bf} and b_{boxts} are deduced from reference projects, see Figure 18 [6].

B.3.3 Concrete cover

The concrete cover for this box girder made of C50/60 is:

$$c_{nom} = c_{\min} + \Delta c_{dev} = 40mm$$

Where:

Minimum cover

$$c_{\min} = \max\{c_{\min,b}; c_{\min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add}; 10mm\} = 35mm$$

$c_{\min,b}$ Minimum cover due to bond requirement

Minimum cover = diameter of bar = 16 mm.

$c_{\min,dur}$ Minimum cover due to environmental conditions

It is assumed that exposure class XF1 and XF3 can be classified as exposure class XD1 for the determination of the concrete cover.

With the recommended structural class S4, exposure class XD1, a design working life of 100 years, strength class C50 and an ensured special quality control of the concrete production table 4.3N and 4.4N [11] give that $c_{\min,dur} = 35mm$.

$\Delta c_{dur,\gamma}$ Additive safety element

Recommended value is 0 mm.

$c_{dur,st}$ Reduction for use of stainless steel
Recommended value is 0 mm.

$c_{dur,add}$ Reduction for use of additional protection
Recommended value is 0 mm.

Allowance in design for deviation
$\Delta c_{dev} = 5mm$
(Precast element)

B.3.4 Effective width of flanges

The effective width of the flanges is based on the distance l_0 between points of zero moment, see Figure 71. However, with a structural schematisation as given in Figure 69 the distance l_0 is 45 metres.

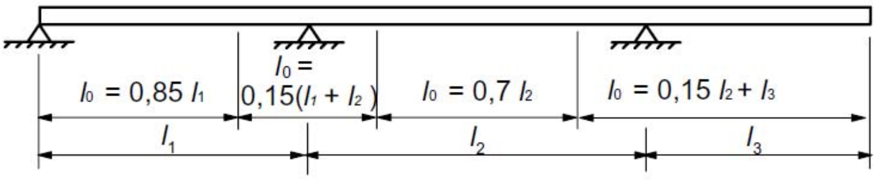

Figure 71: Definition of l_0 , for calculation of effective flange width

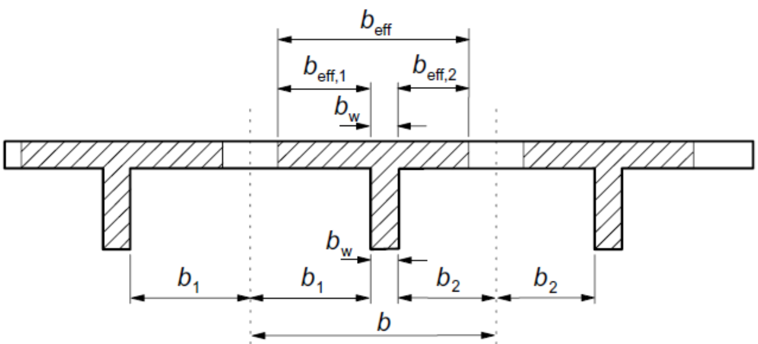

Figure 72: Effective flange width parameters

This gives:
$l_0 = 45m$

Cantilever length top flange	$b_1 = L_{cant} = 1.98m$
Width inner top flange	$b_2 = b_{boxts} / 2 - b_w = 2.34m$
Width bottom flange	$b_3 = b_{bf} / 2 - b_w = 1.84m$

Effective flange width:
$b_{eff} = \sum b_{eff,i} + b_w \leq b$

Where:
$b_{eff,i} = 0.2b_i + 0.1l_0 \leq 0.2l_0$
And
$b_{eff,i} \leq b_i$

Effective width of flanges

		Value	
Effective width cantilever length top flange	$b_{eff,1}$	1.98	m
Effective width inner top flange	$b_{eff,2}$	2.34	m
Effective width bottom flange	$b_{eff,3}$	1.84	m

Total effective flange width

		Value	
Effective width top flange	$b_{eff,t}$	8.96	m
Effective width bottom flange	$b_{eff,b}$	4	m

B.3.5 Cross-sectional properties

Cross-sectional area of concrete

$$A_c = b_{tf} * t_{tf} + 2 * b_w * H_{box} + b_{bf} * t_{bf}$$

Distance from bottom to centroidal axis

$$Z_{cb} = (b_{tf} * t_{tf} * (H - t_{tf}/2) + 2 * b_w * H_{box} * (H_{box}/2 + t_{bf}) + b_{bf} * t_{bf} * (t_{tf}/2)) / A_c$$

Distance from top to centroidal axis

$$Z_{ct} = H - Z_{cb}$$

Moment of inertia of concrete section

$$I_c = \frac{1}{12} * b_{eff,t} * t_{tf}^3 + b_{eff,t} * t_{tf} * (Z_{ct} - t_{tf}/2)^2 +$$
$$2 * \frac{1}{12} * b_w * H_{box}^3 + 2 * b_w * H_{box} * (Z_{cb} - H_{box}/2 - t_{bf})^2 +$$
$$\frac{1}{12} * b_{eff,b} * t_{bf}^3 + b_{eff,b} * t_{bf} * (Z_{cb} - t_{bf}/2)^2$$

Section modulus bottom

$$W_b = I_c / Z_{cb}$$

Section modulus top

$$W_t = I_c / Z_{ct}$$

Perimeter concrete box girder

$$u = b_{tf} + 2 * t_{tf} + 2 * L_{cant} + 2 * \sqrt{((b_{boxts} - b_{bf})/2)^2 + (H_{box} + t_{bf})^2} + b_{bf}$$

Values cross-sectional properties box girder

		Value	
Cross-sectional area of concrete	A_c	4.16	m^2
Distance from bottom to centroidal axis	Z_{cb}	1.730	m
Distance from top to centroidal axis	Z_{ct}	1.070	m
Second moment of area of the concrete section	I_c	5.387	m^4
Section modulus bottom	W_b	3.114	m^3
Section modulus top	W_t	5.036	m^3
Perimeter concrete box girder	u	22.617	m

B.4 Loads

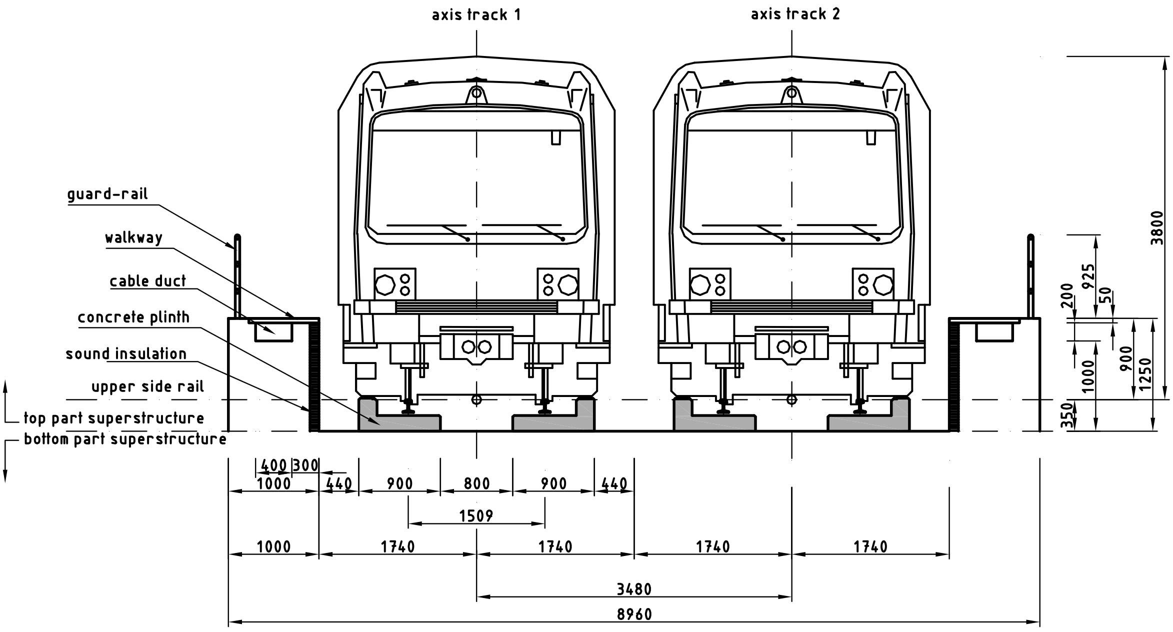

Figure 73: Cross-section top part superstructure

B.4.1 General

Acceleration due to gravity	g	9.81	m/s^2
Dynamic factor [8]	$\phi = 1 + 4/(10 + L)$	1.07	
Partial factor for permanent actions, favourable [9]	$\gamma_{G,fav}$	1.0	
Partial factor for permanent actions, unfavourable [9]	$\gamma_{G,unfav}$	1.35	
Partial factor for variable actions, favourable [9]	$\gamma_{Q,fav}$	0	
Partial factor for variable actions, unfavourable [9]	$\gamma_{Q,unfav}$	1.5	
Partial factor for prestress, favourable	$\gamma_{P,fav}$	1	
Partial factor for prestress, unfavourable	$\gamma_{P,unfav}$	1.3	
Factor for combination value of snow load [9]	$\psi_{0,snow}$	0.8	
Factor for combination value of wind load [9]	$\psi_{0,wind}$	0.75	
Factor for combination value of sideward force [9]	$\psi_{0,sidewf}$	0.8	

B.4.2 Vertical loads

Dead load box girder	$g_{dead} = A_c * \rho_c * g$	102.02	kN/m
Permanent loads [8]:			
Concrete plinths		10	kN/m per track
Rail (S49)		0.97	kN/m per track
Cables		1.2	kN/m per cable duct
Walkway + guard-rail		2	kN/m per walkway
Sound insulation		1.3	kN/m per walkway

Concrete slope (drainage between walkways)		0.5	kN/m^2
Variable loads [8]:			
Mobile load (metros)	q_{mob}	25.5	kN/m per track
Snow load	q_{snow}	0.5	kN/m^2
Concentrated load due to the metro (for local schematisation)	$Q_{mob,loc}$	130	kN per track

B.4.3 Horizontal loads

Wind load[4] [8]	q_{wind}	1.5	kN/m^2
Sideward force due to the metro[5] [8]	Q_{sidewf}	30	kN per track

B.4.4 Load schematisation in longitudinal direction

Serviceability limit state (SLS)

Vertical loads

Dead load box girder	g_{dead}	102.02	kN/m
Permanent loads:			
Concrete plinths	(* 2 tracks)	20	kN/m
Rail (S49)	(* 2 tracks)	1.94	kN/m
Cables	(* 2 cable ducts)	2.4	kN/m
Walkways + guardrails	(* 2 walkways)	4	kN/m
Sound insulation	(* 2 walkways)	2.6	kN/m
Concrete slope (drainage between walkways)	(*(b_{tf} - 2 * 1 m))	3.48	kN/m
Total permanent load	g_{perm}	34.42	kN/m
Variable loads:			
Mobile load (metros)	q_{mob} * 2 tracks * ϕ	54.71	kN/m
Snow load	$\psi_{0,snow} * q_{snow} * b_{tf}$	3.58	kN/m
Total variable load	q_{var}	58.29	kN/m

Ultimate limit state (ULS)

Vertical loads

Dead load box girder	$\gamma_{G,unfav} * g_{dead}$	137.73	kN/m
Total permanent load	$\gamma_{G,unfav} * g_{perm}$	46.47	kN/m
Total variable load	$\gamma_{Q,unfav} * q_{var}$	87.44	kN/m

[4] The viaduct is subjected to wind forces up to a height of 3.6 metres above the upper side of the rail.
[5] The sideward force acts at 1.5 metres above the upper side of the rail, in the centre of the track.

B.4.5 Load schematisation in transversal direction

Serviceability limit state (SLS)

Vertical loads

Permanent loads:			
Concrete plinths	(/ 2 plinths per track / 0.9 m (width plinth))	5.56	kN/m
Rail (S49)	(/ 2 rails per track)	0.485	kN per rail
Cables		1.2	kN per cable duct
Walkway + guardrail	(/ 1 m (width walkway))	2	kN/m
Sound insulation		1.3	kN per walkway
Concrete slope (drainage between walkways)		0.5	kN/m
Variable loads:			
Concentrated load due to the metro (for local schematisation)	$Q_{mob,loc}$ / 2 rails per track * ϕ	69.73	kN per rail
Snow load	$\psi_{0,snow} * q_{snow}$	0.4	kN/m

Horizontal loads

Wind load	$\psi_{0,wind} * q_{wind}$	1.125	kN/m^2
Sideward force due to the metro	$\psi_{0,sidewf} * Q_{sidewf}$	24	kN per track

Ultimate limit state (ULS)

Vertical loads

Permanent loads:			
Concrete plinths	5.56 * $\gamma_{G,unfav}$	7.5	kN/m
Rail (S49)	0.485 * $\gamma_{G,unfav}$	0.65	kN per rail
Cables	1.2 * $\gamma_{G,unfav}$	1.62	kN per cable duct
Walkway + guardrail	2 * $\gamma_{G,unfav}$	2.7	kN/m
Sound insulation	1.3 * $\gamma_{G,unfav}$	1.76	kN per walkway
Concrete slope (drainage between walkways)	0.5 * $\gamma_{G,unfav}$	0.68	kN/m
Variable loads:			
Concentrated load due to the metro (for local schematisation)	69.73 * $\gamma_{Q,unfav}$	104.59	kN per rail
Snow load	0.4 * $\gamma_{Q,unfav}$	0.6	kN/m

Horizontal loads

Wind load	1.125 * $\gamma_{Q,unfav}$	1.69	kN/m^2
Sideward force due to the metro	24 * $\gamma_{Q,unfav}$	36	kN per track

B.5 Prestressing tendons

B.5.1 Layout prestressing tendons

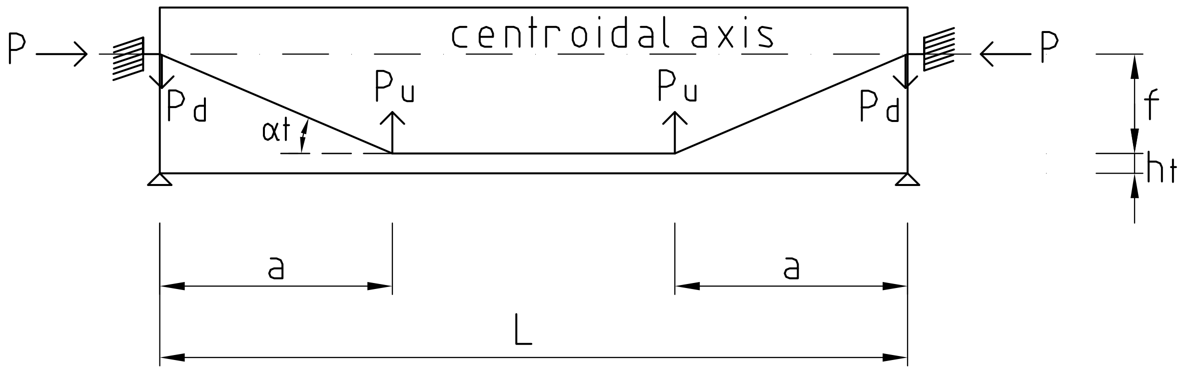

Figure 74: Layout external prestressing tendons

Distance between the centre of the tendons and bottom side at mid-span

	h_t	0.5	m
Tendon eccentricity at mid-span	$f = Z_{cb} - h_t$	1.230	m
Distance of deviation blocks to supports	a	15	m

Angle between prestressing tendon and the centroidal axis $\alpha_t = \tan^{-1}(f/a) = 4.69°$

The tendon eccentricity at the support is 0 metre as the tendon anchorage coincides with the centroidal axis.

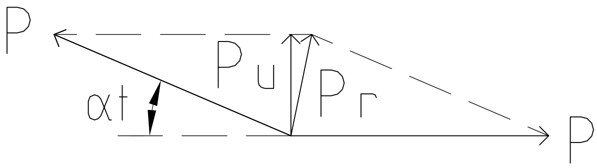

Figure 75: Polygon of prestressing forces

The resulting prestressing force $P_r = 2 * P * \sin(\alpha_t / 2)$ has a small angle with the vertical axis. As the angle is very small the horizontal force of P_r is small. For simplification reasons it is chosen to take into account only the vertical upward prestressing force. The upward prestressing force P_u is dependent of the prestressing force P and the angle a_t

$$P_u = P * \sin \alpha_t$$

B.5.2 Bending moments due to prestressing

The moment diagram and structural schematisation due to prestressing is shown in Figure 76. The elevated metro structure consists of statically determinate box girders supported by columns. Due to symmetry of loading the downward prestressing force at the supports is equal to the upward prestressing force at the deviation blocks.

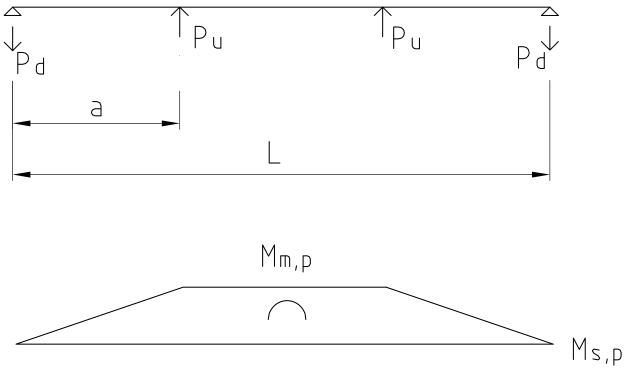

Figure 76: Structural schematisation of the box girder subjected to prestressing forces

Where:

$P_d = P_u$

$M_{s,p} = 0kNm$

$M_{m,p} = P_d * a = P_u * a$

The box girder has 6 tendons externally placed inside the girder according the layout shown in Figure 74. One tendon consists of 37 strands with a diameter of 15.7 mm and a cross-sectional area of 150 mm^2 per strand. The cross-sectional area of one tendon is:

$A_p = 37 * 150 = 5550mm^2$

The number of tendons is:

$n = 6 \ \ tendons$

The estimated prestressing losses are 20% at t = ∞
The working prestress at t = ∞ then becomes:

$\sigma_{pm\infty} = 0.8 * \sigma_{pm0} = 1088N / mm^2$

Hereunder the prestressing forces and bending moments are calculated for the two phases: the construction phase at t = 0 and the end phase at t = ∞.

Construction phase at t = 0

Total prestressing force:

$$P_0 = n * A_p * \sigma_{pm0} = 45288kN \quad (1)$$

Total upward prestressing force:

$$P_{u0} = P_0 * \sin \alpha_t = 3702kN \quad (2)$$

Bending moment between the two deviation blocks

$M_{m,p0} = P_{u0} * a = 55531kNm(\frown)$

End phase at t = ∞

The estimated prestressing losses are 20%
The working prestress at t = ∞ then becomes:

$\sigma_{pm\infty} = 0.8 * \sigma_{pm0} = 1088N / mm^2$

Total prestressing force:

$$P_\infty = n * A_p * \sigma_{pm\infty} = 36230kN \quad (3)$$

Total upward prestressing force:

$$P_{u\infty} = P_{\infty} * \sin \alpha_t = 2962kN$$

(4)

Bending moment between the two deviation blocks

$$M_{m,p\infty} = P_{u\infty} * a = 44425kNm(\cap)$$

B.5.3 Bending moments due to loads

The bending moments due to the loads are determined according the structural load schematisation shown in Figure 77.

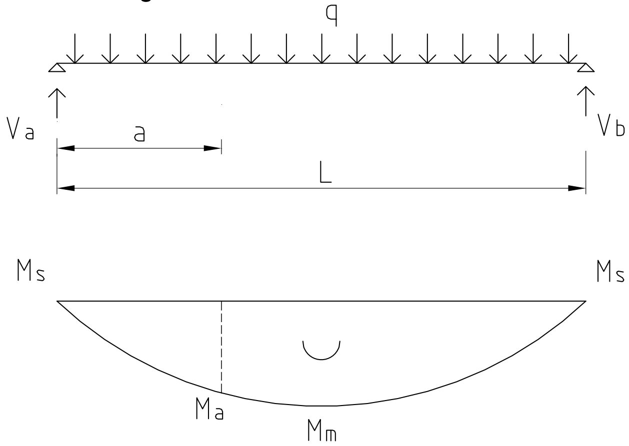

Figure 77: Structural schematisation of the box girder subjected to loads

Where:

$$V_a = V_b = \frac{1}{2} qL$$

$$M_s = 0kNm$$

$$M_m = \frac{1}{8} qL^2$$

$$M_a = \frac{1}{2} qL * a - 0.5 * q * a^2$$

Bending moments in the construction phase at t = 0

At deviation blocks

$$M_{a,0} = \frac{1}{2} * g_{dead} * L * a - 0.5 * g_{dead} * a^2 = 22955kNm(\cup)$$

At mid-span

$$M_{m,0} = \frac{1}{8} * g_{dead} * L^2 = 25825kNm(\cup)$$

Bending moments in the end phase at t = ∞

At deviation blocks

$$M_{a,\infty} = \frac{1}{2} * (g_{dead} + g_{perm} + q_{var}) * L * a - 0.5 * (g_{dead} + g_{perm} + q_{var}) * a^2 = 43816kNm(\cup)$$

At mid-span

$$M_{m,\infty} = \frac{1}{8} * (g_{dead} + g_{perm} + q_{var}) * L^2 = 49293kNm(\cup)$$

Bending moments due to the variable load
At deviation blocks

$$M_{a,v} = \frac{1}{2} * q_{\text{var}} * L * a - 0.5 * q_{\text{var}} * a^2 = 13116kNm\,(\cup)$$

At mid-span

$$M_{m,v} = \frac{1}{8} * q_{\text{var}} * L^2 = 14755kNm\,(\cup)$$

B.5.4 Stresses due to loading

As the railway girder is a prefabricated segmental box girder the joints between the segments cannot resist tensile stresses without opening of the joints. Opening of the joints is however not allowed so the concrete cannot resist tensile stresses: $\sigma_c \leq 0N/mm^2$. Furthermore the concrete stress may not become too large. In order to rule out the non-linearity of creep the concrete compressive stress should not exceed $\sigma_c \geq -0.45 * f_{ck} = -22.5N/mm^2$. Beneath the stresses at the top and bottom side of the box girder are calculated for different phases. The negative stresses refer to compression and positive stresses to tension.

Construction phase at t = 0

At deviation block, top side

$$\sigma_{ct} = -\frac{P_0}{A_c} + \frac{M_{m,p0}}{W_t} - \frac{M_{a,0}}{W_t} = -4.42N/mm^2$$

$$-22.5N/mm^2 \leq \sigma_{ct} = -4.42N/mm^2 \leq 0N/mm^2 \rightarrow Ok \qquad \textbf{(5)}$$

At deviation block, bottom side

$$\sigma_{cb} = -\frac{P_0}{A_c} - \frac{M_{m,p0}}{W_b} + \frac{M_{a,0}}{W_b} = -21.35N/mm^2$$

$$-22.5N/mm^2 \leq \sigma_{cb} = -21.35N/mm^2 \leq 0N/mm^2 \rightarrow Ok \qquad \textbf{(6)}$$

At mid-span, top side

$$\sigma_{ct} = -\frac{P_0}{A_c} + \frac{M_{m,p0}}{W_t} - \frac{M_{m,0}}{W_t} = -4.99N/mm^2$$

$$-22.5N/mm^2 \leq \sigma_{ct} = -4.99N/mm^2 \leq 0N/mm^2 \rightarrow Ok \qquad \textbf{(7)}$$

At mid-span, bottom side

$$\sigma_{cb} = -\frac{P_0}{A_c} - \frac{M_{m,p0}}{W_b} + \frac{M_{m,0}}{W_b} = -20.43N/mm^2$$

$$-22.5N/mm^2 \leq \sigma_{cb} = -20.43N/mm^2 \leq 0N/mm^2 \rightarrow Ok \qquad \textbf{(8)}$$

End phase at t = ∞ fully loaded

At deviation block, top side

$$\sigma_{ct} = -\frac{P_\infty}{A_c} + \frac{M_{m,p\infty}}{W_t} - \frac{M_{a,\infty}}{W_t} = -8.59N/mm^2$$

(9)

$$-22.5N/mm^2 \leq \sigma_{ct} = -8.59N/mm^2 \leq 0N/mm^2 \rightarrow Ok$$

At deviation block, bottom side

$$\sigma_{cb} = -\frac{P_\infty}{A_c} - \frac{M_{m,p\infty}}{W_b} + \frac{M_{a,\infty}}{W_b} = -8.90N/mm^2$$

$$-22.5N/mm^2 \leq \sigma_{cb} = -8.90N/mm^2 \leq 0N/mm^2 \rightarrow Ok \quad (10)$$

At mid-span, top side

$$\sigma_{ct} = -\frac{P_\infty}{A_c} + \frac{M_{m,p\infty}}{W_t} - \frac{M_{m,\infty}}{W_t} = -9.68N/mm^2$$

$$-22.5N/mm^2 \leq \sigma_{ct} = -9.68N/mm^2 \leq 0N/mm^2 \rightarrow Ok \quad (11)$$

At mid-span, bottom side

$$\sigma_{cb} = -\frac{P_\infty}{A_c} - \frac{M_{m,p\infty}}{W_b} + \frac{M_{m,\infty}}{W_b} = -7.15N/mm^2$$

$$-22.5N/mm^2 \leq \sigma_{cb} = -7.15N/mm^2 \leq 0N/mm^2 \rightarrow Ok \quad (12)$$

End phase at t = ∞ without variable load

At deviation block, top side

$$\sigma_{ct} = -\frac{P_\infty}{A_c} + \frac{M_{m,p\infty}}{W_t} - \frac{M_{a,\infty} - M_{a,v}}{W_t} = -5.98N/mm^2$$

$$-22.5N/mm^2 \leq \sigma_{ct} = -5.98N/mm^2 \leq 0N/mm^2 \rightarrow Ok \quad (13)$$

At deviation block, bottom side

$$\sigma_{cb} = -\frac{P_\infty}{A_c} - \frac{M_{m,p\infty}}{W_b} + \frac{M_{a,\infty} - M_{s,v}}{W_b} = -13.12N/mm^2$$

$$-22.5N/mm^2 \leq \sigma_{cb} = -13.12N/mm^2 \leq 0N/mm^2 \rightarrow Ok \quad (14)$$

At mid-span, top side

$$\sigma_{ct} = -\frac{P_\infty}{A_c} + \frac{M_{m,p\infty}}{W_t} - \frac{M_{m,\infty} - M_{m,v}}{W_t} = -6.75N/mm^2$$

$$-22.5N/mm^2 \leq \sigma_{ct} = -6.75N/mm^2 \leq 0N/mm^2 \rightarrow Ok \quad (15)$$

At mid-span, bottom side

$$\sigma_{cb} = -\frac{P_\infty}{A_c} - \frac{M_{m,p\infty}}{W_b} + \frac{M_{m,\infty} - M_{m,v}}{W_b} = -11.88N/mm^2$$

$$-22.5N/mm^2 \leq \sigma_{cb} = -11.88N/mm^2 \leq 0N/mm^2 \rightarrow Ok \quad (16)$$

B.5.5 Prestressing losses

Losses due to the instantaneous deformation of concrete

During tensioning the box girder will shorten. As the tendons are prestressed successively there arises an immediate prestressing loss which can be calculated for each tendon with the following formula:

$$\Delta P_{el} = A_p * E_p * \sum \left[\frac{j * \Delta\sigma_c(t)}{E_{cm}} \right]$$

Where:

$A_p = 5550 mm^2$	Cross-sectional area per prestressing tendon
$E_p = 200{,}000 N/mm^2$	Modulus of elasticity of prestressing steel
$E_{cm} = 37277.87 N/mm^2$	Secant modulus of elasticity of concrete
$\Delta\sigma_c(t) = \sigma_{pm0} * A_p / A_c = 1.81 N/mm^2$	Is the variation of stress in the concrete at the centre of gravity of the tendons applied at time t.
$A_c = 4160000 mm^2$	Cross-sectional area of concrete
$\sigma_{pm0} = 1360 N/mm^2$	Maximum initial tensile stress in the tendon
$j = (n-1)/2n$	Is a coefficient where n is the number of identical tendons successively prestressed.

This prestressing loss taking into account the order in which the tendons are stressed can be compensated by slightly overstressing the tendons. The maximum overstress is needed in the first prestressed tendon as this tendon has the largest loss due the instantaneous deformation of concrete. The required overstress $\sigma_{overstr}$ in the first prestressed tendon to compensate the losses due to instantaneous deformation of concrete can be calculated out of the formula below:

$$\Delta P_{el,1} = A_p * E_p * \frac{j * \Delta\sigma_{overstr}}{E_{cm}} = A_p * (\sigma_{overstr} - \sigma_{pm0}) \qquad \textbf{(17)}$$

Where:

$\Delta\sigma_{c,overstr}(t) = \sigma_{overstr} * A_p / A_c$ Variation of stress in the concrete

For the first prestressed tendon $n = 6 \rightarrow$

$j = (n-1)/2n = 5/12 = 0.4167$

Now fill in formula (17):

$$A_p * E_p * \frac{j * \Delta\sigma_{overstr}}{E_{cm}} = A_p * (\sigma_{overstr} - \sigma_{pm0}) \rightarrow E_p * \frac{j * \sigma_{overstr} * A_p}{E_{cm} * A_c} = \sigma_{overstr} - \sigma_{pm0} \rightarrow$$

$$E_p * \frac{j * A_p}{E_{cm} * A_c} = 1 - \frac{\sigma_{pm0}}{\sigma_{overstr}} \rightarrow \sigma_{overstr} = \frac{\sigma_{pm0}}{1 - E_p * \frac{j * A_p}{E_{cm} * A_c}} = 1364.07 N/mm^2$$

The maximum allowed tensile stress of the tendons during tensioning is $\sigma_{p,\max} = 1440 N/mm^2$. The stress caused by overstressing is far below this value and as also the concrete compressive stress during tensioning is limited to $\sigma_c \leq 0.6 * f_{ck} = 30 N/mm^2$ this small overstressing will not cause any problems for the structure. It can be concluded that the losses due to the instantaneous deformation of concrete can be compensated by overstressing the tendons. By overstressing the tendons the initial tensile stress in all the tendons after tensioning can be the maximum tensile stress $\sigma_{pm0} = 1360 N/mm^2$.

Losses due to friction

The loss due to friction in post-tensioned tendons is:

$$\Delta P_{\mu}(x) = P_{\max}(1 - e^{-\mu(\theta + kx)})$$

Where:

θ	Is the sum of the angular displacement over a distance x (irrespective of direction or sign).
μ	Is the coefficient of friction between the tendon and its duct.
k	Is the unintentional angular displacement for internal tendons (per unit length).
x	Is the distance along the tendon from the point where the prestressing force is equal to $P_{\max}$ (the force at the active end during tensioning).

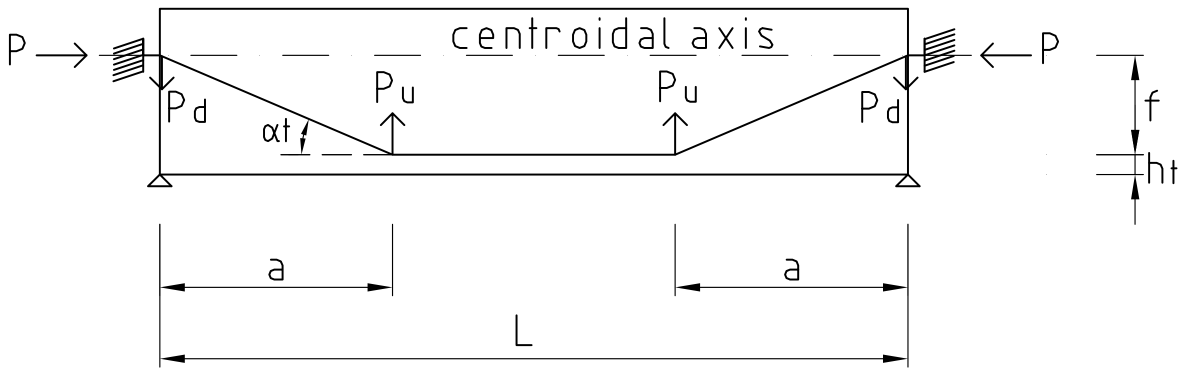

Figure 78: Layout prestressing tendons

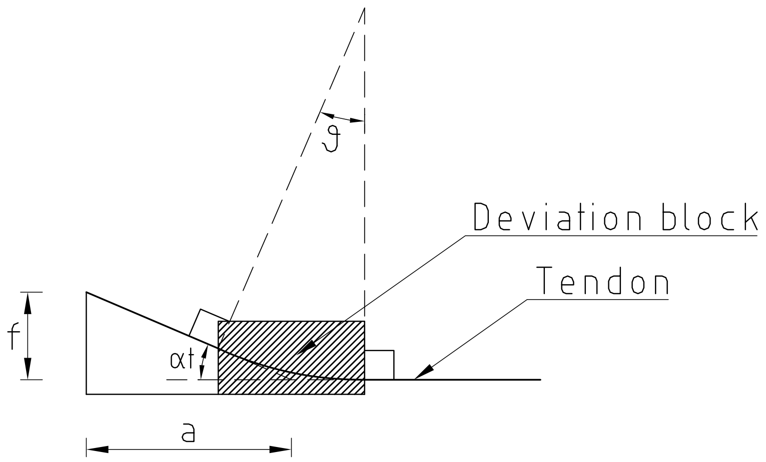

Figure 79: Angular displacement at a deviation block

There are four places where tendon deviation takes place, namely: at the two supports and at the two deviation blocks at a distance a of the supports.

The angular displacement of the tendon per deviation is: $\theta = \alpha_t = \dfrac{f}{a} = 0.08 rad$

For external tendons, the losses of prestress due to unintentional angles may be ignored [11], so the loss due to friction per deviation is:

$$\Delta P_{\mu}(x) = P_{\max}(1 - e^{-\mu * \theta}) = 369.11 kN$$

Where

$P_{\max} = P_0 = n * A_p * \sigma_{pm0} = 45288 kN$

$\mu = 0.1$	See table 5.1 [11] (external unbonded tendons; HDPE duct / lubricated; strand)

Time dependent losses of prestress for post-tensioning

The time dependent loss of prestress for post-tensioning at a location x is calculated according the formula below:

$$\Delta P_{c+s+r} = A_p \Delta\sigma_{p,c+s+r} = A_p \frac{\varepsilon_{cs} E_p + 0.8\Delta\sigma_{pr} + \frac{E_p}{E_{cm}}\varphi(\infty,t_0)*\sigma_{c,QP}}{1+\frac{E_p}{E_{cm}}\frac{n*A_p}{A_c}(1+\frac{A_c}{I_c}z_{cp}^2)[1+0.8\varphi(t,t_0)]}$$

Where:

Creep

$\varphi(\infty,t_0) = 1.2$ Is the final creep coefficient according Figure 3.1b [11] (outside conditions; C50/60; Class R; $t_0 = 28$ days; $h_0 = 2*A_c/u = 368mm$)

Shrinkage

$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} = 0.1651‰ = 0.0001651$ Is the estimated shrinkage strain in absolute value

Where

$\varepsilon_{cd}(\infty) = \beta_{ds}(\infty,t_s)*k_h*\varepsilon_{cd,0} = 0.165‰$ Is the drying shrinkage strain

$\varepsilon_{cd,0} = 0.22‰$ Is the nominal unrestrained drying shrinkage value according Table 3.2 [11] (Relative humidity 80%; C50/60)

$\beta_{ds}(t,t_s) = \frac{(t-t_s)}{(t-t_s)+0.04\sqrt{h_0^3}}$ With $t=\infty \rightarrow \beta_{ds}(\infty,t_s) = 1.0$

$k_h = 0.75$ Is a coefficient depending on the notional size h_0 according to Table 3.3 [11]

$\varepsilon_{ca}(\infty) = \beta_{as}(\infty)*\varepsilon_{ca}(\infty) = 0.0001‰$ Is the autogenous shrinkage

$\beta_{as}(t) = 1-\exp(-0.2t^{0.5})$ With $t=\infty \rightarrow \beta_{as}(\infty) = 1.0$

$\varepsilon_{ca}(\infty) = 2.5(f_{ck}-10)*10^{-6} = 0.0001‰$

Relaxation

Relaxation class 2 (wire or strand):

$$\Delta\sigma_{pr} = \sigma_{pi}*0.66*\rho_{1000}*e^{9.1\mu}\left(\frac{t}{1000}\right)^{0.75(1-\mu)}*10^{-5} = 60.93N/mm^2$$

Where:

$\sigma_{pi} = \sigma_{pm0} = 1360N/mm^2$

Relaxation class 2 $\rightarrow \rho_{1000} = 2.5\%$

$\mu = \sigma_{pi}/f_{pk} = 0.73$

$f_{pk} = 1860N/mm^2$

The long term (final) values of the relaxation losses may be estimated for a time equal to: $t = 500{,}000\,hours$.

Concrete stress

$\sigma_{c,QP}$ Is the stress in the concrete adjacent to the tendons, due to self-weight and initial prestress and other quasi-permanent actions where relevant. The value of $\sigma_{c,QP}$ may be the effect of part of self-weight and initial prestress or the effect of a full quasi-permanent combination of action $\sigma_c = \sigma_c(G + P_{m0} + \psi_2 Q)$, depending on the stage of construction considered.

This means that $\sigma_{c,QP}$ is the stress at the centroidal axis at t=0.
This gives:

$$\sigma_{c,QP} = -\frac{P_0}{A_c} = -10.89 N/mm^2$$

Where:

$$P_0 = n * A_p * \sigma_{pm0} = 45288 kN$$

Other values

$A_c = 4160000 mm^2$	Cross-sectional area of concrete
$A_p = 5550 mm^2$	Cross-sectional area per prestressing tendon
$n = 6$	Number of tendons
$E_{cm} = 37278 N/mm^2$	Secant modulus of elasticity of concrete
$E_p = 200,000 N/mm^2$	Modulus of elasticity of prestressing steel
$I_c = 5.387 * 10^{12} mm^4$	Moment of inertia of concrete section

Time dependent loss of prestress for post-tensioning at support

$$\Delta P_{c+s+r,s} = nA_p \Delta\sigma_{p,c+s+r} = nA_p \frac{\varepsilon_{cs} E_p + 0.8\Delta\sigma_{pr} + \frac{E_p}{E_{cm}} \varphi(\infty, t_0) * \sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}} \frac{n * A_p}{A_c} (1 + \frac{A_c}{I_{c,s}} z_{cp,s}^2)[1 + 0.8\varphi(t, t_0)]} = 4664 kN$$

Where:

$z_{cp,s} = 0 mm$ The tendon eccentricity at the support is 0 m as the tendons anchorage coincides with the centroidal axis.

Time dependent loss of prestress for post-tensioning at mid-span

$$\Delta P_{c+s+r,m} = nA_p \Delta\sigma_{p,c+s+r} = nA_p \frac{\varepsilon_{cs} E_p + 0.8\Delta\sigma_{pr} + \frac{E_p}{E_{cm}} \varphi(\infty, t_0) * \sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}} \frac{n * A_p}{A_c} (1 + \frac{A_c}{I_{c,m}} z_{cp,m}^2)[1 + 0.8\varphi(t, t_0)]} = 4276 kN$$

Where:

$$z_{cp,m} = f = 1230 mm$$

Total prestressing losses

The box girder segments are tensioned from one side from a practical point of view. This is because the construction of the metro system concerns a continuous placement of the segments from one column to the next column. This means that there is only one end well accessible to tension the tendons. The total prestressing losses hereby become, see Table 19:

Place	*Prestressing loss*	*Value*		*Percentage of loss*	*Value*	
At the first support	$\Delta P_{c+s+r,s} + \Delta P_{\mu}$	5033	kN	$\frac{\Delta P_{c+s+r,s} + \Delta P_{\mu}}{n * A_p * \sigma_{pm0}}$	11.11	%
After the first deviation block (at mid-span)	$\Delta P_{c+s+r,m} + 2 * \Delta P_{\mu}$	5014	kN	$\frac{\Delta P_{c+s+r,m} + 2 * \Delta P_{\mu}}{n * A_p * \sigma_{pm0}}$	11.07	%
After the second deviation block (at mid-span)	$\Delta P_{c+s+r,m} + 3 * \Delta P_{\mu}$	5384	kN	$\frac{\Delta P_{c+s+r,m} + 3 * \Delta P_{\mu}}{n * A_p * \sigma_{pm0}}$	11.89	%
At the second support	$\Delta P_{c+s+r,s} + 4 * \Delta P_{\mu}$	6141	kN	$\frac{\Delta P_{c+s+r,s} + 4 * \Delta P_{\mu}}{n * A_p * \sigma_{pm0}}$	13.56	%

Table 19: Total prestressing losses

The maximum prestressing loss arises at the end of the span, at the other end where the tensioning takes place. This loss = 13.56 % which is smaller than the assumed prestressing loss of 20 %. This assumption is thus a safe value for the prestressing losses and has not to be taken any larger. To take into account other unexpected losses and other expected losses like for instance thermal losses and slip of the anchorage it is decided to keep the expected final prestressing loss of 20 %. In the continuation of this design the prestressing loss in the end phase at t = ∞ is thus 20 %.

B.6 Deflection

The bending moments due to the loads are determined according the structural load schematisation shown in Figure 80. This schematisation means a deflection at mid-span of: $w = \frac{5}{384}\frac{qL^4}{EI}$

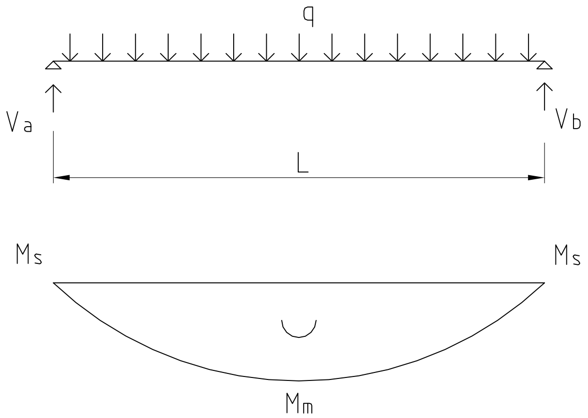

Figure 80: Structural schematisation of the box girder subjected to loads

The moment diagram and structural schematisation due to prestressing is given in Figure 81. The exact upward deflection of this schematisation is more difficult to determine. Therefore it is chosen to re-schematise the schematisation into a more easy and conservative schematisation to calculate the deflection. It can be seen that the moment diagram due to prestressing looks like the one due to the loads but then upside-down and angular. It is therefore chosen to change the structural schematisation of the box girder subjected to prestressing forces into a schematisation with a uniform distributed load like in Figure 80, but then with an upward uniform distributed load.

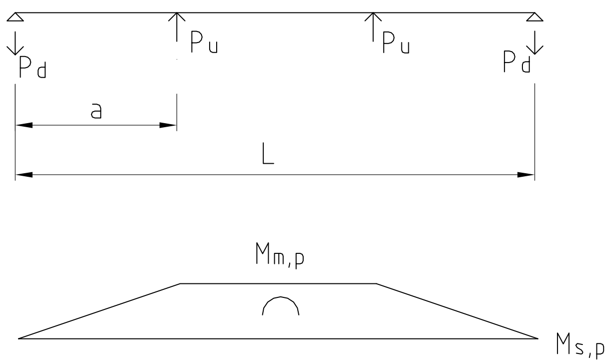

Figure 81: Structural schematisation of the box girder subjected to prestressing forces

For the new schematisation the corresponding uniform distributed load has to be determined:

At t=0:
The bending moment generated by prestressing at mid-span:
$$M_{m,p0} = P_{u0} * a = 55531kNm$$
The bending moment due to a uniform distributed load at mid-span:
$$M_m = \frac{1}{8} * q * L^2$$
For the new schematisation those two moments have to be the same value:
$$M_{m,p0} = M_m \rightarrow q_{pt0} = 219kN/m$$

At t=∞:
The bending moment generated by prestressing at mid-span:

$$M_{m,p\infty} = P_{u\infty} * a = 44425 kNm$$

The bending moment due to a uniform distributed load at mid-span:

$$M_m = \frac{1}{8} * q * L^2$$

For the new schematisation those two moments have to be the same value:

$$M_{m,p\infty} = M_m \rightarrow q_{pt\infty} = 176 kN/m$$

Notice that this new schematisation causes a smaller upward deflection than in the real schematisation. With the requirement of a limited downward deflection this verification thus becomes more conservative.

The deflection is determined with the formula:

$$w = \frac{5}{384} \frac{qL^4}{EI_c}$$

Where:

$L = 45m$	Length span
$I_c = 5.387 m^4$	Moment of inertia of concrete section
$E = E_{c,eff} = \frac{E_{cm}}{1+\varphi(\infty,t_0)} = 16944 N/mm^2$	Effective modulus of elasticity of concrete for deflection at t=0 and at t=∞ without variable load
$E = E_{cm} = 37277.87 N/mm^2$	Secant modulus of elasticity of concrete for additional deflection under mobile load
$\varphi(\infty,t_0) = 1.2$	Creep coefficient, see B.5.5: creep

The deflections and unity checks at mid-span for different phases are:

Time	*Load q*	*Deflection w*	*value*	*Maximum allowed deflection* w_{max}	*Unity check* w/w_{max}
At t=0	$g_{dead} - q_{pt0}$	-68.6	mm	$L/250 = -180mm$ annotation [6]	0.38
At t=∞ without variable load	$g_{dead} + g_{perm} - q_{pt\infty}$	-22.8	mm	$L/500 = 90mm$ annotation [7]	-0.25
Additional deflection under mobile load	q_{var}	15.5	mm	$L/1500 = 30mm$ annotation [8]	0.52
At t=∞ fully loaded	$g_{dead} + g_{perm} + q_{var} - q_{pt\infty}$	-22.8 + 15.5 = -7.3	mm	$L/500 = 90mm$ annotation [7]	-0.08

Table 20: The deflections and unity checks at mid-span for different phases

An upward deflection has a negative sign and a downward deflection has a positive sign. As the unity checks show, the construction satisfies with respect to deflection for all phases. The normative deflection is the additional deflection under mobile load.

[6] The pre-camber may not exceed $L/250$ see 7.4 [11].
[7] The final deflection may not exceed $L/500$ see 7.4 [11].
[8] The maximum deflection under mobile load is $L/1500$ [8].

B.7 Shear + torsion

B.7.1 Shear + torsion in webs

General

The webs have to resist the vertical shear and torsion. As it concerns a segmental box girder the joints between the segments consists of shear keys, see Figure 82 and Figure 83.

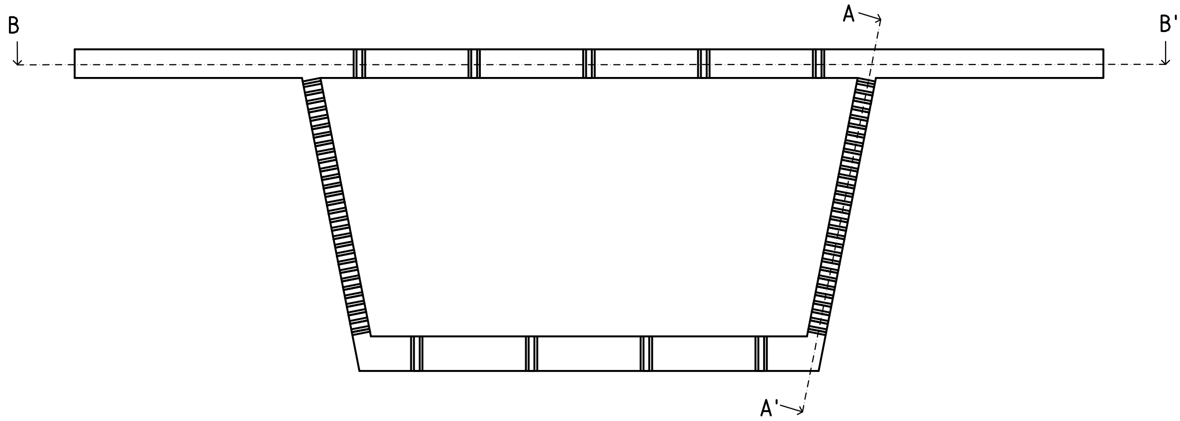

Figure 82: Shear keys in the flanges and in the webs

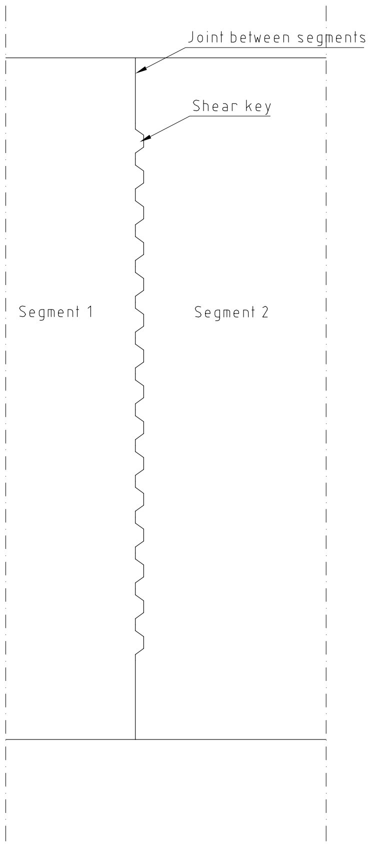

Figure 83: Section A-A'

Each web has 15 shear keys with a height H_s of 150 mm per shear key, see Figure 84. The shear force is taken by compression in the sloped part of the shear key, see Figure 85. Friction of the remaining parts of the shear keys and flanges is not taken into account.

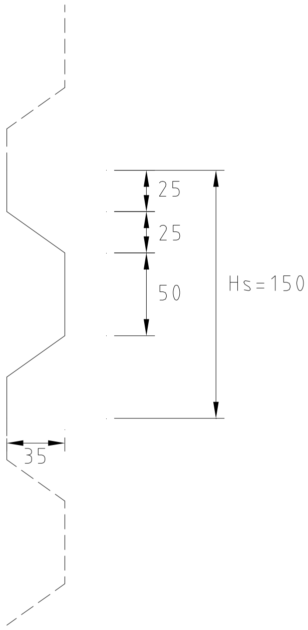

Figure 84: Dimensions shear key in mm

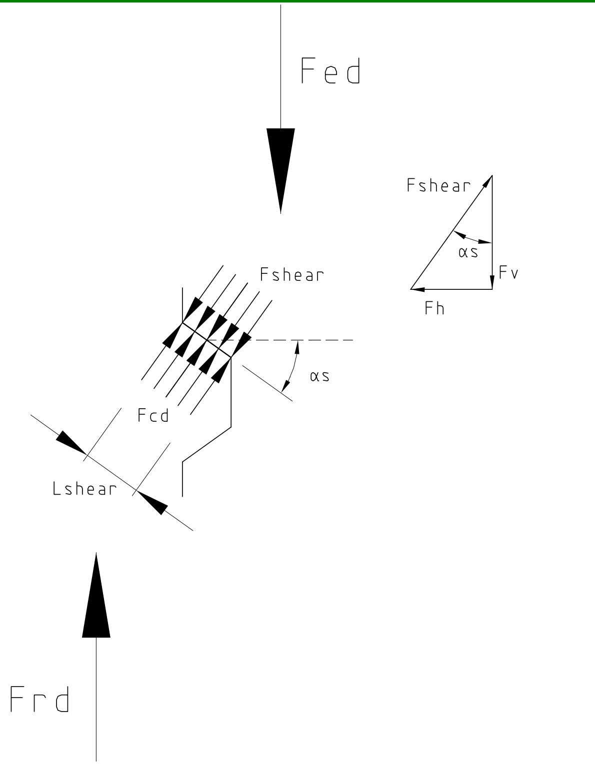

Figure 85: Schematisation of the shear resistance in a shear key by compression in the sloped part L_{shear}

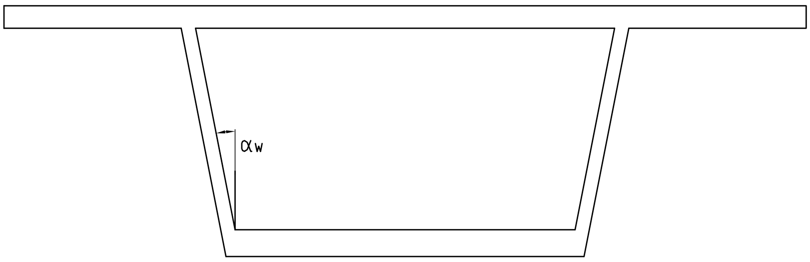

Figure 86: Angle of the webs with the vertical axis

The shear strength of the webs:
The vertical shear strength of one web is:

$$V_{Rd,1} = f_{cd} * L_{shear} * \cos\alpha_s * \cos\alpha_w * t_w * n_s = 2292kN$$

Where:

$f_{cd} = 28.33N/mm^2$	Design value of concrete compressive strength
$L_{shear} = \sqrt{25^2 + 35^2} = 43mm$	Sloped part of a shear key under compression
$\alpha_s = \tan^{-1}(25/35) = 35.5°$	Angle between shear key and beam axis
$\alpha_w = \tan^{-1}\left(\frac{(b_{boxts} - b_{bf})/2}{H_{box} + t_{bf}}\right) = 11.1°$	Angle of webs with vertical axis
$t_w = b_w * \cos\alpha_w = 157mm$	Is the thickness of the web, see Figure 70
$n_s = H_{box}/H_s = 15$	Is the number of shear keys per web
$H_s = 150mm$	Height shear key, see Figure 84

The vertical shear strength of two webs is:

$$V_{Rd,2} = 2 * V_{Rd,1} = 4584kN$$

Shear resistance at t=0 in the webs

Shear forces
The shear diagram at t=0 is shown in Figure 87.

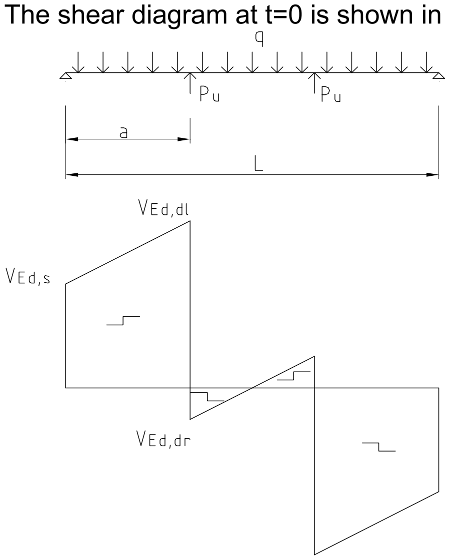

Figure 87: Shear force diagram at t=0

With:

$L = 45m$

$a = 15m$

$q = \gamma_{G,fav} * g_{dead} = 102.02kN/m$

$P_u = \gamma_{P,unfav} * P_{u0} = 4813kN$

$P_{u0} = 3702kN$ See Eq. (2)

This gives the following shear forces

$V_{Ed,dr0} = q * (L/2 - a) = 765kN$

$V_{Ed,dl0} = V_{Ed,dr0} - P_u = -4047kN$

$V_{Ed,s0} = V_{Ed,dl0} + q * a = -2517kN$

The maximum shear force is (absolute value):

$$V_{Ed,dl0} = 4047kN \tag{18}$$

Unity check
The unity check for shear in the webs at t=0 is:

$$V_{Ed,dl0} / V_{Rd,2} = 0.88 \leq 1.0 \rightarrow Ok$$

Shear and torsion resistance at t=∞ in the webs

Shear forces
The shear diagram at t=∞ is shown in Figure 88

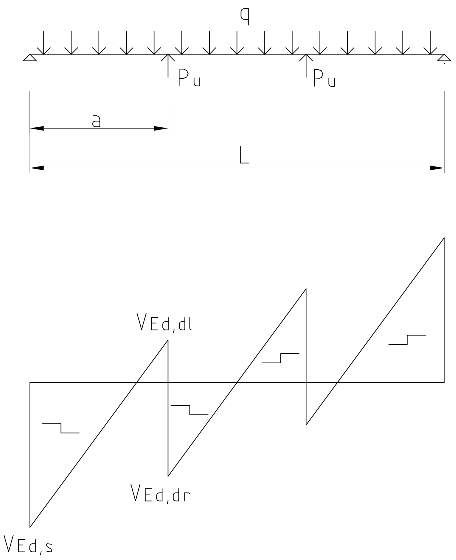

Figure 88: Shear force diagram at t=∞

With:

$L = 45m$

$a = 15m$

$q = \gamma_{G,unfav} * g_{dead} + \gamma_{G,unfav} * g_{perm} + \gamma_{Q,unfav} * q_{var} = 271.64kN/m$

$P_u = \gamma_{P,fav} * P_{u\infty} = 2962kN$

$P_{u\infty} = 2962kN$ See Eq. (4)

This gives the following shear forces

$V_{Ed,dr\infty} = q*(L/2-a) = 2037kN$

$V_{Ed,dl\infty} = V_{Ed,dr\infty} - P_u = -924kN$

$V_{Ed,s\infty} = V_{Ed,dl\infty} + q*a = 3150kN$

The maximum shear force is (absolute value):

$$V_{Ed,s\infty} = 3150kN \quad (19)$$

Torsional moment

The maximum torsional moment is a result of wind load, the sideward force of a metro and the box girder eccentrically loaded by one metro, see Figure 89 and Figure 90.

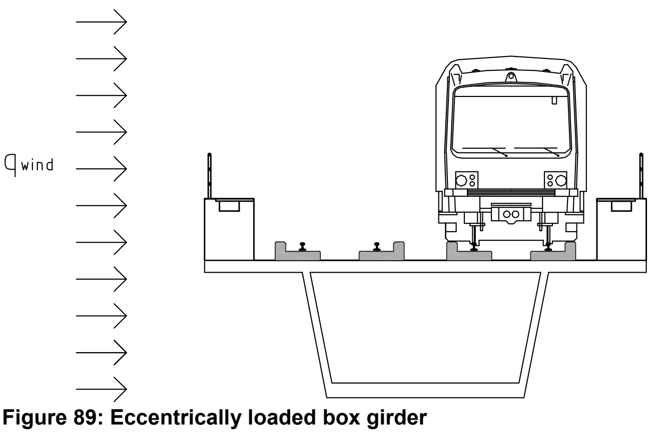

Figure 89: Eccentrically loaded box girder

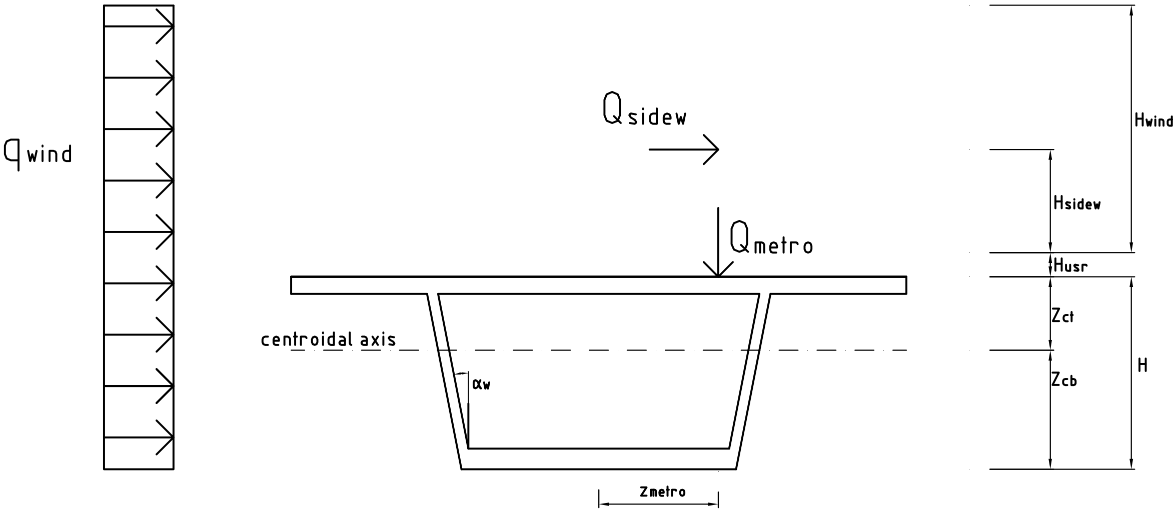

Figure 90: Load schematisation for maximum torsional moment

The maximum torsional moment is:

$$T_{Ed} = q_{wind} * L/2 * (H_{wind} + H_{usr} + Z_{ct} - Z_{cb}) * (((H_{wind} + H_{usr} + Z_{ct} - Z_{cb})/2) + Z_{cb}) + Q_{sidew} * (H_{sidew} + H_{usr} + Z_{ct}) + Q_{metro} * z_{metro} = 2133 kNm \quad (20)$$

Where:

$q_{wind} = 1.69 kN/m^2$	See B.4.5 ULS
$Q_{sidew} = 36 kN$	See B.4.5 ULS
$Q_{metro} = \gamma_{Q,unfav} * \phi * q_{mob} * L/2 = 923 kN$	See B.4.1 and B.4.2, divided by 2 as half the torsion goes to the support of one span
$H_{wind} = 3.6 m$	See B.4.3
$H_{sidew} = 1.5 m$	See B.4.3
$H_{usr} = 0.35 m$	Height upper side rail, see Figure 73
$z_{metro} = 1.74 m$	See Figure 73
$Z_{ct} = 1.07 m$	
$Z_{cb} = 1.73 m$	

The lever arm of the webs is:

$z_{webs} = (b_{boxts} + b_{bf})/2 - b_w = 4.34 m$ See Figure 70

The extra shear force in the webs due to torsion is:

$$V_{Ed+w} = T_{ed} / z_{webs} = 491 kN \quad (21)$$

Unity checks

The unity check for shear in the webs at t=∞ is:

$$V_{Ed,s\infty} / V_{Rd,2} = 0.69 \leq 1.0 \rightarrow Ok$$

The unity check for shear + torsion in the webs at t=∞ is:

$$V_{Ed,s\infty} / V_{Rd,2} + V_{ed+w} / V_{Rd,1} = 0.90 \leq 1.0 \rightarrow Ok$$

The webs satisfy with respect to shear and torsion. The shear and torsion resistance is more than what is required and friction of the remaining parts of the shear keys and flanges is not even taken along. When this verification is not satisfied, the depth of the webs H_{box} should be increased to place more shear keys in the webs. Also increasing the web thickness is an option. For this design this is however not necessary as the verification is satisfied.

B.7.2 Shear + torsion in flanges

General

The flanges have to resist the horizontal shear and torsion. As it concerns a segmental box girder the joints between the segments consists of shear keys see, Figure 91 and Figure 92.

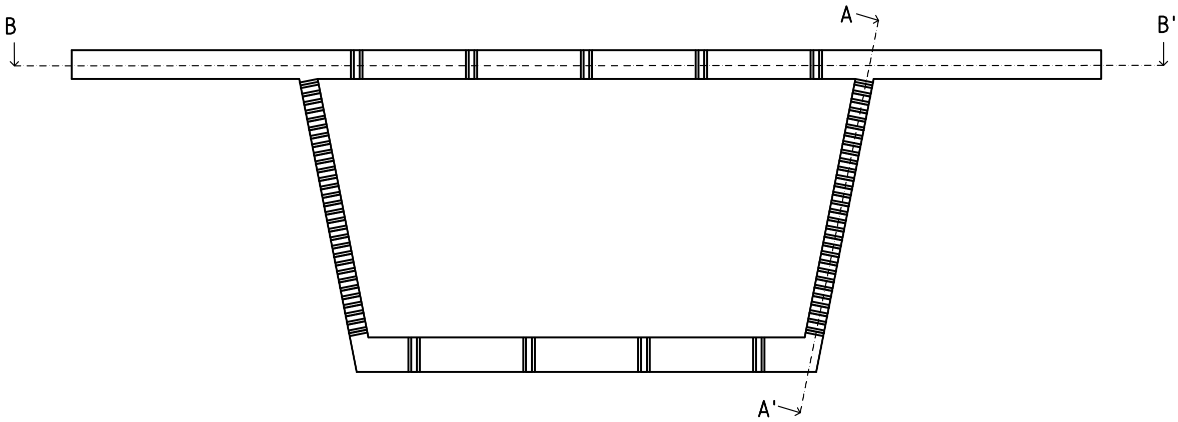

Figure 91: Shear keys in the flanges and in the webs

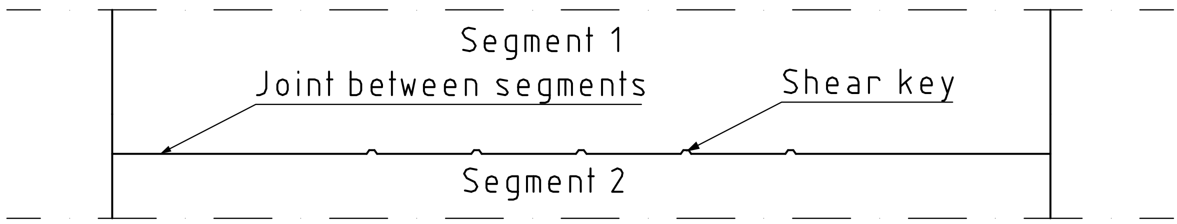

Figure 92: Section B-B'

The top flange has 5 shear keys and the bottom flange has 4 shear keys with a thickness which is the same as the flange thickness, see Figure 91. The shear force is taken by compression in the sloped part of the shear key, see Figure 94. Friction of the remaining parts of the shear keys and flanges and webs is not taken into account.

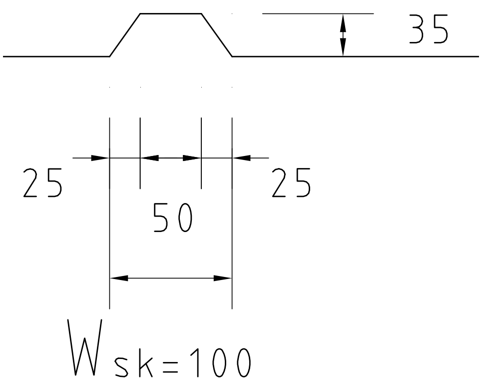

Figure 93: Dimensions shear key in mm

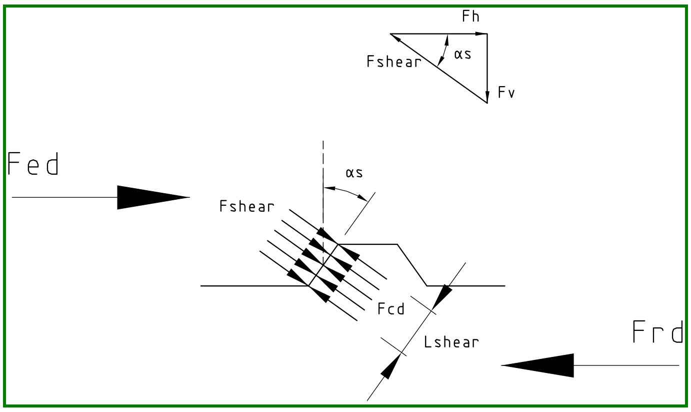

Figure 94: Schematisation of the shear resistance in a shear key by compression in the sloped part L_{shear}

The shear strength of the flanges:
The horizontal shear strength of the top flange is:

$$V_{Rd,tf} = f_{cd} * L_{shear} * \cos\alpha_s * t_{tf} * n_{s,tf} = 1240kN$$

The horizontal shear strength of the bottom flange is:

$$V_{Rd,bf} = f_{cd} * L_{shear} * \cos\alpha_s * t_{bf} * n_{s,bf} = 1190kN$$

Where:

$f_{cd} = 28.33N/mm^2$	Design value of concrete compressive strength
$L_{shear} = \sqrt{25^2 + 35^2} = 43mm$	Sloped part of a shear key under compression
$\alpha_s = \tan^{-1}(25/35) = 35.5°$	Angle between shear key and beam axis
$t_{tf} = 0.25m$	Is the thickness of the top flange
$t_{bf} = 0.3m$	Is the thickness of the bottom flange
$n_{s,tf} = 5$	Is the number of shear keys in the top flange
$n_{s,bf} = 4$	Is the number of shear keys in the bottom flange

Shear resistance at t=∞ in the flanges

Shear forces in the flanges

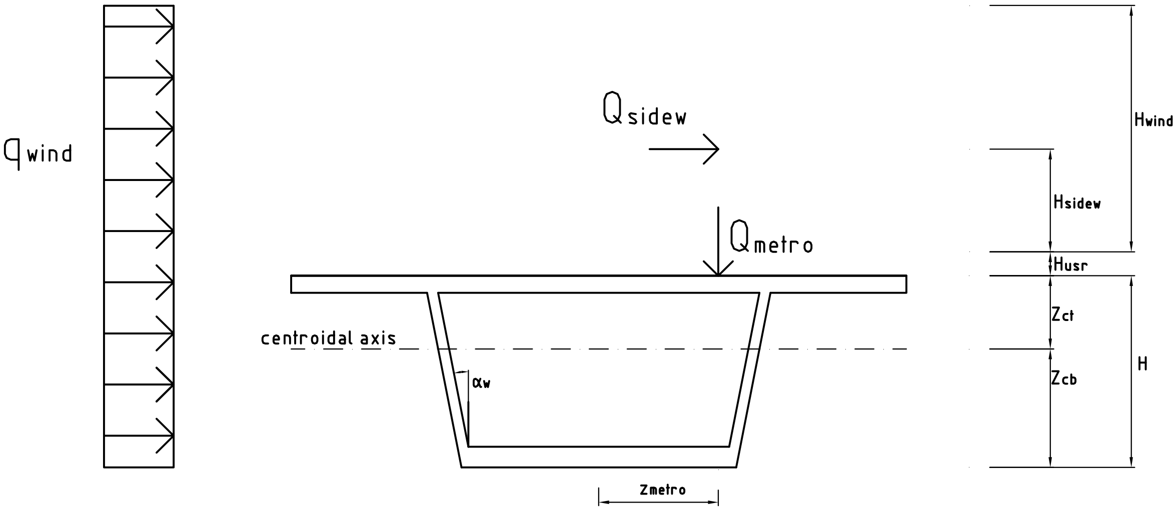

Figure 95: Load schematisation for maximum torsional moment

The shear force in the top flange is:

$$V_{Ed,tf\infty} = q_{wind} * L/2*(H_{wind} + H_{usr} + H/2) + Q_{sidew} = 239kN$$

The shear force in the bottom flange is:

$$V_{Ed,bf\infty} = q_{wind} * L/2*H/2 = 53kN$$

Where:

$q_{wind} = 1.69kN/m^2$	See B.4.5 ULS
$Q_{sidew} = 36kN$	See B.4.5 ULS
$H_{wind} = 3.6m$	See B.4.3
$H_{sidew} = 1.5m$	See B.4.3
$H_{usr} = 0.35m$	See Figure 73

Torsional moment

The maximum torsional moment is a result of wind load, the sideward force of a metro and the box girder eccentrically loaded by one metro, see Figure 95.

The lever arm of the flanges is:

$z_f = H + t_{tf}/2 + t_{bf}/2 = 2.53m$ See Figure 70

The extra shear force in the flanges due to torsion is:

$$V_{Ed+f} = T_{ed}/z_f = 845kN$$

Where:

$T_{Ed} = 2133kNm$ See Eq. (20)

Unity checks

Top flange
The unity check for shear in the top flange at t=∞ is:

$$V_{Ed,tf\infty} / V_{Rd,tf} = 0.19 \leq 1.0 \rightarrow Ok$$

The unity check for shear + torsion in the top flange at t=∞ is:

$$V_{Ed,tf\infty} / V_{Rd,tf} + V_{Ed+f} / V_{Rd,tf} = 0,87 \leq 1.0 \rightarrow Ok$$

Bottom flange
The unity check for shear in the bottom flange at t=∞ is:

$$V_{Ed,bf\infty} / V_{Rd,bf} = 0.04 \leq 1.0 \rightarrow Ok$$

The unity check for shear + torsion in the bottom flange at t=∞ is:

$$V_{Ed,bf\infty} / V_{Rd,bf} + V_{Ed+f} / V_{Rd,bf} = 0,75 \leq 1.0 \rightarrow Ok$$

The flanges satisfy with respect to shear and torsion. The shear and torsion resistance is not much more than what is required. Friction of the remaining parts of the shear keys and flanges is however not even taken along. When this verification is not satisfied, more shear keys should be placed in the flanges. As the flanges offer enough space for additional shear keys this verification will never be normative for the design and will easily satisfy.

B.8 Ultimate resistance moment

B.8.1 General

In all phases during the lifetime of the box girder the concrete force N_c due to the compressive stresses in the concrete should balance the prestressing force P , see Figure 96.

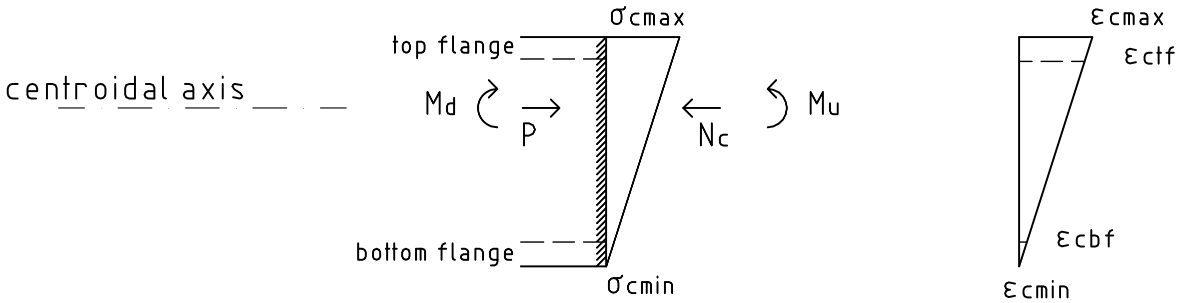

Figure 96: Equilibrium between axial forces P and N_c in the cross-section of the box girder

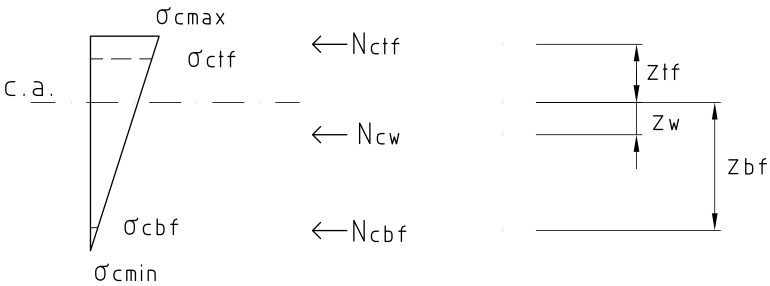

Figure 97: Overview for the calculation of the ultimate resistance moment

At the same time the bending moment M_d due to loading should be resisted by the ultimate resistance moment M_u of the box girder. The ultimate resistance moment arises when the strain difference between the top and bottom flange is as large as possible taking into account that tensile stresses are not allowed. This means that $\sigma_{c\min} = 0N/mm^2$. In which flange the maximum strain arises depends on the stage of loading. For the example given above it would mean that:
The concrete force $N_c = N_{ctf} + N_{cw} + N_{cbf}$ and should be equal to P , see Figure 97.

The ultimate resistance moment $M_u = N_{ctf} * z_{tf} + N_{cw} * z_w + N_{cbf} * z_{bf}$ and should be larger than the bending moment M_d . Where z_{tf}, z_w *and* z_{bf} are positive or negative values considering the location of the force with regard to the centroidal axis.

For this calculation there is made use of the Bi-linear stress-strain relation, see Figure 98.

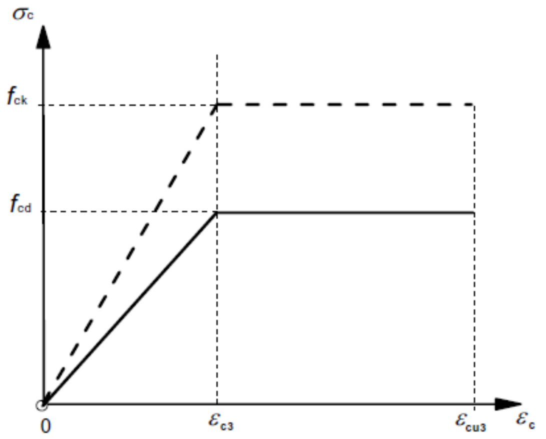

Figure 98: Bi-linear stress-strain relation

Where:

$\varepsilon_{c3} = 1.75‰$	Is the maximum elastic compressive strain in the concrete
$\varepsilon_{cu3} = 3.5‰$	Is the ultimate compressive strain in the concrete

B.8.2 Bending moments due to the loads and prestressing

Bending moment M_d at t=0

In the construction phase at t=0 the loads on the box girder are the dead load and the prestressing force. As the permanent and variable loads are missing and the initial prestressing force is large the box girder has a camber. The normative hogging moment in this phase arises at the deviation blocks, see Figure 99. The maximum strain arises in the bottom flange.

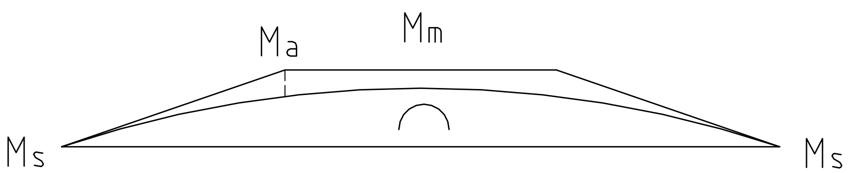

Figure 99: The bending moments due to prestressing minus the bending moments due to dead load results in the largest bending moment M_a at the deviation blocks

At deviation blocks

$$M_{da,0} = \gamma_{P,unfav} * P_{u0} * a - \frac{1}{2} * \gamma_{G,fav} * g_{dead} * L * a - 0.5 * \gamma_{G,fav} * g_{dead} * a^2 = 49235 kNm (\frown)$$

Where:

$P_{u0} = 3702 kN$	See Eq. (2)

Bending moment M_d at t=∞

In the end phase at t=∞ the box girder is fully loaded by the dead, permanent and variable load and is partly resisted by the prestressing force. This load case causes a downward deflection, which means that the normative sagging moment arises at mid-span, see Figure 100. The maximum strain arises in the top flange.

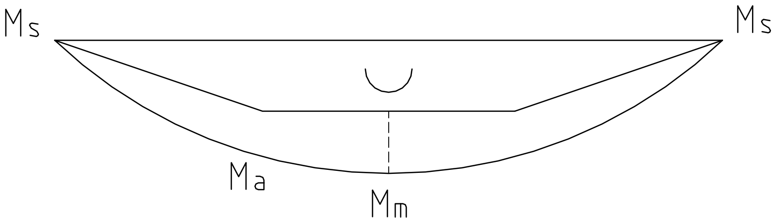

Figure 100: The bending moments due to dead, permanent and variable load minus the bending moments due to prestressing results in the largest bending moment M_m at mid-span

At mid-span

$$M_{dm,\infty} = \frac{1}{8} * (\gamma_{G,unfav} * g_{dead} + \gamma_{G,unfav} * g_{perm} + \gamma_{Q,unfav} * q_{var}) * L^2 - \gamma_{P,fav} * P_{u\infty} * a$$
$$= 24334 kNm (\cup)$$

Where:

$P_{u\infty} = 2962 kN$ See Eq. (4)

B.8.3 Ultimate resistance moment at t=0

Ultimate resistance moment at deviation blocks

The prestressing force at t=0 is:

$P_0 = 45288 kN$ See Eq. (1)

$M_{da,0} = 49235 kNm (\cap)$ means that the maximum compressive strain arises in the bottom flange. The schematisation of the forces in the cross-section is shown in Figure 101 and Figure 102.

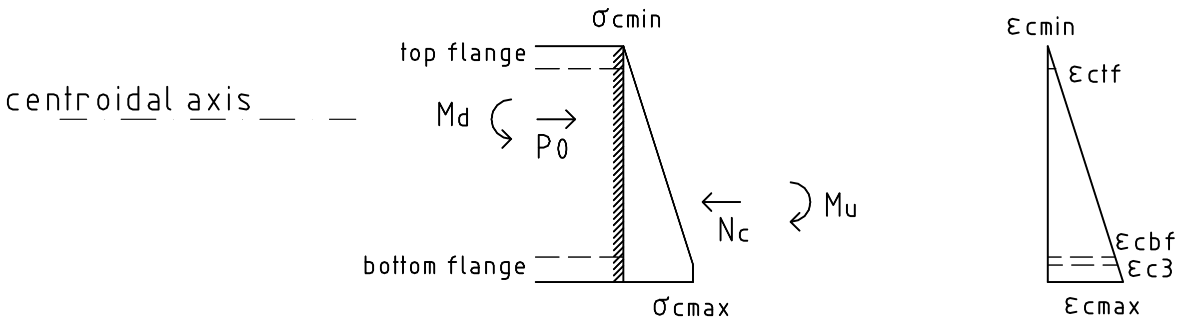

Figure 101: Stress and strain schematisation in the cross-section at the deviation blocks at t=0

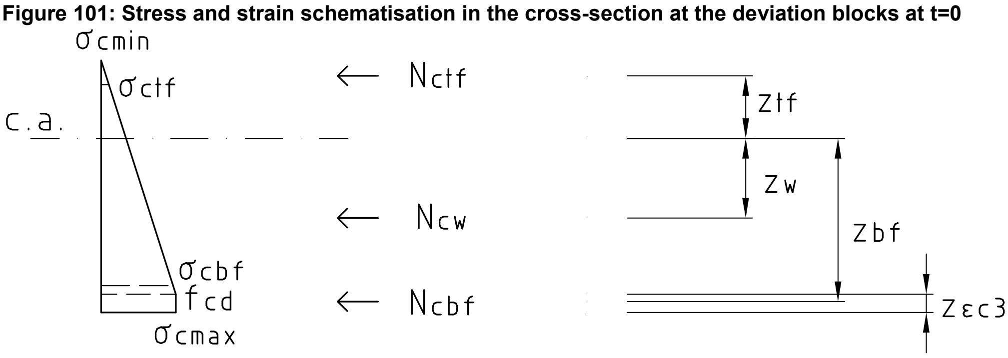

Figure 102: Concrete forces and lever arms in the cross-section at the deviation blocks at t=0

To determine the maximum strain for which holds that $N_c = P_0$ everything is filled in a spreadsheet program (Microsoft Excel) and solved with the function goal seek. With the function goal seek the concrete force N_c is set to be equal to the prestressing force P_0 by changing the maximum compressive strain in the cross-section $\varepsilon_{c\max}$.

The maximum strain in the cross-section which causes equilibrium between N_c and P_0 is:

$\varepsilon_{c\max} = 1.760‰$

This gives:

$\varepsilon_{ctf} = \varepsilon_{c\max} * t_{tf} / H = 0.157‰$

$\varepsilon_{cbf} = \varepsilon_{c\max} * (H - t_{bf}) / H = 1.572‰$

$\varepsilon_{c\min} = 0N / mm^2$

$z_{\varepsilon c3} = H - \varepsilon_{c3} / \varepsilon_{c\max} * H = 0.016m$

$$N_{ctf} = \frac{((\varepsilon_{ctf} + \varepsilon_{c\min})/2)}{\varepsilon_{c3}} * f_{cd} * b_{eff,t} * t_{tf} = 2849kN$$

$$N_{cw} = \frac{((\varepsilon_{ctf} + \varepsilon_{cbf})/2)}{\varepsilon_{c3}} * f_{cd} * 2b_w * H_{box} = 10077kN$$

$$N_{cbf} = f_{cd} * b_{eff,b} * z_{\varepsilon c3} + (\frac{\varepsilon_{cbf}}{\varepsilon_{c3}} * f_{cd} + f_{cd})/2 * b_{eff,b} * (t_{bf} - z_{\varepsilon c3}) = 32362kN$$

Where:

$b_{eff,t} = 8.96m$ See B.3.4

$b_{eff,b} = 4m$ See B.3.4

The total concrete compressive force is:

$N_c = N_{ctf} + N_{cw} + N_{cbf} = 45288kN = P_0$

The lever arms of the concrete forces are:

$$z_{tf} = t_{tf} * \frac{2}{3} - Z_{ct} = -0.90m$$

$$z_w = Z_{cb} - \left(\varepsilon_{ctf} * (H_{box} * \frac{1}{2} + t_{bf}) + \frac{\varepsilon_{cbf} - \varepsilon_{ctf}}{2} * (H_{box} * \frac{1}{3} + t_{bf}) \right) / \left(\varepsilon_{ctf} + \frac{\varepsilon_{cbf} - \varepsilon_{ctf}}{2} \right) = 0.61m$$

$$z_{bf} = Z_{cb} - \frac{f_{cd} * z_{\varepsilon c3} * z_{\varepsilon c3} / 2 + (f_{cd} - \frac{\varepsilon_{cbf}}{\varepsilon_{c3}} * f_{cd})/2 * (t_{bf} - z_{\varepsilon c3}) * ((t_{bf} - z_{\varepsilon c3}) * \frac{1}{3} + z_{\varepsilon c3}) + \frac{\varepsilon_{cbf}}{\varepsilon_{c3}} * f_{cd} * (t_{bf} - z_{\varepsilon c3}) * ((t_{bf} - z_{\varepsilon c3})/2 + z_{\varepsilon c3})}{f_{cd} * z_{\varepsilon c3} + (f_{cd} - \frac{\varepsilon_{cbf}}{\varepsilon_{c3}} * f_{cd})/2 * (t_{bf} - z_{\varepsilon c3}) + \frac{\varepsilon_{cbf}}{\varepsilon_{c3}} * f_{cd} * (t_{bf} - z_{\varepsilon c3})} = 1.58m$$

The ultimate resistance moment is:

$M_u = N_{ctf} * z_{tf} + N_{cw} * z_w + N_{cbf} * z_{bf} = 54826kNm$

Unity check of the ultimate resistance moment:

$M_{da,0} / M_u = 0.90 \leq 1.0 \rightarrow Ok$

The ultimate resistance moment of the box girder is thus large enough to resist the bending moments in the construction phase at t=0. The unity check however approaches the limit 1.0, so this verification needs attention. When this verification is not satisfied the depth of the webs H_{box} should be decreased, see Figure 70. This way the upward prestressing force becomes smaller, see Figure 74, and thus the hogging moment due to prestressing decreases. Another option is to make the box girder heavier such that the hogging moment M_d becomes smaller.

B.8.4 Ultimate resistance moment at t=∞

Ultimate resistance moment at mid-span

The prestressing force at t=∞ is:

$P_\infty = 36230kN$ See Eq. (3)

$M_{dm,\infty} = 24334kNm(\cup)$ means that the maximum compressive strain arises in the top flange.

The schematisation of the forces in the cross-section is shown in Figure 103 and Figure 104.

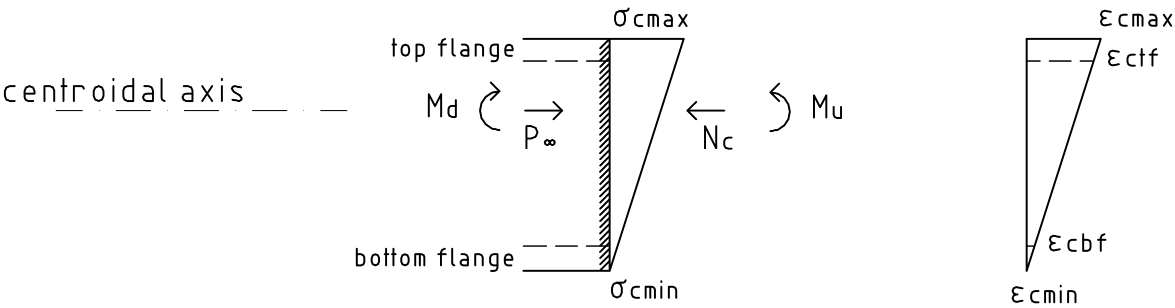

Figure 103: Stress and strain schematisation in the cross-section at mid-span at=∞

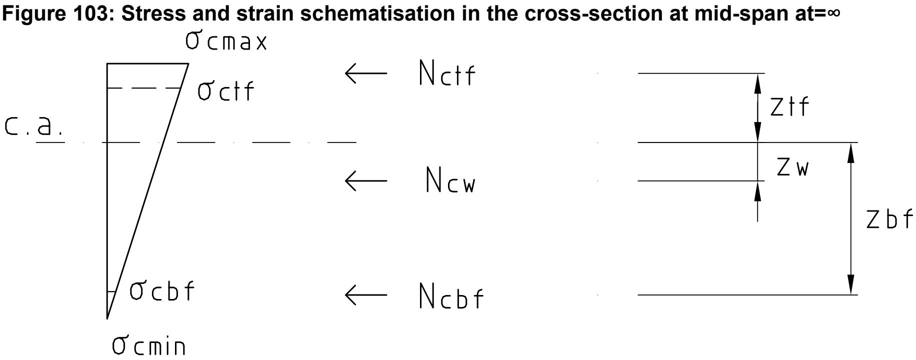

Figure 104: Concrete forces and lever arms in the cross-section at mid-span at=∞

To determine the maximum strain for which holds that $N_c = P_\infty$ everything is filled in a spreadsheet program (Microsoft Excel) and solved with the function goal seek. With the function goal seek the concrete force N_c is set to be equal to the prestressing force P_∞ by changing the maximum compressive strain in the cross-section $\varepsilon_{c\max}$.

The maximum strain in the cross-section which causes equilibrium between N_c and P_∞ is:

$\varepsilon_{c\max} = 0.870‰$

This gives:

$\varepsilon_{ctf} = \varepsilon_{c\max} * (H - t_{tf}) / H = 0.793‰$

$\varepsilon_{cbf} = \varepsilon_{c\max} * t_{bf} / H = 0.093‰$

$\varepsilon_{c\min} = 0N / mm^2$

$$N_{ctf} = \frac{((\varepsilon_{c\max} + \varepsilon_{ctf})/2)}{\varepsilon_{c3}} * f_{cd} * b_{eff,t} * t_{tf} = 30160kN$$

$$N_{cw} = \frac{((\varepsilon_{ctf} + \varepsilon_{cbf})/2)}{\varepsilon_{c3}} * f_{cd} * 2b_w * H_{box} = 5164kN$$

$$N_{cbf} = \frac{((\varepsilon_{cbf} + \varepsilon_{c\min})/2)}{\varepsilon_{c3}} * f_{cd} * b_{eff,b} * t_{bf} = 906kN$$

Where:

$b_{eff,t} = 8.96m$ See B.3.4

$b_{eff,b} = 4m$ See B.3.4

The total concrete compressive force is:

$$N_c = N_{ctf} + N_{cw} + N_{cbf} = 36230kN = P_\infty$$

The lever arms of the concrete forces are:

$$z_{tf} = Z_{ct} - \left(\frac{\varepsilon_{c\max} - \varepsilon_{ctf}}{2} * t_{tf} * \frac{1}{3} + \varepsilon_{ctf} * t_{tf} * \frac{1}{2} \right) / \left(\frac{\varepsilon_{c\max} - \varepsilon_{ctf}}{2} + \varepsilon_{ctf} \right) = 0.95m$$

$$z_w = \left(\frac{\varepsilon_{ctf} - \varepsilon_{cbf}}{2} * (H_{box} * \frac{2}{3} + t_{bf}) + \varepsilon_{cbf} * (H_{box} * \frac{1}{2} + t_{bf}) \right) / \left(\frac{\varepsilon_{ctf} - \varepsilon_{cbf}}{2} + \varepsilon_{cbf} \right) - Z_{cb} = -0.01m$$

$$z_{bf} = t_{bf} * \frac{2}{3} - Z_{cb} = -1.53m$$

The ultimate resistance moment is:

$$M_u = N_{ctf} * z_{tf} + N_{cw} * z_w + N_{cbf} * z_{bf} = 27117kNm$$

Unity check of the ultimate resistance moment:

$$M_{dm,\infty} / M_u = 0.90 \leq 1.0 \rightarrow Ok$$

The ultimate resistance moment of the box girder is thus enough to resist the bending moments in the end phase at t=∞. The unity check however approaches the limit 1.0, so this verification needs attention. When this verification is not satisfied the depth of the webs H_{box} should be increased, see Figure 70. This way the lever arms z become larger which has a positive effect on the ultimate resistance moment. Also the upward prestressing force then becomes larger, see Figure 74. The bending moment M_d should be kept as small as possible by creating a light as possible box girder.

B.9 Deck

To determine if the thickness of the top flange / deck meet the requirements of shear and bending moments, the local schematisation is considered. The deck is schematised in the transversal direction as a floor of 1 metre wide with two fixed supports (the webs). The width of 1 metre in longitudinal direction comes from [8], which says that for the calculation of the deck the wheel pressure in longitudinal direction of the track may be spread to two sides over a distance of 1 metre + twice the height of the concrete plinth. For a more conservative calculation only the width of 1 metre is taken. To calculate the shear and bending moments in the deck there is made use of the program Scia Engineer. In the next section the input in Scia Engineer is given. For the geometry of the deck the assumption was made that the web width should be 0.2 metres. With this width the geometry in Figure 105 becomes:

$$L_{cant,centre} = L_{cant} + b_w / 2 = 2.08m$$

$$L_{span} = b_{tf} - 2 * L_{cant,centre} = 4.8m$$

For the load schematisation in the next section reference is made to Section B.4.5.
With the shear and bending moments due to loading as result from the input in Scia Engineer next the verification of shear and ultimate resistance moment for the deck is done.

B.9.1 Schematisation load input in Scia Engineer

- ***Geometry deck***

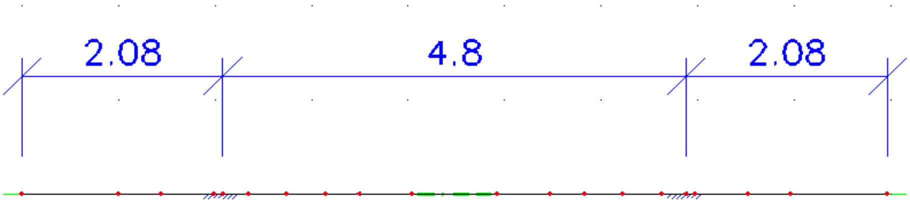

Figure 105: Structural schematisation deck box girder

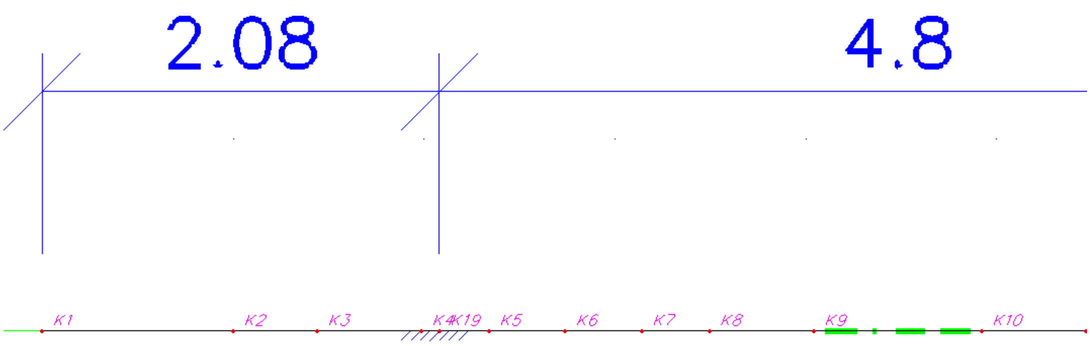

Figure 106: Nodes left side of the deck

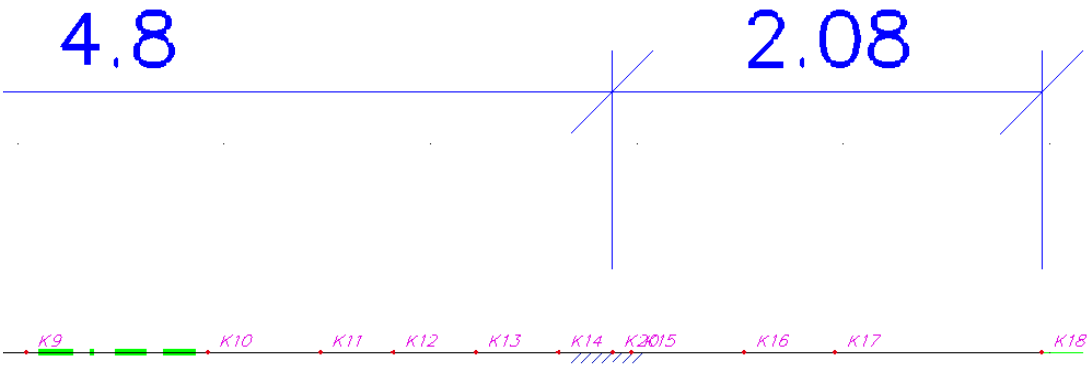

Figure 107: Nodes right side of the deck

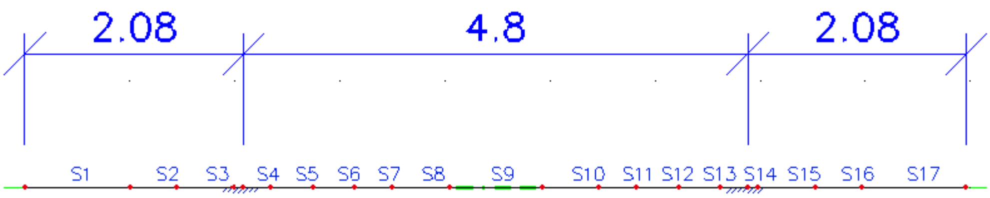

Figure 108: Bars

Nodes

Name	*Coordinate X [m]*	*Coordinate Z [m]*
K1	0,000	6,000
K2	1,000	6,000
K3	1,440	6,000
K4	1,986	6,000
K5	2,340	6,000
K6	2,740	6,000
K7	3,140	6,000
K8	3,494	6,000
K9	4,040	6,000
K10	4,920	6,000
K11	5,466	6,000
K12	5,820	6,000
K13	6,220	6,000
K14	6,620	6,000
K15	6,974	6,000
K16	7,520	6,000
K17	7,960	6,000
K18	8,960	6,000
K19	2,080	6,000
K20	6,880	6,000

1D-bar

Name	*Cross-section*	*Length [m]*	*Form*	*Start node*	*End node*	*Type*	*EEM-type*	*Layer*
S1	CS1 - Rectangle (250; 1000)	1,000	Line	K1	K2	floor strip (99)	standard	Layer1
S2	CS1 - Rectangle (250; 1000)	0,440	Line	K2	K3	floor strip (99)	standard	Layer1

S3	CS1 - Rectangle (250; 1000)	0,546	Line	K3	K4	floor strip (99)	standard	Layer1
S4	CS1 - Rectangle (250; 1000)	0,354	Line	K4	K5	floor strip (99)	standard	Layer1
S5	CS1 - Rectangle (250; 1000)	0,400	Line	K5	K6	floor strip (99)	standard	Layer1
S6	CS1 - Rectangle (250; 1000)	0,400	Line	K6	K7	floor strip (99)	standard	Layer1
S7	CS1 - Rectangle (250; 1000)	0,354	Line	K7	K8	floor strip (99)	standard	Layer1
S8	CS1 - Rectangle (250; 1000)	0,546	Line	K8	K9	floor strip (99)	standard	Layer1
S9	CS1 - Rectangle (250; 1000)	0,880	Line	K9	K10	floor strip (99)	standard	Layer1
S10	CS1 - Rectangle (250; 1000)	0,546	Line	K10	K11	floor strip (99)	standard	Layer1
S11	CS1 - Rectangle (250; 1000)	0,354	Line	K11	K12	floor strip (99)	standard	Layer1
S12	CS1 - Rectangle (250; 1000)	0,400	Line	K12	K13	floor strip (99)	standard	Layer1
S13	CS1 - Rectangle (250; 1000)	0,400	Line	K13	K14	floor strip (99)	standard	Layer1
S14	CS1 - Rectangle (250; 1000)	0,354	Line	K14	K15	floor strip (99)	standard	Layer1
S15	CS1 - Rectangle (250; 1000)	0,546	Line	K15	K16	floor strip (99)	standard	Layer1
S16	CS1 - Rectangle (250; 1000)	0,440	Line	K16	K17	floor strip (99)	standard	Layer1
S17	CS1 - Rectangle (250; 1000)	1,000	Line	K17	K18	floor strip (99)	standard	Layer1

Node support

Name	*Node*	*System*	*Type*	*X*	*Z*	*Ry*
Sn1	K19	GCS	Standard	Fixed	Fixed	Fixed
Sn2	K20	GCS	Standard	Fixed	Fixed	Fixed

- ***Load input***

The values of the loads are taken from section: B.4.5 Load schematisation in transversal direction in the SLS.

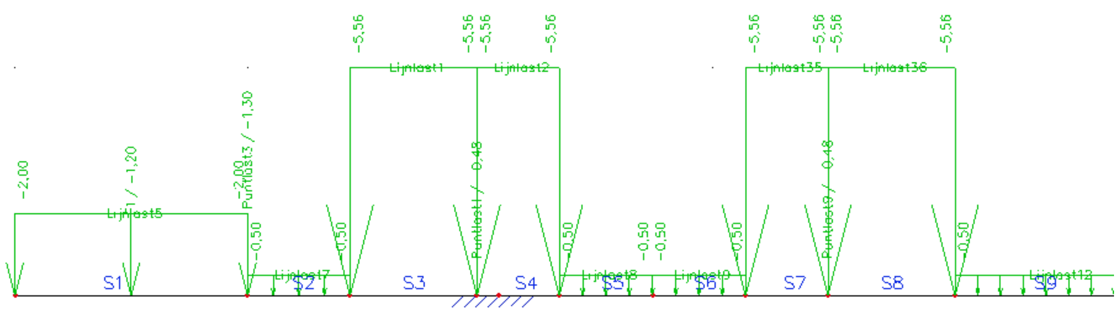

Figure 109: Permanent loads left side of the deck

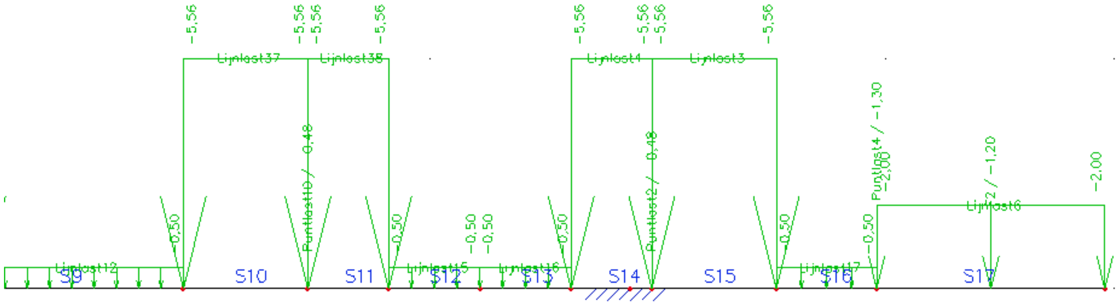

Figure 110: Permanent loads right side of the deck

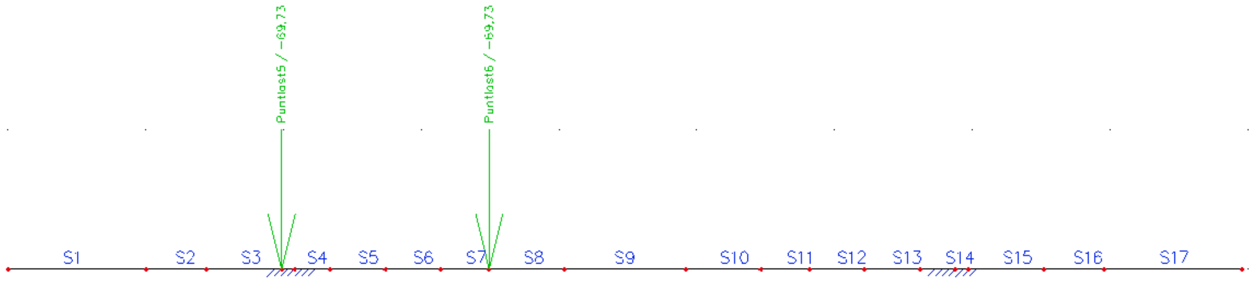

Figure 111: Metro load on the left side of the deck

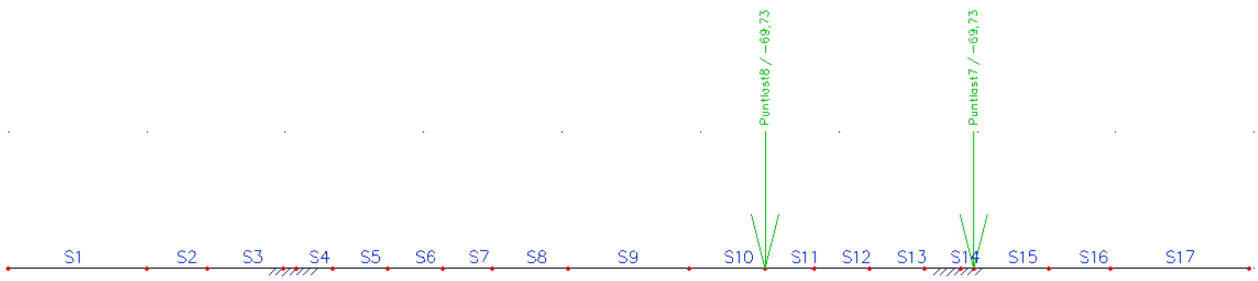

Figure 112: Metro load on the right side of the deck

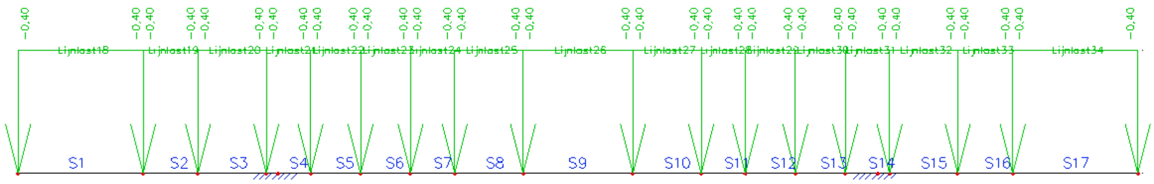

Figure 113: Snow load

Line loads on bars

Name	Bar	Type	Direction	P1 [kN/m]	x1	Coordinate definition	Origin	Exc ey [m]
	Load case	System	Distribution	P2 [kN/m]	x2	Loc	Angle [deg]	Exc ez [m]
Lijnlast1	S3	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast2	S4	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast3	S15	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast4	S14	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast5	S1	Force	Z	-2,00	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000

Lijnlast6	S17	Force	Z	-2,00	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast7	S2	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast8	S5	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast9	S6	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast12	S9	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast15	S12	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast16	S13	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast17	S16	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast35	S7	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast36	S8	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast37	S10	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast38	S11	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000

Concentrated loads on nodes

Name	*Node*	*Load case*	*System*	*Direction*	*Type*	*Value - F [kN]*
Puntlast1	K4	BG2 - Permanent load	GCS	Z	Force	-0,48
Puntlast2	K15	BG2 - Permanent load	GCS	Z	Force	-0,48
Puntlast3	K2	BG2 - Permanent load	GCS	Z	Force	-1,30
Puntlast4	K17	BG2 - Permanent load	GCS	Z	Force	-1,30
Puntlast9	K8	BG2 - Permanent load	GCS	Z	Force	-0,48
Puntlast10	K11	BG2 - Permanent load	GCS	Z	Force	-0,48

Concentrated loads on bars

Name	Bar	System	F [kN]	x	Coordinate	Repeat (n)
	Load case	Direction	Type	Angle [deg]	Origin	dx
F1	S1	GCS	-1,20	0,500	Rela	1
	BG2 - Permanent load	Z	Force		From start	
F2	S17	GCS	-1,20	0,500	Rela	1
	BG2 - Permanent load	Z	Force		From start	

Concentrated loads on nodes

Name	*Node*	*Load case*	*System*	*Direction*	*Type*	*Value - F [kN]*
Puntlast5	K4	BG3 – Metro left	GCS	Z	Force	-69,73
Puntlast6	K8	BG3 – Metro left	GCS	Z	Force	-69.73
Puntlast7	K15	BG3 – Metro right	GCS	Z	Force	-69,73
Puntlast8	K11	BG3 – Metro right	GCS	Z	Force	-69.73

Line loads on bars

Name	Bar	Type	Direction	P1 [kN/m]	x1	Coordinate definition	Origin	Exc ey [m]
	Load case	System	Distribution	P2 [kN/m]	x2	Loc	Angle [deg]	Exc ez [m]
Lijnlast18	S1	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast19	S2	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast20	S3	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast21	S4	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast22	S5	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast23	S6	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast24	S7	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast25	S8	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast26	S9	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast27	S10	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast28	S11	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast29	S12	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast30	S13	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast31	S14	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast32	S15	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast33	S16	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast34	S17	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000

- ***Results***

The shear forces and moments in the deck due to the permanent and variable loads in the ULS are shown below. This does not include the dead load of the deck.

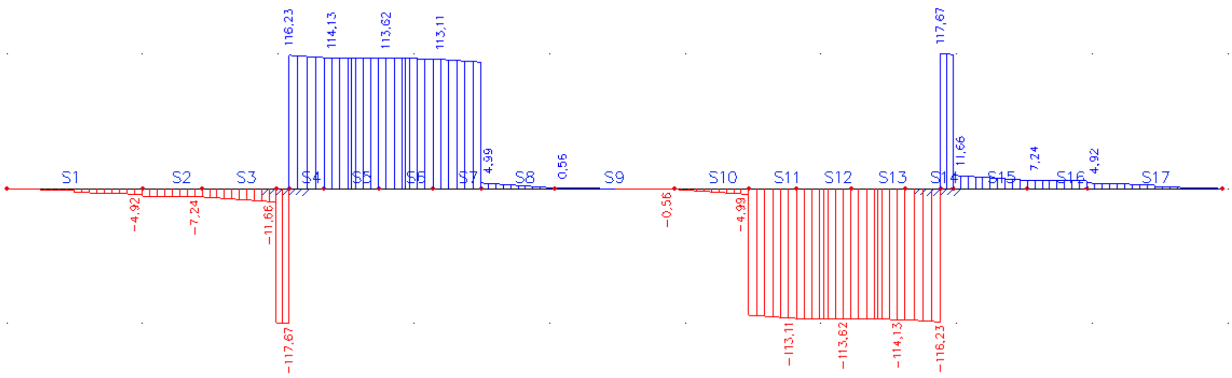

Figure 114: Shear forces in the deck due to permanent and variable loads

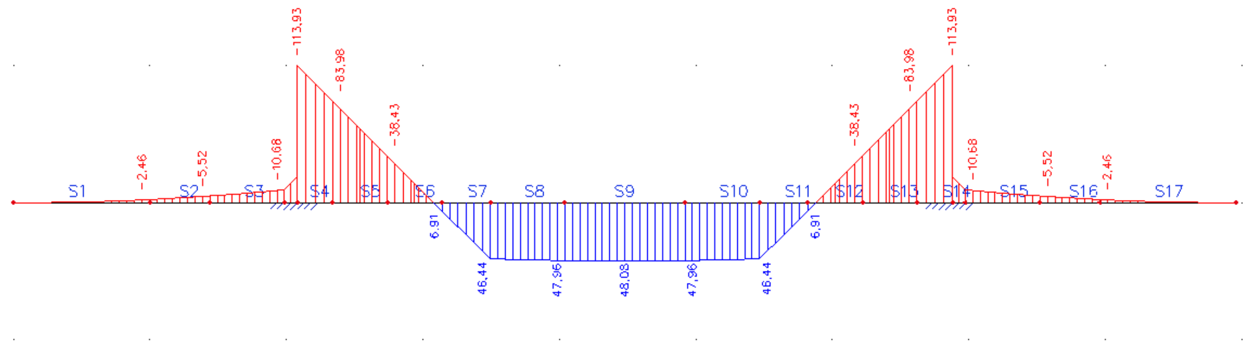

Figure 115: Moments in the deck due to permanent and variable loads

B.9.2 Verifications of shear and ultimate resistance moment

Total shear force and bending moment

The maximum shear force due to permanent and variable loads in the ULS is:

$V_{Ed,perm+var} = 117.67kN$ See Figure 114

The extra shear force due to the dead load is:

$$V_{Ed,dead} = t_{tf} * b_{deck} * L_{cant,centre} * \rho_c * g * \gamma_{G,unfav} = 17.22kN$$

Where:

$b_{deck} = 1.0m$

$L_{cant,centre} = 2.08m$

The total shear force is:

$$V_{Ed} = V_{Ed,perm+var} + V_{Ed,dead} = 134.89kN$$

The maximum bending moment due to permanent and variable loads in the ULS is:

$M_{d,perm+var} = 113.93kNm$ See Figure 115

The extra bending moment due to the dead load at the webs is:

$$M_{d,dead} = \frac{1}{12}(t_{tf} * b_{deck} * \rho_c * g * \gamma_{G,unfav}) * L_{span}^{2} = 15.89kNm$$

Where:

$L_{span} = 4.8m$

The total bending moment is:

$$M_d = M_{d,dead} + M_{d,perm+var} = 129.82kNm$$

Shear resistance

The minimum shear strength of concrete is:

$$V_{Rd,c1} = (v_{\min} + k_1\sigma_{cp})b_{deck}d = 135.8kN$$

Where:

$d = t_{tf} - c_{nom} - \frac{1}{2}\phi_{reinf} - \phi_{stirrups} = 194mm$ Is the effective structural depth

$c_{nom} = 40mm$ Is the concrete cover, see B.3.3

$\phi_{reinf} = 16mm$ Is the diameter of the longitudinal reinforcement

$\phi_{stirrups} = 8mm$ Is the diameter of the stirrups

$\sigma_{cp} = 0N/mm^2$ No axial force in this direction due to prestressing

$v_{\min} = 0.035 * k^{3/2} * f_{ck}^{1/2} = 0.7N/mm^2$

$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0 = 2$

Unity check:

$$V_{Ed} / V_{Rd,c1} = 0.99 \leq 1.0 \rightarrow Ok$$

As the unity check shows, the minimum shear strength of concrete is already sufficient. For the ultimate resistance moment the deck however needs longitudinal reinforcement, see Figure 116.

The shear strength of concrete with longitudinal reinforcement is:

$$V_{Rd,c2} = [C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}]b_{deck}d = 167.59kN$$

Where:

$C_{Rd,c} = 0.18/\gamma_c = 0.12$

$\rho_l = \frac{A_{sl}}{b_{deck} * d} < 0.02 = 0.009$

$A_{sl} = n_{reinf} * \pi * (\frac{1}{2} * \phi_{reinf})^2 = 1810mm^2$ Is the area of tensile reinforcement, see Figure 116

$n_{reinf} = b_{deck} / S_{reinf} = 9.09 \rightarrow 9bars/m$ Is the number of reinforcement bars

$S_{reinf} = 110mm$ Is the spacing of the reinforcement bars

Unity check:

$$V_{Ed} / V_{Rd,c2} = 0.80 \leq 1.0 \rightarrow Ok$$

With longitudinal reinforcement the deck easily satisfies with respect to local shear.

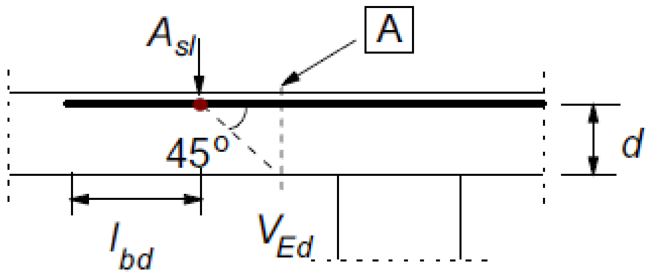

Figure 116: Definition of A_{sl}

Furthermore the shear force in the deck should always satisfy the condition:

$$V_{Ed} \leq 0.5 * b_{deck} * d * \nu * f_{cd}$$

Where:

$$\nu = 0.6\left[1 - \frac{f_{ck}}{250}\right] = 0.48$$

Is a strength reduction factor for concrete cracked in shear

Filling in the formula gives:

$$V_{Ed} = 134.89kN \leq 0.5 * b_{deck} * d * \nu * f_{cd} = 1319kN \rightarrow Ok$$

Ultimate resistance moment

The ultimate resistance moment of the deck is calculated according to the schematisation in Figure 117. In this case however the schematisation should be mirrored along the centre line as the tension arises at the top side and the compression zone is at the bottom side of the deck, see Figure 116.

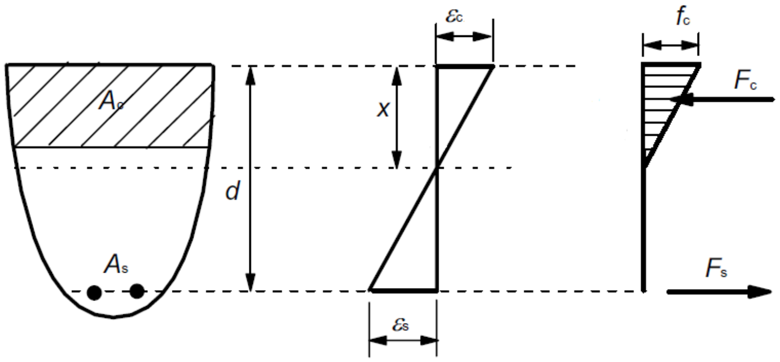

Figure 117: Rectangular stress distribution

The two horizontal forces F_c and F_s should be in equilibrium:

$$F_c - F_s = 0$$

This is the same as:

$$0.5 * x * b_{deck} * \frac{\varepsilon_c}{\varepsilon_{c3}} * f_{cd} - \varepsilon_s * E_s * A_{sl} = 0$$

Where:

$$x = \frac{\varepsilon_c}{\varepsilon_{tot}} * t_{tf}$$ Concrete compressive zone

$$\varepsilon_{tot} = \varepsilon_c + \varepsilon_s$$ Total strain in the deck

$\varepsilon_s = \frac{f_{yd}}{E_s} = 2.174‰$ Tensile strain in the reinforcement

$\varepsilon_{c3} = 1.75‰$ Compressive strain in the concrete at the end of the linear part

$A_{sl} = 1810mm^2$ Area of tensile reinforcement

Solving the formula gives the compressive strain in the concrete:

$F_c - F_s = 0 \rightarrow \varepsilon_c = 1.134‰$

The concrete compressive zone is:

$$x = \frac{\varepsilon_c}{\varepsilon_{tot}} * t_{tf} = 85.7mm$$

The ultimate resistance moment of the deck is:

$$M_u = \varepsilon_s * E_s * A_{sl} * (d - \frac{1}{3}x) = 130.16kNm$$

Unity check for the ultimate resistance moment of the deck is:

$$M_d / M_u = 0.997 \leq 1.0 \rightarrow Ok$$

The ultimate resistance moment of the deck is thus just enough to resist the bending moments. If this verification is not satisfied the lever arm between the two forces F_c and F_s should be increased. This means that the deck becomes thicker. Another option is to add more reinforcement bars. This however has a strong influence on the rotation capacity, see hereunder.

Furthermore:

The cracking moment is:

$$M_r = f_{ctm,fl} * \frac{1}{6} * b_{deck} * t_{tf}^2 = 57.26kNm$$

Where:

$$f_{ctm,fl} = \max\{(1.6 - h/1000) f_{ctm}; f_{ctm}\} = 5.50N/mm^2$$

$$h = t_{tf} = 250mm$$

Because $M_r \leq M_u$, the deck satisfies with respect to the minimum required percentage of reinforcement.

The cross-sectional area of longitudinal reinforcement is:

$$A_{sl} = n_{reinf} * \pi * (\frac{1}{2} * \phi_{reinf})^2 = 1810mm^2$$

The maximum allowed cross-sectional area of longitudinal reinforcement is:

$$A_{s,\max} = 0.04 * A_c = 0.04 * t_{tf} * b_{deck} = 10000mm^2 \rightarrow Ok$$

The minimum required cross-sectional area of longitudinal reinforcement is:

$$A_{s,\min} = 0.26 * \frac{f_{ctm}}{f_{yk}} b_{deck} * d = 411mm^2 \rightarrow Ok$$

The height of the compression zone should satisfy: See 5.6.3 [11]

$$x/d = 0.44 \leq 0.45 \rightarrow Ok$$

This verification considers the rotation capacity of the deck at the supports (the webs). It shows that the rotation capacity of the deck is sufficient, but is very close to the limit so attention is needed. If this verification is not satisfied the thickness of the deck should be increased. Another option is to diminish the number of reinforcement bars which will result in a smaller compressive zone x . This will however also reduce the ultimate resistance moment.

The maximum deflection under dead load, permanent load and variable load in the SLS is calculated with the program Scia Engineer and is:

$w_z = 1.4mm$ See Figure 118

The maximum allowed deflection is:

$w_{\max} = L_{span} / 500 = 9.6mm \rightarrow Ok$

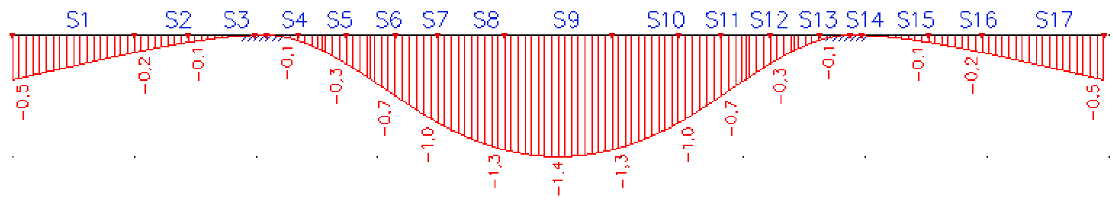

Figure 118: Deflection of the deck in mm under dead load, permanenet load and variable load.

B.10 Fatigue + vibration

B.10.1 Fatigue prestressing steel

For prestressing steel adequate fatigue resistance should be assumed if the following expression is satisfied:

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

Where:

$\gamma_{F,fat} = 1.0$	Is the partial factor for fatigue loads
$\gamma_{s,fat} = \gamma_s = 1.15$	Is the partial factor for prestressing steel for the fatigue verification
$\Delta\sigma_{Rsk}(N^*) = 150 N/mm^2$	Is the stress range at N* cycles, see table 6.4N [11]: straight tendons or curved tendons in plastic ducts
$k_1 = 5$	See Figure 119
$k_2 = 10$	See Figure 119
$N^* = 1000000\, loading\ cycles$	See Figure 119

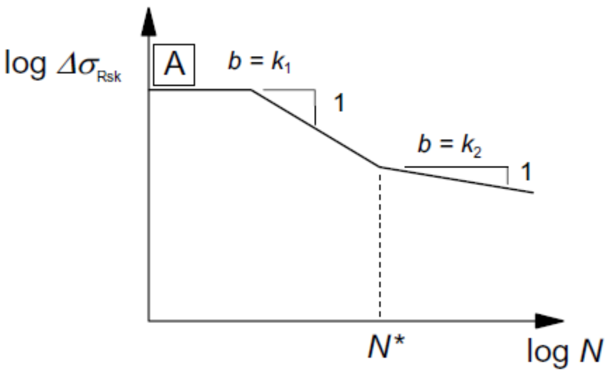

Figure 119: Shape of the characteristic fatigue strength curve (S-N-curves for prestressing steel)

The damage equivalent stress range for prestressing steel is calculated according to Equation NN.106 [12]:

$$\Delta\sigma_{S,equ}(N^*) = \lambda_s * \phi * \Delta\sigma_s$$

Where:

$\Delta\sigma_s$	Is the steel stress range due to the variable load
ϕ	Is the dynamic factor
$\lambda_s = \lambda_{s,1} * \lambda_{s,2} * \lambda_{s,3} * \lambda_{s,4}$	Is a correction factor to calculate the damage equivalent stress range from the stress range caused by $\phi * \Delta\sigma_s$

$\gamma_{s,1}$	Is a factor accounting for element type (eg. continuous beam) and takes into account the damaging effect of traffic depending on the length of the influence line or area.
$\gamma_{s,2}$	Is a factor taking into account the traffic volume
$\gamma_{s,3}$	Is a factor that takes into account the design life of the bridge
$\gamma_{s,4}$	Is a factor to be applied when the structural element is loaded by more than one track

$\gamma_{s,1} = 0.65$ See Table NN.2 [12]: (1) post tensioning straight tendons, s* standard traffic mix and simply supported beam

$$\gamma_{s,2} = \sqrt[k_2]{\frac{Vol}{25*10^6}} = 0.96$$

Where:

$k_2 = 10$ See Figure 119

$Vol = Q_{metro} / g * 6 * 24 * 365 = 15848367 tonnes / year / track$ Assumption of 6 metros per hour

$Q_{metro} = q_{mob} * 116m = 2958kN$ 116 metres is the length of a metro

$$\gamma_{s,3} = \sqrt[k_2]{\frac{N_{years}}{100}} = 1$$

Where:

$N_{years} = 100 years$ Is the design life of the viaduct

$$\gamma_{s,4} = \sqrt[k_2]{n + (1-n) * s_1^{k_2} + (1-n) * s_2^{k_2}} = 0.81$$

Where:

$n = 0.12$ Is the proportion of traffic that crosses the bridge simultaneously, 0.12 is the suggested value

$s_j = 0$ Only compressive stresses occur under traffic loads on a track

$$\lambda_s = \lambda_{s,1} * \lambda_{s,2} * \lambda_{s,3} * \lambda_{s,4} = 0.50$$

The dynamic factor is determined according [8] and not according [10] as this dynamic factor is normative (larger):

$\phi = 1 + 4/(10 + L) = 1.07$

The deflection at mid-span at t=∞ due to the dead load, the permanent load and prestressing is:

$w_1 = -23mm$ Upwards, see Table 20

The deflection at mid-span at t=∞ due to the dead load, the permanent load, the variable load and prestressing is: $w_2 = -7mm$ Downwards, see Table 20

It is assumed that the deflection at the deviation blocks is the same as at mid-span (conservative assumption).

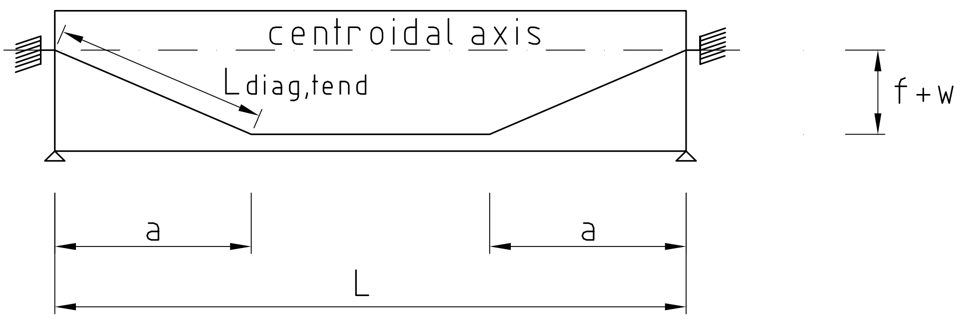

Figure 120: Schematisation for determining the elongation of the tendons

With:

$f = 1230mm$ See Figure 120

$a = 15m$

The length of one diagonal part of the tendon without variable load at t=∞:

$$L_{diag,tend1} = \sqrt{(f + w_1)^2 + a^2} = 15.049m$$

The length of one diagonal part of the tendon with variable load at t=∞:

$$L_{diag,tend2} = \sqrt{(f + w_2)^2 + a^2} = 15.050m$$

The elongation due to the variable load for one diagonal part of the tendon is:

$$\Delta L = \frac{L_{diag,tend1}}{L_{diag,tend2}} = 1.3mm$$

The strain in the tendon due to the variable load for two diagonal parts of the tendon is:

$$\varepsilon_s = 2 * \frac{\Delta L}{L_{diag,tend1}} = 0.0002$$

The steel stress range due to the variable load is:

$$\Delta\sigma_s = \varepsilon_s * E_s = 33.26N/mm^2$$

The damage equivalent stress range for prestressing steel is:

$$\Delta\sigma_{S,equ}(N^*) = \lambda_s * \phi * \Delta\sigma_s = 17.93N/mm^2$$

The fatigue verification for prestressing steel is:

$$\gamma_{F,fat} * \Delta\sigma_{S,equ}(N^*) \le \frac{\Delta\sigma_{Rsk}(N^*)}{\gamma_{s,fat}} \rightarrow \frac{\gamma_{F,fat} * \Delta\sigma_{S,equ}(N^*) * \gamma_{s,fat}}{\Delta\sigma_{Rsk}(N^*)} = 0.137 \le 1.0 \rightarrow Ok$$

The fatigue verification for prestressing steel is easily satisfied and as the standard [12] (6.8.4) says: "Fatigue verification for external and unbonded tendons, lying within the depth of the concrete section, is not necessary" this could also be expected. This calculation with a rough estimation of the elongation of the tendons is however done to confirm the assumption. Fatigue of the prestressing tendons is not an issue in the design.

B.10.2 Fatigue concrete

The fatigue verification for concrete is calculated according to Equation NN.112 [12]:
For concrete subjected to compression adequate fatigue resistance may be assumed if the following expression is satisfied:

$$14 * \frac{1 - E_{cd,\max,equ}}{\sqrt{1 - R_{equ}}} \ge 6$$

Fatigue at mid-span, at the top

$$14 * \frac{1 - E_{cd,\max,equ}}{\sqrt{1 - R_{equ}}} \ge 6 \rightarrow \frac{6}{14} * \sqrt{1 - R_{equ}} + E_{cd,\max,equ} = 0.74 \le 1.0 \rightarrow Ok$$

Where:

$R_{equ} = \dfrac{E_{cd,\min,equ}}{E_{cd,\max,equ}} = 0.76$ Stress ratio

$E_{cd,\min,equ} = \gamma_{sd} \dfrac{\sigma_{cd,\min,equ}}{f_{cd,fat}} = 0.40$ Minimum compressive stress level

$E_{cd,\max,equ} = \gamma_{sd} \dfrac{\sigma_{cd,\max,equ}}{f_{cd,fat}} = 0.53$ Maximum compressive stress level

$f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd} \left(1 - \dfrac{f_{ck}}{250}\right) = 19.27 N/mm^2$ Design fatigue strength of concrete

$\beta_{cc}(t) = \exp\left\{ s\left[1 - \left(\dfrac{28}{t}\right)^{1/2}\right]\right\} \rightarrow \beta_{cc}(t28) = 1.0$ Coefficient for concrete strength at first load application

$k_1 = 0.85$ Recommended value for $N = 10^6$ cycles

$\gamma_{sd} = 1.15$ Is the partial factor for model uncertainty for action/action effort

$\sigma_{cd,\max,equ} = \sigma_{c,perm} - \lambda_c(\sigma_{c,\max} - \sigma_{c,perm}) = 8.91 N/mm^2$ Upper stress of the ultimate amplitude for N cycles

$\sigma_{cd,\min,equ} = \sigma_{c,perm} - \lambda_c(\sigma_{c,perm} - \sigma_{c,\min}) = 6.75 N/mm^2$ Lower stress of the ultimate amplitude for N cycles

$\sigma_{c,perm} = 6.75 N/mm^2$ Permanent stress, without variable load, see Eq. (15)

$\sigma_{c,\max} = 9.68 N/mm^2$ Maximum compressive stress, with variable load, see Eq. (11)

$\sigma_{c,\min} = 6.75 N/mm^2$ Minimum compressive stress, without variable load, see Eq. (15)

$\lambda_c = \lambda_{c,0} * \lambda_{c,1} * \lambda_{c,2,3} * \lambda_{c,4} = 0.74$ Correction factor to calculate the upper and lower stresses of the damage equivalent stress

$\lambda_{c,0} = 0.94 + 0.2 \dfrac{\sigma_{c,perm}}{f_{cd,fat}} \geq 1 = 1.01$ Is a factor to take account of the permanent stress

$\lambda_{c,1} = 0.75$ Is a factor accounting for element type, see Table NN.3 [12]: (1) compression zone, s* standard traffic mix and simply supported beam

$\lambda_{c,2,3} = 1 + \dfrac{1}{8}\log\left[\dfrac{Vol}{25*10^6}\right] + \dfrac{1}{8}\log\left[\dfrac{N_{years}}{100}\right] = 0.98$ Is a factor to take account of the traffic volume and the design life of the bridge

$Vol = Q_{metro} / g * 6 * 24 * 365 = 15848367 tonnes/year/track$ Assumption of 6 metros per hour

$Q_{metro} = q_{mob} * 116m = 2958kN$ 116 metres is the length of a metro

$N_{years} = 100 years$ Is the design life of the viaduct

$\lambda_{c,4} = 1.0$ Is a factor to be applied when the structure is loaded by more than one track, is the most conservative value

Fatigue at mid-span, at the bottom

$$14*\frac{1-E_{cd,\max,equ}}{\sqrt{1-R_{equ}}}\geq 6\rightarrow\frac{6}{14}*\sqrt{1-R_{equ}}+E_{cd,\max,equ}=0.95\leq 1.0\rightarrow Ok$$

Where:

$R_{equ}=\frac{E_{cd,\min,equ}}{E_{cd,\max,equ}}=0.69$ Stress ratio

$E_{cd,\min,equ}=\gamma_{sd}\frac{\sigma_{cd,\min,equ}}{f_{cd,fat}}=0.49$ Minimum compressive stress level

$E_{cd,\max,equ}=\gamma_{sd}\frac{\sigma_{cd,\max,equ}}{f_{cd,fat}}=0.71$ Maximum compressive stress level

$f_{cd,fat}=k_1\beta_{cc}(t_0)\,f_{cd}\left(1-\frac{f_{ck}}{250}\right)=19.27N/mm^2$ Design fatigue strength of concrete

$\beta_{cc}(t)=\exp\left\{s\left[1-\left(\frac{28}{t}\right)^{1/2}\right]\right\}\rightarrow\beta_{cc}(t28)=1.0$ Coefficient for concrete strength at first load application

$k_1=0.85$ Recommended value for $N=10^6$ cycles

$\gamma_{sd}=1.15$ Is the partial factor for model uncertainty for action/action effort

$\sigma_{cd,\max,equ}=\sigma_{c,perm}-\lambda_c(\sigma_{c,\max}-\sigma_{c,perm})=11.88N/mm^2$ Upper stress of the ultimate amplitude for N cycles

$\sigma_{cd,\min,equ}=\sigma_{c,perm}-\lambda_c(\sigma_{c,perm}-\sigma_{c,\min})=8.20N/mm^2$ Lower stress of the ultimate amplitude for N cycles

$\sigma_{c,perm}=11.88N/mm^2$ Permanent stress, without variable load, see Eq. (16)

$\sigma_{c,\max}=11.88N/mm^2$ Maximum compressive stress, without variable load, see Eq. (16)

$\sigma_{c,\min}=7.15N/mm^2$ Minimum compressive stress, with variable load, see Eq. (12)

$\lambda_c=\lambda_{c,0}*\lambda_{c,1}*\lambda_{c,2,3}*\lambda_{c,4}=0.78$ Correction factor to calculate the upper and lower stresses of the damage equivalent stress

$\lambda_{c,0}=0.94+0.2\frac{\sigma_{c,perm}}{f_{cd,fat}}\geq 1=1.06$ Is a factor to take account of the permanent stress

$\lambda_{c,1}=0.75$ Is a factor accounting for element type, see Table NN.3 [12]: (1) compression zone, s* standard traffic mix and simply supported beam

$\lambda_{c,2,3}=1+\frac{1}{8}\log\left[\frac{Vol}{25*10^6}\right]+\frac{1}{8}\log\left[\frac{N_{years}}{100}\right]=0.98$ Is a factor to take account of the traffic volume and the design life of the bridge

$Vol=Q_{metro}/g*6*24*365=15848367tonnes/year/track$ Assumption of 6 metros per hour

$Q_{metro} = q_{mob} * 116m = 2958kN$ 116 metres is the length of a metro

$N_{years} = 100\,years$ Is the design life of the viaduct

$\lambda_{c,4} = 1.0$ Is a factor to be applied when the structure is loaded by more than one track, is the most conservative value

Fatigue at the deviation blocks, at the top

$$14 * \frac{1 - E_{cd,\max,equ}}{\sqrt{1 - R_{equ}}} \geq 6 \rightarrow \frac{6}{14} * \sqrt{1 - R_{equ}} + E_{cd,\max,equ} = 0.68 \leq 1.0 \rightarrow Ok$$

Where:

$R_{equ} = \frac{E_{cd,\min,equ}}{E_{cd,\max,equ}} = 0.76$ Stress ratio

$E_{cd,\min,equ} = \gamma_{sd} \frac{\sigma_{cd,\min,equ}}{f_{cd,fat}} = 0.36$ Minimum compressive stress level

$E_{cd,\max,equ} = \gamma_{sd} \frac{\sigma_{cd,\max,equ}}{f_{cd,fat}} = 0.47$ Maximum compressive stress level

$f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd} \left(1 - \frac{f_{ck}}{250}\right) = 19.27 N/mm^2$ Design fatigue strength of concrete

$\beta_{cc}(t) = \exp\left\{ s \left[1 - \left(\frac{28}{t} \right)^{1/2} \right] \right\} \rightarrow \beta_{cc}(t28) = 1.0$ Coefficient for concrete strength at first load application

$k_1 = 0.85$ Recommended value for $N = 10^6$ cycles

$\gamma_{sd} = 1.15$ Is the partial factor for model uncertainty for action/action effort

$\sigma_{cd,\max,equ} = \sigma_{c,perm} - \lambda_c (\sigma_{c,\max} - \sigma_{c,perm}) = 7.89 N/mm^2$ Upper stress of the ultimate amplitude for N cycles

$\sigma_{cd,\min,equ} = \sigma_{c,perm} - \lambda_c (\sigma_{c,perm} - \sigma_{c,\min}) = 5.98 N/mm^2$ Lower stress of the ultimate amplitude for N cycles

$\sigma_{c,perm} = 5.98 N/mm^2$ Permanent stress, without variable load, see Eq. (13)

$\sigma_{c,\max} = 8.59 N/mm^2$ Maximum compressive stress, with variable load, see Eq. (9)

$\sigma_{c,\min} = 5.98 N/mm^2$ Minimum compressive stress, without variable load, see Eq. (13)

$\lambda_c = \lambda_{c,0} * \lambda_{c,1} * \lambda_{c,2,3} * \lambda_{c,4} = 0.73$ Correction factor to calculate the upper and lower stresses of the damage equivalent stress

$\lambda_{c,0} = 0.94 + 0.2 \frac{\sigma_{c,perm}}{f_{cd,fat}} \geq 1 = 1.00$ Is a factor to take account of the permanent stress

$\lambda_{c,1} = 0.75$ Is a factor accounting for element type, see Table NN.3 [12]: (1) compression zone, s* standard traffic mix and simply supported beam

$$\lambda_{c,2,3} = 1 + \frac{1}{8}\log\left[\frac{Vol}{25*10^6}\right] + \frac{1}{8}\log\left[\frac{N_{years}}{100}\right] = 0.98$$ Is a factor to take account of the traffic volume and the design life of the bridge

$Vol = Q_{metro} / g * 6 * 24 * 365 = 15848367 tonnes / year / track$ Assumption of 6 metros per hour

$Q_{metro} = q_{mob} * 116m = 2958kN$ 116 metres is the length of a metro

$N_{years} = 100 years$ Is the design life of the viaduct

$\lambda_{c,4} = 1.0$ Is a factor to be applied when the structure is loaded by more than one track, is the most conservative value

Fatigue at the deviation blocks, at the bottom

$$14 * \frac{1 - E_{cd,\max,equ}}{\sqrt{1 - R_{equ}}} \geq 6 \rightarrow \frac{6}{14} * \sqrt{1 - R_{equ}} + E_{cd,\max,equ} = 0.998 \leq 1.0 \rightarrow Ok$$

Where:

$R_{equ} = \frac{E_{cd,\min,equ}}{E_{cd,\max,equ}} = 0.75$ Stress ratio

$E_{cd,\min,equ} = \gamma_{sd} \frac{\sigma_{cd,\min,equ}}{f_{cd,fat}} = 0.59$ Minimum compressive stress level

$E_{cd,\max,equ} = \gamma_{sd} \frac{\sigma_{cd,\max,equ}}{f_{cd,fat}} = 0.78$ Maximum compressive stress level

$f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd}\left(1 - \frac{f_{ck}}{250}\right) = 19.27 N / mm^2$ Design fatigue strength of concrete

$\beta_{cc}(t) = \exp\left\{s\left[1 - \left(\frac{28}{t}\right)^{1/2}\right]\right\} \rightarrow \beta_{cc}(t28) = 1.0$ Coefficient for concrete strength at first load application

$k_1 = 0.85$ Recommended value for $N = 10^6$ cycles

$\gamma_{sd} = 1.15$ Is the partial factor for model uncertainty for action/action effort

$\sigma_{cd,\max,equ} = \sigma_{c,perm} - \lambda_c(\sigma_{c,\max} - \sigma_{c,perm}) = 13.12 N / mm^2$ Upper stress of the ultimate amplitude for N cycles

$\sigma_{cd,\min,equ} = \sigma_{c,perm} - \lambda_c(\sigma_{c,perm} - \sigma_{c,\min}) = 9.80 N / mm^2$ Lower stress of the ultimate amplitude for N cycles

$\sigma_{c,perm} = 13.12 N / mm^2$ Permanent stress, without variable load, see Eq. (14)

$\sigma_{c,\max} = 13.12 N / mm^2$ Maximum compressive stress, without variable load, see Eq. (14)

$\sigma_{c,\min} = 8.90 N/mm^2$ — Minimum compressive stress, with variable ` load, see Eq. (10)

$\lambda_c = \lambda_{c,0} * \lambda_{c,1} * \lambda_{c,2,3} * \lambda_{c,4} = 0.79$ — Correction factor to calculate the upper and lower stresses of the damage equivalent stress

$\lambda_{c,0} = 0.94 + 0.2 \frac{\sigma_{c,perm}}{f_{cd,fat}} \geq 1 = 1.08$ — Is a factor to take account of the permanent stress

$\lambda_{c,1} = 0.75$ — Is a factor accounting for element type, see Table NN.3 [12]: (1) compression zone, s* standard traffic mix and simply supported beam

$\lambda_{c,2,3} = 1 + \frac{1}{8}\log\left[\frac{Vol}{25*10^6}\right] + \frac{1}{8}\log\left[\frac{N_{years}}{100}\right] = 0.98$ — Is a factor to take account of the traffic volume and the design life of the bridge

$Vol = Q_{metro} / g * 6 * 24 * 365 = 15848367 tonnes / year / track$ Assumption of 6 metros per hour

$Q_{metro} = q_{mob} * 116m = 2958kN$ — 116 metres is the length of a metro

$N_{years} = 100 years$ — Is the design life of the viaduct

$\lambda_{c,4} = 1.0$ — Is a factor to be applied when the structure is loaded by more than one track, is the most conservative value

Conclusion

The fatigue verification for concrete is satisfied but it is an important issue in the design of the box girder as some unity checks are near the limit 1.0. When this verification is not satisfied the best way is to increase the thickness of the bottom flange which decreases the compressive stress in the concrete at the bottom and also brings down the centroidal axis of the box girder and thus reduces the upward prestressing force, see Figure 74.

B.10.3 Vibration

For the box girder only the static analysis is considered. The dynamic metro load is multiplied by the dynamic factor ϕ to take into account the dynamic loading. This method of calculation holds when the first natural frequency of the box girder stays within the prescribed limits [10]. When the limits are exceeded a dynamic analysis is required. A dynamic analysis can prove that the box girder is still determined against the dynamic effects. Such an analysis is however extensive and more difficult and is therefore left out of the design of the box girder. For this design the first natural frequency of the box girder should stay within the limits such that a static analysis is sufficient and a dynamic analysis is not necessary. The check for determining whether a dynamic analysis is required is done according two verifications which are elaborated below.

Verification according Annex F [10]

The first natural bending frequency of the box girder is [20]:

$$n_0 = \frac{C_{end}}{2\pi}\sqrt{\frac{E_{cm} I_c}{A_c \rho_c L^4}} = 3.43 Hz$$

Where:

$C_{end} = 9.94$ — Boundary condition coefficient [20]

$I_c = 5.387 m^4$ — Moment of inertia

The velocity of the metros is:

$$v = 100 km/h = 27.78 m/s$$

Table 21 gives the maximum value of the velocity divided by the first natural frequency of:

$$(v/n_0)_{\lim} = 14.73m$$

Mass m 10^3 kg/m		≥5,0 <7,0	≥7,0 <9,0	≥9,0 <10,0	≥10,0 <13,0	≥13,0 <15,0	≥15,0 <18,0	≥18,0 <20,0	≥20,0 <25,0	≥25,0 <30,0	≥30,0 <40,0	≥40,0 <50,0	≥50,0 -
Span $L \in$ m [a]	ζ %	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m
[5,00,7,50)	2	1,71	1,78	1,88	1,88	1,93	1,93	2,13	2,13	3,08	3,08	3,54	3,59
	4	1,71	1,83	1,93	1,93	2,13	2,24	3,03	3,08	3,38	3,54	4,31	4,31
[7,50,10,0)	2	1,94	2,08	2,64	2,64	2,77	2,77	3,06	5,00	5,14	5,20	5,35	5,42
	4	2,15	2,64	2,77	2,98	4,93	5,00	5,14	5,21	5,35	5,62	6,39	6,53
[10,0,12,5)	1	2,40	2,50	2,50	2,50	2,71	6,15	6,25	6,36	6,36	6,45	6,45	6,57
	2	2,50	2,71	2,71	5,83	6,15	6,25	6,36	6,36	6,45	6,45	7,19	7,29
[12,5,15,0)	1	2,50	2,50	3,58	3,58	5,24	5,24	5,36	5,36	7,86	9,14	9,14	9,14
	2	3,45	5,12	5,24	5,24	5,36	5,36	7,86	8,22	9,53	9,76	10,36	10,48
[15,0,17,5)	1	3,00	5,33	5,33	5,33	6,33	6,33	6,50	6,50	6,50	7,80	7,80	7,80
	2	5,33	5,33	6,33	6,33	6,50	6,50	10,17	10,33	10,33	10,50	10,67	12,40
[17,5,20,0)	1	3,50	6,33	6,33	6,33	6,50	6,50	7,17	7,17	10,67	12,80	12,80	12,80
[20,0,25,0)	1	5,21	5,21	5,42	7,08	7,50	7,50	13,54	13,54	13,96	14,17	14,38	14,38
[25,0,30,0)	1	6,25	6,46	6,46	10,21	10,21	10,21	10,63	10,63	12,75	12,75	12,75	12,75
[30,0,40,0)	1				10,56	18,33	18,33	18,61	18,61	18,89	19,17	19,17	19,17
≥40,0	1				14,73	15,00	15,56	15,56	15,83	18,33	18,33	18,33	18,33

[a] $L \in [a,b)$ means $a \le L < b$

NOTE 1 Table F.1 includes a safety factor of 1.2 on $(v/n_0)_{\lim}$ for acceleration, deflection and strength criteria and a safety factor of 1,0 on the $(v/n_0)_{\lim}$ for fatigue.

NOTE 2 Table F.1 includes an allowance of $(1+\varphi''/2)$ for track irregularities.

Table 21: Maximum value of $(v/n_0)_{\lim}$ for a simply supported beam or slab and a maximum permitted acceleration of $\alpha_{\max} < 3.5m/s^2$, Table F.1 [10]

With:

$$m = A_c * \rho_c = 10.4*10^3 kg/m$$

$$L = 45m$$

The verification of the ratio of the velocity over the first natural frequency is:

$$v/n_0 = 8.09m \le 14.73m \rightarrow Ok$$

Verification according to Figure 6.10 [10]

Limits of natural frequency n_0 (Hz) as a function of L (m)

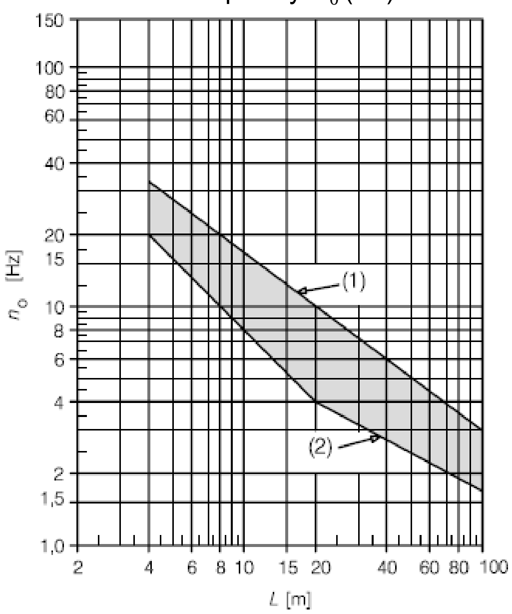

Figure 121: Limits of bridge natural frequency n_0 (Hz) as a function of L (m)

According this verification the first natural frequency of the box girder should be in the grey area, see Figure 121.

Where:
The upper limit of natural frequency is governed by dynamic enhancements due to track irregularities and is given by:

$$n_{0\max} = 94.76 * L^{-0.748} = 5.5Hz$$

The lower limit of natural frequency is governed by dynamic impact criteria and is given by:

$$n_{0\min} = 23.58 * L^{-0.592} = 2.48Hz$$

The first natural frequency of the box girder is:

$$n_0 = \frac{C_{end}}{2\pi}\sqrt{\frac{E_{cm}I_c}{A_c\rho_c L^4}} = 3.43Hz \rightarrow Ok$$

Conclusion

Both verifications show that the box girder does not require a dynamic analysis and a static analysis is sufficient. As the first natural frequency of the girder easily stays within the limits, the box girder is well determined against the dynamic effects. The increasing and decreasing of static stresses and deformations under the effects of moving traffic should, considering the calculations, not give any problems for this box girder.

B.11 Buckling webs

Verification of buckling is needed for the webs of the box girder. The buckling strength of the webs should meet the requirement:

$$F_k \geq \alpha_{cr} * F_d$$

Where:

$F_k = \dfrac{\pi^2 EI}{l_0^2}$ Euler buckling force

$\alpha_{cr} = 10$ Force amplifier to reach the elastic critical buckling

$F_d = Max\{V_{Ed,dl0}/2; V_{Ed,s\infty}/2 + V_{Ed+w}\} * \cos\alpha_w = 2106kN$ Buckling force in one web, see B.7

$V_{Ed,dl0} = 4047kN$ See Eq. (18)

$V_{Ed,s\infty} = 3150kN$ See Eq. (19)

$V_{Ed+w} = 491kN$ See Eq. (21)

$\alpha_w = 11.1°$ Angle of webs with vertical axis, see Figure 123

$E_{cm} = 37278N/mm^2$ Secant modulus of elasticity of concrete

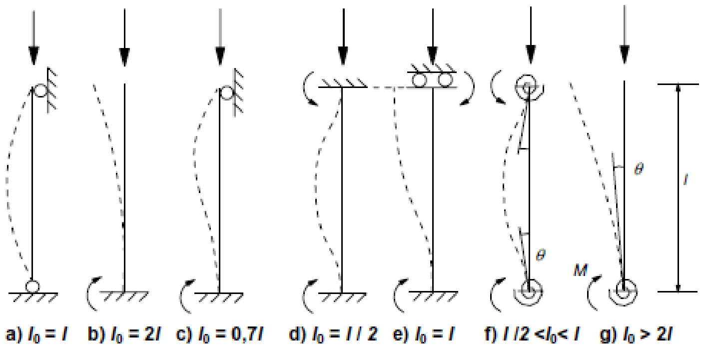

Figure 122: Examples of different buckling modes and corresponding effective lengths for isolated members

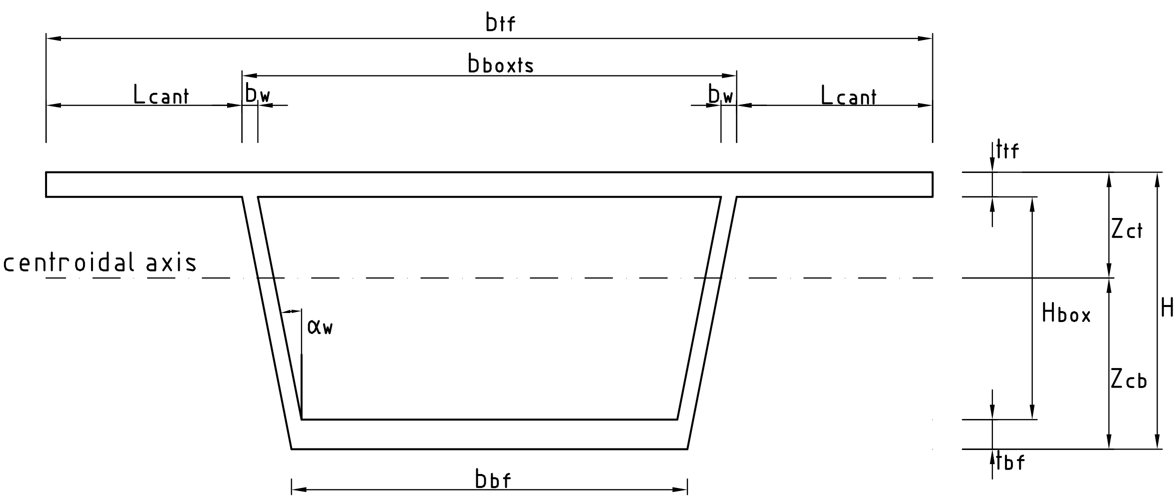

Figure 123: Cross-section of the box girder

The webs are fixed to the flanges. This would mean that buckling mode d, see Figure 122, can be considered to determine the effective buckling length. But as the webs and flanges are relatively slender full rotation stiffness is not likely to occur. In reality buckling mode f should be taken to calculate the effective buckling length of the webs. The rotation stiffness is dependent on the stiffness of the flanges. To determine this rotation stiffness a more extensive calculation is necessary. To be able to make a simple verification of buckling it is therefore chosen to schematise the webs as buckling mode a, see Figure 122. This is the most conservative buckling mode for the webs, where the effective buckling length equals the length of the webs:

$$l_0 = \sqrt{H_{box}^2 + ((b_{boxts} - b_{bf})/2)^2} = 2.305m$$

Now the required moment of inertia of the webs can be calculated:

$$\frac{\pi^2 EI}{l_0^2} \geq \alpha_{cr} * F_d \rightarrow I \geq \frac{l_0^2 * \alpha_{cr} * F_d}{\pi^2 * E} = 304085832mm^4$$

The formula for the moment of inertia of the web is:

$$I = \frac{1}{12} * L_{webs} * t_w^3 \geq 304085832mm^4$$

In this formula L_{webs} is the effective length of the webs in longitudinal direction of the box girder which can be taken for the buckling resistance. It is hard to determine this effective length, especially for a segmental box girder with its joints between the segments creating discontinuities in the webs. For this calculation it is chosen to take effective length of the webs as 1 metre. This is chosen as in the local schematisation of the deck, see B.9.2, the local metro point load is distributed over 1 metre in the longitudinal direction of the box girder. In the deck schematisation it is therefore chosen to take a deck width of 1 metre. This local deck load should be taken by the webs. For this reason an effective length of the webs of 1 metre is chosen in respect of buckling of the webs. Besides, this assumption is considered as quite conservative as buckling of the webs will probably concern more than 1 metre. Most likely the effective length of the webs equals the length of a segmental box girder, which means a length of 3 metres. However, in this buckling verification a safe assumption of the effective length is taken:

$$L_{webs} = 1m$$

The minimum required thickness of the webs hereby becomes:

$$t_{w,req} = \sqrt[3]{\frac{304085832 * 12}{L_{webs}}} = 154mm$$

The minimum required width of the webs hereby becomes:

$$b_{w,req} = t_w / \cos\alpha_w = 157mm$$

Verification of buckling of the webs:

$$b_w = 160mm \geq b_{w,req} = 157mm \rightarrow Ok$$

The webs thus satisfy with respect to buckling. The thickness/width of the webs is however just enough to resist buckling. When this verification is not satisfied the thickness of the webs should be increased.

Appendix C: Calculations UHPC box girder C180

C.1 Introduction

This Appendix presents the calculations of the optimal box girder in Ultra High Performance Concrete (UHPC) C180. First the material characteristics of UHPC and steel are described. Paragraph 3 deals with the geometry and the structural schematisation of the box girder and its characteristics. The loads to which the box girder is subjected are treated in paragraph 4. In the next paragraph the layout of the external prestressing tendons is shown and the stresses in the box girder due to loading and prestressing are calculated. It also contains the calculations of the prestressing losses. Furthermore this Appendix describes the calculations on deflection, shear and torsion and the ultimate resistance moment of the box girder in respectively the paragraphs 6, 7 and 8. The calculations on the deck thickness are treated in paragraph 9. Finally this Appendix deals with the calculations on fatigue and vibration of the box girder and buckling of the webs.

The formulas and values used in the calculations are taken from [11] and other references, which are then stated in the text.

C.2 Material characteristics

C.2.1 UHPC C180: Ductal®-AF

Density of UHPC [8] [i5]	ρ_c	2600 kg/m^3
Partial factor for UHPC	γ_c	1.5
Additional partial safety factor [16]	$\gamma_c^{'}$	1.25
Characteristic compressive cylinder strength of UHPC [i5]	f_{ck}	180 N/mm^2
Characteristic axial tensile strength of UHPC [i5]	f_{ctk}	8.0 N/mm^2
Secant modulus of elasticity of UHPC [i5]	E_{cm}	50000 N/mm^2
Orientation coefficient global effects [18]	K	1.25
Orientation coefficient local effects [18]	K	1.75
Partial safety factor fundamental combinations [18]	γ_{bf}	1.3
Design value of UHPC compressive strength, SLS [18]	$f_{cd,sls} = 0.6 * f_{ck}$	108 N/mm^2
Design value of UHPC compressive strength, ULS [18] [16]	$f_{cd,uls} = 0.85 * f_{ck} / (\gamma_c * \gamma_c^{'})$	81.6 N/mm^2
Design value of UHPC tensile strength, SLS global effects [18]	$f_{ctd,slsG} = f_{ctk} / K$	6.4 N/mm^2
Design value of UHPC tensile strength, ULS global effects [18]	$f_{ctd,ulsG} = f_{ctk} / (K * \gamma_{bf})$	4.92 N/mm^2
Design value of UHPC tensile strength, ULS local effects [18]	$f_{ctd,ulsL} = f_{ctk} / (K * \gamma_{bf})$	3.52 N/mm^2
Flexural tensile strength UHPC [i5]	$f_{ctm,fl}$	30 N/mm^2
Compressive strain in the UHPC at the end of the linear part [18]	$\varepsilon_{c3} = f_{cd,uls} / E_{cm}$	1.632 ‰
Ultimate compressive strain in the UHPC [18]	ε_{cu3}	3.0 ‰

C.2.2 Reinforcing steel FeB 500

Density of reinforcing steel	ρ_s	7850 kg/m^3
Characteristic yield strength of reinforcement	f_{yk}	500 N/mm^2
Partial factor for reinforcing steel	γ_s	1.15
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_s$	435 N/mm^2
Design value of modulus of elasticity of reinforcing steel	E_s	200,000 N/mm^2

C.2.3 Prestressing steel FeP 1860

Density of prestressing steel	ρ_p	7850 kg/m^3
Characteristic tensile strength of prestressing steel [24]	f_{pk}	1860 N/mm^2
Characteristic 0.1% proof-stress of prestressing steel [24]	$f_{p0.1k}$	1600 N/mm^2
Partial factor for prestressing steel	γ_s	1.15
Design tensile strength of prestressing steel	$f_{pd} = f_{p0.1k} / \gamma_s$	1391 N/mm^2
Ultimate tensile strength of prestressing steel	f_{pk} / γ_s	1617 N/mm^2
Design value of modulus of elasticity of prestressing steel	E_p	200,000 N/mm^2
Factor	k_1	0.8
Factor	k_2	0.9
Factor	k_7	0.75
Factor	k_8	0.85
Maximum tensile stress in the tendon (during tensioning)	$\sigma_{p,\max} = \min\{k_1 * f_{pk}; k_2 * f_{p0.1k}\}$	1440 N/mm^2
Maximum tensile stress in the tendon (after tensioning, initial stress at t=0)	$\sigma_{pm0} = \min\{k_7 * f_{pk}; k_8 * f_{p0.1k}\}$	1360 N/mm^2

C.3 Geometry box girder

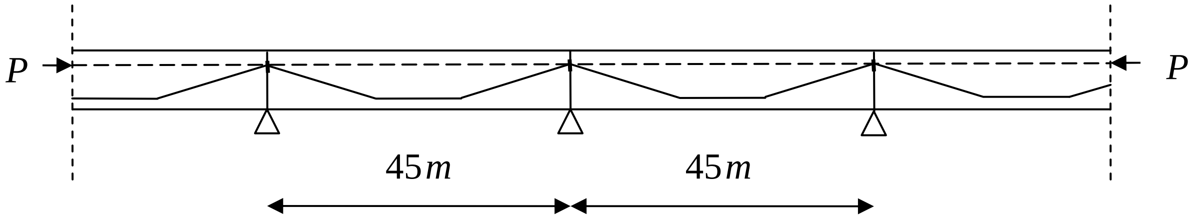

Figure 124: Statically determinate box girders supported by columns

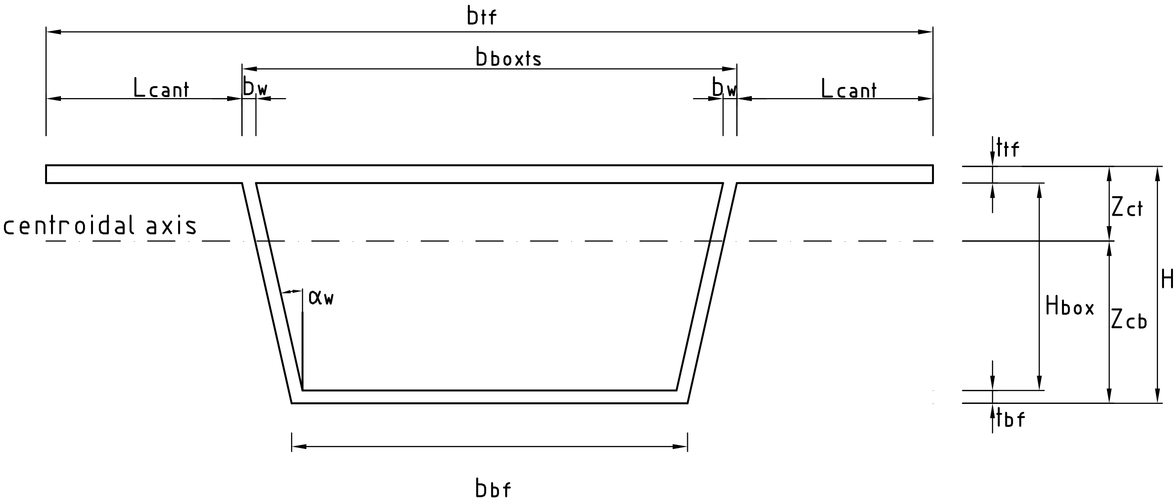

Figure 125: Cross-section of the box girder

C.3.1 General

Length span	L	45	m
Depth box girder	H	2.41	m
Width top flange	b_{tf}	8.96	m
Thickness top flange	t_{tf}	0.18	m
Width web	b_w	0.14	m
Width bottom flange	b_{bf}	4	m
Thickness bottom flange	t_{bf}	0.13	m
Width box top side	b_{boxts}	5	m
Cantilever length top flange	L_{cant}	1.98	m
Depth webs	H_{box}	2.1	m

Angle of webs with vertical axis $\alpha_w = \tan^{-1}\left(\frac{(b_{boxts} - b_{bf})/2}{H_{box} + t_{bf}}\right) = 12.64°$

C.3.2 Determination of cantilever length

The rules of thumbs to determine the top flange thickness given below are true for normal concrete.

$$t_{tf} = \frac{1}{10} L_{cant}$$

$$t_{tf} = \frac{1}{30} * (b_{tf} - 2(b_w + L_{cant}))$$

For UHPC these rules are not applicable. But to be able to make a good comparison between the two materials (C50/60 versus C180) it is chosen to take the same geometry for the box as is chosen for concrete C50/60:

b_{boxts}	5	m
L_{cant}	1.98	m
b_{bf}	4	m

The width of the bottom flange b_{bf} is chosen smaller than the width of the box top side b_{boxts}. This way the railway girder requires a smaller support and the angle α_w is still small enough for the webs to transfer the vertical loads mainly by normal forces than by bending. The dimensions b_{bf} and b_{boxts} are deduced from reference projects, see Figure 18 [6].

C.3.3 Concrete cover

The very dense material structure of UHPC results in a higher durability and smaller concrete cover compared with concrete C50/60.
The concrete cover for the box girder made of C50/60 is:

$$c_{nom} = c_{\min} + \Delta c_{dev} = 40mm$$

The concrete cover for UHPC is assumed to be half of this value:

$$c = 20mm$$

C.3.4 Effective width of flanges

The effective width of the flanges is based on the distance l_0 between points of zero moment, see Figure 126. However, with a structural schematisation as given in Figure 124 the distance l_0 is 45 metres.

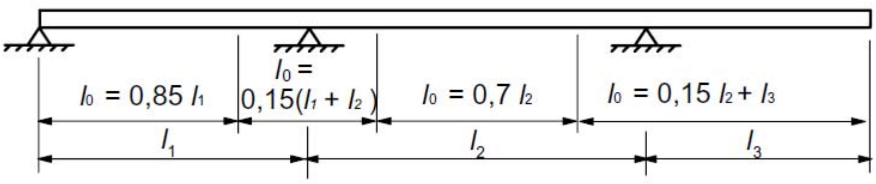

Figure 126: Definition of l_0, for calculation of effective flange width

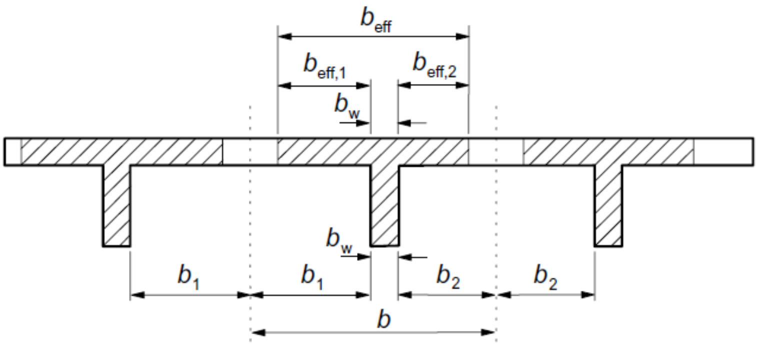

Figure 127: Effective flange width parameters

This gives:
$l_0 = 45m$

Cantilever length top flange	$b_1 = L_{cant} = 1.98m$
Width inner top flange	$b_2 = b_{boxts} / 2 - b_w = 2.36m$
Width bottom flange	$b_3 = b_{bf} / 2 - b_w = 1.86m$

Effective flange width:
$b_{eff} = \sum b_{eff,i} + b_w \leq b$

Where:
$b_{eff,i} = 0.2b_i + 0.1l_0 \leq 0.2l_0$
And
$b_{eff,i} \leq b_i$

Effective width of flanges

		Value	
Effective width cantilever length top flange	$b_{eff,1}$	1.98	m
Effective width inner top flange	$b_{eff,2}$	2.36	m
Effective width bottom flange	$b_{eff,3}$	1.86	m

Total effective flange width

		Value	
Effective width top flange	$b_{eff,t}$	8.96	m
Effective width bottom flange	$b_{eff,b}$	4	m

C.3.5 Cross-sectional properties

Cross-sectional area of UHPC
$A_c = b_{tf} * t_{tf} + 2 * b_w * H_{box} + b_{bf} * t_{bf}$

Distance from bottom to centroidal axis
$Z_{cb} = (b_{tf} * t_{tf} * (H - t_{tf} / 2) + 2 * b_w * H_{box} * (H_{box} / 2 + t_{bf}) + b_{bf} * t_{bf} * (t_{tf} / 2)) / A_c$

Distance from top to centroidal axis
$Z_{ct} = H - Z_{cb}$

Moment of inertia of UHPC section

$$I_c = \frac{1}{12} * b_{eff,t} * t_{tf}^3 + b_{eff,t} * t_{tf} * (Z_{ct} - t_{tf}/2)^2 +$$

$$2 * \frac{1}{12} * b_w * H_{box}^3 + 2 * b_w * H_{box} * (Z_{cb} - H_{box}/2 - t_{bf})^2 +$$

$$\frac{1}{12} * b_{eff,b} * t_{bf}^3 + b_{eff,b} * t_{bf} * (Z_{cb} - t_{bf}/2)^2$$

Section modulus bottom

$$W_b = I_c / Z_{cb}$$

Section modulus top

$$W_t = I_c / Z_{ct}$$

Perimeter UHPC box girder

$$u = b_{tf} + 2 * t_{tf} + 2 * L_{cant} + 2 * \sqrt{((b_{boxts} - b_{bf})/2)^2 + (H_{box} + t_{bf})^2} + b_{bf}$$

Values cross-sectional properties box girder

		Value	
Cross-sectional area of UHPC	A_c	2.721	m^2
Distance from bottom to centroidal axis	Z_{cb}	1.643	m
Distance from top to centroidal axis	Z_{ct}	0.767	m
Second moment of area of the UHPC section	I_c	2.381	m^4
Section modulus bottom	W_b	1.450	m^3
Section modulus top	W_t	3.103	m^3
Perimeter UHPC box girder	u	21.851	m

C.4 Loads

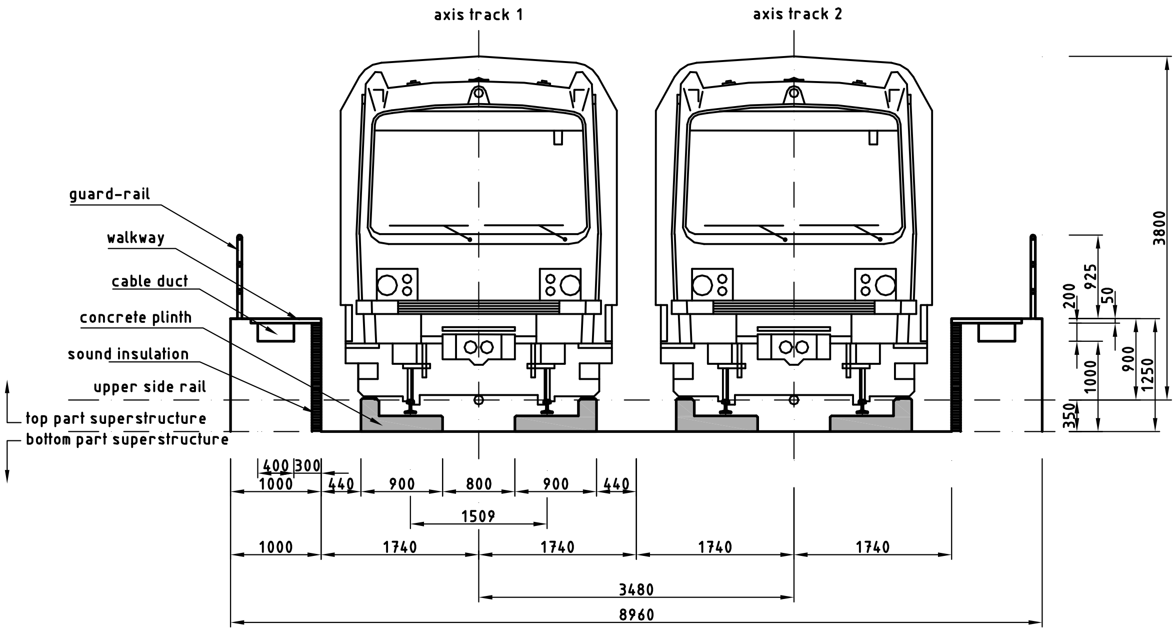

Figure 128: Cross-section top part superstructure

C.4.1 General

Acceleration due to gravity	g	9.81	m/s^2
Dynamic factor [8]	$\phi = 1 + 4/(10 + L)$	1.07	
Partial factor for permanent actions, favourable [9]	$\gamma_{G,fav}$	1.0	
Partial factor for permanent actions, unfavourable [9]	$\gamma_{G,unfav}$	1.35	
Partial factor for variable actions, favourable [9]	$\gamma_{Q,fav}$	0	
Partial factor for variable actions, unfavourable [9]	$\gamma_{Q,unfav}$	1.5	
Partial factor for prestress, favourable	$\gamma_{P,fav}$	1	
Partial factor for prestress, unfavourable	$\gamma_{P,unfav}$	1.3	
Factor for combination value of snow load [9]	$\psi_{0,snow}$	0.8	
Factor for combination value of wind load [9]	$\psi_{0,wind}$	0.75	
Factor for combination value of sideward force [9]	$\psi_{0,sidewf}$	0.8	

C.4.2 Vertical loads

Dead load box girder	$g_{dead} = A_c * \rho_c * g$	69.4	kN/m
Permanent loads [8]:			
Concrete plinths		10	kN/m per track
Rail (S49)		0.97	kN/m per track
Cables		1.2	kN/m per cable duct
Walkway + guard-rail		2	kN/m per walkway
Sound insulation		1.3	kN/m per walkway

Concrete slope (drainage between walkways)		0.5	kN/m^2
Variable loads [8]:			
Mobile load (metros)	q_{mob}	25.5	kN/m per track
Snow load	q_{snow}	0.5	kN/m^2
Concentrated load due to the metro (for local schematisation)	$Q_{mob,loc}$	130	kN per track

C.4.3 Horizontal loads

Wind load[9] [8]	q_{wind}	1.5	kN/m^2
Sideward force due to the metro[10] [8]	Q_{sidewf}	30	kN per track

C.4.4 Load schematisation in longitudinal direction

Serviceability limit state (SLS)

Vertical loads

Dead load box girder	g_{dead}	69.4	kN/m
Permanent loads:			
Concrete plinths	(* 2 tracks)	20	kN/m
Rail (S49)	(* 2 tracks)	1.94	kN/m
Cables	(* 2 cable ducts)	2.4	kN/m
Walkways + guardrails	(* 2 walkways)	4	kN/m
Sound insulation	(* 2 walkways)	2.6	kN/m
Concrete slope (drainage between walkways)	(*(b_{tf} - 2 * 1 m))	3.48	kN/m
Total permanent load	g_{perm}	34.42	kN/m
Variable loads:			
Mobile load (metros)	q_{mob} * 2 tracks * ϕ	54.71	kN/m
Snow load	$\psi_{0,snow} * q_{snow} * b_{tf}$	3.58	kN/m
Total variable load	q_{var}	58.29	kN/m

Ultimate limit state (ULS)

Vertical loads

Dead load box girder	$\gamma_{G,unfav} * g_{dead}$	93.69	kN/m
Total permanent load	$\gamma_{G,unfav} * g_{perm}$	46.47	kN/m
Total variable load	$\gamma_{Q,unfav} * q_{var}$	87.44	kN/m

[9] The viaduct is subjected to wind forces up to a height of 3.6 metres above the upper side of the rail.
[10] The sideward force acts at 1.5 metres above the upper side of the rail, in the centre of the track.

C.4.5 Load schematisation in transversal direction

Serviceability limit state (SLS)

Vertical loads

Permanent loads:			
Concrete plinths	(/ 2 plinths per track / 0.9 m (width plinth))	5.56	kN/m
Rail (S49)	(/ 2 rails per track)	0.485	kN per rail
Cables		1.2	kN per cable duct
Walkway + guardrail	(/ 1 m (width walkway))	2	kN/m
Sound insulation		1.3	kN per walkway
Concrete slope (drainage between walkways)		0.5	kN/m
Variable loads:			
Concentrated load due to the metro (for local schematisation)	$Q_{mob,loc}$ / 2 rails per track * ϕ	69.73	kN per rail
Snow load	$\psi_{0,snow} * q_{snow}$	0.4	kN/m

Horizontal loads

Wind load	$\psi_{0,wind} * q_{wind}$	1.125	kN/m^2
Sideward force due to the metro	$\psi_{0,sidewf} * Q_{sidewf}$	24	kN per track

Ultimate limit state (ULS)

Vertical loads

Permanent loads:			
Concrete plinths	5.56 * $\gamma_{G,unfav}$	7.5	kN/m
Rail (S49)	0.485 * $\gamma_{G,unfav}$	0.65	kN per rail
Cables	1.2 * $\gamma_{G,unfav}$	1.62	kN per cable duct
Walkway + guardrail	2 * $\gamma_{G,unfav}$	2.7	kN/m
Sound insulation	1.3 * $\gamma_{G,unfav}$	1.76	kN per walkway
Concrete slope (drainage between walkways)	0.5 * $\gamma_{G,unfav}$	0.68	kN/m
Variable loads:			
Concentrated load due to the metro (for local schematisation)	69.73 * $\gamma_{Q,unfav}$	104.59	kN per rail
Snow load	0.4 * $\gamma_{Q,unfav}$	0.6	kN/m

Horizontal loads

Wind load	1.125 * $\gamma_{Q,unfav}$	1.69	kN/m^2
Sideward force due to the metro	24 * $\gamma_{Q,unfav}$	36	kN per track

C.5 Prestressing tendons

C.5.1 Layout prestressing tendons

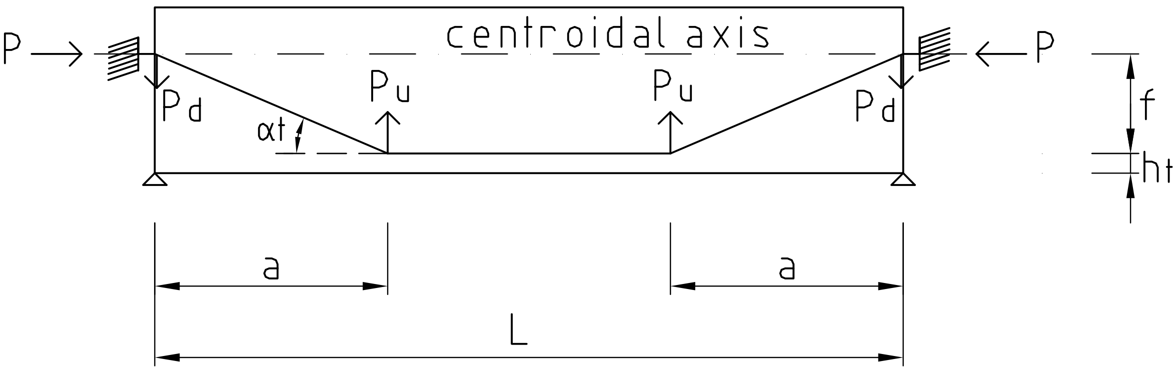

Figure 129: Layout prestressing tendons

Distance between the centre of the tendons and bottom side at mid-span

	h_t	0.5	m
Tendon eccentricity at mid-span	$f = Z_{cb} - h_t$	1.143	m
Distance of deviation blocks to supports	a	17	m

Angle between prestressing tendon and the centroidal axis $\alpha_t = \tan^{-1}(f/a) = 3.845°$

The tendon eccentricity at the support is 0 m as the tendon anchorage coincides with the centroidal axis.

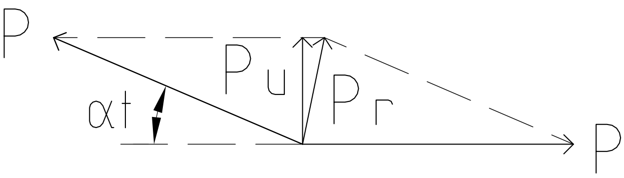

Figure 130: Polygon of prestressing forces

The resulting prestressing force $P_r = 2 * P * \sin(\alpha_t / 2)$ has a small angle with the vertical axis. As the angle is very small the horizontal force of P_r is small. For simplification reasons it is chosen to take into account only the vertical upward prestressing force. The upward prestressing force P_u is dependent of the prestressing force P and the angle a_t

$$P_u = P * \sin \alpha_t$$

C.5.2 Bending moments due to prestressing

The moment diagram and structural schematisation due to prestressing is shown in Figure 131. The elevated metro structure consists of statically determinate box girders supported by columns. Due to symmetry of loading the downward prestressing force at the supports is equal to the upward prestressing force at the deviation blocks.

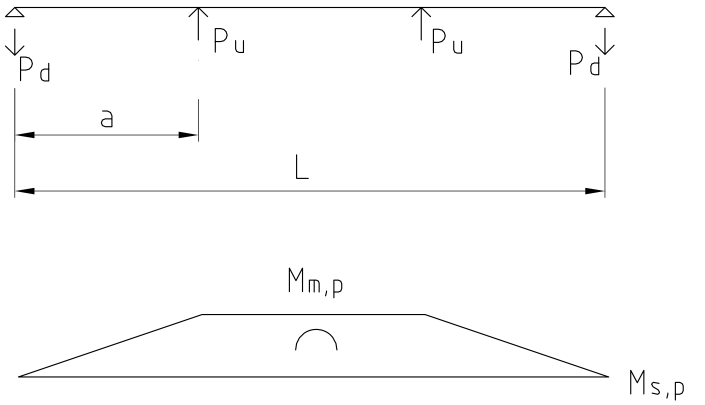

Figure 131: Structural schematisation of the box girder subjected to prestressing forces

Where:

$$P_d = P_u$$

$$M_{s,p} = 0kNm$$

$$M_{m,p} = P_d * a = P_u * a$$

The box girder has 6 tendons externally placed inside the girder according the layout shown in Figure 129. One tendon consists of 37 strands with a diameter of 15.7 mm and a cross-sectional area of 150 mm^2 per strand. The cross-sectional area of one tendon is:

$$A_p = 37 * 150 = 5550mm^2$$

The number of tendons is:

$$n = 6 \ \ tendons$$

The estimated prestressing losses are 20% at t = ∞
The working prestress at t = ∞ then becomes:

$$\sigma_{pm\infty} = 0.8 * \sigma_{pm0} = 1088N/mm^2$$

Hereunder the prestressing forces and bending moments are calculated for the two phases: the construction phase at t = 0 and the end phase at t = ∞.

Construction phase at t = 0

Total prestressing force:

$$P_0 = n * A_p * \sigma_{pm0} = 45288kN \tag{22}$$

Total upward prestressing force:

$$P_{u0} = P_0 * \sin\alpha_t = 3037kN \tag{23}$$

Bending moment between the two deviation blocks

$$M_{m,p0} = P_{u0} * a = 51632kNm(\frown)$$

End phase at t = ∞

The estimated prestressing losses are 20%
The working prestress at t = ∞ then becomes:

$$\sigma_{pm\infty} = 0.8 * \sigma_{pm0} = 1088N/mm^2$$

Total prestressing force:

$$P_\infty = n * A_p * \sigma_{pm\infty} = 36230kN \tag{24}$$

Total upward prestressing force:

$$P_{u\infty} = P_{\infty} * \sin\alpha_t = 2430kN$$

(25)

Bending moment between the two deviation blocks

$$M_{m,p\infty} = P_{u\infty} * a = 41306kNm(\cap)$$

C.5.3 Bending moments due to loads

The bending moments due to the loads are determined according the structural load schematisation shown in Figure 132.

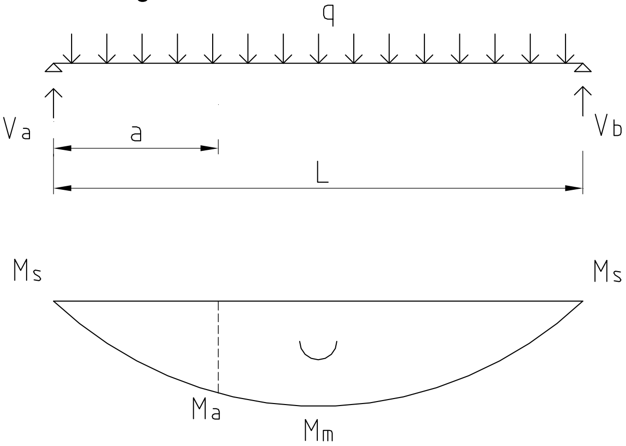

Figure 132: Structural schematisation of the box girder subjected to loads

Where:

$$V_a = V_b = \frac{1}{2}qL$$

$$M_s = 0kNm$$

$$M_m = \frac{1}{8}qL^2$$

$$M_a = \frac{1}{2}qL * a - 0.5 * q * a^2$$

Bending moments in the construction phase at t = 0

At deviation blocks

$$M_{a,0} = \frac{1}{2} * g_{dead} * L * a - 0.5 * g_{dead} * a^2 = 16516kNm(\cup)$$

At mid-span

$$M_{m,0} = \frac{1}{8} * g_{dead} * L^2 = 17566Nm(\cup)$$

Bending moments in the end phase at t = ∞

At deviation blocks

$$M_{a,\infty} = \frac{1}{2} * (g_{dead} + g_{perm} + q_{var}) * L * a - 0.5 * (g_{dead} + g_{perm} + q_{var}) * a^2 = 38582kNm(\cup)$$

At mid-span

$$M_{m,\infty} = \frac{1}{8} * (g_{dead} + g_{perm} + q_{var}) * L^2 = 41034kNm(\cup)$$

Bending moments due to the variable load
At deviation blocks

$$M_{a,v} = \frac{1}{2} * q_{\text{var}} * L * a - 0.5 * q_{\text{var}} * a^2 = 13874 kNm (\cup)$$

At mid-span

$$M_{m,v} = \frac{1}{8} * q_{\text{var}} * L^2 = 14755 kNm (\cup)$$

C.5.4 Stresses due to loading

As the railway girder is a prefabricated segmental box girder the joints between the segments cannot resist tensile stresses without opening of the joints. Opening of the joints is however not allowed so the concrete cannot resist tensile stresses: $\sigma_c \leq 0 N/mm^2$. Furthermore the concrete stress may not become too large. In order to rule out the non-linearity of creep the concrete compressive stress should not exceed $\sigma_c \geq -0.45 * f_{ck} = -81 N/mm^2$. Beneath the stresses at the top and bottom side of the box girder are calculated for different phases. The negative stresses refer to compression and positive stresses to tension.

Construction phase at t = 0

At deviation block, top side

$$\sigma_{ct} = -\frac{P_0}{A_c} + \frac{M_{m,p0}}{W_t} - \frac{M_{a,0}}{W_t} = -5.33 N/mm^2$$

$$-81 N/mm^2 \leq \sigma_{ct} = -5.33 N/mm^2 \leq 0 N/mm^2 \rightarrow Ok \tag{26}$$

At deviation block, bottom side

$$\sigma_{cb} = -\frac{P_0}{A_c} - \frac{M_{m,p0}}{W_b} + \frac{M_{a,0}}{W_b} = -40.87 N/mm^2$$

$$-81 N/mm^2 \leq \sigma_{cb} = -40.87 N/mm^2 \leq 0 N/mm^2 \rightarrow Ok \tag{27}$$

At mid-span, top side

$$\sigma_{ct} = -\frac{P_0}{A_c} + \frac{M_{m,p0}}{W_t} - \frac{M_{m,0}}{W_t} = -5.67 N/mm^2$$

$$-81 N/mm^2 \leq \sigma_{ct} = -5.67 N/mm^2 \leq 0 N/mm^2 \rightarrow Ok \tag{28}$$

At mid-span, bottom side

$$\sigma_{cb} = -\frac{P_0}{A_c} - \frac{M_{m,p0}}{W_b} + \frac{M_{m,0}}{W_b} = -40.14 N/mm^2$$

$$-81 N/mm^2 \leq \sigma_{cb} = -40.14 N/mm^2 \leq 0 N/mm^2 \rightarrow Ok \tag{29}$$

End phase at t = ∞ fully loaded

At deviation block, top side

$$\sigma_{ct} = -\frac{P_\infty}{A_c} + \frac{M_{m,p\infty}}{W_t} - \frac{M_{a,\infty}}{W_t} = -12.44 N/mm^2$$

(30)

$$-81N/mm^2 \le \sigma_{ct} = -12.44N/mm^2 \le 0N/mm^2 \rightarrow Ok$$

At deviation block, bottom side

$$\sigma_{cb} = -\frac{P_\infty}{A_c} - \frac{M_{m,p\infty}}{W_b} + \frac{M_{a,\infty}}{W_b} = -15.19N/mm^2$$

$$-81N/mm^2 \le \sigma_{cb} = -15.19N/mm^2 \le 0N/mm^2 \rightarrow Ok \qquad (31)$$

At mid-span, top side

$$\sigma_{ct} = -\frac{P_\infty}{A_c} + \frac{M_{m,p\infty}}{W_t} - \frac{M_{m,\infty}}{W_t} = -13.23N/mm^2$$

$$-81N/mm^2 \le \sigma_{ct} = -13.23N/mm^2 \le 0N/mm^2 \rightarrow Ok \qquad (32)$$

At mid-span, bottom side

$$\sigma_{cb} = -\frac{P_\infty}{A_c} - \frac{M_{m,p\infty}}{W_b} + \frac{M_{m,\infty}}{W_b} = -13.50N/mm^2$$

$$-81N/mm^2 \le \sigma_{cb} = -13.50N/mm^2 \le 0N/mm^2 \rightarrow Ok \qquad (33)$$

End phase at t = ∞ without variable load

At deviation block, top side

$$\sigma_{ct} = -\frac{P_\infty}{A_c} + \frac{M_{m,p\infty}}{W_t} - \frac{M_{a,\infty} - M_{a,v}}{W_t} = -7.97N/mm^2$$

$$-81N/mm^2 \le \sigma_{ct} = -7.97N/mm^2 \le 0N/mm^2 \rightarrow Ok \qquad (34)$$

At deviation block, bottom side

$$\sigma_{cb} = -\frac{P_\infty}{A_c} - \frac{M_{m,p\infty}}{W_b} + \frac{M_{a,\infty} - M_{s,v}}{W_b} = -24.77N/mm^2$$

$$-81N/mm^2 \le \sigma_{cb} = -24.77N/mm^2 \le 0N/mm^2 \rightarrow Ok \qquad (35)$$

At mid-span, top side

$$\sigma_{ct} = -\frac{P_\infty}{A_c} + \frac{M_{m,p\infty}}{W_t} - \frac{M_{m,\infty} - M_{m,v}}{W_t} = -8.47N/mm^2$$

$$-81N/mm^2 \le \sigma_{ct} = -8.47N/mm^2 \le 0N/mm^2 \rightarrow Ok \qquad (36)$$

At mid-span, bottom side

$$\sigma_{cb} = -\frac{P_\infty}{A_c} - \frac{M_{m,p\infty}}{W_b} + \frac{M_{m,\infty} - M_{m,v}}{W_b} = -23.68N/mm^2$$

$$-81N/mm^2 \le \sigma_{cb} = -23.68N/mm^2 \le 0N/mm^2 \rightarrow Ok \qquad (37)$$

C.5.5 Prestressing losses

Losses due to the instantaneous deformation of concrete

During tensioning the box girder will shorten. As the tendons are prestressed successively there arises an immediate prestressing loss which can be calculated for each tendon with the following formula:

$$\Delta P_{el} = A_p * E_p * \sum \left[\frac{j * \Delta\sigma_c(t)}{E_{cm}} \right]$$

Where:

$A_p = 5550 mm^2$	Cross-sectional area per prestressing tendon
$E_p = 200{,}000 N/mm^2$	Modulus of elasticity of prestressing steel
$E_{cm} = 50000 N/mm^2$	Secant modulus of elasticity of UHPC
$\Delta\sigma_c(t) = \sigma_{pm0} * A_p / A_c = 2.77 N/mm^2$	Is the variation of stress in the UHPC at the centre of gravity of the tendons applied at time t.
$A_c = 2720800 mm^2$	Cross-sectional area of UHPC
$\sigma_{pm0} = 1360 N/mm^2$	Maximum initial tensile stress in the tendon
$j = (n-1)/2n$	Is a coefficient where n is the number of identical tendons successively prestressed.

This prestressing loss taking into account the order in which the tendons are stressed can be compensated by slightly overstressing the tendons. The maximum overstress is needed in the first prestressed tendon as this tendon has the largest loss due the instantaneous deformation of UHPC. The required overstress $\sigma_{overstr}$ in the first prestressed tendon to compensate the losses due to instantaneous deformation of UHPC can be calculated out of the formula below:

$$\Delta P_{el,1} = A_p * E_p * \frac{j * \Delta\sigma_{overstr}}{E_{cm}} = A_p * (\sigma_{overstr} - \sigma_{pm0}) \tag{38}$$

Where:

$\Delta\sigma_{c,overstr}(t) = \sigma_{overstr} * A_p / A_c$ Variation of stress in the concrete

For the first prestressed tendon $n = 6 \rightarrow$

$j = (n-1)/2n = 5/12 = 0.4167$

Now fill in formula (38):

$$A_p * E_p * \frac{j * \Delta\sigma_{overstr}}{E_{cm}} = A_p * (\sigma_{overstr} - \sigma_{pm0}) \rightarrow E_p * \frac{j * \sigma_{overstr} * A_p}{E_{cm} * A_c} = \sigma_{overstr} - \sigma_{pm0} \rightarrow$$

$$E_p * \frac{j * A_p}{E_{cm} * A_c} = 1 - \frac{\sigma_{pm0}}{\sigma_{overstr}} \rightarrow \sigma_{overstr} = \frac{\sigma_{pm0}}{1 - E_p * \frac{j * A_p}{E_{cm} * A_c}} = 1364.64 N/mm^2$$

The maximum allowed tensile stress of the tendons during tensioning is $\sigma_{p,\max} = 1440 N/mm^2$. The stress caused by overstressing is far below this value and as also the UHPC compressive stress during tensioning is limited to $\sigma_c \leq 0.6 * f_{ck} = 108 N/mm^2$ this small overstressing will not cause any problems for the structure. It can be concluded that the losses due to the instantaneous deformation of UHPC can be compensated by overstressing the tendons. By overstressing the tendons the initial tensile stress in all the tendons after tensioning can be the maximum tensile stress $\sigma_{pm0} = 1360 N/mm^2$.

Losses due to friction

The loss due to friction in post-tensioned tendons is:

$$\Delta P_{\mu}(x) = P_{\max}(1 - e^{-\mu(\theta + kx)})$$

Where:

θ	Is the sum of the angular displacement over a distance x (irrespective of direction or sign).
μ	Is the coefficient of friction between the tendon and its duct.
k	Is the unintentional angular displacement for internal tendons (per unit length).
x	Is the distance along the tendon from the point where the prestressing force is equal to $P_{\max}$ (the force at the active end during tensioning).

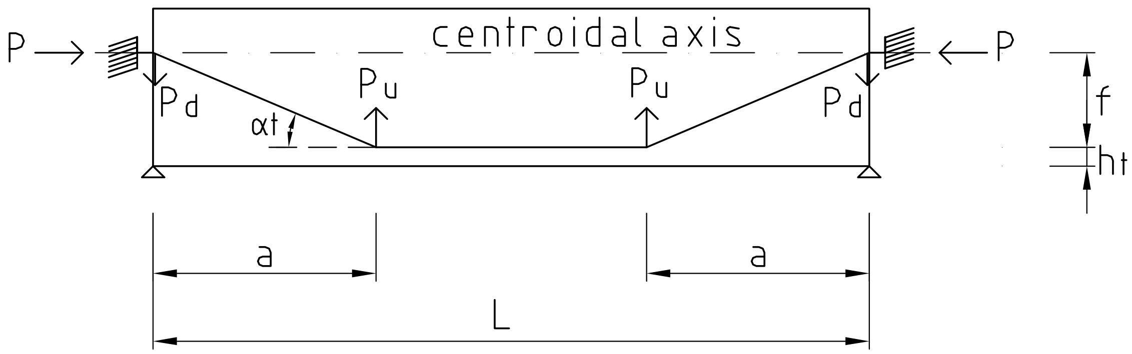

Figure 133: Layout prestressing tendons

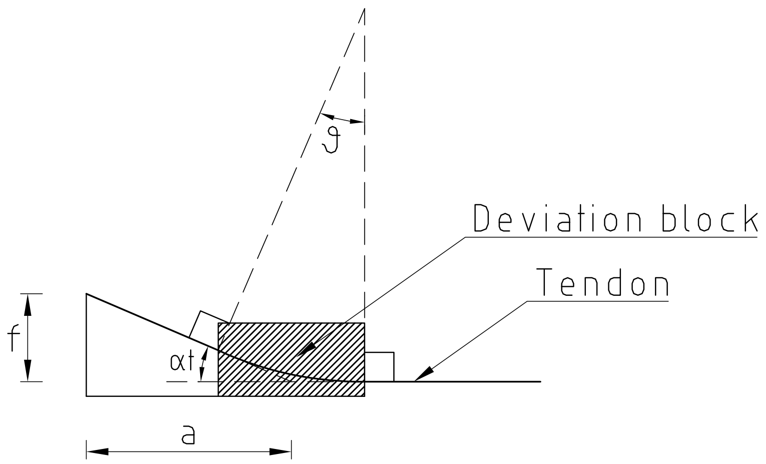

Figure 134: Angular displacement at a deviation block

There are four places where tendon deviation takes place, namely: at the two supports and at the two deviation blocks at a distance a from the supports.

The angular displacement per deviation is: $\theta = \alpha_t = \dfrac{f}{a} = 0.07 rad$

For external tendons, the losses of prestress due to unintentional angles may be ignored [11], so the loss due to friction per deviation is:

$$\Delta P_{\mu}(x) = P_{\max}(1 - e^{-\mu * \theta}) = 302.93 kN$$

Where

$P_{\max} = P_0 = n * A_p * \sigma_{pm0} = 45288 kN$

$\mu = 0.1$ See table 5.1 [11] (external unbonded tendons; HDPE duct / lubricated; strand)

Time dependent losses of prestress for post-tensioning

The time dependent loss of prestress for post-tensioning at a location x is calculated according the formula below:

$$\Delta P_{c+s+r} = A_p \Delta\sigma_{p,c+s+r} = A_p \frac{\varepsilon_{cs} E_p + 0.8\Delta\sigma_{pr} + \frac{E_p}{E_{cm}}\varphi(\infty,t_0) * \sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}} \frac{n * A_p}{A_c}(1 + \frac{A_c}{I_c} z_{cp}^2)[1 + 0.8\varphi(t,t_0)]}$$

Where:

Creep

$\varphi(\infty,t_0) = 0.3$ Is the final creep coefficient [i5]

Shrinkage

$\varepsilon_{cs} = 0.01‰ = 0.00001$ Total shrinkage strain in absolute value [i5]

Relaxation

Relaxation class 2 (wire or strand):

$$\Delta\sigma_{pr} = \sigma_{pi} * 0.66 * \rho_{1000} * e^{9.1\mu}\left(\frac{t}{1000}\right)^{0.75(1-\mu)} * 10^{-5} = 60.93 N/mm^2$$

Where:

$\sigma_{pi} = \sigma_{pm0} = 1360 N/mm^2$

Relaxation class 2 $\rightarrow \rho_{1000} = 2.5\%$

$\mu = \sigma_{pi} / f_{pk} = 0.73$

$f_{pk} = 1860 N/mm^2$

The long term (final) values of the relaxation losses may be estimated for a time equal to: $t = 500{,}000\, hours$.

Concrete stress

$\sigma_{c,QP}$ Is the stress in the UHPC adjacent to the tendons, due to self-weight and initial prestress and other quasi-permanent actions where relevant. The value of $\sigma_{c,QP}$ may be the effect of part of self-weight and initial prestress or the effect of a full quasi-permanent combination of action $\sigma_c = \sigma_c(G + P_{m0} + \psi_2 Q)$, depending on the stage of construction considered.

This means that $\sigma_{c,QP}$ is the stress at the centroidal axis at t=0.

This gives:

$$\sigma_{c,QP} = -\frac{P_0}{A_c} = -16.65 N/mm^2$$

Where:

$P_0 = n * A_p * \sigma_{pm0} = 45288 kN$

Other values

$A_c = 2720800 mm^2$ Cross-sectional area of UHPC

$A_p = 5550 mm^2$ Cross-sectional area per prestressing tendon

$n = 6$	Number of tendons
$E_{cm} = 50000 N/mm^2$	Secant modulus of elasticity of UHPC
$E_p = 200{,}000 N/mm^2$	Modulus of elasticity of prestressing steel
$I_c = 2.381*10^{12} mm^4$	Moment of inertia of UHPC section

Time dependent loss of prestress for post-tensioning at support

$$\Delta P_{c+s+r,s} = nA_p \Delta\sigma_{p,c+s+r} = nA_p \frac{\varepsilon_{cs} E_p + 0.8\Delta\sigma_{pr} + \frac{E_p}{E_{cm}}\varphi(\infty,t_0) * \sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}}\frac{n*A_p}{A_c}(1 + \frac{A_c}{I_{c,s}} z_{cp,s}^2)[1 + 0.8\varphi(t,t_0)]} = 2220kN$$

Where:

$z_{cp,s} = 0mm$	The tendon eccentricity at the support is 0 m as the tendons anchorage coincides with the centroidal axis.

Time dependent loss of prestress for post-tensioning at mid-span

$$\Delta P_{c+s+r,m} = nA_p \Delta\sigma_{p,c+s+r} = nA_p \frac{\varepsilon_{cs} E_p + 0.8\Delta\sigma_{pr} + \frac{E_p}{E_{cm}}\varphi(\infty,t_0) * \sigma_{c,QP}}{1 + \frac{E_p}{E_{cm}}\frac{n*A_p}{A_c}(1 + \frac{A_c}{I_{c,m}} z_{cp,m}^2)[1 + 0.8\varphi(t,t_0)]} = 2046kN$$

Where:

$z_{cp,m} = f = 1143mm$

Total prestressing losses

The box girder segments are tensioned from one side from a practical point of view. This is because the construction of the metro system concerns a continuous placement of the segments from one column to the next column. This means that there is only one end well accessible to tension the tendons. The total prestressing losses hereby become, see Table 22:

Place	*Prestressing loss*	*Value*		*Percentage of loss*	*Value*	
At the first support	$\Delta P_{c+s+r,s} + \Delta P_\mu$	2523	kN	$\frac{\Delta P_{c+s+r,s} + \Delta P_\mu}{n*A_p*\sigma_{pm0}}$	5.57	%
After the first deviation block (at mid-span)	$\Delta P_{c+s+r,m} + 2*\Delta P_\mu$	2651	kN	$\frac{\Delta P_{c+s+r,m} + 2*\Delta P_\mu}{n*A_p*\sigma_{pm0}}$	5.85	%
After the second deviation block (at mid-span)	$\Delta P_{c+s+r,m} + 3*\Delta P_\mu$	2954	kN	$\frac{\Delta P_{c+s+r,m} + 3*\Delta P_\mu}{n*A_p*\sigma_{pm0}}$	6.52	%
At the second support	$\Delta P_{c+s+r,s} + 4*\Delta P_\mu$	3432	kN	$\frac{\Delta P_{c+s+r,s} + 4*\Delta P_\mu}{n*A_p*\sigma_{pm0}}$	7.58	%

Table 22: Total prestressing losses

The maximum prestressing loss arises at the end of the span, at the other end where the tensioning takes place. This loss = 7.58 % which is smaller than the assumed prestressing loss of 20 %. This assumption is thus a safe value for the prestressing losses and has not to be taken any larger. To take into account other unexpected losses and other expected losses like for instance thermal losses and slip of the anchorage it is decided to keep the expected final prestressing losses of 20 %. Notice that these formulas for prestressing losses are from [11] which can be used for concrete C50/60. For UHPC there are no formulas to determine the prestressing losses, but as the method should be quite similar this should give an impression of the losses. As the calculations show that the losses are far below the expected loss of 20 %, it is assumed to be on the safe side. In the continuation of this design the prestressing loss in the end phase at t = ∞ is thus 20 %.

C.6 Deflection

The bending moments due to the loads are determined according the structural load schematisation shown in Figure 135. This schematisation means a deflection at mid-span of: $w = \frac{5}{384}\frac{qL^4}{EI}$

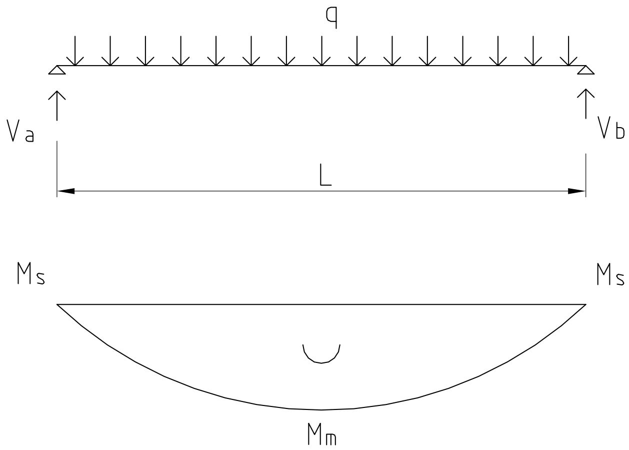

Figure 135: Structural schematisation of the box girder subjected to loads

The moment diagram and structural schematisation due to prestressing is given in Figure 136. The exact upward deflection of this schematisation is more difficult to determine. Therefore it is chosen to re-schematise the schematisation into a more easy and conservative schematisation to calculate the deflection. It can be seen that the moment diagram due to prestressing looks like the one due to the loads but then upside-down and angular. It is therefore chosen to change the structural schematisation of the box girder subjected to prestressing forces into a schematisation with a uniform distributed load like in Figure 135, but then with an upward uniform distributed load.

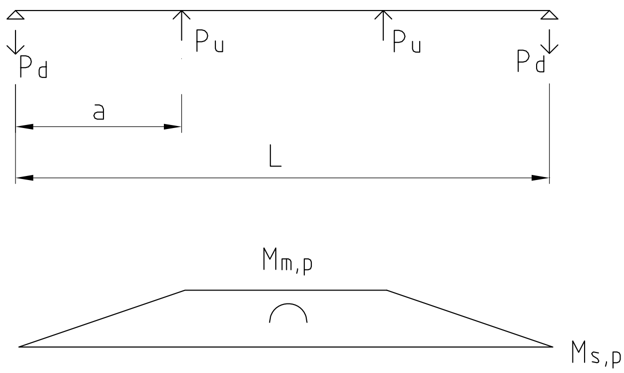

Figure 136: Structural schematisation of the box girder subjected to prestressing forces

For the new schematisation the corresponding uniform distributed load has to be determined:

At t=0:
The bending moment generated by prestressing at mid-span:
$$M_{m,p0} = P_{u0} * a = 51632kNm$$
The bending moment due to a uniform distributed load at mid-span:
$$M_m = \frac{1}{8} * q * L^2$$
For the new schematisation those two moments have to be the same value:
$$M_{m,p0} = M_m \rightarrow q_{pt0} = 204kN/m$$

At t=∞:
The bending moment generated by prestressing at mid-span:
$$M_{m,p\infty} = P_{u\infty} * a = 41306kNm$$
The bending moment due to a uniform distributed load at mid-span:
$$M_m = \frac{1}{8} * q * L^2$$
For the new schematisation those two moments have to be the same value:
$$M_{m,p\infty} = M_m \rightarrow q_{pt\infty} = 163kN/m$$

Notice that this new schematisation causes a smaller upward deflection than in the real schematisation. With the requirement of a limited downward deflection this verification thus becomes more conservative.

The deflection is determined with the formula:
$$w = \frac{5}{384}\frac{qL^4}{EI_{c,m}}$$
Where:

$L = 45m$	Length span
$I_c = 2.381m^4$	Moment of inertia of UHPC section
$E = E_{c,eff} = \frac{E_{cm}}{1+\varphi(\infty,t_0)} = 38462N/mm^2$	Effective modulus of elasticity of UHPC for deflection at t=0 and at t=∞ without variable load
$E = E_{cm} = 50000N/mm^2$	Secant modulus of elasticity of UHPC for additional deflection under mobile load
$\varphi(\infty,t_0) = 0.3$	Creep coefficient, see C.5.5: creep

The deflections and unity checks at mid-span for different phases are:

Time	*Load* q	*Deflection* w	*value*	*Maximum allowed deflection* $w_{\max}$	*Unity check* $w/w_{\max}$
At t=0	$g_{dead} - q_{pt0}$	-78.5	mm	$L/250 = -180mm$ annotation [11]	0.44
At t=∞ without variable load	$g_{dead} + g_{perm} - q_{pt\infty}$	-34.6	mm	$L/500 = 90mm$ annotation [12]	-0.38
Additional deflection under mobile load	q_{var}	26.1	mm	$L/1500 = 30mm$ annotation [13]	0.87
At t=∞ fully loaded	$g_{dead} + g_{perm} + q_{\text{var}} - q_{pt\infty}$	-34.6 + 26.1 = -8.5	mm	$L/500 = 90mm$ annotation [12]	-0.09

Table 23: The deflections and unity checks at mid-span for different phases

An upward deflection has a negative sign and a downward deflection has a positive sign. As the unity checks show the construction satisfies with respect to deflection for all phases and always has a camber. The normative deflection is the additional deflection under mobile load.

[11] The pre-camber may not exceed $L/250$ see 7.4 [11].
[12] The final deflection may not exceed $L/500$ see 7.4 [11].
[13] The maximum deflection under mobile load is $L/1500$ [8].

C.7 Shear + torsion

C.7.1 Shear + torsion in webs

General

The webs have to resist the vertical shear and torsion. As it concerns a segmental box girder the joints between the segments consists of shear keys, see Figure 137 and Figure 138.

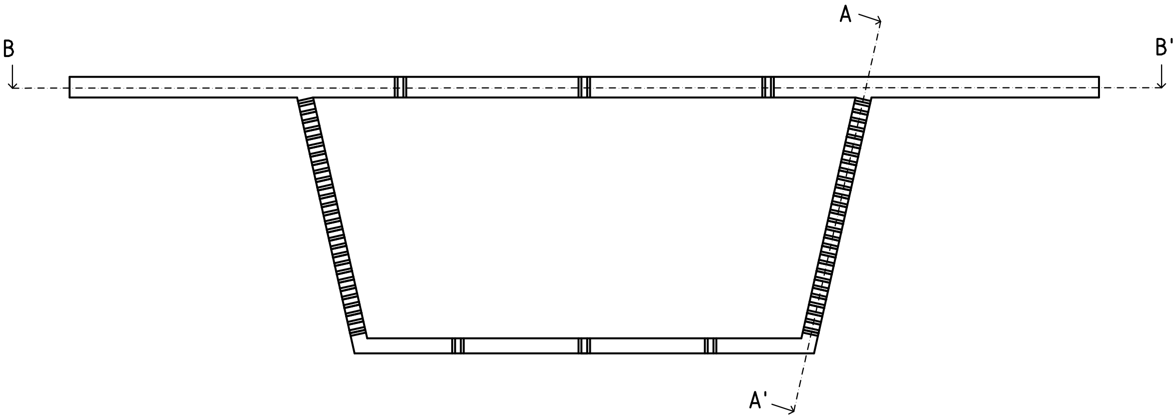

Figure 137: Shear keys in the flanges and in the webs

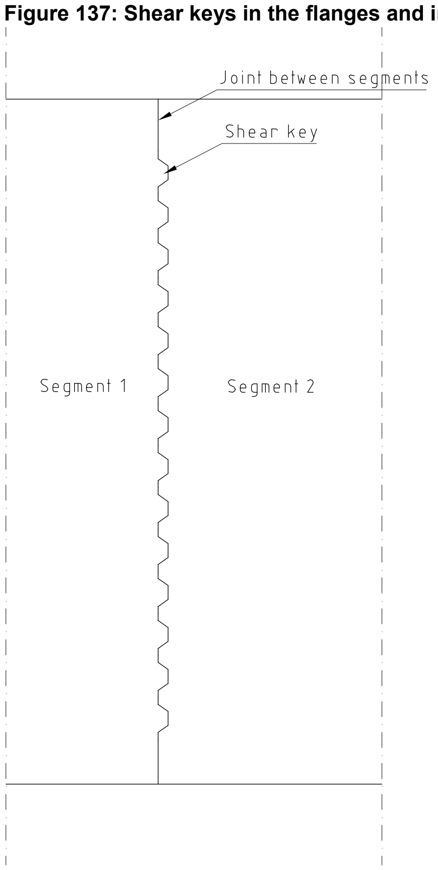

Figure 138: Section A-A'

Each web has 14 shear keys with a height H_s of 150 mm per shear key, see Figure 139. The shear force is taken by compression in the sloped part of the shear key, see Figure 140. Friction of the remaining parts of the shear keys and flanges is not taken into account.

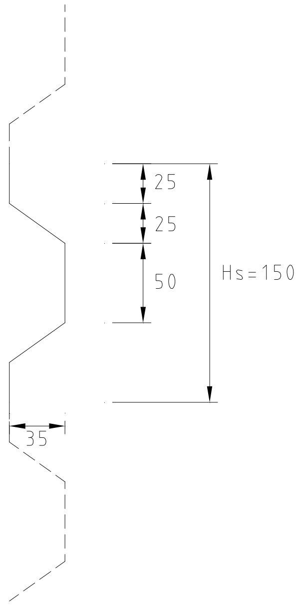

Figure 139: Dimensions shear key in mm

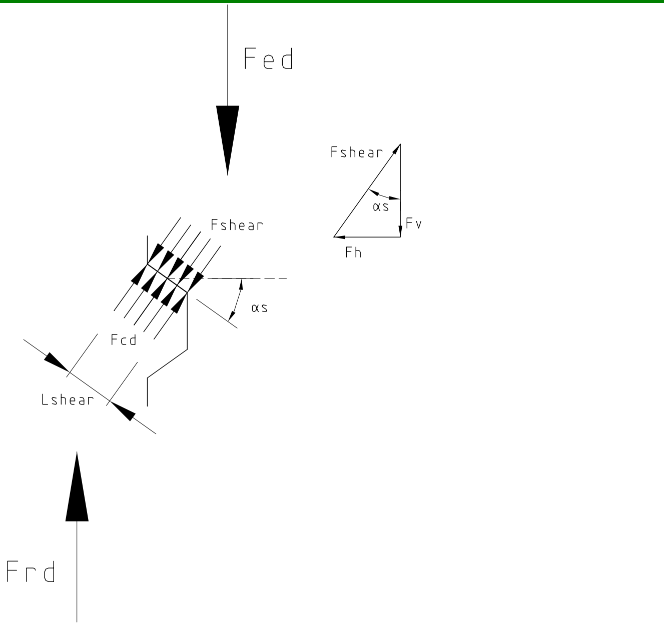

Figure 140: Schematisation of the shear resistance in a shear key by compression in the sloped part L_{shear}

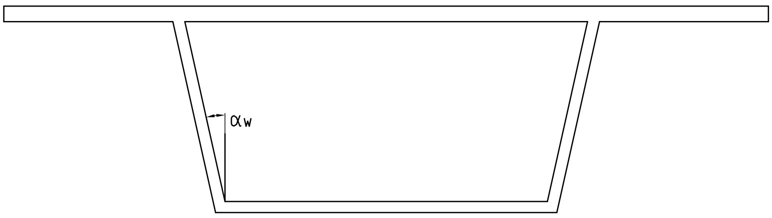

Figure 141: Angle of the webs with the vertical axis

The shear strength of the webs:

The vertical shear strength of one web is:

$$V_{Rd,1} = f_{cd,uls} * L_{shear} * \cos\alpha_s * \cos\alpha_w * t_w * n_s = 5330kN$$

Where:

$f_{cd,uls} = 81.6N/mm^2$	Design value of UHPC compressive strength
$L_{shear} = \sqrt{25^2 + 35^2} = 43mm$	Sloped part of a shear key under compression
$\alpha_s = \tan^{-1}(25/35) = 35.5°$	Angle between shear key and beam axis
$\alpha_w = \tan^{-1}\left(\frac{(b_{boxts} - b_{bf})/2}{H_{box} + t_{bf}}\right) = 12.64°$	Angle of webs with vertical axis
$t_w = b_w * \cos\alpha_w = 137mm$	Is the thickness of the web, see Figure 125
$n_s = H_{box}/H_s = 14$	Is the number of shear keys per web
$H_s = 150mm$	Height shear key, see Figure 139

The vertical shear strength of two webs is:

$$V_{Rd,2} = 2 * V_{Rd,1} = 10660kN$$

Shear resistance at t=0 in the webs

Shear forces
The shear diagram at t=0 is shown in Figure 142.

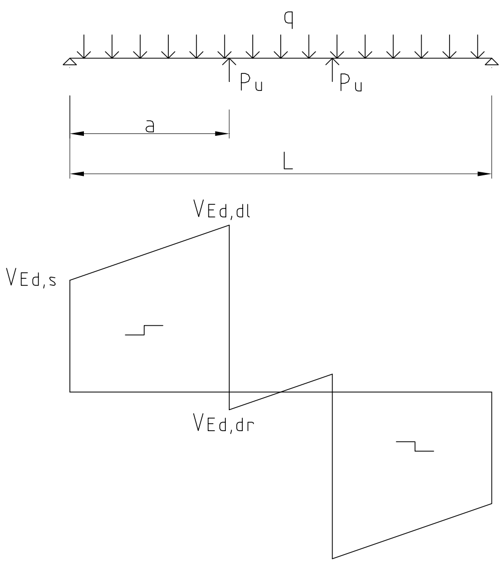

Figure 142: Shear force diagram at t=0

With:

$L = 45m$

$a = 17m$

$q = \gamma_{G,fav} * g_{dead} = 69.4kN/m$

$P_u = \gamma_{P,unfav} * P_{u0} = 3948kN$

$P_{u0} = 3037kN$ See Eq. (23)

This gives the following shear forces

$V_{Ed,dr0} = q*(L/2-a) = 382kN$

$V_{Ed,dl0} = V_{Ed,dr0} - P_u = -3567kN$

$V_{Ed,s0} = V_{Ed,dl0} + q*a = -2387kN$

The maximum shear force is (absolute value):

$$V_{Ed,dl0} = 3567kN \quad (39)$$

Unity check
The unity check for shear in the webs at t=0 is:

$$V_{Ed,dl0}/V_{Rd,2} = 0.33 \leq 1.0 \rightarrow Ok$$

Shear and torsion resistance at t=∞ in the webs

Shear forces
The shear diagram at t=∞ is shown in Figure 143.

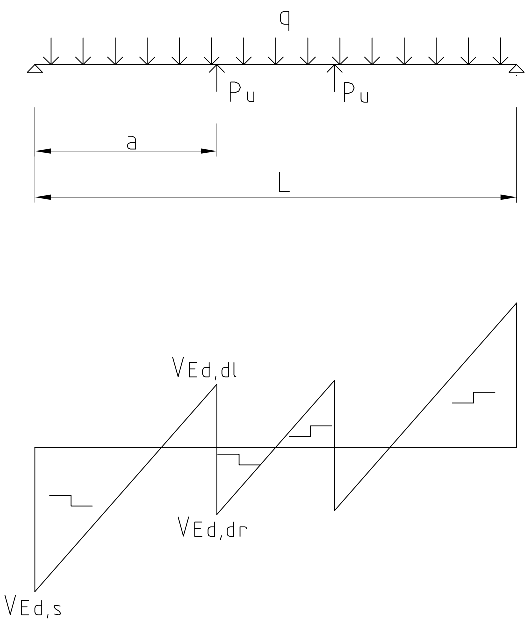

Figure 143: Shear force diagram at t=∞

With:

$L = 45m$

$a = 17m$

$q = \gamma_{G,unfav} * g_{dead} + \gamma_{G,unfav} * g_{perm} + \gamma_{Q,unfav} * q_{\text{var}} = 227.59kN/m$

$P_u = \gamma_{P,fav} * P_{u\infty} = 2430kN$

$P_{u\infty} = 2430kN$ See Eq. (25)

This gives the following shear forces

$V_{Ed,dr\infty} = q*(L/2-a) = 1252kN$

$V_{Ed,dl\infty} = V_{Ed,dr\infty} - P_u = -1178kN$

$V_{Ed,s\infty} = V_{Ed,dl\infty} + q*a = 2691kN$

The maximum shear force is (absolute value):

$$V_{Ed,s\infty} = 2691kN \tag{40}$$

Torsional moment

The maximum torsional moment is a result of wind load, the sideward force of a metro and the box girder eccentrically loaded by one metro, see Figure 144 and Figure 145.

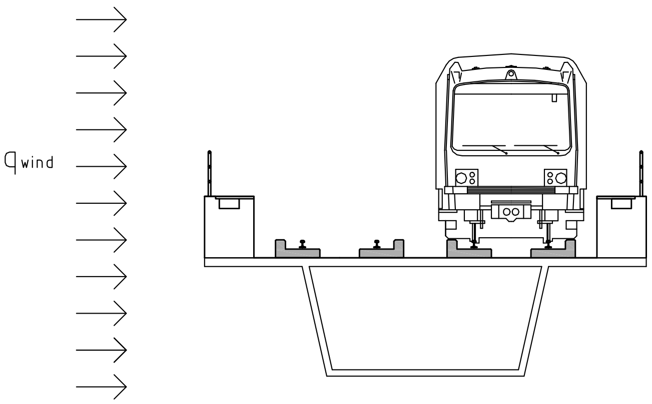

Figure 144: Eccentrically loaded box girder

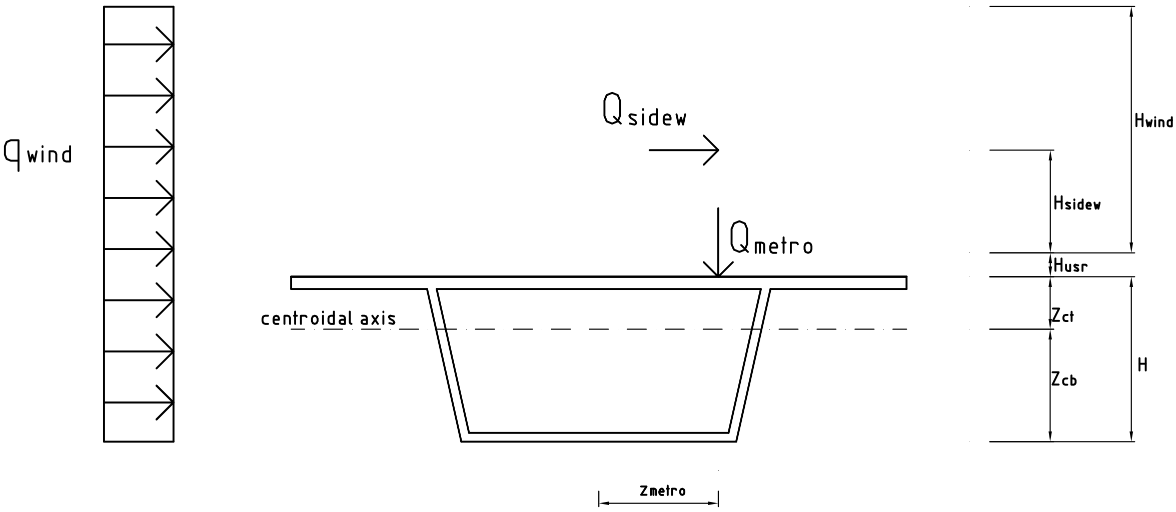

Figure 145: Load schematisation for maximum torsional moment

The maximum torsional moment is:

$$T_{Ed} = q_{wind} * L/2 * (H_{wind} + H_{usr} + Z_{ct} - Z_{cb}) * (((H_{wind} + H_{usr} + Z_{ct} - Z_{cb})/2) + Z_{cb})$$
$$+ Q_{sidew} * (H_{sidew} + H_{usr} + Z_{ct}) + Q_{metro} * z_{metro} = 2072 kNm$$

(41)

Where:

$q_{wind} = 1.69 kN/m^2$	See C.4.5 ULS
$Q_{sidew} = 36 kN$	See C.4.5 ULS
$Q_{metro} = \gamma_{Q,unfav} * \phi * q_{mob} * L/2 = 923 kN$	See C.4.1 and C.4.2, divided by 2 as half the torsion goes to the support of one span
$H_{wind} = 3.6 m$	See C.4.3
$H_{sidew} = 1.5 m$	See C.4.3
$H_{usr} = 0.35 m$	Height upper side rail, see Figure 128
$z_{metro} = 1.74 m$	See Figure 128
$Z_{ct} = 0.767 m$	
$Z_{cb} = 1.643 m$	

The lever arm of the webs is:

$z_{webs} = (b_{boxts} + b_{bf})/2 - b_w = 4.36 m$ See Figure 125

The extra shear force in the webs due to torsion is:

$$V_{Ed+w} = T_{ed} / z_{webs} = 475 kN$$

(42)

Unity checks

The unity check for shear in the webs at t=∞ is:

$$V_{Ed,s\infty} / V_{Rd,2} = 0.25 \leq 1.0 \rightarrow Ok$$

The unity check for shear + torsion in the webs at t=∞ is:

$$V_{Ed,s\infty} / V_{Rd,2} + V_{ed+w} / V_{Rd,1} = 0.34 \leq 1.0 \rightarrow Ok$$

The webs satisfy with respect to shear and torsion. The shear and torsion resistance is much more than what is required and friction of the remaining parts of the shear keys and flanges is not even taken along. It is thus possible to have less shear keys in the webs. When this verification is not satisfied, the depth of the webs H_{box} should be increased to place more shear keys in the webs. Also increasing the web thickness is an option. For this design this is however not necessary as the verification is easily satisfied.

C.7.2 Shear + torsion in flanges

General

The flanges have to resist the horizontal shear and torsion. As it concerns a segmental box girder the joints between the segments consists of shear keys see, Figure 146 and Figure 147.

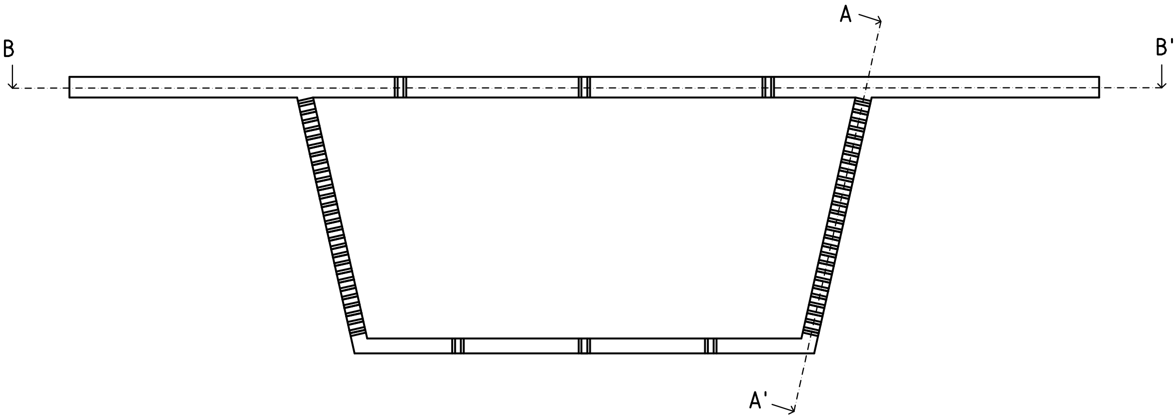

Figure 146: Shear keys in the flanges and in the webs

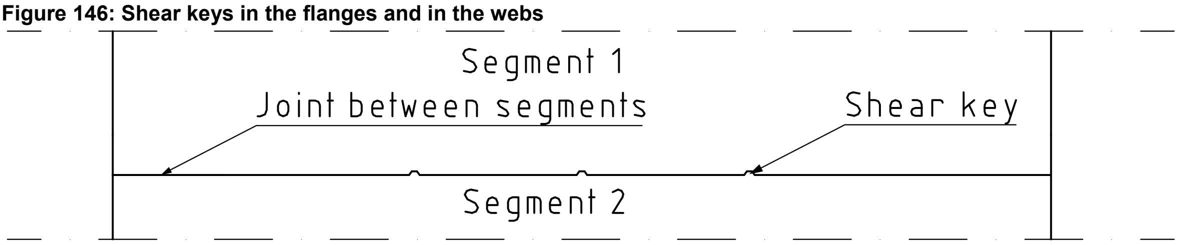

Figure 147: Section B-B'

The top flange and the bottom flange both have 3 shear keys with a thickness which is the same as the flange thickness, see Figure 146. The shear force is taken by compression in the sloped part of the shear key, see Figure 149. Friction of the remaining parts of the shear keys and flanges and webs is not taken into account.

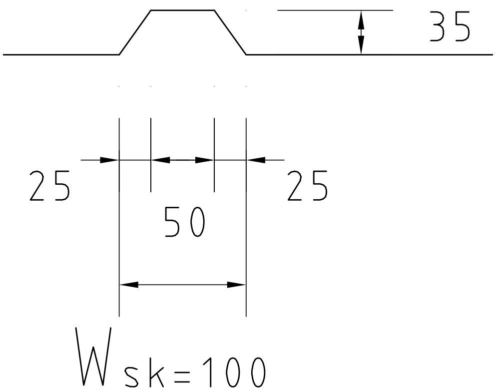

Figure 148: Dimensions shear key in mm

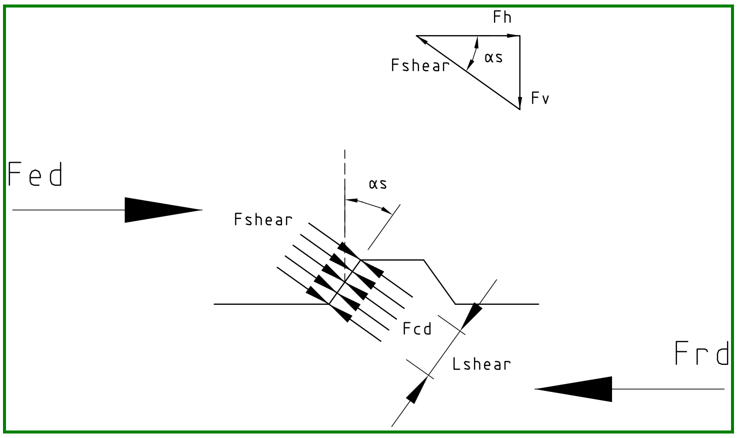

Figure 149: Schematisation of the shear resistance in a shear key by compression in the sloped part L_{shear}

The shear strength of the flanges:

The horizontal shear strength of the top flange is:

$$V_{Rd,tf} = f_{cd} * L_{shear} * \cos\alpha_s * t_{tf} * n_{s,tf} = 1028kN$$

The horizontal shear strength of the bottom flange is:

$$V_{Rd,bf} = f_{cd} * L_{shear} * \cos\alpha_s * t_{bf} * n_{s,bf} = 1114kN$$

Where:

$f_{cd,uls} = 81.6N/mm^2$	Design value of UHPC compressive strength
$L_{shear} = \sqrt{25^2 + 35^2} = 43mm$	Sloped part of a shear key under compression
$\alpha_s = \tan^{-1}(25/35) = 35.5°$	Angle between shear key and beam axis
$t_{tf} = 0.18m$	Is the thickness of the top flange
$t_{bf} = 0.13m$	Is the thickness of the bottom flange
$n_{s,tf} = 3$	Is the number of shear keys in the top flange
$n_{s,bf} = 3$	Is the number of shear keys in the bottom flange

Shear resistance at t=∞ in the flanges

Shear forces in the flanges

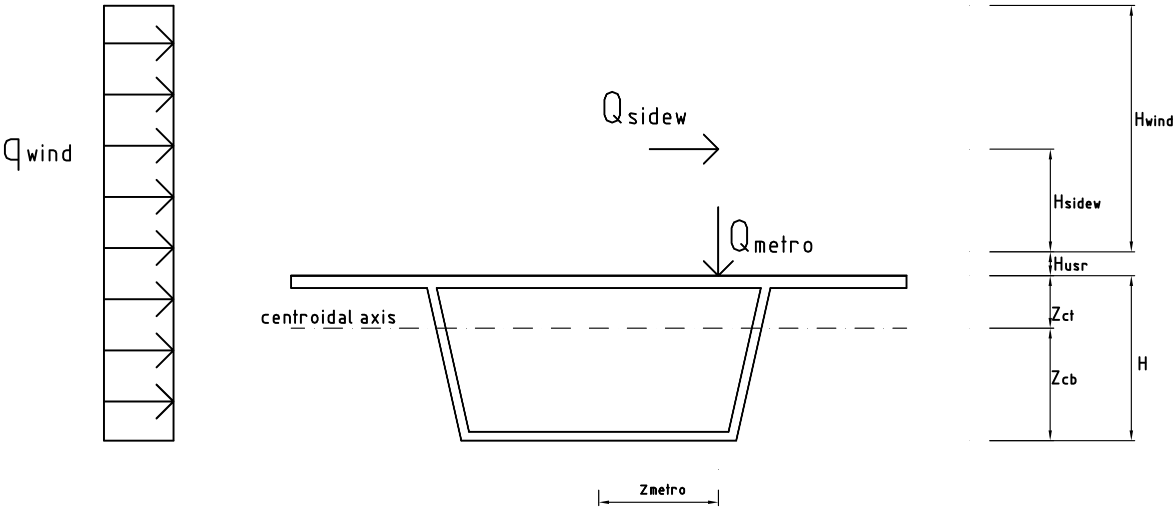

Figure 150: Load schematisation for maximum torsional moment

The shear force in the top flange is:

$$V_{Ed,tf\infty} = q_{wind} * L/2 * (H_{wind} + H_{usr} + H/2) + Q_{sidew} = 232kN$$

The shear force in the bottom flange is:

$$V_{Ed,bf\infty} = q_{wind} * L/2 * H/2 = 46kN$$

Where:

$q_{wind} = 1.69kN/m^2$	See C.4.5 ULS
$Q_{sidew} = 36kN$	See C.4.5 ULS
$H_{wind} = 3.6m$	See C.4.3
$H_{sidew} = 1.5m$	See C.4.3
$H_{usr} = 0.35m$	See Figure 128

Torsional moment

The maximum torsional moment is a result of wind load, the sideward force of a metro and the box girder eccentrically loaded by one metro, see Figure 150.

The lever arm of the flanges is:

$z_f = H + t_{tf}/2 + t_{bf}/2 = 2.26m$ See Figure 125

The extra shear force in the flanges due to torsion is:

$$V_{Ed+f} = T_{ed}/z_f = 919kN$$

Where:

$T_{Ed} = 2072kNm$ See Eq. (41)

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Unity checks

Top flange
The unity check for shear in the top flange at t=∞ is:

$$V_{Ed,tf\infty} / V_{Rd,tf} = 0.15 \leq 1.0 \rightarrow Ok$$

The unity check for shear + torsion in the top flange at t=∞ is:

$$V_{Ed,tf\infty} / V_{Rd,tf} + V_{Ed+f} / V_{Rd,tf} = 0,75 \leq 1.0 \rightarrow Ok$$

Bottom flange
The unity check for shear in the bottom flange at t=∞ is:

$$V_{Ed,bf\infty} / V_{Rd,bf} = 0.04 \leq 1.0 \rightarrow Ok$$

The unity check for shear + torsion in the bottom flange at t=∞ is:

$$V_{Ed,bf\infty} / V_{Rd,bf} + V_{Ed+f} / V_{Rd,bf} = 0,87 \leq 1.0 \rightarrow Ok$$

The flanges satisfy with respect to shear and torsion. The shear and torsion resistance is not much more than what is required. Friction of the remaining parts of the shear keys and flanges is however not even taken along. When this verification is not satisfied, more shear keys should be placed in the flanges. As the flanges offer enough space for additional shear keys this verification will never be normative for the design and will easily satisfy.

C.8 Ultimate resistance moment

C.8.1 General

In all phases during the lifetime of the box girder the concrete force N_c due to the compressive stresses in the UHPC should balance the prestressing force P , see Figure 151.

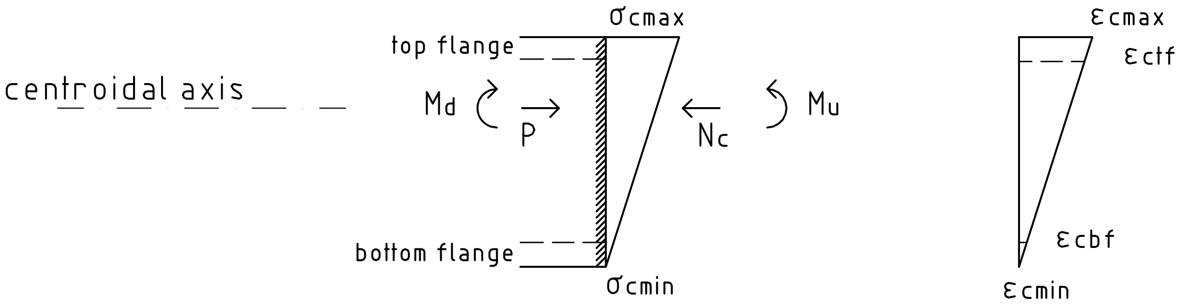

Figure 151: Equilibrium between axial forces P and N_c in the cross-section of the box girder

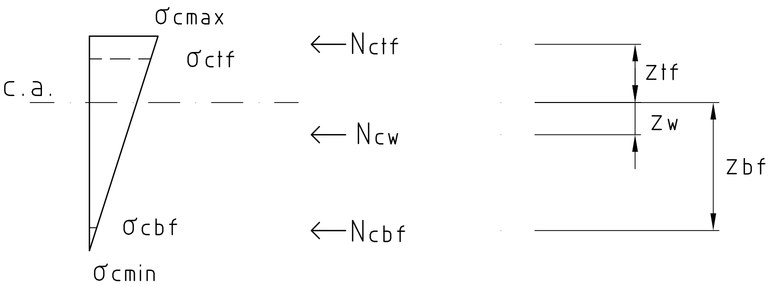

Figure 152: Overview for the calculation of the ultimate moment

At the same time the bending moment M_d due to loading should be resisted by the ultimate resistance moment M_u of the box girder. The ultimate resistance moment arises when the strain difference between the top and bottom flange is as large as possible taking into account that tensile stresses are not allowed. This means that $\sigma_{c\min} = 0N/mm^2$. In which flange the maximum strain arises depends on the stage of loading. For the example given above it would mean that:
The concrete force $N_c = N_{ctf} + N_{cw} + N_{cbf}$ and should be equal to P , see Figure 152.

The ultimate resistance moment $M_u = N_{ctf} * z_{tf} + N_{cw} * z_w + N_{cbf} * z_{bf}$ and should be larger than the bending moment M_d . Where z_{tf}, z_w *and* z_{bf} are positive or negative values considering the location of the force with regard to the centroidal axis.

For this calculation there is made use of the stress-strain relation for UHPC according the strain softening law, see Figure 153.

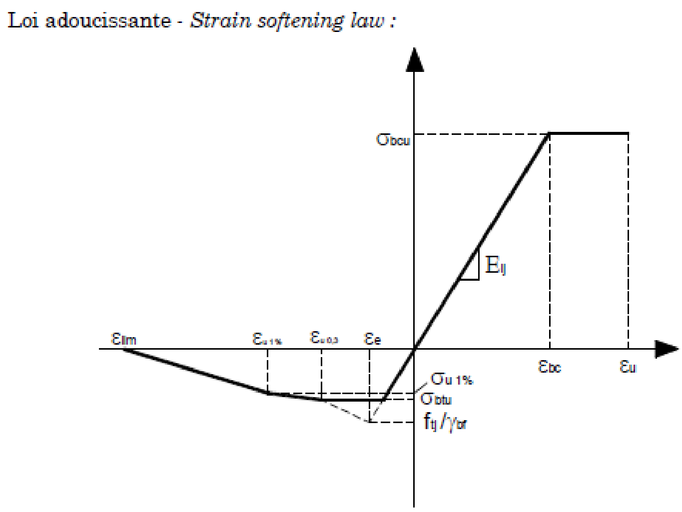

Figure 153: Stress-strain relation for UHPC [18]

Where:

$\varepsilon_{c3} = \varepsilon_{bc} = 1.632‰$ Is the maximum elastic compressive strain in the UHPC

$\varepsilon_{cu3} = \varepsilon_{u} = 3.0‰$ Is the ultimate compressive strain in the UHPC

C.8.2 Bending moments due to the loads and prestressing

Bending moment M_d at t=0

In the construction phase at t=0 the loads on the box girder are the dead load and the prestressing force. As the permanent and variable loads are missing and the initial prestressing force is large the box girder has a camber. The normative bending moment in this phase arises at the deviation blocks, see Figure 154. The maximum strain arises in the bottom flange.

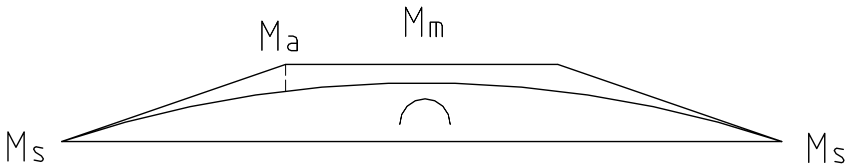

Figure 154: The bending moments due to prestressing minus the bending moments due to dead load results in the largest bending moment M_a at the deviation blocks

At deviation blocks

$$M_{da,0} = \gamma_{P,unfav} * P_{u0} * a - \frac{1}{2} * \gamma_{G,fav} * g_{dead} * L * a - 0.5 * \gamma_{G,fav} * g_{dead} * a^2 = 50605 kNm (\frown)$$

Where:

$P_{u0} = 3037 kN$ See Eq. (23)

Bending moment M_d at t=∞

In the end phase at t=∞ the box girder is fully loaded by the dead, permanent and variable load and is partly resisted by the prestressing force. This load case causes a downward deflection, which means that the normative bending moment arises at mid-span, see Figure 155. The maximum strain arises in the top flange.

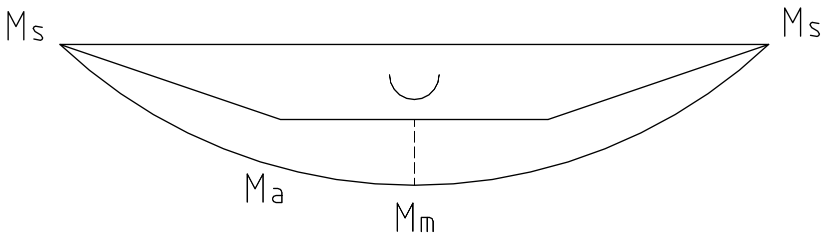

Figure 155: The bending moments due to dead, permanent and variable load minus the bending moments due to prestressing results in the largest bending moment M_m at mid-span

At mid-span

$$M_{dm,\infty} = \frac{1}{8} * (\gamma_{G,unfav} * g_{dead} + \gamma_{G,unfav} * g_{perm} + \gamma_{Q,unfav} * q_{var}) * L^2 - \gamma_{P,fav} * P_{u\infty} * a$$
$$= 16304 kNm (\cup)$$

Where:

$P_{u\infty} = 2430 kN$ — See Eq. (25)

C.8.3 Ultimate resistance moment at t=0

Ultimate resistance moment at deviation blocks

The prestressing force at t=0 is:

$P_0 = 45288 kN$ — See Eq. (22)

$M_{da,0} = 50605 kNm (\cap)$ means that the maximum compressive strain arises in the bottom flange. The schematisation of the forces in the cross-section is shown in Figure 156 and Figure 157.

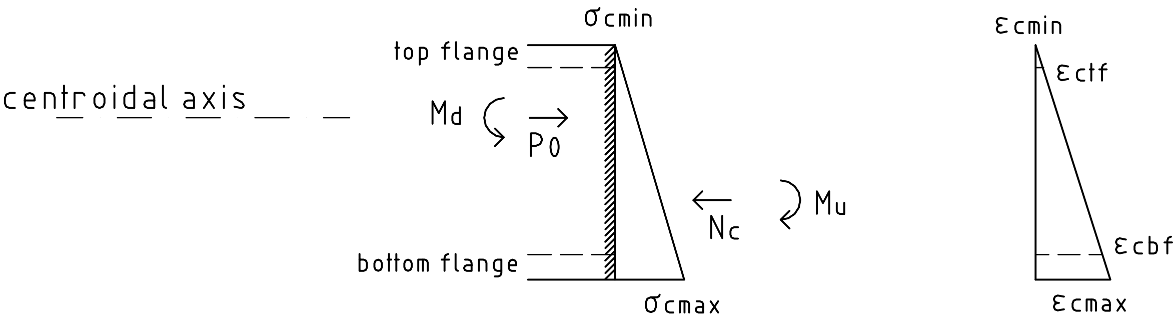

Figure 156: Stress and strain schematisation in the cross-section at the deviation blocks at t=0

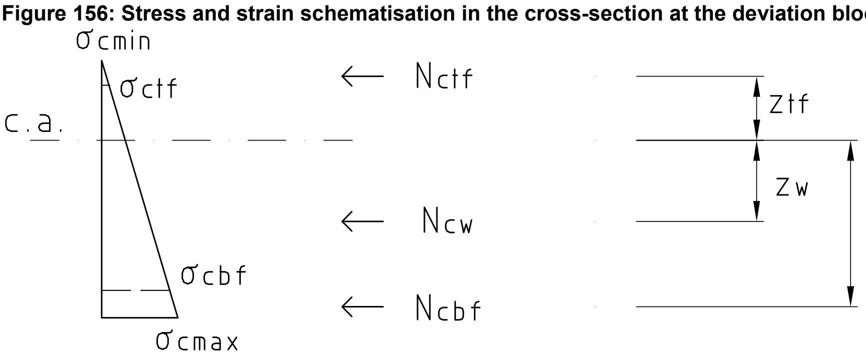

Figure 157: Concrete forces and lever arms in the cross-section at the deviation blocks at t=0

To determine the maximum strain for which holds that $N_c = P_0$ everything is filled in a spreadsheet program (Microsoft Excel) and solved with the function goal seek. With the function goal seek the concrete force N_c is set to be equal to the prestressing force P_0 by changing the maximum compressive strain in the cross-section $\varepsilon_{c\max}$.

The maximum strain in the cross-section which causes equilibrium between N_c and P_0 is:

$$\varepsilon_{c\max} = 1.046‰$$

This gives:

$$\varepsilon_{ctf} = \varepsilon_{c\max} * t_{tf} / H = 0.078‰$$

$$\varepsilon_{cbf} = \varepsilon_{c\max} * (H - t_{bf}) / H = 0.989‰$$

$$\varepsilon_{c\min} = 0 N / mm^2$$

$$N_{ctf} = \frac{((\varepsilon_{ctf} + \varepsilon_{c\min})/2)}{\varepsilon_{c3}} * f_{cd,uls} * b_{eff,t} * t_{tf} = 3148 kN$$

$$N_{cw} = \frac{((\varepsilon_{ctf} + \varepsilon_{cbf})/2)}{\varepsilon_{c3}} * f_{cd,uls} * 2b_w * H_{box} = 15688 kN$$

$$N_{cbf} = \frac{((\varepsilon_{cbf} + \varepsilon_{c\max})/2)}{\varepsilon_{c3}} * f_{cd,uls} * b_{eff,b} * t_{bf} = 26451 kN$$

Where:

$b_{eff,t} = 8.96 m$ See C.3.4

$b_{eff,b} = 4 m$ See C.3.4

The total concrete compressive force is:

$$N_c = N_{ctf} + N_{cw} + N_{cbf} = 45288 kN = P_0$$

The lever arms of the concrete forces are:

$$z_{tf} = t_{tf} * \frac{2}{3} - Z_{ct} = -0.65 m$$

$$z_w = Z_{cb} - \left(\varepsilon_{ctf} * (H_{box} * \frac{1}{2} + t_{bf}) + \frac{\varepsilon_{cbf} - \varepsilon_{ctf}}{2} * (H_{box} * \frac{1}{3} + t_{bf}) \right) / \left(\varepsilon_{ctf} + \frac{\varepsilon_{cbf} - \varepsilon_{ctf}}{2} \right) = 0.76 m$$

$$z_{bf} = Z_{cb} - \left(\frac{\varepsilon_{c\max} - \varepsilon_{cbf}}{2} * t_{bf} * \frac{1}{3} + \varepsilon_{cbf} * t_{bf} * \frac{1}{2} \right) / \left(\frac{\varepsilon_{c\max} - \varepsilon_{cbf}}{2} + \varepsilon_{cbf} \right) = 1.58 m$$

The ultimate resistance moment is:

$$M_u = N_{ctf} * z_{tf} + N_{cw} * z_w + N_{cbf} * z_{bf} = 51654 kNm$$

Unity check of the ultimate resistance moment:

$$M_{da,0} / M_u = 0.98 \leq 1.0 \rightarrow Ok$$

The ultimate resistance moment of the box girder is thus enough to resist the bending moments in the construction phase at t=0. The unity check however approaches the limit 1.0, so this verification needs attention. When this verification is not satisfied the depth of the webs H_{box} should be decreased, see Figure 125. This way the upward prestressing force becomes smaller, see Figure 129, and thus the hogging moment due to prestressing decreases. Another option is to make the box girder heavier such that the hogging moment M_d becomes smaller.

C.8.4 Ultimate resistance moment at t=∞

Ultimate resistance moment at mid-span

The prestressing force at t=∞ is:

$P_\infty = 36230 kN$ See Eq. (24)

$M_{dm,\infty} = 16304 kNm(\cup)$ means that the maximum compressive strain arises in the top flange.
The schematisation of the forces in the cross-section is shown in Figure 158 and Figure 159.

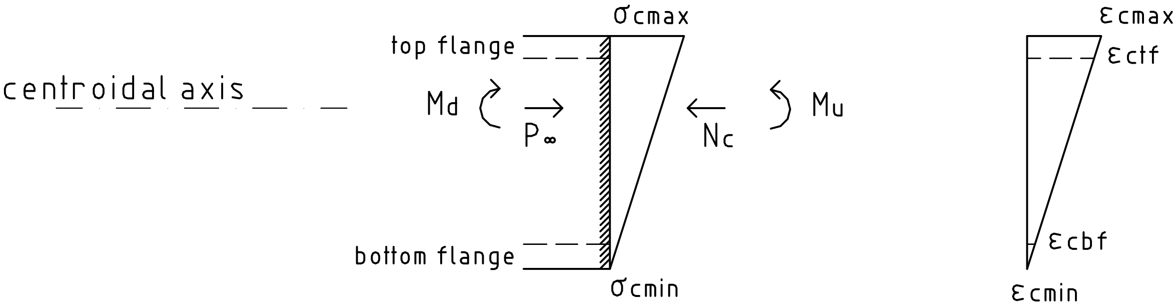

Figure 158: Stress and strain schematisation in the cross-section at mid-span at=∞

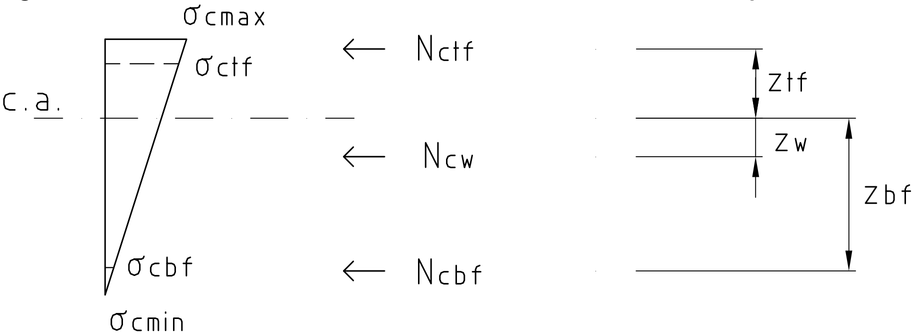

Figure 159: Concrete forces and lever arms in the cross-section at mid-span at=∞

To determine the maximum strain for which holds that $N_c = P_\infty$ everything is filled in a spreadsheet program (Microsoft Excel) and solved with the function goal seek. With the function goal seek the concrete force N_c is set to be equal to the prestressing force P_∞ by changing the maximum compressive strain in the cross-section $\varepsilon_{c\max}$.

The maximum strain in the cross-section which causes equilibrium between N_c and P_∞ is:

$\varepsilon_{c\max} = 0.391‰$

This gives:

$\varepsilon_{ctf} = \varepsilon_{c\max} * (H - t_{tf}) / H = 0.362‰$

$\varepsilon_{cbf} = \varepsilon_{c\max} * t_{bf} / H = 0.021‰$

$\varepsilon_{c\min} = 0 N / mm^2$

$$N_{ctf} = \frac{((\varepsilon_{c\max} + \varepsilon_{ctf})/2)}{\varepsilon_{c3}} * f_{cd,uls} * b_{eff,t} * t_{tf} = 30332 kN$$

$$N_{cw} = \frac{((\varepsilon_{ctf} + \varepsilon_{cbf})/2)}{\varepsilon_{c3}} * f_{cd,uls} * 2b_w * H_{box} = 5625 kN$$

$$N_{cbf} = \frac{((\varepsilon_{cbf} + \varepsilon_{c\min})/2)}{\varepsilon_{c3}} * f_{cd,uls} * b_{eff,b} * t_{bf} = 274kN$$

Where:

$b_{eff,t} = 8.96m$ See C.3.4

$b_{eff,b} = 4m$ See C.3.4

The total concrete compressive force is:

$$N_c = N_{ctf} + N_{cw} + N_{cbf} = 36230kN = P_\infty$$

The lever arms of the concrete forces are:

$$z_{tf} = Z_{ct} - \left(\frac{\varepsilon_{c\max} - \varepsilon_{ctf}}{2} * t_{tf} * \frac{1}{3} + \varepsilon_{ctf} * t_{tf} * \frac{1}{2}\right) / \left(\frac{\varepsilon_{c\max} - \varepsilon_{ctf}}{2} + \varepsilon_{ctf}\right) = 0.68m$$

$$z_w = \left(\frac{\varepsilon_{ctf} - \varepsilon_{cbf}}{2} * (H_{box} * \frac{2}{3} + t_{bf}) + \varepsilon_{cbf} * (H_{box} * \frac{1}{2} + t_{bf})\right) / \left(\frac{\varepsilon_{ctf} - \varepsilon_{cbf}}{2} + \varepsilon_{cbf}\right) - Z_{cb} = -0.15m$$

$$z_{bf} = t_{bf} * \frac{2}{3} - Z_{cb} = -1.56m$$

The ultimate resistance moment is:

$$M_u = N_{ctf} * z_{tf} + N_{cw} * z_w + N_{cbf} * z_{bf} = 19304kNm$$

Unity check of the ultimate resistance moment:

$$M_{dm,\infty} / M_u = 0.84 \leq 1.0 \rightarrow Ok$$

The ultimate resistance moment of the box girder is thus enough to resist the bending moments in the end phase at t=∞. The unity check however approaches the limit 1.0, so this verification needs attention. When this verification is not satisfied the depth of the webs H_{box} should be increased, see Figure 125. This way the lever arms z become larger which has a positive effect on the ultimate resistance moment. Also the upward prestressing force then becomes larger, see Figure 129. The bending moment M_d should be kept as small as possible by creating a light as possible box girder.

C.9 Deck

To determine if the thickness of the top flange / deck meet the requirements of shear and bending moments, the local schematisation is considered. The deck is schematized in the transversal direction as a floor of 1 metre wide with two fixed supports (the webs). The width of 1 metre in longitudinal direction comes from [8], which says that for the calculation of the deck the wheel pressure in longitudinal direction of the track may be spread to two sides over a distance of 1 metre + twice the height of the concrete plinth. For a more conservative calculation only the width of 1 metre is taken. To calculate the shear and bending moments in the deck there is made use of the program Scia Engineer. In the next section the input in Scia Engineer is given. For the geometry of the deck the assumption was made that the web width should be 0.2 metres. With this width the geometry in Figure 160 becomes:

$$L_{cant,centre} = L_{cant} + b_w / 2 = 2.08m$$

$$L_{span} = b_{tf} - 2 * L_{cant,centre} = 4.8m$$

For the load schematisation in the next section reference is made to Section C.4.5.
With the shear and bending moments due to loading as result from the input in Scia Engineer next the verification of shear and ultimate resistance moment for the deck is done.

C.9.1 Schematisation load input in Scia Engineer

- ***Geometry deck***

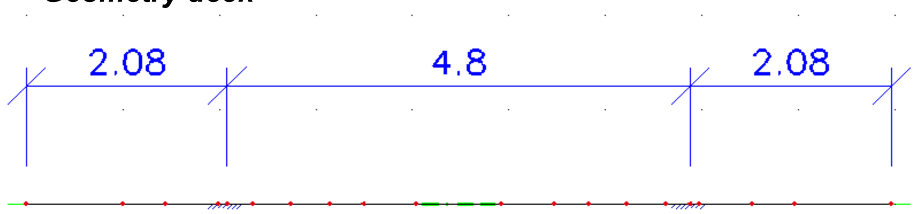

Figure 160: Structural schematisation deck box girder

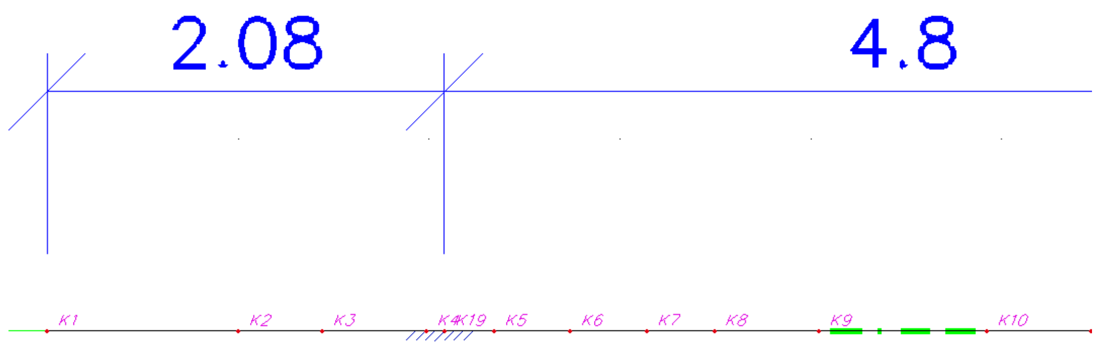

Figure 161: Nodes left side of the deck

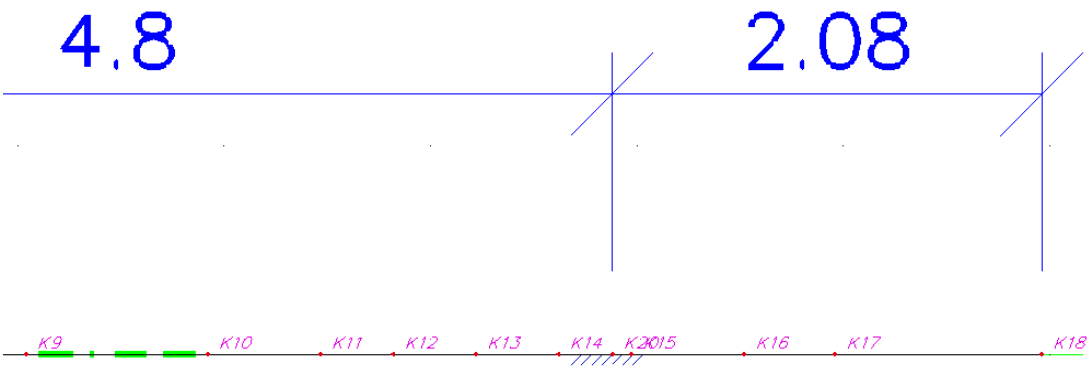

Figure 162: Nodes right side of the deck

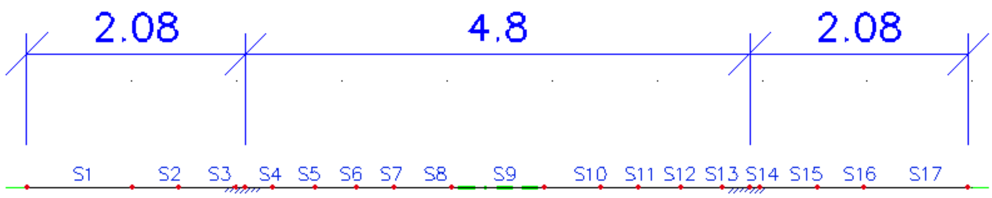

Figure 163: Bars

Nodes

Name	*Coordinate X [m]*	*Coordinate Z [m]*
K1	0,000	6,000
K2	1,000	6,000
K3	1,440	6,000
K4	1,986	6,000
K5	2,340	6,000
K6	2,740	6,000
K7	3,140	6,000
K8	3,494	6,000
K9	4,040	6,000
K10	4,920	6,000
K11	5,466	6,000
K12	5,820	6,000
K13	6,220	6,000
K14	6,620	6,000
K15	6,974	6,000
K16	7,520	6,000
K17	7,960	6,000
K18	8,960	6,000
K19	2,080	6,000
K20	6,880	6,000

1D-bar

Name	*Cross-section*	*Length [m]*	*Form*	*Start node*	*End node*	*Type*	*EEM-type*	*Layer*
S1	CS1 - Rectangle (180; 1000)	1,000	Line	K1	K2	floor strip (99)	standard	Layer1
S2	CS1 - Rectangle (180; 1000)	0,440	Line	K2	K3	floor strip (99)	standard	Layer1

S3	CS1 - Rectangle (180; 1000)	0,546	Line	K3	K4	floor strip (99)	standard	Layer1
S4	CS1 - Rectangle (180; 1000)	0,354	Line	K4	K5	floor strip (99)	standard	Layer1
S5	CS1 - Rectangle (180; 1000)	0,400	Line	K5	K6	floor strip (99)	standard	Layer1
S6	CS1 - Rectangle (180; 1000)	0,400	Line	K6	K7	floor strip (99)	standard	Layer1
S7	CS1 - Rectangle (180; 1000)	0,354	Line	K7	K8	floor strip (99)	standard	Layer1
S8	CS1 - Rectangle (180; 1000)	0,546	Line	K8	K9	floor strip (99)	standard	Layer1
S9	CS1 - Rectangle (180; 1000)	0,880	Line	K9	K10	floor strip (99)	standard	Layer1
S10	CS1 - Rectangle (180; 1000)	0,546	Line	K10	K11	floor strip (99)	standard	Layer1
S11	CS1 - Rectangle (180; 1000)	0,354	Line	K11	K12	floor strip (99)	standard	Layer1
S12	CS1 - Rectangle (180; 1000)	0,400	Line	K12	K13	floor strip (99)	standard	Layer1
S13	CS1 - Rectangle (180; 1000)	0,400	Line	K13	K14	floor strip (99)	standard	Layer1
S14	CS1 - Rectangle (180; 1000)	0,354	Line	K14	K15	floor strip (99)	standard	Layer1
S15	CS1 - Rectangle (180; 1000)	0,546	Line	K15	K16	floor strip (99)	standard	Layer1
S16	CS1 - Rectangle (180; 1000)	0,440	Line	K16	K17	floor strip (99)	standard	Layer1
S17	CS1 - Rectangle (180; 1000)	1,000	Line	K17	K18	floor strip (99)	standard	Layer1

Node support

Name	*Node*	*System*	*Type*	*X*	*Z*	*Ry*
Sn1	K19	GCS	Standard	Fixed	Fixed	Fixed
Sn2	K20	GCS	Standard	Fixed	Fixed	Fixed

- ***Load input***

The values of the loads are taken from section: C.4.5 Load schematisation in transversal directionin the SLS.

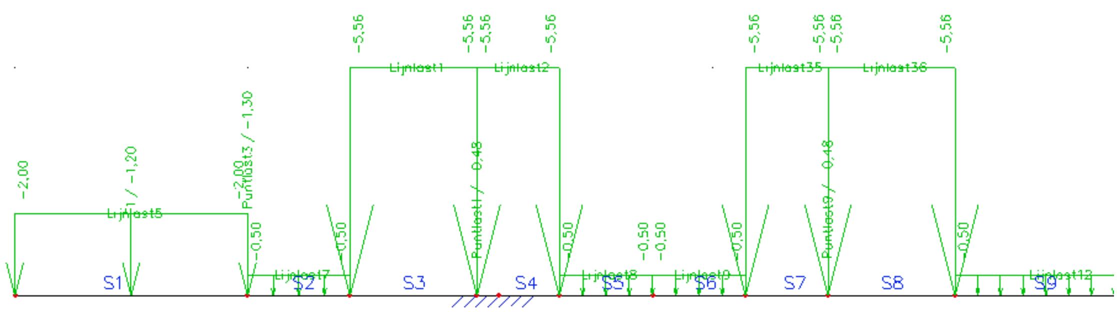

Figure 164: Permanent loads left side of the deck

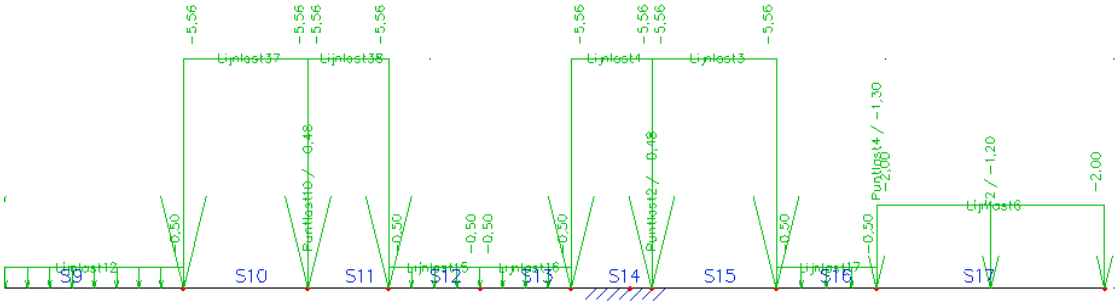

Figure 165: Permanent loads right side of the deck

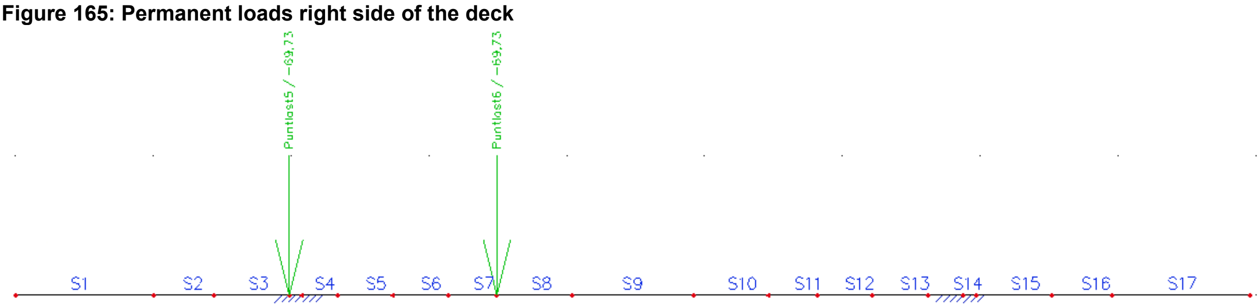

Figure 166: Metro load on the left side of the deck

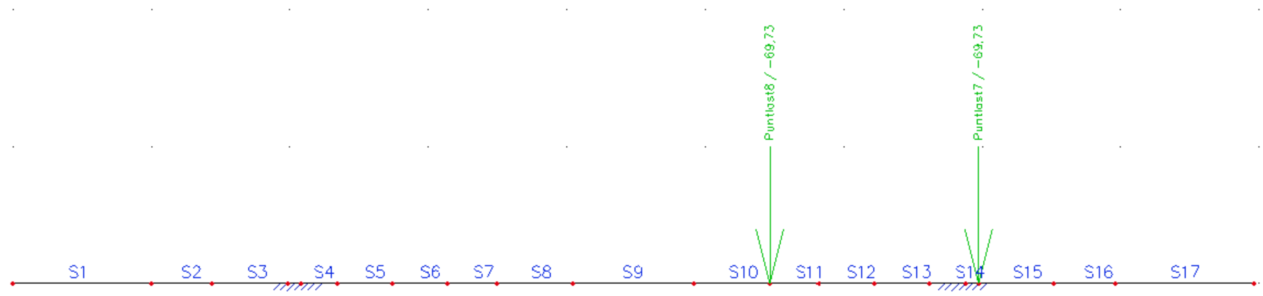

Figure 167: Metro load on the right side of the deck

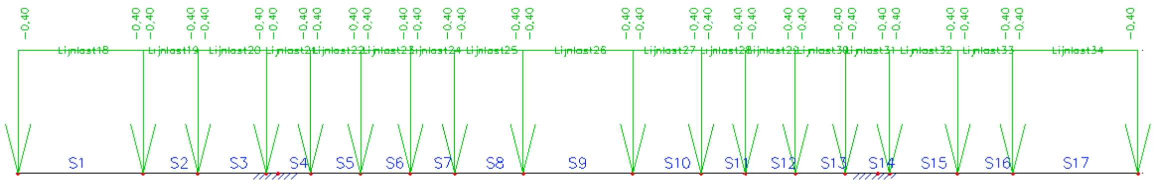

Figure 168: Snow load

Line loads on bars

Name	Bar	Type	Direction	P1 [kN/m]	x1	Coordinate definition	Origin	Exc ey [m]
	Load case	System	Distribution	P2 [kN/m]	x2	Loc	Angle [deg]	Exc ez [m]
Lijnlast1	S3	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast2	S4	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast3	S15	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast4	S14	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast5	S1	Force	Z	-2,00	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000

Lijnlast6	S17	Force	Z	-2,00	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast7	S2	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast8	S5	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast9	S6	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast12	S9	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast15	S12	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast16	S13	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast17	S16	Force	Z	-0,50	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast35	S7	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast36	S8	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast37	S10	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000
Lijnlast38	S11	Force	Z	-5,56	0,000	Rela	From start	
	BG2 - Permanent load	LCS	Uniform		1,000	Length		0,000

Concentrated loads on nodes

Name	*Node*	*Load case*	*System*	*Direction*	*Type*	*Value - F [kN]*
Puntlast1	K4	BG2 - Permanent load	GCS	Z	Force	-0,48
Puntlast2	K15	BG2 - Permanent load	GCS	Z	Force	-0,48
Puntlast3	K2	BG2 - Permanent load	GCS	Z	Force	-1,30
Puntlast4	K17	BG2 - Permanent load	GCS	Z	Force	-1,30
Puntlast9	K8	BG2 - Permanent load	GCS	Z	Force	-0,48
Puntlast10	K11	BG2 - Permanent load	GCS	Z	Force	-0,48

Concentrated loads on bars

Name	Bar	System	F [kN]	x	Coordinate	Repeat (n)
	Load case	Direction	Type	Angle [deg]	Origin	dx
F1	S1	GCS	-1,20	0,500	Rela	1
	BG2 - Permanent load	Z	Force		From start	
F2	S17	GCS	-1,20	0,500	Rela	1
	BG2 - Permanent load	Z	Force		From start	

Concentrated loads on nodes

Name	*Node*	*Load case*	*System*	*Direction*	*Type*	*Value - F [kN]*
Puntlast5	K4	BG3 – Metro left	GCS	Z	Force	-69,73
Puntlast6	K8	BG3 – Metro left	GCS	Z	Force	-69.73
Puntlast7	K15	BG3 – Metro right	GCS	Z	Force	-69,73
Puntlast8	K11	BG3 – Metro right	GCS	Z	Force	-69.73

Line loads on bars

Name	Bar	Type	Direction	P1 [kN/m]	x1	Coordinate definition	Origin	Exc ey [m]
	Load case	System	Distribution	P2 [kN/m]	x2	Loc	Angle [deg]	Exc ez [m]
Lijnlast18	S1	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast19	S2	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast20	S3	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast21	S4	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast22	S5	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast23	S6	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast24	S7	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast25	S8	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast26	S9	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast27	S10	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast28	S11	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast29	S12	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast30	S13	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast31	S14	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast32	S15	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast33	S16	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000
Lijnlast34	S17	Force	Z	-0,4	0,000	Rela	From start	
	BG5 – Snow load	LCS	Uniform		1,000	Length		0,000

- ***Results***

The shear forces and moments in the deck due to the permanent and variable loads in the ULS are shown below. This does not include the dead load of the deck.

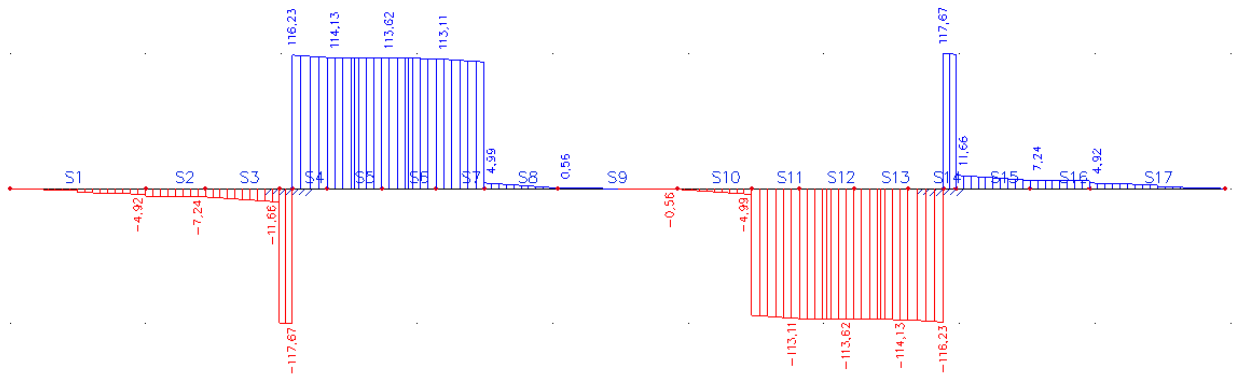

Figure 169: Shear forces in the deck due to permanent and variable loads

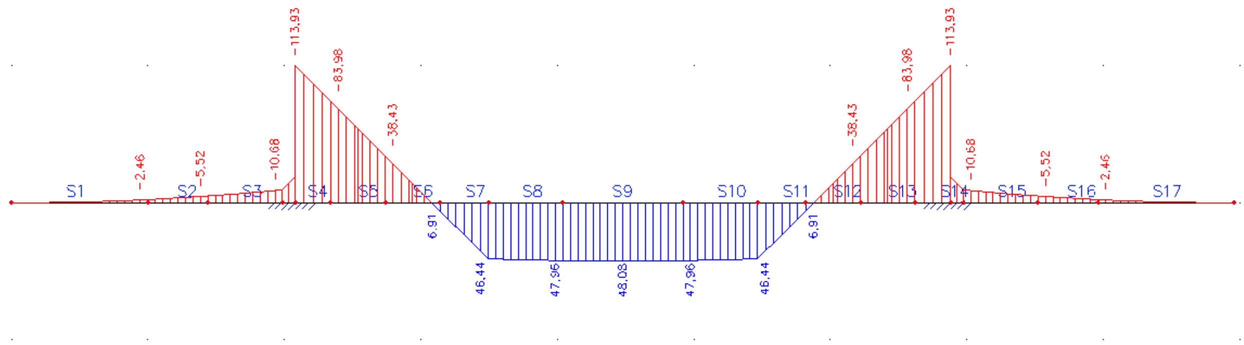

Figure 170: Moments in the deck due to permanent and variable loads

C.9.2 Verifications of shear and ultimate resistance moment

Total shear force and bending moment

The maximum shear force due to permanent and variable loads in the ULS is:

$V_{Ed,perm+var} = 117.67kN$ See Figure 169

The extra shear force due to the dead load is:

$$V_{Ed,dead} = t_{tf} * b_{deck} * L_{cant,centre} * \rho_c * g * \gamma_{G,unfav} = 12.89kN$$

Where:

$b_{deck} = 1.0m$

$L_{cant,centre} = 2.08m$

The total shear force is:

$$V_{Ed} = V_{Ed,perm+var} + V_{Ed,dead} = 130.56kN$$

The maximum bending moment due to permanent and variable loads in the ULS is:

$M_{d,perm+var} = 113.93kNm$ See Figure 170

The extra bending moment due to the dead load at the webs is:

$$M_{d,dead} = \frac{1}{12}(t_{tf} * b_{deck} * \rho_c * g * \gamma_{G,unfav}) * L_{span}^{2} = 11.90kNm$$

Where:
$L_{span} = 4.8m$

The total bending moment is:
$$M_d = M_{d,dead} + M_{d,perm+\mathrm{var}} = 125.83kNm$$

Shear resistance

The total shear strength of the UHPC deck is calculated according [18] with one exception which is explicitly stated below. The total shear strength of the UHPC deck is:
$$V_{Rd} = V_{Rd,c} + V_{Rd,s} + V_{Rd,f} = 733.30kN$$

Where:

$V_{Rd,c} = \frac{1}{\gamma_E} * \frac{0.24}{\gamma_b} * \sqrt{f_{ck}} * b_{deck} * z = 278.20kN$ — Shear strength due to participation of the concrete, reinforced concrete according [1]

$V_{Rd,s} = 0.9 * d * \frac{A_{sw}}{s} * f_{yd} * (\sin\alpha + \cos\alpha) = 0kN$ — Shear strength due to participation of the stirrup reinforcement. As there are no stirrups in the deck this does not contribute to the shear strength of the deck.

$V_{Rd,f} = \frac{S * \sigma_p}{\gamma_{bf} * \tan\theta} = 455.10kN$ — Shear strength due to participation of the fibres

$\gamma_E * \gamma_b = 1.5$	Safety coefficient
$f_{ck} = 180N/mm^2$	Caracteristic compressive cylinder strength of UHPC
$b_{deck} = 1.0m$	Width of the deck
$z = 0.9 * d = 129.6mm$	Lever arm of internal forces
$d = t_{tf} - c - \phi_{re\inf}/2 - \phi_{stirrups} = 144mm$	Effective depth of a cross-section
$c = 20mm$	Concrete cover, see C.3.3
$\phi_{re\inf} = 16mm$	Diameter longitudinal reinforcement
$\phi_{stirrups} = 8mm$	Diameter stirrups
$t_{tf} = 180mm$	Thickness of the top flange/deck
$A_{sw} = 0mm^2$	Cross-sectional area of shear reinforcement per stirrup, there are no stirrups
s	Spacing of the stirrups
$f_{yd} = \frac{f_{yk}}{\gamma_s} = 435N/mm^2$	Design yield strength of reinforcement
α	Angle between shear reinforcement and the beam axis perpendicular to the shear force (α may not be smaller than $45°$)

$S = b_{deck} * z = 129600 mm^2$ Area of fibre effect

$\sigma_p = \frac{1}{K} * \frac{1}{w_{\lim}} * \int_0^{w_{\lim}} \sigma(w) dw = 4.57 N/mm^2$ Residual tensile strength

For calculation of the residual tensile strength there is made use of the stress-strain relation for UHPC according the strain softening law, see Figure 171

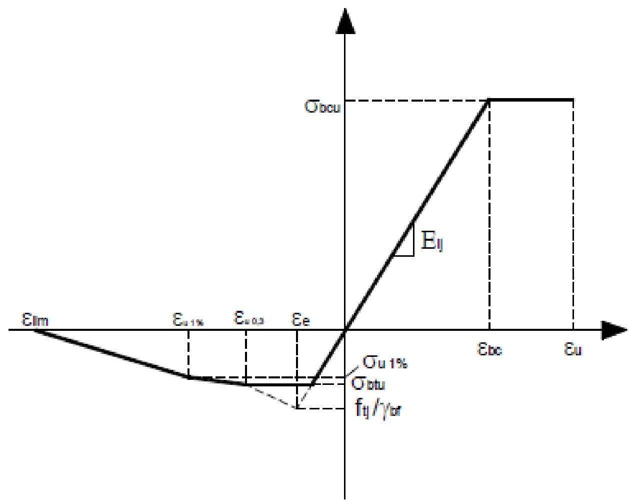

Figure 171: Stress-strain relation for UHPC [18]

$\sigma_p = \frac{1}{K} * \frac{1}{\varepsilon_{u0.3}} * (0.5 * \varepsilon_{lin} * f_{ctk} + (\varepsilon_{u0.3} - \varepsilon_{lin}) * f_{ctk} = 4.57 N/mm^2$

$K = 1.75$ Orientation coefficient local effects

$\varepsilon_{lin} = \frac{f_{ctk}}{\gamma_{bf} * K * E_{cm}} = 0.00007$ Strain at the end of the linear part

$\varepsilon_{u0.3} = \frac{w_{\lim}}{l_c} + \frac{f_{ctk}}{\gamma_{bf} * E_{cm}} = 0.0251$ Strain for the limited crack width

$w_{\lim} = 3mm$ Limited crack width

$l_c = \frac{2}{3} * t_{tf} = 120mm$ Characteristic length

$f_{ctk} = 8 N/mm^2$ Characteristic axial tensile strength of UHPC

$E_{cm} = 50000 N/mm^2$ Secant modulus of elasticity of UHPC

$\gamma_{bf} = 1.3$ Partial safety factor fundamental combinations

$\theta = 45°$ Angle between the concrete compression strut and the beam axis perpendicular to the shear force (θ is minimal $30°$)

The ultimate shear stress must be no more than:

$$\tau_u \leq 1.14 \frac{0.85}{\gamma_E * \gamma_c} f_{ck}^{2/3} \sin(2\theta)$$

This gives:

$$V_{Ed} = 130.56 \leq 1.14 \frac{0.85}{\gamma_E * \gamma_c} f_{ck}^{2/3} \sin(2\theta) * b_{deck} * d = 2966 kN \rightarrow Ok$$

Unity check:

$$V_{Ed} / V_{Rd} = 0.18 \leq 1.0 \rightarrow Ok$$

Without stirrup reinforcement the deck easily satisfies with respect to local shear.

Ultimate resistance moment

The ultimate resistance moment of the deck is calculated according to the schematisation in Figure 172. In this case however the schematisation should be mirrored along the centre line as the tension arises at the top side and the compression zone is at the bottom side of the deck, see Figure 173.

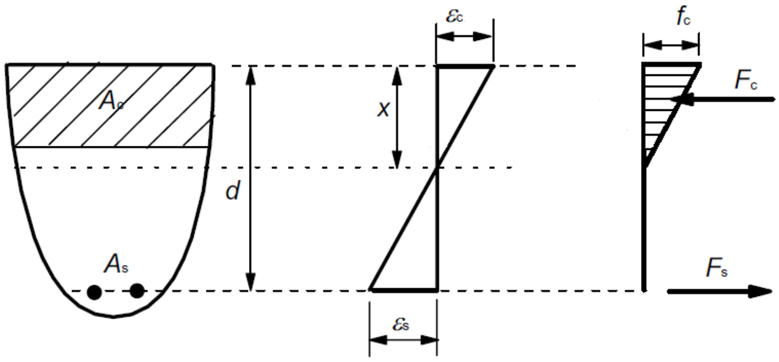

Figure 172: Rectangular stress distribution

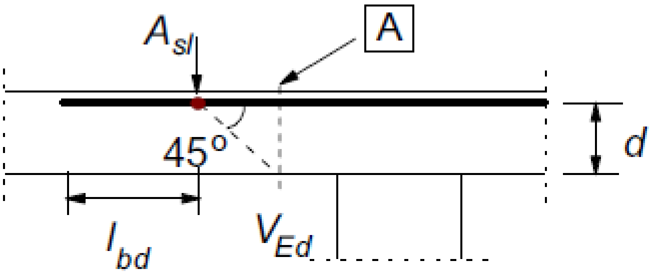

Figure 173: Definition of A_{sl}

The reinforcement in the deck is:

$A_{sl} = n_{reinf} * \pi * (\frac{1}{2} * \phi_{reinf})^2 = 2413mm^2$ Is the area of tensile reinforcement, see Figure 173

$n_{reinf} = b_{deck} / S_{reinf} = 12.5 \rightarrow 12 bars / m$ Is the number of reinforcement bars

$S_{reinf} = 80mm$ Is the spacing of the reinforcement bars

The two horizontal forces F_c and F_s should be in equilibrium:

$$F_c - F_s = 0$$

This is the same as:

$$0.5 * x * b_{deck} * \frac{\varepsilon_c}{\varepsilon_{c3}} * f_{cd} - \varepsilon_s * E_s * A_{sl} = 0$$

Where:

$x = \frac{\varepsilon_c}{\varepsilon_{tot}} * t_{tf}$ Concrete compressive zone

$\varepsilon_{tot} = \varepsilon_c + \varepsilon_s$	Total strain in the deck
$\varepsilon_s = \frac{f_{yd}}{E_s} = 2.174‰$	Tensile strain in the reinforcement
$\varepsilon_{c3} = 1.632‰$	Compressive strain in the concrete at the end of the linear part
$A_{sl} = 2413mm^2$	Area of tensile reinforcement

Solving the formula gives:
$F_c - F_s = 0 \rightarrow \varepsilon_c = 0.84‰$

The concrete compressive zone is:

$$x = \frac{\varepsilon_c}{\varepsilon_{tot}} * t_{tf} = 50.1mm$$

The ultimate resistance moment of the deck is:

$$M_u = \varepsilon_s * E_s * A_{sl} * (d - \frac{1}{3}x) = 133.55kNm$$

Unity check for the ultimate resistance moment of the deck is:

$$M_d / M_u = 0.94 \leq 1.0 \rightarrow Ok$$

The ultimate resistance moment of the deck is thus just enough to resist the bending moments. If this verification is not satisfied the lever arm between the two forces F_c and F_s should be increased. This means that the deck becomes thicker. Another option is to add more reinforcement bars. This however has a strong influence on the rotation capacity, see hereunder.

Furthermore:

The cracking moment is:

$$M_r = f_{ctm,fl} * \frac{1}{6} * b_{deck} * t_{tf}^2 = 162kNm$$

Because $M_r \geq M_u$ brittle failure can occur. Due to the fibres the flexural tensile strength of this material is much larger than for conventional concrete. The ultimate resisting moment M_u should therefore be larger than the bending moment M_d at all times as brittle failure caused by failure of the reinforcement should be excluded.

The height of the compression zone should satisfy: See 5.6.3 [11]

$$x / d = 0.3478 \leq 0.35 \rightarrow Ok$$

This verification considers the rotation capacity of the deck at the supports (the webs) is sufficient. It shows that the verification is satisfied but is very close to the limit so attention is needed. If this verification is not satisfied the thickness of the deck should be increased. Another option is to diminish the number of reinforcement bars which will result in a smaller compressive zone x. This will however also reduce the ultimate resistance moment.

C.10 Fatigue + vibration

C.10.1 Fatigue prestressing steel

For prestressing steel adequate fatigue resistance should be assumed if the following expression is satisfied:

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

Where:

$\gamma_{F,fat} = 1.0$	Is the partial factor for fatigue loads
$\gamma_{s,fat} = \gamma_s = 1.15$	Is the partial factor for prestressing steel for the fatigue verification
$\Delta\sigma_{Rsk}(N^*) = 150 N/mm^2$	Is the stress range at N* cycles, see table 6.4N [11]: straight tendons or curved tendons in plastic ducts
$k_1 = 5$	See Figure 174
$k_2 = 10$	See Figure 174
$N^* = 1000000\, loading\ cycles$	See Figure 174

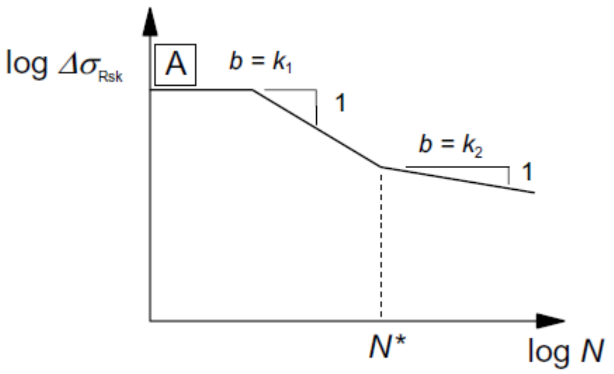

Figure 174: Shape of the characteristic fatigue strength curve (S-N-curves for prestressing steel)

The damage equivalent stress range for prestressing steel is calculated according to Equation NN.106 [12]:

$$\Delta\sigma_{S,equ}(N^*) = \lambda_s * \phi * \Delta\sigma_s$$

Where:

$\Delta\sigma_s$	Is the steel stress range due to the variable load
ϕ	Is the dynamic factor
$\lambda_s = \lambda_{s,1} * \lambda_{s,2} * \lambda_{s,3} * \lambda_{s,4}$	Is a correction factor to calculate the damage equivalent stress range from the stress range caused by $\phi * \Delta\sigma_s$

$\gamma_{s,1}$	Is a factor accounting for element type (eg. continuous beam) and takes into account the damaging effect of traffic depending on the length of the influence line or area.
$\gamma_{s,2}$	Is a factor taking into account the traffic volume
$\gamma_{s,3}$	Is a factor that takes into account the design life of the bridge
$\gamma_{s,4}$	Is a factor to be applied when the structural element is loaded by more than one track

$\gamma_{s,1} = 0.65$ See Table NN.2 [12]: (1) post tensioning straight tendons, s* standard traffic mix and simply supported beam

$$\gamma_{s,2} = \sqrt[k_2]{\frac{Vol}{25*10^6}} = 0.96$$

Where:

$k_2 = 10$ See Figure 174

$Vol = Q_{metro} / g * 6 * 24 * 365 = 15848367 tonnes / year / track$ Assumption of 6 metros per hour

$Q_{metro} = q_{mob} * 116m = 2958kN$ 116 metres is the length of a metro

$$\gamma_{s,3} = \sqrt[k_2]{\frac{N_{years}}{100}} = 1$$

Where:

$N_{years} = 100 years$ Is the design life of the viaduct

$$\gamma_{s,4} = \sqrt[k_2]{n + (1-n) * s_1^{k_2} + (1-n) * s_2^{k_2}} = 0.81$$

Where:

$n = 0.12$ Is the proportion of traffic that crosses the bridge simultaneously, 0.12 is the suggested value

$s_j = 0$ Only compressive stresses occur under traffic loads on a track

$$\lambda_s = \lambda_{s,1} * \lambda_{s,2} * \lambda_{s,3} * \lambda_{s,4} = 0.50$$

The dynamic factor is determined according [8] and not according [10] as this dynamic factor is normative (larger):

$\phi = 1 + 4/(10 + L) = 1.07$

The deflection at mid-span at t=∞ due to the dead load, the permanent load and prestressing is:

$w_1 = -35mm$ Upwards, see Table 23

The deflection at mid-span at t=∞ due to the dead load, the permanent load, the variable load and prestressing is: $w_2 = -8mm$ Upwards, see Table 23

It is assumed that the deflection at the deviation blocks is the same as at mid-span (conservative assumption).

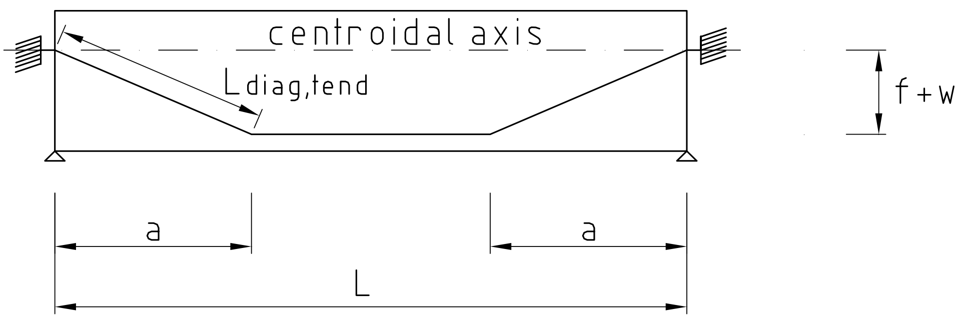

Figure 175: Schematisation for determining the elongation of the tendons

With:

$f = 1143mm$ See Figure 175

$a = 17m$

The length of one diagonal part of the tendon without variable load at t=∞:

$$L_{diag,tend1} = \sqrt{(f + w_1)^2 + a^2} = 17.036m$$

The length of one diagonal part of the tendon with variable load at t=∞:

$$L_{diag,tend2} = \sqrt{(f + w_2)^2 + a^2} = 17.038m$$

The elongation due to the variable load for one diagonal part of the tendon is:

$$\Delta L = \frac{L_{diag,tend1}}{L_{diag,tend2}} = 1.7mm$$

The strain in the tendon due to the variable load for two diagonal parts of the tendon is:

$$\varepsilon_s = 2 * \frac{\Delta L}{L_{diag,tend1}} = 0.0002$$

The steel stress range due to the variable load is:

$$\Delta\sigma_s = \varepsilon_s * E_s = 40.39N/mm^2$$

The damage equivalent stress range for prestressing steel is:

$$\Delta\sigma_{S,equ}(N*) = \lambda_s * \phi * \Delta\sigma_s = 21.77N/mm^2$$

The fatigue verification for prestressing steel is:

$$\gamma_{F,fat} * \Delta\sigma_{S,equ}(N*) \le \frac{\Delta\sigma_{Rsk}(N*)}{\gamma_{s,fat}} \rightarrow \frac{\gamma_{F,fat} * \Delta\sigma_{S,equ}(N*) * \gamma_{s,fat}}{\Delta\sigma_{Rsk}(N*)} = 0.167 \le 1.0 \rightarrow Ok$$

The fatigue verification for prestressing steel is easily satisfied and as the standard [12] (6.8.4) says: "Fatigue verification for external and unbonded tendons, lying within the depth of the concrete section, is not necessary" this could also be expected. This calculation with a rough estimation of the elongation of the tendons is however done to confirm the assumption. Fatigue of the prestressing tendons is not an issue for the design.

C.10.2 Fatigue concrete

The fatigue verification of concrete C50/60 is calculated according Equation NN.112 [12]:
For concrete C50/60 subjected to compression adequate fatigue resistance may be assumed if the following expression is satisfied:

$$14 * \frac{1 - E_{cd,\max,equ}}{\sqrt{1 - R_{equ}}} \ge 6$$

For UHPC there is no fatigue verification and the verification for concrete C50/60 given above cannot be used as the design fatigue strength of the UHPC then becomes:

$$f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd,uls} \left(1 - \frac{f_{ck}}{250}\right) = 19.42N/mm^2$$

Where:

$$\beta_{cc}(t) = \exp\left\{s\left[1 - \left(\frac{28}{t}\right)^{1/2}\right]\right\} \rightarrow \beta_{cc}(t28) = 1.0$$ Coefficient for concrete strength at first load application

$k_1 = 0.85$ Recommended value for $N = 10^6$ cycles

Some stresses in the concrete at t=∞ are larger than this design fatigue strength, see Eq. (35) and (37). According this verification the stresses are thus too large and should be smaller. But as UHPC contains steel fibres which makes the concrete more ductile it is expected that the design fatigue strength is much larger than the value calculated above. The maximum stress in the box girder is:

At deviation block, bottom side

$$\sigma_{cb} = -\frac{P_\infty}{A_c} - \frac{M_{m,p\infty}}{W_b} + \frac{M_{a,\infty} - M_{s,v}}{W_b} = -24.77 N/mm^2$$ See Eq. (35)

The design value of UHPC compressive strength is:

$$f_{cd,uls} = 81.6 N/mm^2$$

The maximum compressive stress is thus much smaller than the design compressive strength of UHPC. As the strong UHPC contains steel fibres the design fatigue strength is assumed to be at least 30.0 N/mm^2. It is expected that the design fatigue strength is even more than this value. My assumption is that even half of the design value of the UHPC compressive strength is still a safe assumption:

$$f_{cd,fat} \approx \frac{1}{2} * f_{cd,uls} = 40.8 N/mm^2$$

Considering the material UHPC with its fibres it is thus expected that the fatigue verification for UHPC is satisfied and will never become an issue for this design. This is however a very critical assumption for the design and should be validated in order to present this design as a good design.

C.10.3 Vibration

For the box girder only the static analysis is considered. The dynamic metro load is multiplied by the dynamic factor ϕ to take into account the dynamic loading. This method of calculation holds when the first natural frequency of the box girder stays within the prescribed limits [10]. When the limits are exceeded a dynamic analysis is required. A dynamic analysis can prove that the box girder is still determined against the dynamic effects. Such an analysis is however extensive and more difficult and is therefore left out of the design of the box girder. For this design the first natural frequency of the box girder should stay within the limits such that a static analysis is sufficient and a dynamic analysis is not necessary. The check for determining whether a dynamic analysis is required is done according two verifications which are elaborated below.

Verification according Annex F [10]

The first natural bending frequency of the box girder is [20]:

$$n_0 = \frac{C_{end}}{2\pi} \sqrt{\frac{E_{cm} I_c}{A_c \rho_c L^4}} = 3.21 Hz$$

Where:

$C_{end} = 9.94$ Boundary condition coefficient [20]

$I_c = 2.381 m^4$ Moment of inertia

The velocity of the metros is:

$$v = 100 km/h = 27.78 m/s$$

Mass m 10^3 kg/m		≥5,0 <7,0	≥7,0 <9,0	≥9,0 <10,0	≥10,0 <13,0	≥13,0 <15,0	≥15,0 <18,0	≥18,0 <20,0	≥20,0 <25,0	≥25,0 <30,0	≥30,0 <40,0	≥40,0 <50,0	≥50,0 -
Span $L \in$ m [a]	ζ %	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m
[5,00,7,50)	2	1,71	1,78	1,88	1,88	1,93	1,93	2,13	2,13	3,08	3,08	3,54	3,59
	4	1,71	1,83	1,93	1,93	2,13	2,24	3,03	3,08	3,38	3,54	4,31	4,31
[7,50,10,0)	2	1,94	2,08	2,64	2,64	2,77	2,77	3,06	5,00	5,14	5,20	5,35	5,42
	4	2,15	2,64	2,77	2,98	4,93	5,00	5,14	5,21	5,35	5,62	6,39	6,53
[10,0,12,5)	1	2,40	2,50	2,50	2,50	2,71	6,15	6,25	6,36	6,36	6,45	6,45	6,57
	2	2,50	2,71	2,71	5,83	6,15	6,25	6,36	6,36	6,45	6,45	7,19	7,29
[12,5,15,0)	1	2,50	2,50	3,58	3,58	5,24	5,24	5,36	5,36	7,86	9,14	9,14	9,14
	2	3,45	5,12	5,24	5,24	5,36	5,36	7,86	8,22	9,53	9,76	10,36	10,48
[15,0,17,5)	1	3,00	5,33	5,33	5,33	6,33	6,33	6,50	6,50	6,50	7,80	7,80	7,80
	2	5,33	5,33	6,33	6,33	6,50	6,50	10,17	10,33	10,33	10,50	10,67	12,40
[17,5,20,0)	1	3,50	6,33	6,33	6,33	6,50	6,50	7,17	7,17	10,67	12,80	12,80	12,80
[20,0,25,0)	1	5,21	5,21	5,42	7,08	7,50	7,50	13,54	13,54	13,96	14,17	14,38	14,38
[25,0,30,0)	1	6,25	6,46	6,46	10,21	10,21	10,21	10,63	10,63	12,75	12,75	12,75	12,75
[30,0,40,0)	1				10,56	18,33	18,33	18,61	18,61	18,89	19,17	19,17	19,17
≥40,0	1				14,73	15,00	15,56	15,56	15,83	18,33	18,33	18,33	18,33

[a] $L \in [a,b)$ means $a \leq L < b$

NOTE 1 Table F.1 includes a safety factor of 1.2 on $(v/n_0)_{lim}$ for acceleration, deflection and strength criteria and a safety factor of 1,0 on the $(v/n_0)_{lim}$ for fatigue.

NOTE 2 Table F.1 includes an allowance of $(1+\varphi''/2)$ for track irregularities.

Table 24: Maximum value of $(v/n_0)_{\lim}$ for a simply supported beam or slab and a maximum permitted acceleration of $\alpha_{\max} < 3.5 m/s^2$, Table F.1 [10]

Table 24 gives no maximum value of the velocity divided by the first natural frequency for:

$$m = A_c * \rho_c = 7.074 * 10^3 kg/m$$

$$L = 45m$$

Therefore there is made an extrapolation of the table to determine the maximum value of the velocity divided by the first natural frequency. This is a rough extrapolation as the difference of the values is large between masses above and below $m = 10 * 10^3 kg/m$, see Table 24.

The extrapolated maximum value of the velocity divided by the first natural frequency is:

$$(v/n_0)_{\lim} = 10.0m$$

The verification of the ratio of the velocity over the first natural frequency is:

$$v/n_0 = 8.67m \leq 10.0m \rightarrow Ok$$

Verification according to Figure 6.10 [10]

Limits of natural frequency n_0 (Hz) as a function of L (m)

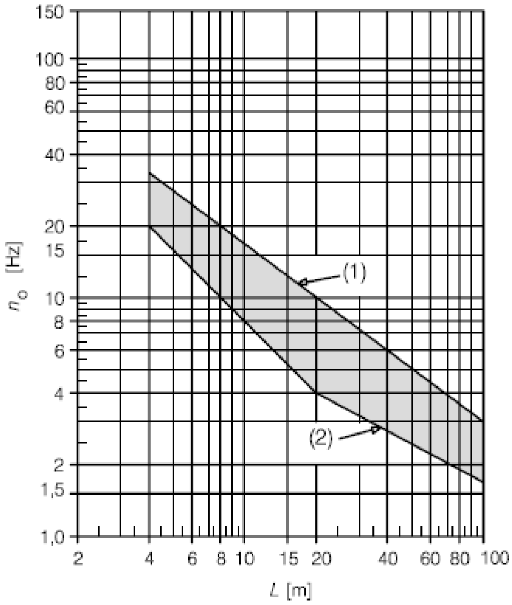

Figure 176: Limits of bridge natural frequency n_0 (Hz) as a function of L (m)

According this verification the first natural frequency of the box girder should be in the grey area, see Figure 176.

Where:
The upper limit of natural frequency is governed by dynamic enhancements due to track irregularities and is given by:

$$n_{0\max} = 94.76 * L^{-0.748} = 5.5Hz$$

The lower limit of natural frequency is governed by dynamic impact criteria and is given by:

$$n_{0\min} = 23.58 * L^{-0.592} = 2.48Hz$$

The first natural frequency of the box girder is:

$$n_0 = \frac{C_{end}}{2\pi}\sqrt{\frac{E_{cm}I_c}{A_c\rho_c L^4}} = 3.21Hz \rightarrow Ok$$

Conclusion

Both verifications show that the box girder does not require a dynamic analysis and a static analysis is sufficient. As the first natural frequency of the girder easily stays within the limits, the box girder is well determined against the dynamic effects. The increasing and decreasing of static stresses and deformations under the effects of moving traffic should, considering the calculations, not give any problems for this box girder. The roughly extrapolated maximum value of the velocity divided by the first natural frequency in the first verification is considered as a safe value as Table 24 is valid for trains and in this design, metros cause the dynamic loading which in general is less than the dynamic loading by trains. Besides the first natural frequency of the box girder is still large enough to satisfy with respect to the roughly extrapolated maximum value of the velocity divided by the first natural frequency from Table 24. It is thus concluded that the box girder has a good resistance against vibration.

C.11 Buckling webs

Verification of buckling is needed for the webs of the box girder. The buckling strength of the webs should meet the requirement:

$$F_k \geq \alpha_{cr} * F_d$$

Where:

$F_k = \dfrac{\pi^2 EI}{l_0^2}$	Euler buckling force
$\alpha_{cr} = 10$	Force amplifier to reach the elastic critical buckling
$F_d = Max\{V_{Ed,dl0}/2; V_{Ed,s\infty}/2 + V_{Ed+w}\}/\cos\alpha_w = 1866kN$	Buckling force in one web, see C.7

$V_{Ed,dl0} = 3567kN$	See Eq. (39)
$V_{Ed,s\infty} = 2691kN$	See Eq. (40)
$V_{Ed+w} = 475kN$	See Eq. (42)
$\alpha_w = 12.64°$	Angle of webs with vertical axis, see Figure 178
$E_{cm} = 50000N/mm^2$	Secant modulus of elasticity of UHPC

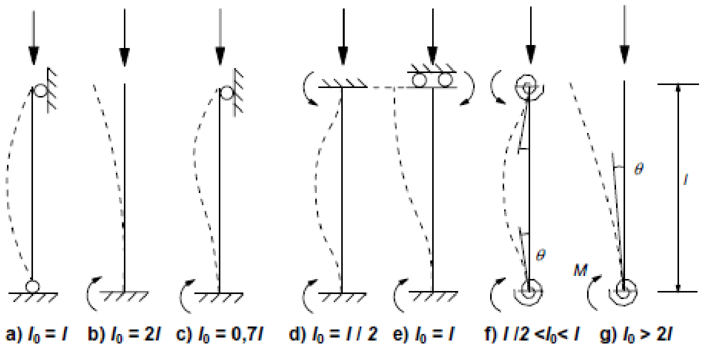

Figure 177: Examples of different buckling modes and corresponding effective lengths for isolated members

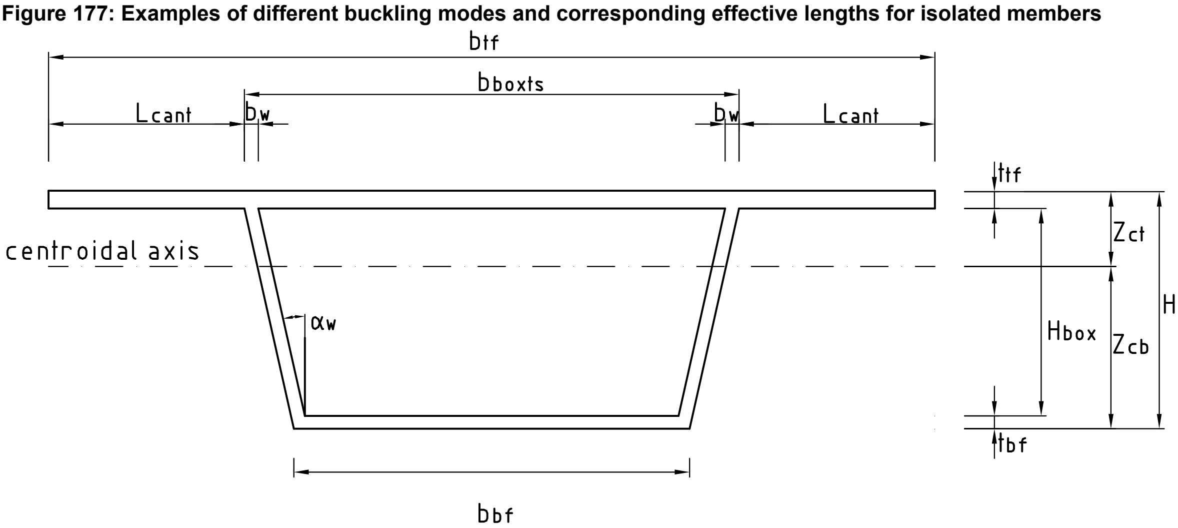

Figure 178: Cross-section of the box girder

The webs are fixed to the flanges. This would mean that buckling mode d, see Figure 177, can be considered to determine the effective buckling length. But as the webs and flanges are relatively slender full rotation stiffness is not likely to occur. In reality buckling mode f should be taken to calculate the effective buckling length of the webs. The rotation stiffness is dependent on the stiffness of the flanges. To determine this rotation stiffness a more extensive calculation is necessary. To be able to make a simple verification of buckling it is therefore chosen to schematise the webs as buckling mode a, see Figure 177. This is the most conservative buckling mode for the webs, where the effective buckling length equals the length of the webs:

$$l_0 = \sqrt{H_{box}^2 + ((b_{boxts} - b_{bf})/2)^2} = 2.159m$$

Now the required moment of inertia of the webs can be calculated:

$$\frac{\pi^2 EI}{l_0^2} \geq \alpha_{cr} * F_d \rightarrow I \geq \frac{l_0^2 * \alpha_{cr} * F_d}{\pi^2 * E} = 176203498mm^4$$

The formula for the moment of inertia of the web is:

$$I = \frac{1}{12} * L_{webs} * t_w^3 \geq 176203498mm^4$$

In this formula L_{webs} is the effective length of the webs in longitudinal direction of the box girder which can be taken for the buckling resistance. It is hard to determine this effective length, especially for a segmental box girder with its joints between the segments creating discontinuities in the webs. For this calculation it is chosen to take effective length of the webs as 1 metre. This is chosen as in the local schematisation of the deck, see C.9.2, the local metro point load is distributed over 1 metre in the longitudinal direction of the box girder. In the deck schematisation it is therefore chosen to take a deck width of 1 metre. This local deck load should be taken by the webs. For this reason an effective length of the webs of 1 metre is chosen in respect of buckling of the webs. Besides, this assumption is considered as quite conservative as buckling of the webs will probably concern more than 1 metre. Most likely the effective length of the webs equals the length of a segmental box girder, which means a length of 3 metres. However, in this buckling verification a safe assumption of the effective length is taken:

$$L_{webs} = 1m$$

The minimum required thickness of the webs hereby becomes:

$$t_{w,req} = \sqrt[3]{\frac{176203498 * 12}{L_{webs}}} = 128mm$$

The minimum required width of the webs hereby becomes:

$$b_{w,req} = t_w / \cos\alpha_w = 132mm$$

Verification of buckling of the webs:

$$b_w = 140mm \geq b_{w,req} = 132mm \rightarrow Ok$$

The webs thus satisfy with respect to buckling. The thickness of the webs is however just enough to resist buckling. When this verification is not satisfied the thickness of the webs should be increased.

Appendix D: Results optimisation process

D.1 Results optimisation process concrete box girder C50/60

D.1.1 Box girder with 6 tendons

Depth webs		2.1	2.25	2.4	2.55	2.7	2.85	3	3.15	3.3	m
Number of shear keys		14	15	16	17	18	19	20	21	22	
Thickness top flange		0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	m
Number of tendons		6	6	6	6	6	6	6	6	6	
Depth box girder		2.65	2.8	2.95	3.1	3.25	3.41	3.6	3.79	3.99	m
Thickness bottom flange		0.3	0.3	0.3	0.3	0.3	0.31	0.35	0.39	0.44	m
Width web		0.16	0.16	0.17	0.18	0.18	0.19	0.2	0.21	0.21	m
Distance of deviation blocks to supports		14	15	16	17	18	19	19	19	19	m
Dead load box girder		100.85	102.02	104.38	106.88	108.20	111.91	118.70	125.64	132.09	kN/m
Ultimate resistance moment at t=0 bottom side		0.870853	0.898022	0.925355	0.952394	0.978226	0.996586	0.996742	0.998938	0.994933	Unity check
Ultimate resistance moment at t=∞ top side		1.067195	0.897363	0.769691	0.658194	0.538472	0.464228	0.436005	0.414428	0.390166	Unity check
Stresses at t=0	deviation block top side	-4.715889	-4.418485	-4.103952	-3.800871	-3.536554	-3.298061	-3.145133	-2.992994	-2.89869	N/mm^2
	deviation block bottom side	-21.23773	-21.3488	-21.12039	-20.85923	-20.90323	-20.11878	-18.23029	-16.66678	-15.29903	N/mm^2
	mid-span top side	-5.491809	-4.988227	-4.513265	-4.082364	-3.716359	-3.40354	-3.248545	-3.09462	-2.998141	N/mm^2
	mid-span bottom side	-19.97805	-20.42722	-20.46422	-20.41219	-20.61892	-19.95653	-18.08257	-16.53075	-15.17465	N/mm^2
Stresses at t=∞ without variable load	deviation block top side	-6.295698	-5.984093	-5.664535	-5.353059	-5.065025	-4.797469	-4.559416	-4.336756	-4.166093	N/mm^2
	deviation block bottom side	-12.89422	-13.11722	-13.07876	-13.0151	-13.1874	-12.77399	-11.6655	-10.73349	-9.929176	N/mm^2
	mid-span top side	-7.336447	-6.74605	-6.208824	-5.725205	-5.302027	-4.93539	-4.692815	-4.466222	-4.291458	N/mm^2
	mid-span bottom side	-11.2046	-11.88473	-12.20622	-12.42409	-12.81265	-12.56184	-11.47495	-10.5602	-9.772393	N/mm^2
Stresses at t=∞fuly loaded	deviation block top side	-8.989862	-8.588345	-8.174995	-7.768909	-7.389832	-7.013183	-6.607394	-6.238173	-5.935957	N/mm^2
	deviation block bottom side	-8.52035	-8.904766	-9.054282	-9.178469	-9.511447	-9.365744	-8.740106	-8.188363	-7.715772	N/mm^2
	mid-span top side	-10.47912	-9.675834	-8.947877	-8.294583	-7.723701	-7.206049	-6.791578	-6.41479	-6.105211	N/mm^2
	mid-span bottom side	-6.102598	-7.145723	-7.815288	-8.343641	-8.983539	-9.069075	-8.477012	-7.951953	-7.504103	N/mm^2
Prestressing losses at the first support		11.15423	11.11402	11.0274	10.93832	10.89514	10.76665	10.58308	10.41728	10.28006	%
Prestressing losses at mid-span after the first deviation block		11.12307	11.07223	10.98597	10.89991	10.85274	10.75272	10.69101	10.62863	10.58635	%
Prestressing losses at mid-span after the second deviation block		11.93218	11.88726	11.80371	11.71951	11.67572	11.57198	11.53671	11.50115	11.48361	%
Prestressing losses at the second support		13.58159	13.55908	13.48063	13.39709	13.36408	13.22442	13.12019	13.03483	12.97185	%
Deflection at t=0		0.378062	0.381341	0.373953	0.364311	0.359685	0.335718	0.284977	0.245182	0.211059	Unity check
Deflection without variable load		-0.201902	-0.25385	-0.281205	-0.298899	-0.319651	-0.30935	-0.260584	-0.222405	-0.190005	Unity check
Additional deflection under mobile load		0.586826	0.516595	0.455878	0.404604	0.362771	0.318792	0.267035	0.226779	0.192968	Unity check
Final deflection fully loaded (t=∞)		-0.006293	-0.081651	-0.129246	-0.164031	-0.198727	-0.203086	-0.171572	-0.146812	-0.125682	Unity check
Vertical shear in webs	at t=0	0.920781	0.883005	0.795721	0.722152	0.699394	0.636519	0.59001	0.549604	0.537309	Unity check
Vertical shear in webs	at t=∞	0.736511	0.687258	0.61579	0.557271	0.529241	0.491774	0.456815	0.426601	0.418137	Unity check
Vertical shear + torsion in webs	at t=∞	0.966541	0.901702	0.805451	0.726368	0.689049	0.636085	0.588759	0.547896	0.53545	Unity check
Horizontal shear in top flange		0.190617	0.192914	0.195211	0.197508	0.199806	0.202256	0.205166	0.208076	0.211139	Unity check
Horizontal shear + torsion in top flange		0.912548	0.874403	0.841097	0.811618	0.785043	0.762128	0.743346	0.72678	0.712492	Unity check
Horizontal shear in bottom flange		0.042276	0.044669	0.047062	0.049455	0.051848	0.052646	0.049227	0.04651	0.0434	Unity check
Horizontal shear + torsion in bottom flange		0.794288	0.754554	0.71986	0.689152	0.66147	0.617032	0.529745	0.462138	0.399475	Unity check
Fatigue prestressing steel		0.166741	0.137437	0.11388	0.095261	0.080934	0.067138	0.058201	0.051091	0.044774	Unity check
Fatigue concrete deviation block top side		0.703712	0.681884	0.659761	0.638493	0.618683	0.59845	0.574681	0.553056	0.534609	Unity check
Fatigue concrete deviation block bottom side		0.990855	0.998424	0.991537	0.983196	0.987956	0.958542	0.885371	0.823682	0.769402	Unity check
Fatigue concrete mid-span top side		0.787934	0.743081	0.702748	0.666959	0.636664	0.608771	0.584513	0.562448	0.543598	Unity check
Fatigue concrete mid-span bottom side		0.92299	0.948056	0.955623	0.958755	0.972329	0.949759	0.87773	0.816947	0.763511	Unity check
First natural bending frequency n0		3.239833	3.433065	3.613091	3.790051	3.978044	4.172774	4.426872	4.669173	4.936596	Hz
Ratio v/no		8.57383	8.091248	7.688093	7.329131	6.982772	6.656909	6.274809	5.949185	5.626909	m
Mass box girder m		10.28	10.4	10.64	10.895	11.03	11.4075	12.1	12.8075	13.465	10^3 kg/m
Buckling webs		150.4414	156.8861	163.9233	171.0415	178.4204	185.4541	193.2599	201.1085	208.808	mm

D.1.2 Box girder with 8 tendons

Depth webs		1.65	1.8	1.95	2.1	2.25	2.4	2.55	2.7	2.85	m
Number of shear keys		11	12	13	14	15	16	17	18	19	
Thickness top flange		0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	m
Number of tendons		8	8	8	8	8	8	8	8	8	
Depth box girder		2.35	2.51	2.67	2.83	2.99	3.14	3.29	3.43	3.6	m
Thickness bottom flange		0.45	0.46	0.47	0.48	0.49	0.49	0.49	0.48	0.5	m
Width web		0.21	0.2	0.18	0.17	0.17	0.17	0.18	0.19	0.2	m
Distance of deviation blocks to supports		12	13	15	16	17	19	20	22	22.5	m
Dead load box girder		116.08	117.72	118.26	119.53	121.77	123.02	125.52	127.19	131.94	kN/m
Ultimate resistance moment at t=0 bottom side		0.798788	0.826345	0.835166	0.860645	0.88578	0.907148	0.938517	0.972639	0.99691	Unity check
Ultimate resistance moment at t=∞ top side		1.046222	0.863562	0.695275	0.563076	0.46048	0.363565	0.288101	0.211599	0.168796	Unity check
Stresses at t=0	deviation block top side	-6.225412	-5.85016	-5.749615	-5.417912	-5.072714	-4.816712	-4.424327	-4.041282	-3.694648	N/mm^2
	deviation block bottom side	-21.20692	-21.13925	-21.00045	-20.97449	-20.75221	-20.75268	-20.65905	-20.84766	-20.09877	N/mm^2
	mid-span top side	-7.789031	-7.040237	-6.438602	-5.903579	-5.4025	-4.943383	-4.486274	-4.043657	-3.694648	N/mm^2
	mid-span bottom side	-19.18466	-19.62567	-20.13804	-20.37635	-20.35259	-20.59983	-20.58461	-20.84479	-20.09877	N/mm^2
Stresses at t=∞ without variable load	deviation block top side	-7.768961	-7.381157	-7.306337	-6.936911	-6.562929	-6.304282	-5.889581	-5.49515	-5.10831	N/mm^2
	deviation block bottom side	-13.35894	-13.47614	-13.41242	-13.57429	-13.56666	-13.64458	-13.70332	-13.93943	-13.54165	N/mm^2
	mid-span top side	-9.796237	-8.919199	-8.195858	-7.562425	-6.985937	-6.466396	-5.968514	-5.498167	-5.10831	N/mm^2
	mid-span bottom side	-10.73702	-11.52	-12.29899	-12.80392	-13.05408	-13.44896	-13.60847	-13.93578	-13.54165	N/mm^2
Stresses at t=∞fuly loaded	deviation block top side	-10.58942	-10.09752	-10.02329	-9.537981	-9.047221	-8.724866	-8.191109	-7.698158	-7.171843	N/mm^2
	deviation block bottom side	-9.711187	-10.02137	-10.01156	-10.37086	-10.55635	-10.72362	-10.93779	-11.2723	-11.10926	N/mm^2
	mid-span top side	-13.40193	-12.22487	-11.25243	-10.40034	-9.628106	-8.947004	-8.298812	-7.702264	-7.171843	N/mm^2
	mid-span bottom side	-6.0737	-7.315728	-8.473032	-9.30879	-9.852466	-10.45556	-10.80837	-11.26733	-11.10926	N/mm^2
Prestressing losses at the first support		11.65407	11.59946	11.53942	11.49938	11.42429	11.35154	11.27285	11.19629	11.06166	%
Prestressing losses at mid-span after the first deviation block		11.63009	11.57209	11.46096	11.42379	11.35658	11.24401	11.16787	11.05187	10.96421	%
Prestressing losses at mid-span after the second deviation block		12.31439	12.26484	12.11404	12.08428	12.02283	11.88166	11.81269	11.67566	11.60468	%
Prestressing losses at the second support		13.70699	13.6777	13.49865	13.48085	13.42306	13.26449	13.20733	13.06766	12.98306	%
Deflection at t=0		0.335369	0.346775	0.35485	0.353686	0.345801	0.344839	0.338406	0.338764	0.31514	Unity check
Deflection without variable load		-0.055374	-0.143323	-0.212563	-0.256184	-0.280716	-0.307587	-0.321202	-0.340258	-0.324026	Unity check
Additional deflection under mobile load		0.663693	0.564345	0.486204	0.422092	0.368642	0.328819	0.294128	0.267943	0.235558	Unity check
Final deflection fully loaded (t=∞)		0.165857	0.044792	-0.050495	-0.115486	-0.157836	-0.197981	-0.223159	-0.250944	-0.245507	Unity check
Vertical shear in webs	at t=0	0.961124	0.955118	0.955469	0.971807	0.934503	0.875838	0.805681	0.731982	0.68367	Unity check
Vertical shear in webs	at t=∞	0.74366	0.712496	0.773337	0.757208	0.71196	0.69904	0.626597	0.582439	0.531573	Unity check
Vertical shear + torsion in webs	at t=∞	0.974449	0.933925	0.999524	0.979478	0.919985	0.894481	0.801177	0.739222	0.67392	Unity check
Horizontal shear in top flange		0.186022	0.188472	0.190923	0.193373	0.195824	0.198121	0.200418	0.202562	0.205166	Unity check
Horizontal shear + torsion in top flange		1.051782	0.997202	0.950358	0.909857	0.87457	0.843585	0.816041	0.791212	0.769573	Unity check
Horizontal shear in bottom flange		0.024993	0.026115	0.027188	0.028217	0.029204	0.030669	0.032134	0.0342	0.034459	Unity check
Horizontal shear + torsion in bottom flange		0.626216	0.575524	0.532132	0.494678	0.462078	0.442318	0.424751	0.417435	0.387214	Unity check
Fatigue prestressing steel		0.188834	0.148425	0.103749	0.085076	0.070428	0.053725	0.046157	0.036966	0.032676	Unity check
Fatigue concrete deviation block top side		0.787301	0.759789	0.75601	0.728737	0.701276	0.683655	0.654289	0.628255	0.599675	Unity check
Fatigue concrete deviation block bottom side		0.996299	0.997226	0.992307	0.995352	0.989273	0.990801	0.98923	0.999006	0.969763	Unity check
Fatigue concrete mid-span top side		0.949277	0.881823	0.826333	0.777906	0.734253	0.69621	0.660336	0.628478	0.599675	Unity check
Fatigue concrete mid-span bottom side		0.888571	0.915542	0.945345	0.962608	0.967428	0.982423	0.985157	0.998848	0.969763	Unity check
First natural bending frequency n0		2.839565	3.057814	3.286859	3.50879	3.719991	3.91874	4.101903	4.269392	4.470579	Hz
Ratio v/no		9.782406	9.084196	8.451163	7.916626	7.467163	7.088446	6.771925	6.506261	6.213464	m
Mass box girder m		11.8325	12	12.055	12.185	12.4125	12.54	12.795	12.965	13.45	10^3 kg/m
Buckling webs		131.7484	139.4915	147.3084	153.9059	161.7387	169.5143	176.8395	184.8665	193.0263	mm

D.1.3 Box girder with 4 tendons

Depth webs		3.75	3.9	4.05	4.2	4.35	4.5	4.65	4.8	4.95	m
Number of shear keys		25	26	27	28	29	30	31	32	33	
Thickness top flange		0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	m
Number of tendons		4	4	4	4	4	4	4	4	4	
Depth box girder		4.34	4.52	4.7	4.88	5.06	5.24	5.42	5.6	5.78	m
Thickness bottom flange		0.34	0.37	0.4	0.43	0.46	0.49	0.52	0.55	0.58	m
Width web		0.23	0.23	0.24	0.25	0.26	0.26	0.27	0.28	0.29	m
Distance of deviation blocks to supports		14	14	14	14	14	14	14	14	14	m
Dead load box girder		130.60	135.23	141.85	148.62	155.54	160.39	167.53	174.81	182.25	kN/m
Ultimate resistance moment at t=0 bottom side		0.999156	0.997921	0.996951	0.996519	0.996479	0.997403	0.997946	0.998605	0.999315	Unity check
Ultimate resistance moment at t=∞ top side		1.034306	0.975543	0.954693	0.938688	0.926662	0.89013	0.884705	0.881608	0.880525	Unity check
Stresses at t=0	deviation block top side	-2.043345	-1.986903	-1.923265	-1.861886	-1.803103	-1.753293	-1.699323	-1.648209	-1.599897	N/mm^2
	deviation block bottom side	-10.75834	-10.15668	-9.452613	-8.831987	-8.280407	-7.916622	-7.462069	-7.050599	-6.676235	N/mm^2
	mid-span top side	-2.57085	-2.502386	-2.432876	-2.366002	-2.302015	-2.242915	-2.184367	-2.128796	-2.07612	N/mm^2
	mid-span bottom side	-10.01818	-9.464987	-8.798305	-8.21001	-7.686652	-7.352291	-6.91967	-6.52777	-6.170977	N/mm^2
Stresses at t=∞ without variable load	deviation block top side	-3.103554	-2.99695	-2.893636	-2.796459	-2.705078	-2.622014	-2.540802	-2.464343	-2.392322	N/mm^2
	deviation block bottom side	-6.54564	-6.236801	-5.822325	-5.453077	-5.121712	-4.927858	-4.648622	-4.393995	-4.160761	N/mm^2
	mid-span top side	-3.770089	-3.643637	-3.526901	-3.417327	-3.314398	-3.216707	-3.125502	-3.039556	-2.958487	N/mm^2
	mid-span bottom side	-5.610403	-5.36905	-5.009251	-4.687053	-4.396562	-4.242422	-3.994785	-3.768224	-3.560077	N/mm^2
Stresses at t=∞fuly loaded	deviation block top side	-4.517936	-4.331722	-4.151604	-3.98419	-3.82828	-3.690929	-3.554614	-3.426985	-3.307326	N/mm^2
	deviation block bottom side	-4.561075	-4.445749	-4.207171	-3.987658	-3.784992	-3.695842	-3.514932	-3.346741	-3.18997	N/mm^2
	mid-span top side	-5.41993	-5.200615	-4.994289	-4.802785	-4.624585	-4.463569	-4.308087	-4.162453	-4.025816	N/mm^2
	mid-span bottom side	-3.295459	-3.279833	-3.125216	-2.977678	-2.837311	-2.805307	-2.672364	-2.546629	-2.427673	N/mm^2
Prestressing losses at the first support		9.763572	9.725813	9.661139	9.603766	9.55312	9.534246	9.493843	9.458854	9.428889	%
Prestressing losses at mid-span after the first deviation block		10.7446	10.77768	10.78148	10.78856	10.79925	10.83718	10.85363	10.87394	10.89805	%
Prestressing losses at mid-span after the second deviation block		12.17702	12.24859	12.28834	12.33199	12.37986	12.45622	12.51065	12.56958	12.63294	%
Prestressing losses at the second support		14.06083	14.13856	14.18173	14.23404	14.29496	14.39136	14.4649	14.54578	14.63358	%
Deflection at t=0		0.118676	0.106533	0.093666	0.082822	0.073597	0.067436	0.060423	0.054327	0.048996	Unity check
Deflection without variable load		-0.058652	-0.0528	-0.043625	-0.035989	-0.029582	-0.027076	-0.022184	-0.017998	-0.014396	Unity check
Additional deflection under mobile load		0.17231	0.152154	0.134497	0.119619	0.106962	0.096612	0.087175	0.078966	0.071783	Unity check
Final deflection fully loaded (t=∞)		-0.001216	-0.002082	0.001207	0.003884	0.006072	0.005128	0.006874	0.008324	0.009531	Unity check
Vertical shear in webs	at t=0	0.403441	0.397049	0.372743	0.351027	0.331536	0.327425	0.310321	0.294821	0.280723	Unity check
Vertical shear in webs	at t=∞	0.312114	0.303801	0.28904	0.275838	0.263975	0.258534	0.248351	0.239106	0.230683	Unity check
Vertical shear + torsion in webs	at t=∞	0.405701	0.394738	0.374111	0.355674	0.339114	0.331985	0.317748	0.304827	0.29306	Unity check
Horizontal shear in top flange		0.216499	0.219256	0.222013	0.224769	0.227526	0.230283	0.233039	0.235796	0.238553	Unity check
Horizontal shear + torsion in top flange		0.664252	0.654529	0.646077	0.638467	0.631606	0.625356	0.619807	0.614793	0.610263	Unity check
Horizontal shear in bottom flange		0.061092	0.058467	0.056235	0.054315	0.052646	0.051181	0.049885	0.04873	0.047695	Unity check
Horizontal shear + torsion in bottom flange		0.472629	0.426096	0.387535	0.354968	0.327156	0.303141	0.282317	0.264069	0.24797	Unity check
Fatigue prestressing steel		0.086719	0.078598	0.071156	0.064797	0.059311	0.05484	0.050617	0.046892	0.043585	Unity check
Fatigue concrete deviation block top side		0.461284	0.449602	0.438147	0.427423	0.417353	0.408571	0.399623	0.391153	0.383121	Unity check
Fatigue concrete deviation block bottom side		0.593323	0.569151	0.54062	0.515495	0.492959	0.477407	0.458478	0.441212	0.425397	Unity check
Fatigue concrete mid-span top side		0.508097	0.494595	0.481593	0.469425	0.458001	0.447863	0.43772	0.42812	0.419014	Unity check
Fatigue concrete mid-span bottom side		0.57032	0.549113	0.523782	0.501115	0.480705	0.466554	0.449331	0.433613	0.419217	Unity check
First natural bending frequency n0		5.253979	5.494511	5.706019	5.911096	6.110502	6.33141	6.521781	6.70809	6.890781	Hz
Ratio v/no		5.286998	5.055551	4.868154	4.69926	4.545908	4.387297	4.259232	4.140937	4.031151	m
Mass box girder m		13.3125	13.785	14.46	15.15	15.855	16.35	17.0775	17.82	18.5775	10^3 kg/m
Buckling webs		222.131	229.1686	236.4649	244.2666	252.1273	258.8415	266.7976	274.8114	282.8829	mm

D.2 Results optimisation process UHPC box girder C180

D.2.1 Box girder with 6 tendons

Depth webs		1.95	2.1	2.25	2.4	2.55	2.7	2.85	3	3.15	m
Number of shear keys		13	14	15	16	17	18	19	20	21	
Thickness top flange		0.18	0.18	0.18	0.18	0.18	0.18	0.18	0.18	0.18	m
Number of tendons		6	6	6	6	6	6	6	6	6	
Depth box girder		2.27	2.41	2.57	2.75	2.93	3.1	3.28	3.47	3.66	m
Thickness bottom flange		0.14	0.13	0.14	0.17	0.2	0.22	0.25	0.29	0.33	m
Width web		0.13	0.14	0.14	0.15	0.16	0.16	0.17	0.18	0.18	m
Distance of deviation blocks to supports		15	17	18	18	18	19	19	19	19	m
Dead load box girder		68.35	69.40	71.49	76.84	82.35	85.62	91.36	98.27	103.73	kN/m
Ultimate resistance moment at t=0 bottom side		0.927842	0.979699	0.998876	0.991904	0.990612	0.996762	0.999957	0.995606	0.991634	Unity check
Ultimate resistance moment at t=∞ top side		1.055392	0.844592	0.655365	0.570374	0.510473	0.426843	0.396686	0.3872	0.362394	Unity check
Stresses at t=0	deviation block top side	-6.059373	-5.329337	-4.927687	-4.711015	-4.458579	-4.243855	-3.996471	-3.819098	-3.687037	N/mm^2
	deviation block bottom side	-39.6572	-40.86877	-39.09953	-33.68821	-29.62629	-27.7247	-24.92017	-22.0586	-20.11599	N/mm^2
	mid-span top side	-6.720959	-5.667572	-5.14163	-4.918385	-4.660884	-4.361718	-4.112333	-3.933729	-3.799385	N/mm^2
	mid-span bottom side	-38.26833	-40.14471	-38.66249	-33.31336	-29.29644	-27.5433	-24.75569	-21.90975	-19.98055	N/mm^2
Stresses at t=∞ without variable load	deviation block top side	-8.571311	-7.967731	-7.441283	-6.993425	-6.567226	-6.256469	-5.891984	-5.598958	-5.359139	N/mm^2
	deviation block bottom side	-23.9083	-24.7653	-24.13164	-21.12164	-18.80897	-17.77569	-16.11049	-14.34397	-13.18806	N/mm^2
	mid-span top side	-9.566056	-8.473726	-7.758235	-7.29368	-6.854085	-6.421715	-6.051498	-5.75374	-5.508767	N/mm^2
	mid-span bottom side	-21.82001	-23.68212	-23.48417	-20.57889	-18.34125	-17.52136	-15.88404	-14.14299	-13.00768	N/mm^2
Stresses at t=∞fuly loaded	deviation block top side	-13.08517	-12.43845	-11.62817	-10.76882	-10.00401	-9.492538	-8.873276	-8.341111	-7.905251	N/mm^2
	deviation block bottom side	-14.43226	-15.19483	-15.57872	-14.2971	-13.20532	-12.79494	-11.87813	-10.78334	-10.11865	N/mm^2
	mid-span top side	-14.64415	-13.22856	-12.11958	-11.22638	-10.43407	-9.738031	-9.106719	-8.563892	-8.118017	N/mm^2
	mid-span bottom side	-11.15947	-13.50344	-14.57488	-13.46999	-12.50412	-12.41709	-11.54673	-10.49407	-9.86216	N/mm^2
Prestressing losses at the first support		5.607631	5.571406	5.547571	5.497114	5.454462	5.412999	5.378921	5.336719	5.315067	%
Prestressing losses at mid-span after the first deviation block		5.936955	5.854651	5.856059	5.890747	5.917116	5.889904	5.90828	5.914	5.933902	%
Prestressing losses at mid-span after the second deviation block		6.625196	6.523545	6.533471	6.59295	6.644349	6.611864	6.653785	6.678139	6.718922	%
Prestressing losses at the second support		7.672354	7.578086	7.579809	7.603724	7.63616	7.578878	7.615436	7.629137	7.670129	%
Deflection at t=0		0.42344	0.435882	0.397408	0.314603	0.256182	0.227843	0.191761	0.157841	0.134705	Unity check
Deflection without variable load		-0.328954	-0.384548	-0.372879	-0.294388	-0.238907	-0.218189	-0.182674	-0.147326	-0.124854	Unity check
Additional deflection under mobile load		0.974998	0.87138	0.726141	0.564627	0.451973	0.381982	0.316943	0.26176	0.221111	Unity check
Final deflection fully loaded (t=∞)		-0.003955	-0.094088	-0.130832	-0.106179	-0.08825	-0.090861	-0.077026	-0.060072	-0.05115	Unity check
Vertical shear in webs	at t=0	0.388812	0.334594	0.319957	0.287651	0.260832	0.247937	0.226988	0.20736	0.201948	Unity check
Vertical shear in webs	at t=∞	0.283538	0.252455	0.236996	0.211682	0.190986	0.187129	0.171244	0.160278	0.156524	Unity check
Vertical shear + torsion in webs	at t=∞	0.38733	0.341614	0.320151	0.285129	0.256475	0.249339	0.227391	0.21142	0.205854	Unity check
Horizontal shear in top flange		0.222797	0.225382	0.228336	0.23166	0.234984	0.238123	0.241446	0.244954	0.248463	Unity check
Horizontal shear + torsion in top flange		1.177299	1.119003	1.069103	1.028657	0.993652	0.961718	0.935089	0.912488	0.892335	Unity check
Horizontal shear in bottom flange		0.035926	0.041076	0.040674	0.035843	0.03246	0.031222	0.02907	0.026512	0.024574	Unity check
Horizontal shear + torsion in bottom flange		0.854071	0.865957	0.761332	0.598429	0.487661	0.42591	0.362019	0.302733	0.25871	Unity check
Fatigue prestressing steel		0.218703	0.166882	0.133056	0.107702	0.089546	0.071256	0.061159	0.051862	0.04505	Unity check
First natural bending frequency n0		3.053051	3.205052	3.459239	3.783739	4.085171	4.358157	4.631756	4.914137	5.204224	Hz
Ratio v/no		9.098365	8.666872	8.030026	7.341357	6.79966	6.373744	5.997246	5.652625	5.337545	m
Mass box girder m		6.96748	7.07408	7.28728	7.83328	8.39488	8.72768	9.31268	10.01728	10.57368	10^3 kg/m
Buckling webs		125.0441	131.5375	137.3142	144.0739	150.914	157.0092	163.6383	171.0248	177.6947	mm

D.2.2 Box girder with 8 tendons

Depth webs		1.5	1.65	1.8	1.95	2.1	2.25	2.4	2.55	2.7	m
Number of shear keys		10	11	12	13	14	15	16	17	18	
Thickness top flange		0.18	0.18	0.18	0.18	0.18	0.18	0.18	0.18	0.18	m
Number of tendons		8	8	8	8	8	8	8	8	8	
Depth box girder		1.97	2.06	2.16	2.27	2.44	2.62	2.81	3	3.2	m
Thickness bottom flange		0.29	0.23	0.18	0.14	0.16	0.19	0.23	0.27	0.32	m
Width web		0.11	0.12	0.13	0.13	0.14	0.15	0.15	0.16	0.17	m
Distance of deviation blocks to supports		11	14	18	22	22.5	22.5	22.5	22.5	22.5	m
Dead load box girder		79.14	74.70	71.44	68.35	72.46	77.74	82.97	89.50	97.20	kN/m
Ultimate resistance moment at t=0 bottom side		0.716334	0.789971	0.875696	0.978491	0.994855	0.999365	0.993497	0.996567	0.995821	Unity check
Ultimate resistance moment at t=∞ top side		1.042509	0.810971	0.570799	0.287966	0.19058	0.14499	0.116281	0.109413	0.11827	Unity check
Stresses at t=0	deviation block top side	-10.07816	-9.48488	-8.467671	-6.946122	-6.303426	-5.852191	-5.603146	-5.221655	-4.900405	N/mm^2
	deviation block bottom side	-32.40296	-38.28005	-45.46096	-55.25489	-49.87362	-43.70412	-38.34478	-33.72793	-29.4314	N/mm^2
	mid-span top side	-12.13241	-10.492	-8.726026	-6.949062	-6.303426	-5.852191	-5.603146	-5.221655	-4.900405	N/mm^2
	mid-span bottom side	-29.56955	-36.68215	-44.98929	-55.24872	-49.87362	-43.70412	-38.34478	-33.72793	-29.4314	N/mm^2
Stresses at t=∞ without variable load	deviation block top side	-11.75105	-11.58532	-11.00179	-9.744118	-8.886603	-8.220356	-7.750053	-7.229484	-6.784516	N/mm^2
	deviation block bottom side	-20.83483	-24.28174	-28.65048	-35.41361	-32.54215	-28.90722	-25.68874	-22.77655	-19.98986	N/mm^2
	mid-span top side	-14.69875	-13.05648	-11.38463	-9.748539	-8.886603	-8.220356	-7.750053	-7.229484	-6.784516	N/mm^2
	mid-span bottom side	-16.7691	-21.94759	-27.95155	-35.40432	-32.54215	-28.90722	-25.68874	-22.77655	-19.98986	N/mm^2
Stresses at t=∞fuly loaded	deviation block top side	-16.03014	-16.30614	-16.06145	-14.81971	-13.46775	-12.34857	-11.48389	-10.63015	-9.884472	N/mm^2
	deviation block bottom side	-14.93274	-16.79168	-19.41324	-24.75833	-23.77431	-21.84208	-19.98996	-18.09054	-16.14195	N/mm^2
	mid-span top side	-20.49096	-18.5632	-16.65511	-14.82663	-13.46775	-12.34857	-11.48389	-10.63015	-9.884472	N/mm^2
	mid-span bottom side	-8.779971	-13.21062	-18.32942	-24.74379	-23.77431	-21.84208	-19.98996	-18.09054	-16.14195	N/mm^2
Prestressing losses at the first support		5.668804	5.705834	5.718967	5.755473	5.692045	5.621334	5.559326	5.488208	5.412577	%
Prestressing losses at mid-span after the first deviation block		6.053562	5.975639	5.843614	5.732338	5.741547	5.757505	5.772572	5.765492	5.746728	%
Prestressing losses at mid-span after the second deviation block		6.63486	6.519037	6.339514	6.202508	6.230017	6.268511	6.302961	6.313837	6.309976	%
Prestressing losses at the second support		7.412699	7.336028	7.206666	7.165982	7.157457	7.154351	7.150493	7.133243	7.102321	%
Deflection at t=0		0.269689	0.387369	0.511526	0.648295	0.544069	0.440191	0.355017	0.289517	0.233572	Unity check
Deflection without variable load		-0.064042	-0.26301	-0.467383	-0.688722	-0.590783	-0.481147	-0.389017	-0.3158	-0.251469	Unity check
Additional deflection under mobile load		0.983757	0.972339	0.969571	0.974998	0.769344	0.600788	0.47205	0.379063	0.305326	Unity check
Final deflection fully loaded (t=∞)		0.263877	0.061103	-0.144193	-0.363723	-0.334335	-0.280885	-0.231667	-0.189445	-0.149694	Unity check
Vertical shear in webs	at t=0	0.6286	0.516942	0.42731	0.401278	0.36	0.325917	0.315377	0.286404	0.260492	Unity check
Vertical shear in webs	at t=∞	0.475012	0.376485	0.332151	0.308113	0.26696	0.235153	0.224118	0.204239	0.190341	Unity check
Vertical shear + torsion in webs	at t=∞	0.640053	0.512597	0.446094	0.411905	0.356763	0.313982	0.298652	0.27083	0.250447	Unity check
Horizontal shear in top flange		0.217258	0.21892	0.220766	0.222797	0.225936	0.22926	0.232768	0.236276	0.239969	Unity check
Horizontal shear + torsion in top flange		1.400894	1.319487	1.245192	1.177299	1.121202	1.073938	1.033572	0.999177	0.969856	Unity check
Horizontal shear in bottom flange		0.015052	0.019845	0.026589	0.035926	0.03379	0.030554	0.02707	0.024619	0.022157	Unity check
Horizontal shear + torsion in bottom flange		0.504832	0.594054	0.70954	0.854071	0.705239	0.564035	0.444881	0.363686	0.295865	Unity check
Fatigue prestressing steel		0.26153	0.185054	0.1285	0.098705	0.079768	0.065705	0.053934	0.045001	0.037388	Unity check
First natural bending frequency n0		2.824662	2.924377	2.994721	3.053051	3.338155	3.646978	3.982579	4.279095	4.575062	Hz
Ratio v/no		9.83402	9.498698	9.275583	9.098365	8.321297	7.616656	6.974821	6.491508	6.071563	m
Mass box girder m		8.06728	7.61488	7.28208	6.96748	7.38608	7.92428	8.45728	9.12288	9.90808	10^3 kg/m
Buckling webs		109.6349	114.6823	121.5154	127.4723	133.7999	141.267	148.4801	155.6115	162.4677	mm

D.2.3 Box girder with 4 tendons

Depth webs		3.3	3.45	3.6	3.75	3.9	4.05	4.2	4.35	4.5	m
Number of shear keys		22	23	24	25	26	27	28	29	30	
Thickness top flange		0.18	0.18	0.18	0.18	0.18	0.18	0.18	0.18	0.18	m
Number of tendons		4	4	4	4	4	4	4	4	4	
Depth box girder		3.72	3.89	4.07	4.24	4.42	4.62	4.8	4.98	5.17	m
Thickness bottom flange		0.24	0.26	0.29	0.31	0.34	0.39	0.42	0.45	0.49	m
Width web		0.18	0.19	0.2	0.2	0.21	0.22	0.22	0.23	0.24	m
Distance of deviation blocks to supports		14	14	14	14	14	13	13	13	13	m
Dead load box girder		95.92	101.10	107.45	111.02	117.60	126.38	131.12	138.08	146.22	kN/m
Ultimate resistance moment at t=0 bottom side		0.990987	0.997026	0.992394	0.998749	0.996191	0.99837	0.998272	0.999748	0.994324	Unity check
Ultimate resistance moment at t=∞ top side		1.033175	0.989082	0.962052	0.901965	0.888308	0.896784	0.86188	0.858377	0.864406	Unity check
Stresses at t=0	deviation block top side	-2.760893	-2.616303	-2.529327	-2.421406	-2.338263	-2.187906	-2.123962	-2.043253	-1.990156	N/mm^2
	deviation block bottom side	-15.52044	-14.38275	-13.0618	-12.46896	-11.45178	-10.33217	-9.788639	-9.144101	-8.443772	N/mm^2
	mid-span top side	-3.375952	-3.223081	-3.130631	-3.008664	-2.92113	-2.915618	-2.839234	-2.753226	-2.696935	N/mm^2
	mid-span bottom side	-14.64555	-13.56179	-12.29742	-11.7493	-10.77285	-9.542388	-9.041588	-8.42737	-7.758381	N/mm^2
Stresses at t=∞ without variable load	deviation block top side	-4.27337	-4.062922	-3.902888	-3.736303	-3.595597	-3.334775	-3.224043	-3.104859	-3.010579	N/mm^2
	deviation block bottom side	-9.479464	-8.840968	-8.060312	-7.770347	-7.152136	-6.546147	-6.238288	-5.831029	-5.379487	N/mm^2
	mid-span top side	-5.109129	-4.87628	-4.696809	-4.505626	-4.349058	-4.260687	-4.127077	-3.991805	-3.883731	N/mm^2
	mid-span bottom side	-8.29063	-7.740501	-7.051079	-6.82757	-6.274497	-5.541265	-5.295133	-4.93564	-4.532757	N/mm^2
Stresses at t=∞fuly loaded	deviation block top side	-6.518609	-6.164498	-5.862412	-5.588498	-5.331079	-4.882004	-4.689797	-4.486389	-4.309364	N/mm^2
	deviation block bottom side	-6.2857	-5.99755	-5.569364	-5.500552	-5.130628	-4.866956	-4.707411	-4.436349	-4.120004	N/mm^2
	mid-span top side	-7.728144	-7.327714	-6.982543	-6.666165	-6.373453	-6.143583	-5.910822	-5.673054	-5.464283	N/mm^2
	mid-span bottom side	-4.565184	-4.423726	-4.145451	-4.179912	-3.916458	-3.497779	-3.432136	-3.238388	-3.000032	N/mm^2
Prestressing losses at the first support		5.50373	5.511169	5.507932	5.530967	5.533493	5.620395	5.639998	5.655553	5.665447	%
Prestressing losses at mid-span after the first deviation block		6.547762	6.603071	6.642069	6.711117	6.753656	6.972592	7.033282	7.091086	7.139281	%
Prestressing losses at mid-span after the second deviation block		7.73799	7.829742	7.897359	8.005792	8.078448	8.4167	8.512751	8.607316	8.687675	%
Prestressing losses at the second support		9.074412	9.19118	9.273801	9.414991	9.507868	9.952718	10.0784	10.20424	10.31063	%
Deflection at t=0		0.092304	0.080979	0.068624	0.06281	0.054125	0.043703	0.03937	0.034716	0.029829	Unity check
Deflection without variable load		-0.052116	-0.044868	-0.035249	-0.033371	-0.026546	-0.016891	-0.014829	-0.011549	-0.00765	Unity check
Additional deflection under mobile load		0.239836	0.208523	0.17937	0.15947	0.13943	0.119512	0.106838	0.095402	0.084682	Unity check
Final deflection fully loaded (t=∞)		0.027829	0.024639	0.024541	0.019786	0.019931	0.022947	0.020784	0.020251	0.020577	Unity check
Vertical shear in webs	at t=0	0.174642	0.162047	0.149475	0.147746	0.137172	0.133973	0.131775	0.123683	0.115699	Unity check
Vertical shear in webs	at t=∞	0.137141	0.126864	0.11989	0.1154	0.109669	0.100002	0.098017	0.093692	0.090881	Unity check
Vertical shear + torsion in webs	at t=∞	0.183255	0.169141	0.158948	0.153242	0.144842	0.132982	0.130214	0.123864	0.119304	Unity check
Horizontal shear in top flange		0.249571	0.252709	0.256033	0.259172	0.262496	0.266189	0.269512	0.272836	0.276344	Unity check
Horizontal shear + torsion in top flange		0.861671	0.844641	0.830449	0.816433	0.805073	0.79673	0.787447	0.779022	0.772231	Unity check
Horizontal shear in bottom flange		0.034344	0.033151	0.031097	0.030305	0.028804	0.026248	0.025323	0.024521	0.023378	Unity check
Horizontal shear + torsion in bottom flange		0.340394	0.30635	0.268786	0.246019	0.220302	0.189491	0.173304	0.159504	0.14482	Unity check
Fatigue prestressing steel		0.100672	0.090181	0.079375	0.072743	0.065067	0.065383	0.059848	0.054741	0.049599	Unity check
First natural bending frequency n0		5.196248	5.428181	5.677106	5.923294	6.154903	6.413093	6.658994	6.866847	7.082864	Hz
Ratio v/no		5.345737	5.117327	4.892947	4.689583	4.513114	4.331417	4.171468	4.045201	3.921829	m
Mass box girder m		9.77808	10.30588	10.95328	11.31728	11.98808	12.88248	13.36608	14.07588	14.90528	10^3 kg/m
Buckling webs		178.9074	185.1942	192.3364	197.7381	204.9531	210.4468	216.8954	223.2039	231.1623	mm

Appendix E: Calculations FRP

E.1 Introduction

This Appendix presents the calculations of the Fibre Reinforced Polymer sandwich girder. First the material characteristics of the FRP are described. Paragraph 3 deals with the geometry and the structural schematisation of the sandwich girder and its characteristics. The loads to which the sandwich girder is subjected and partial factors are treated in paragraph 4. Furthermore this Appendix describes the calculations on deflection, vibration, stresses and shear of the sandwich girder in respectively the paragraphs 5, 6, 7 and 8. Finally this Appendix deals with the calculations on buckling of the core.

The formulas and values used in the calculations are taken from [17] and other references, which are then stated in the text.

E.2 Material characteristics

E.2.1 Carbon fibres, polyacrylonitrile, fibre type: graphite, table 3.3 [7]

Density carbon fibres	ρ_c	1870 kg/m^3
Modulus of elasticity	E_c	345000 N/mm^2
Shear modulus, Appendix A [2]	G_c	5000 N/mm^2
Characteristic tensile strength	f_{ctk}	2600 N/mm^2
Maximum strain	$\varepsilon_{c\max}$	0.74 %
Volume fraction	V_c	0.55

E.2.2 Epoxy, Appendix 8 [17]

Density epoxy	ρ_e	1200 kg/m^3
Modulus of elasticity	E_e	3500 N/mm^2
Shear modulus	G_e	1400 N/mm^2
Characteristic tensile strength	f_{etk}	45 N/mm^2
Maximum strain	$\varepsilon_{e\max}$	4 %
Volume fraction	V_e	0.45

E.3 Geometry sandwich girder

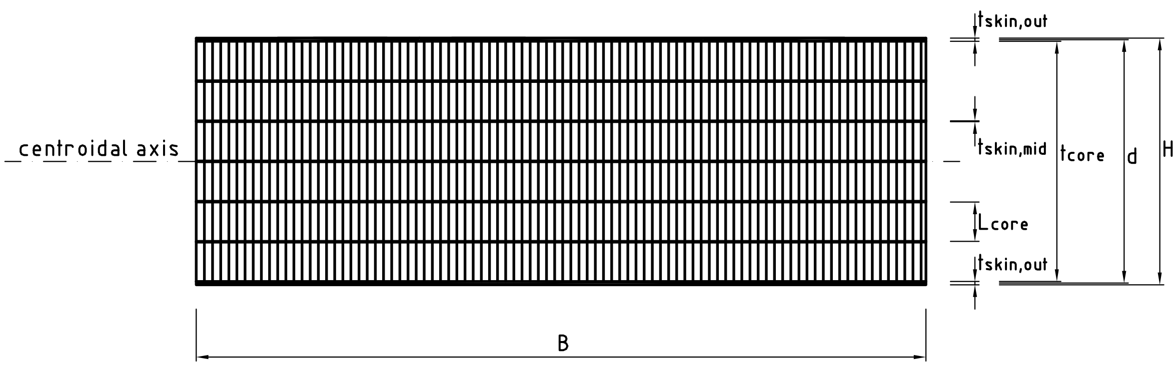

Figure 179: Cross-section of the sandwich girder

E.3.1 General

Length span	L	45	m
Width sandwich girder	B	9	m
Spacing outer skins	d	3	m
Thickness outer skins	$t_{skin,out}$	0.04	m
Thickness middle skins	$t_{skin,mid}$	0.01	m
Core depth	$t_{core} = d - 2*\frac{1}{2}*t_{skin,out}$	2.96	m
Number of core parts	nr	6	
Buckling length core parts	$L_{core} = \frac{t_{core} - (nr-1)*t_{skin,mid}}{nr}$	0.485	m
Sandwich depth	$H = d + 2*\frac{1}{2}*t_{skin,out}$	3.04	m

E.3.2 Skin

For the moment of inertia of the sandwich girder in z-direction (vertical) only the skins are taken into account. As the moment of inertia of the skins self, is small the calculation of the moment of inertia of the girder is only based on the Huygens-Steiner theorem.

Moment of inertia of the sandwich girder in z-direction:

$$I_z = 2*B*t_{skin,out}*\left(\frac{d}{2}\right)^2 + 2*B*t_{skin,nid}*(L_{core}+t_{skin,mid})^2 +$$
$$2*B*t_{skin,mid}*(2*L_{core}+2*t_{skin,mid})^2 = 1.841m^4$$

Fibre layout in the skins:

Percentage of fibres in x-direction (0°)	$v_{s0} = 55\%$
Percentage of fibres in y-direction (90°)	$v_{s90} = 15\%$
Percentage of fibres in xy-direction (45°)	$v_{s45} = 15\%$
Percentage of fibres in xy-direction (-45°)	$v_{s-45} = 15\%$

Effective modulus of elasticity in x-direction:

$$E_x = V_c*\frac{v_{s0}}{100}*E_c + (1-V_c*\frac{v_{s0}}{100})*E_e = 106803.75N/mm^2$$

Bending stiffness of the sandwich girder in x-direction:

$$E_x I_z = 1.966*10^{17}\,Nmm^2$$

E.3.3 Core

Fibre layout in the core:

Percentage of fibres in x-direction (0°)	$v_{c0} = 15\%$
Percentage of fibres in z-direction (90°)	$v_{c90} = 15\%$
Percentage of fibres in xz-direction (45°)	$v_{c45} = 35\%$
Percentage of fibres in xz-direction (-45°)	$v_{c-45} = 35\%$

Effective modulus of elasticity in x-direction:

$$E_{x,core} = V_c * \frac{v_{c0}}{100} * E_c + (1 - V_c * \frac{v_{c0}}{100}) * E_e = 31673.75 N/mm^2$$

Effective modulus of elasticity in z-direction:

$$E_{z,core} = V_c * \frac{v_{c90}}{100} * E_c + (1 - V_c * \frac{v_{c90}}{100}) * E_e = 31673.75 N/mm^2$$

Effective shear modulus in xz-direction:

$$\frac{1}{G_{xz}} = \frac{V_c * \frac{v_{c45}}{100}}{G_c} + \frac{1 - V_c * \frac{v_{c45}}{100}}{G_e} \rightarrow G_{xz} = 1625.26 N/mm^2$$

Core triangles

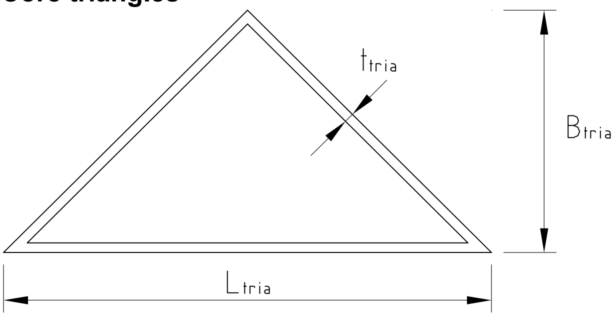

Figure 180: Cross-section of a core triangle

Length core triangle	$L_{tria} = 200mm$
Width core triangle	$B_{tria} = 100mm$
Thickness core triangle	$t_{tria} = 4mm$

Number of triangles in the width of the sandwich girder	$n_{tw} = \frac{B}{B_{tria}} = 90$
Number of triangles per metre length	$n_{tpm} = n_{tw} * \frac{1000}{L_{tria}} = 450$
Total number of triangles in the sandwich girder	$n_{tr} = n_{tw} * \frac{L}{L_{tria}} = 20250$

The cross-sectional area of one triangle in vertical direction:

$$A_{1,tria,v} = \frac{1}{2} * B_{tria}^{\ 2} - \frac{1}{2} * (B_{tria} - t_{tria})^2 = 768 mm^2$$

The cross-sectional area of one triangle in horizontal direction:

$$A_{1,tria,h} = t_{tria} + t_{tria} * \cos\left(\tan^{-1}\left(\frac{B_{tria}}{L_{tria}/2}\right)\right) * nr * L_{core} = 8234.72 mm^2$$

Moment of inertia of one triangle:

$$I_{tria} = \frac{1}{36} * L_{tria} * B_{tria}^{\ 3} - \frac{1}{36} * (L_{tria} - 2 * t_{tria}) * (B_{tria} - 2 * t_{tria})^3 = 1402553 mm^4$$

The shear stiffness of the sandwich girder in x-direction:

$$GA_x = G_{xz} * n_{tw} * A_{1,tria,h} = 1204521813N$$

E.4 Loads and partial factors

E.4.1 General

The sandwich girder has to satisfy [3]:

$$S * \gamma_f \leq R / (\gamma_m * \gamma_c)$$

Where:

S	Is the effect of the representative load
R	Is the representative load carrying capacity and/or strength of the structure
γ_f	Is a load factor
γ_m	Is a material factor
γ_c	Is a conversion factor

The load factors [9]:

Partial factor for permanent actions, unfavourable	$\gamma_{G,unfav} = 1.35$
Partial factor for variable actions, unfavourable	$\gamma_{Q,unfav} = 1.5$

The material factor [3]:

$$\gamma_m = \gamma_{m1} * \gamma_{m2} = 1.62$$

Where:

$\gamma_{m1} = 1.35$	Partial material factor due to uncertainties in obtaining the correct material properties
$\gamma_{m2} = 1.2$	Partial material factor due to uncertainties in the material properties dependent on the production method (vacuum injection table 1 [3])

The conversion factors [3]:

$$\gamma_c = \gamma_{ct} * \gamma_{cm} * \gamma_{cc} * \gamma_{cf}$$

Where:

$\gamma_{ct} = 1.1$	Conversion factor for temperature effects
$\gamma_{cm} = 1.1$	Conversion factor for moisture effects (FRP structure subjected to changing humidity circumstances)
$\gamma_{cc} = t^n = 1.73$	Conversion factor for creep effects
$t = 100 * 365 * 24 = 876000\ hours$	Duration of the loading in hours (design life of 100 years)
$n = 0.04$	Exponent depending on fibre type: woven fabric
$\gamma_{cf} = 1.1$	Conversion factor for fatigue effects

In Table 25 the conversion factors per situation are given.

	Ultimate limit state			Serviceability limit state		
	Strength $\gamma_{c,str}$	Stability $\gamma_{c,sta}$	Fatigue $\gamma_{c,fat}$	Deflection $\gamma_{c,def}$	Vibration $\gamma_{c,vib}$	First cracking $\gamma_{c,fc}$
Conversion factor temperature	1.1	1.1	1.1	1.1	1.1[14]	1.1
Conversion factor moisture	1.1	1.1	1.1	1.1	1.1[14]	1.1
Conversion factor creep [15]	1.73	1.73	-	1.73	-	1.73
Conversion factor fatigue [16]	-	1.1	-	1.1	1.1[14]	1.1
Short term loading	1.21	1.33	1.21	1.33	1.33	1.33
Long term loading	2.09	2.3	1.21	2.3	1.33	2.3

Table 25: Conversion factors

E.4.2 Vertical loads

Dead load skins:

$$g_{dead,skin} = 2*(t_{skin,out} * B * V_c * \rho_c * g + t_{skin,out} * B * V_e * \rho_e * g) +$$
$$(nr-1)*(t_{skin,mid} * B * V_c * \rho_c * g + t_{skin,mid} * B * V_e * \rho_e * g) = 18.0kN/m$$

Dead load core triangles:

$$g_{dead,tria} = A_{1,tria,v} * n_{tpm} * nr * L_{core} * (V_c * \rho_c * g + V_e * \rho_e * g) = 15.48kN/m$$

Dead load foam in core:

$g_{dead,foam} = 1.0kN/m$ Assumption

Total dead load sandwich structure:

$$g_{dead} = g_{dead,skin} + g_{dead,tria} + g_{dead,foam} = 34.48kN/m$$

Total permanent load:

$g_{perm} = 34.42kN/m$ See B.4.4

Total variable load:

$q_{var} = 58.29kN/m$ See B.4.4

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[14] The serviceability limit state vibration should be checked with and without conversion factors.
[15] The conversion factor for creep should only be taken into account for long term loading.
[16] The conversion factor for fatigue should only be taken into account for stiffness related limit states.

E.4.3 Loads in the serviceability limit state

Uniform distributed load:

$$q_{sls} = g_{dead} + g_{perm} + q_{\text{var}} = 127.19 kN/m$$

Maximum shear force:

$$V_{Ed.sls} = 0.5 * L * q_{sls} = 2862 kN$$

Maximum bending moment:

$$M_{Ed,sls} = \frac{1}{8} * q_{sls} * L^2 = 32195 kNm$$

E.4.4 Loads in the ultimate limit state

Uniform distributed load:

$$q_{uls} = \gamma_{G,unfav} * g_{dead} + \gamma_{G,unfav} * g_{perm} + \gamma_{Q,unfav} * q_{\text{var}} = 180.45 kN/m$$

Maximum shear force:

$$V_{Ed.uls} = 0.5 * L * q_{uls} = 4060 kN$$

Maximum bending moment:

$$M_{Ed,uls} = \frac{1}{8} * q_{uls} * L^2 = 45677 kNm$$

E.5 Deflection

The deflection is determined with the formula [3]:

$$w = \frac{5}{384} \frac{qL^4}{E_x I_z / (\gamma_m * \gamma_c)} + \frac{\eta * q * L}{8 * GA_x / (\gamma_m * \gamma_c)}$$

Where:

$L = 45m$	Length span
$E_x I_z = 1.966 * 10^{17} Nmm^2$	Bending stiffness of the sandwich structure in x-direction
$GA_x = 1204521813N$	Shear stiffness of the sandwich structure in x-direction:
$\gamma_m = 1.62$	Material factor
$\gamma_c = \gamma_{cm} = 1.1$	Conversion factor for time independent deflection
$\gamma_c = \gamma_{cm} + \gamma_{cc} = 1.9$	Conversion factor for time dependent deflection
$\eta = 1.2$	Correction factor for the form of the cross-section of the girder: rectangular cross-section [3]

The deflections at mid-span and unity checks for different phases are:

Time	*Load q*	*Deflection w*	*value*	*Maximum allowed deflection* w_{max}	*Unity check* w/w_{max}
At t=∞ without variable load	$g_{dead} + g_{perm}$	58.8	mm	$L/500 = 90mm$ annotation [17]	0.65
Additional deflection under mobile load	q_{var}	28.8	mm	$L/1500 = 30mm$ annotation [18]	0.96
At t=∞ fully loaded	$g_{dead} + g_{perm} + q_{var}$	58.8+28.8= 87.6	mm	$L/500 = 90mm$ annotation [17]	0.97

Table 26: The deflections at mid-span and unity checks for different phases

As the unity checks show the construction satisfies with respect to deflection for all phases. The normative deflections are the additional deflection under mobile load and the deflection at t=∞ fully loaded. The deflection is mostly determined by the deflection due to bending and not by shearing. To decrease the deflection the bending stiffness should thus be increased (enlarge the moment of inertia). Notice that deflection is indeed a very important verification for FRP bridges.

E.6 Vibration

For the sandwich girder only the static analysis is considered. The dynamic metro load is multiplied by the dynamic factor ϕ to take into account the dynamic loading. This method of calculation holds when the first natural frequency of the box girder stays within the prescribed limits [10]. When the limits are exceeded a dynamic analysis is required. A dynamic analysis can prove that the box girder is still determined against the dynamic effects. Such an analysis is however extensive and more difficult and is therefore left out of the design of the box girder. For this design the first natural frequency of the box girder should stay within the limits such that a static analysis is sufficient and a dynamic analysis is not necessary. The check for determining whether a dynamic analysis is required is done according two verifications which are elaborated below.

Verification according Annex F [10]

The first natural bending frequency of the sandwich girder is [20]:

$n_0 = \frac{C_{end}}{2\pi}\sqrt{\frac{E_x I_z}{g_{dead} * L^4 / g}} = 5.84Hz$	Without partial factors
$n_0 = \frac{C_{end}}{2\pi}\sqrt{\frac{E_x I_z / (\gamma_m * \gamma_c)}{g_{dead} * L^4 / g}} = 3.98Hz$	With partial factors

Where:

$C_{end} = 9.94$	Boundary condition coefficient [20]
$E_x I_z = 1.966 * 10^{17} Nmm^2$	Bending stiffness of the sandwich girder in x-direction
$\gamma_m = 1.62$	Material factor
$\gamma_c = \gamma_{c,vib} = 1.33$	Conversion factor for vibration, see Table 25

The velocity of the metros is:

$$v = 100km/h = 27.78m/s$$

[17] The final deflection may not exceed $L/500$ see 7.4 [11].
[18] The maximum deflection under mobile load is $L/1500$ [8].

Mass m 10^3 kg/m		≥5,0 <7,0	≥7,0 <9,0	≥9,0 <10,0	≥10,0 <13,0	≥13,0 <15,0	≥15,0 <18,0	≥18,0 <20,0	≥20,0 <25,0	≥25,0 <30,0	≥30,0 <40,0	≥40,0 <50,0	≥50,0 -
Span $L \in$ m [a]	ζ %	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m	v/n_0 m
[5,00,7,50)	2	1,71	1,78	1,88	1,88	1,93	1,93	2,13	2,13	3,08	3,08	3,54	3,59
	4	1,71	1,83	1,93	1,93	2,13	2,24	3,03	3,08	3,38	3,54	4,31	4,31
[7,50,10,0)	2	1,94	2,08	2,64	2,64	2,77	2,77	3,06	5,00	5,14	5,20	5,35	5,42
	4	2,15	2,64	2,77	2,98	4,93	5,00	5,14	5,21	5,35	5,62	6,39	6,53
[10,0,12,5)	1	2,40	2,50	2,50	2,50	2,71	6,15	6,25	6,36	6,36	6,45	6,45	6,57
	2	2,50	2,71	2,71	5,83	6,15	6,25	6,36	6,36	6,45	6,45	7,19	7,29
[12,5,15,0)	1	2,50	2,50	3,58	3,58	5,24	5,24	5,36	5,36	7,86	9,14	9,14	9,14
	2	3,45	5,12	5,24	5,24	5,36	5,36	7,86	8,22	9,53	9,76	10,36	10,48
[15,0,17,5)	1	3,00	5,33	5,33	5,33	6,33	6,33	6,50	6,50	6,50	7,80	7,80	7,80
	2	5,33	5,33	6,33	6,33	6,50	6,50	10,17	10,33	10,33	10,50	10,67	12,40
[17,5,20,0)	1	3,50	6,33	6,33	6,33	6,50	6,50	7,17	7,17	10,67	12,80	12,80	12,80
[20,0,25,0)	1	5,21	5,21	5,42	7,08	7,50	7,50	13,54	13,54	13,96	14,17	14,38	14,38
[25,0,30,0)	1	6,25	6,46	6,46	10,21	10,21	10,21	10,63	10,63	12,75	12,75	12,75	12,75
[30,0,40,0)	1				10,56	18,33	18,33	18,61	18,61	18,89	19,17	19,17	19,17
≥40,0	1				14,73	15,00	15,56	15,56	15,83	18,33	18,33	18,33	18,33

[a] $L \in [a,b)$ means $a \leq L < b$

NOTE 1 Table F.1 includes a safety factor of 1.2 on $(v/n_0)_{lim}$ for acceleration, deflection and strength criteria and a safety factor of 1,0 on the $(v/n_0)_{lim}$ for fatigue.

NOTE 2 Table F.1 includes an allowance of $(1+\varphi''/2)$ for track irregularities.

Table 27: Maximum value of $(v/n_0)_{\lim}$ for a simply supported beam or slab and a maximum permitted acceleration of $\alpha_{\max} < 3.5 m/s^2$, Table F.1 [10]

Table 27 gives no maximum value of the velocity divided by the first natural frequency for:

$$m = g_{dead} / g = 3.515 * 10^3 kg/m$$

$$L = 45m$$

Therefore there is made an extrapolation of the table to determine the maximum value of the velocity divided by the first natural frequency. This is a rough extrapolation as the difference of the values is large between masses above and below $m = 10 * 10^3 kg/m$, see Table 27.

The extrapolated maximum value of the velocity divided by the first natural frequency is:

$$(v/n_0)_{\lim} = 10.0m$$

The verification of the ratio of the velocity over the first natural frequency is:

$v/n_0 = 4.75m \leq 10.0m \rightarrow Ok$	n_0 without partial factors
$v/n_0 = 6.98m \leq 10.0m \rightarrow Ok$	n_0 with partial factors

Verification according to Figure 6.10 [10]

Limits of natural frequency n_0 (Hz) as a function of L (m)

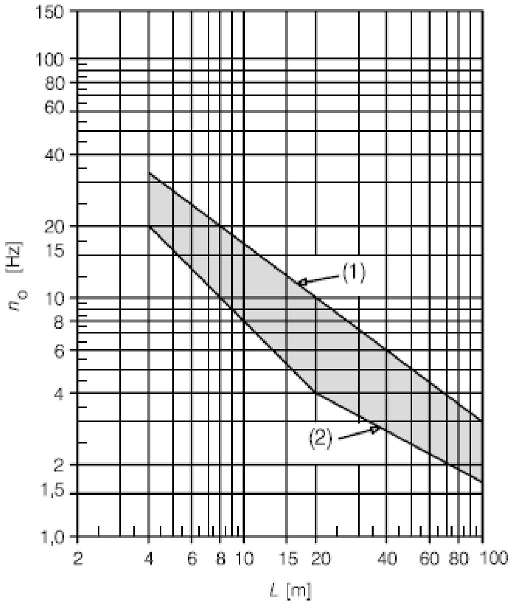


Figure 181: Limits of bridge natural frequency n_0 (Hz) as a function of L (m)

According this verification the first natural frequency of the sandwich girder should be in the grey area, see Figure 181.

Where:
The upper limit of natural frequency is governed by dynamic enhancements due to track irregularities and is given by:

$$n_{0\max} = 94.76 * L^{-0.748} = 5.5Hz$$

The lower limit of natural frequency is governed by dynamic impact criteria and is given by:

$$n_{0\min} = 23.58 * L^{-0.592} = 2.48Hz$$

The first natural frequency of the sandwich girder is:

$n_0 = \frac{C_{end}}{2\pi}\sqrt{\frac{E_x I_z}{g_{dead} * L^4 / g}} = 5.84Hz \rightarrow Not\ ok$	Without partial factors
$n_0 = \frac{C_{end}}{2\pi}\sqrt{\frac{E_x I_z /(\gamma_m * \gamma_c)}{g_{dead} * L^4 / g}} = 3.98Hz \rightarrow Ok$	With partial factors

Conclusion

As the FRP sandwich girder is very light Table 27 does not give a solution for the maximum value of the velocity divided by the first natural frequency. Therefore there is made an extrapolation of this maximum. This extrapolation is however quite rough and the question rises if this extrapolation is valid. For this reason only the second verification is taken into account. This verification shows that the sandwich girder requires a dynamic analysis as the first natural frequency of the structure without partial factors is too high. This means that the frequency approaches the frequency due to track irregularities which causes enhancement of the dynamic loads. This way the vertical forces due to impacts on the rail become larger than just the vertical load. The dynamic factor which is taken into account so far is not sufficient anymore when the upper limit of 5.5 Hz is passed. The structure thus requires a dynamic analysis. It is however expected that the maximum frequency of the structure of 5.84 Hz, which is not much more than the limit, is not very problematic as in reality the amplitude of the acceleration of the metros is small. Besides the damping of the FRP sandwich girder (foam) is not taken into account. It is therefore expected that executing a dynamic analysis will not result in a different design. It is however recommended to make a dynamic analysis to be certain of this assumption. A dynamic analysis is not treated in this design as this is too specific and goes far beyond the purpose to design a global FRP railway girder. For a further elaboration of a FRP metro viaduct it is thus recommended to make a dynamic analysis to check whether the structure is determined against the dynamic effects.

E.7 Stresses

The maximum compressive and tensile stress in the sandwich girder is:

$$\sigma_{x,skin} = \frac{M_{Ed,uls} * \frac{1}{2} H}{I_z} = 37.72 N / mm^2$$

E.7.1 Skin

Tension

The ultimate tensile strength of the skin is:

$$f_{t,skin} = \frac{E_{x,skin}}{\gamma_m * \gamma_c} * \varepsilon_{c\max} = 233.25 N / mm^2$$

Where:

$\gamma_c = \gamma_{c,str,long} = 2.09$ Conversion factor for strength, see Table 25

Unity check tensile stress in the skin:

$$\frac{\sigma_{x,skin}}{f_{t,skin}} = 0.16 \leq 1.0 \rightarrow Ok$$

Compression

There are two failure modes in compression which apply to the skin, see Figure 182.

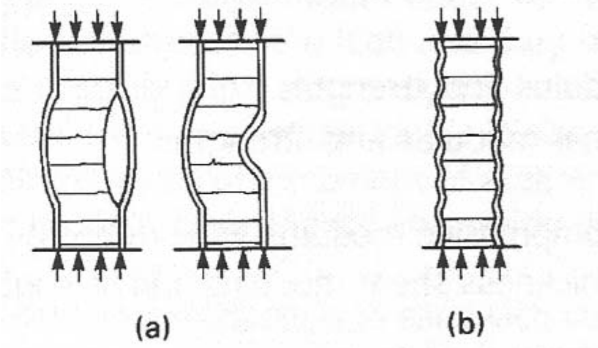


Figure 182: Sandwich material failure modes in compression: (a) skin wrinkling; (b) skin dimpling

Skin wrinkling [7]:

$$f_{c,wrinkling} = \frac{1}{2} \left(\frac{E_{x,skin}}{\gamma_m * \gamma_c} * \frac{E_{x,core}}{\gamma_m * \gamma_c} * \frac{G_{xz}}{\gamma_m * \gamma_c} \right)^{1/3} = 2367.63 N / mm^2$$

Skin dimpling [7]:

$$f_{c,\dim pling} = 0.75 * \frac{E_{x,skin}}{\gamma_m * \gamma_c} * (t_{skin,out} / L_{tria})^{3/2} = 1922.2 N / mm^2$$

Where:

$\gamma_c = \gamma_{c,sta,long} = 2.3$ Conversion factor for stability, see Table 25

The compressive strength of the skin equals the smallest strength. Dimpling of the skin is thus the normative compressive failure mode.

Unity check compressive stress in the skin:

$$\frac{\sigma_{x,skin}}{f_{c,\dim pling}} = 0.02 \leq 1.0 \rightarrow Ok$$

E.7.2 Core

The ultimate tensile strength of the core is:

$$f_{t,core} = \frac{E_{x,core}}{\gamma_m * \gamma_c} * \varepsilon_{c\max} = 69.17 N/mm^2$$

Where:

$\gamma_c = \gamma_{c,str,long} = 2.09$ Conversion factor for strength, see Table 25

Unity check tensile stress in the core:

$$\frac{\sigma_{x,skin}}{f_{t,core}} = 0.55 \leq 1.0 \rightarrow Ok$$

E.7.3 Flexural strength

The flexural strength of the sandwich girder is [7]:

$$M_{Rd} = B * t_{skin,out} * t_{core} * \frac{E_x}{\gamma_m * \gamma_c} * f_{t,core} / \frac{E_{x,core}}{\gamma_m * \gamma_c} = 248548.92 kNm$$

Where:

$\gamma_c = \gamma_{c,str,long} = 2.09$ Conversion factor for strength, see Table 25

Unity check flexural strength:

$$\frac{M_{Ed,uls}}{M_{Rd}} = 0.18 \leq 1.0 \rightarrow Ok$$

E.8 Shear

It is expected that all the shear stresses are carried in the core. The shear strength of this material with its volume fraction of fibres and fibre orientation is not known and should be determined by experiments. For this design it is however assumed that a quite conservative shear strength of:
$\tau = 50 N/mm^2$ will do, considering the shear strengths given in [i7].

The design shear strength then becomes:

$$\tau_{Rd} = \frac{50}{\gamma_m * \gamma_c} = 14.76 N/mm^2$$

Where:

$\gamma_c = \gamma_{c,str,long} = 2.09$ Conversion factor for strength, see Table 25

E.8.1 Transverse shear

The transverse shear force is:

$$\tau_{Ed} = \frac{V_{Ed,uls}}{A_{1,tria,h} * n_{tw}} = 5.48N/mm^2$$

Unity check transverse shear:

$$\frac{\tau_{Ed}}{\tau_{Rd}} = 0.37 \leq 1.0 \rightarrow Ok$$

E.8.2 Parallel shear

The parallel shear force is:

$$\tau_{Ed} = \frac{V_{Ed,uls} * S}{I_z * \frac{A_{1,tria,h}}{t_{core}} * n_{tw}} = 10.77N/mm^2$$

Where:

$$S = t_{skin,out} * B * \frac{d}{2} + t_{skin,mid} * B * (2L_{core} + 2t_{skin,mid}) + t_{skin,mid} * B * (L_{core} + t_{skin,mid}) +$$

$$A_{1,tria,h} * n_{tw} * \frac{t_{core}}{4} = 1.222m^3$$

Unity check parallel shear:

$$\frac{\tau_{Ed}}{\tau_{Rd}} = 0.73 \leq 1.0 \rightarrow Ok$$

E.9 Buckling of the core

The critical buckling force of the core is:

$$F_{cr} = \frac{\pi^2 * \frac{E_{z,core}}{\gamma_m * \gamma_c} * I_{tria} * n_{tw}}{L_{buc,core}{}^2} = 45007.44kN$$

Where:

$L_{buc,core} = L_{core} = 0.485m$

$\gamma_c = \gamma_{c,sta,long} = 2.3$ Conversion factor for stability, see Table 25

The maximum buckling force is:

$V_{Ed.uls} = 0.5 * L * q_{uls} = 4060.15kN$

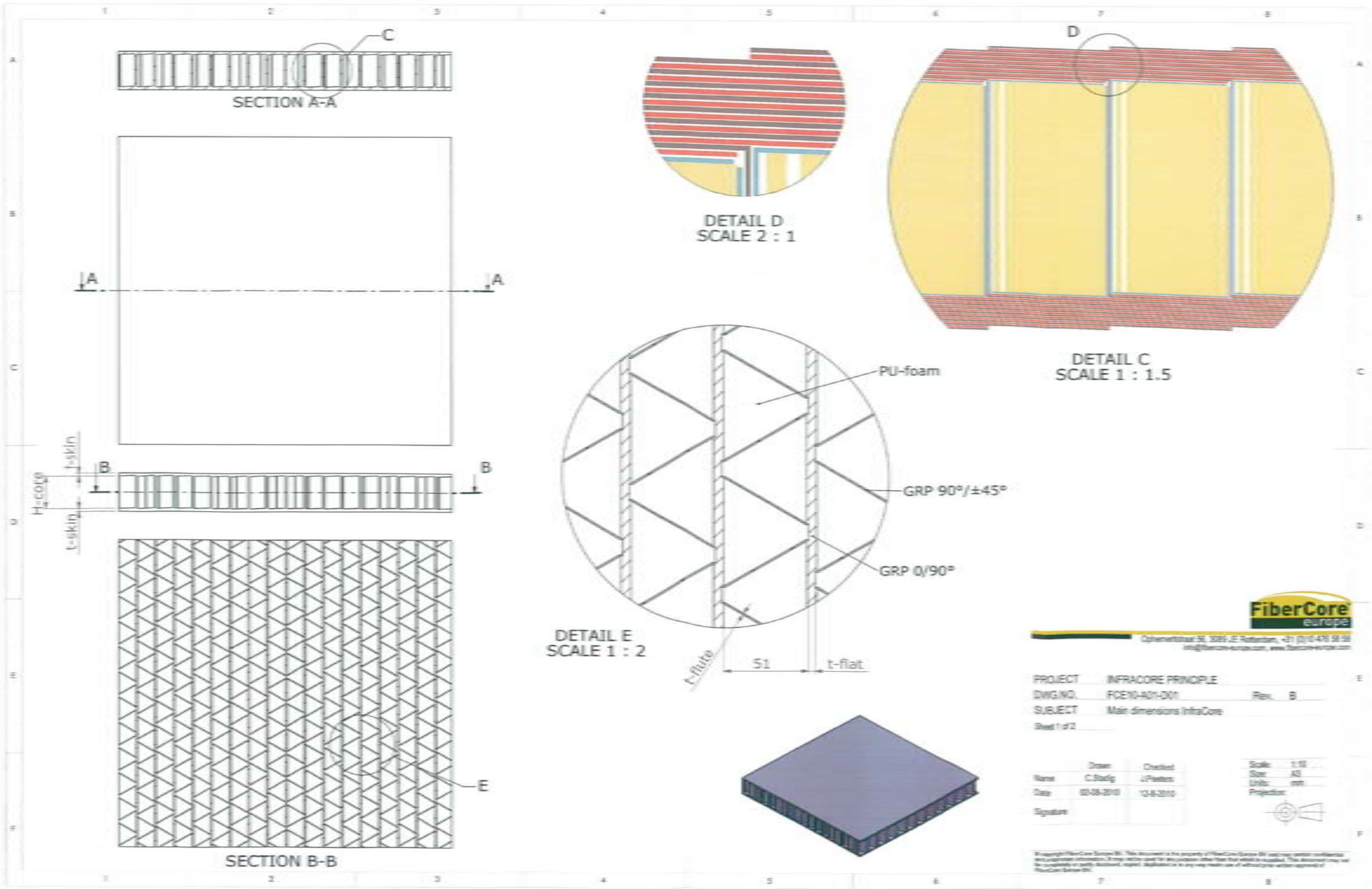
Unity check buckling of the core:

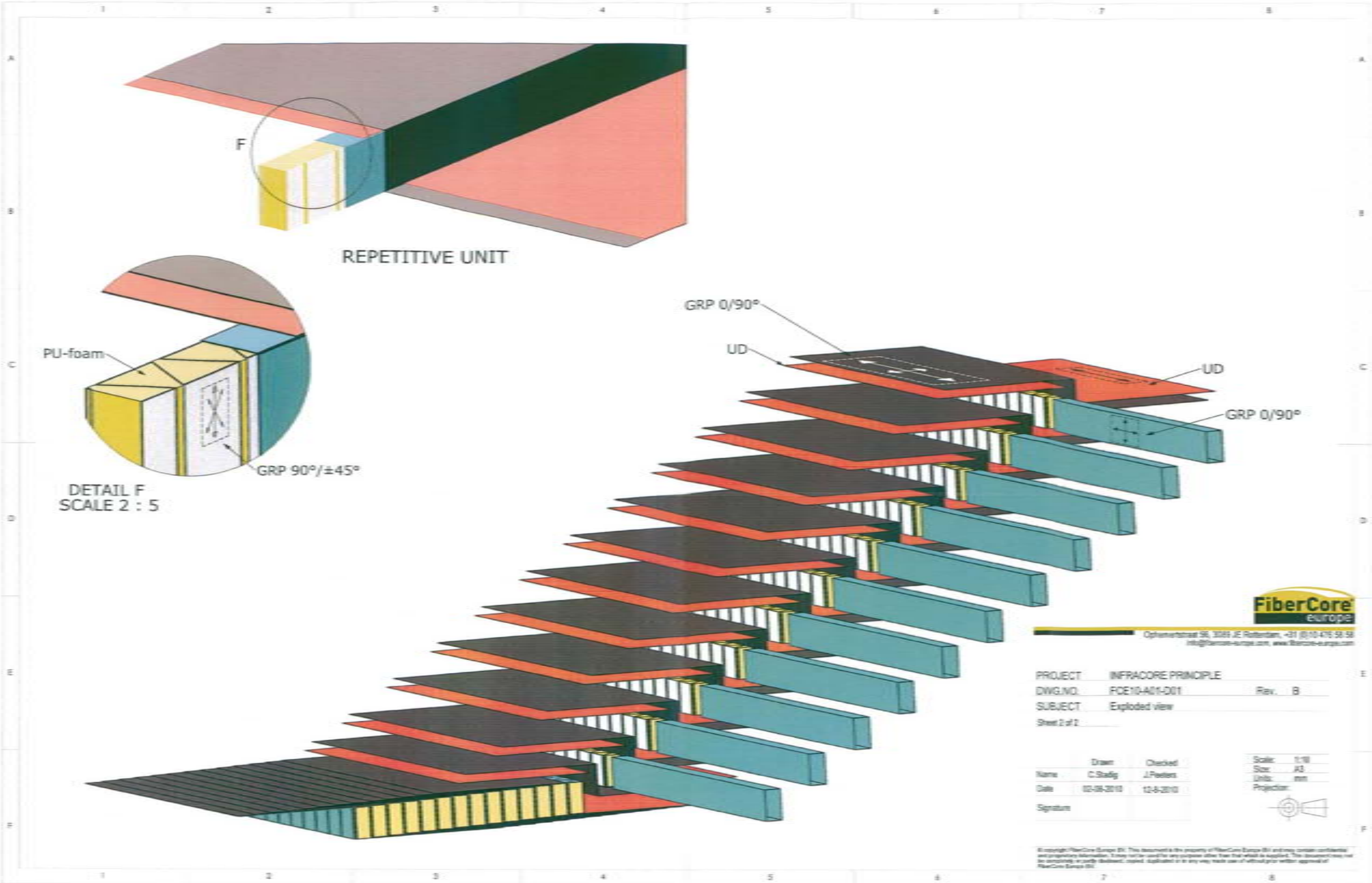
$$\frac{V_{Ed,uls} * \alpha_{cr}}{F_{cr}} = 0.90 \leq 1.0 \rightarrow Ok$$

Where:

$\alpha_{cr} = 10$ Factor by which the design loading would have to be increased to cause elastic instability in a global mode

Appendix F: Infracore principle




F
REPETITIVE UNIT
PU-foam
GRP 90°/±45°
DETAIL F
SCALE 2 : 5
GRP 0/90°
UD
UD
GRP 0/90°
FiberCore
europe
PROJECT
INFRACORE PRINCIPLE
DWG.NO.
FCE10-A01-D01
Rev. B
SUBJECT
Exploded view
Drawn
Checked
Name
Date
Signature
Scale:
Size:
A3
Units:
mm
Projection:

Appendix G: Calculations column + foundation

G.1 Column + foundation in combination with a concrete box girder

G.1.1 General

This Appendix presents the calculations of the column + foundation of the elevated metro structure with a concrete box girder. First the cross-sectional properties of the column are described. Paragraph 3 deals with the loads to which the column is subjected. The geometry and characteristics of the foundation are treated in paragraph 4. Furthermore this Appendix describes the calculations on stability and stiffness of the structure in respectively the paragraphs 5 and 6. The forces in the piles and stresses in the column are treated in paragraph 7 and 8. Finally this Appendix gives an overview of the weight contribution of the different elements of the structure.

The formulas and values used in the calculations are taken from [11] and other references, which are then stated in the text.

G.1.2 Column C50/60

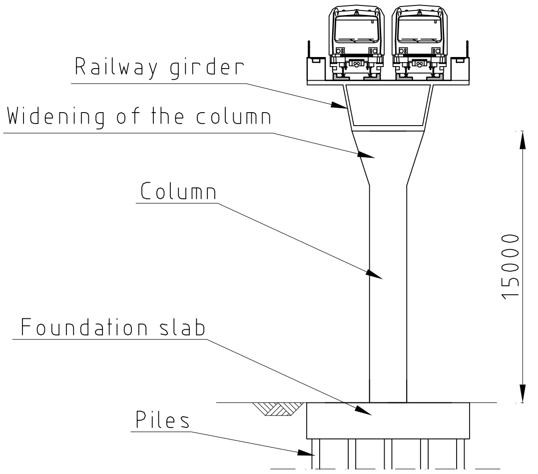


Figure 183: Schematisation of the elevated metro structure with a concrete box girder

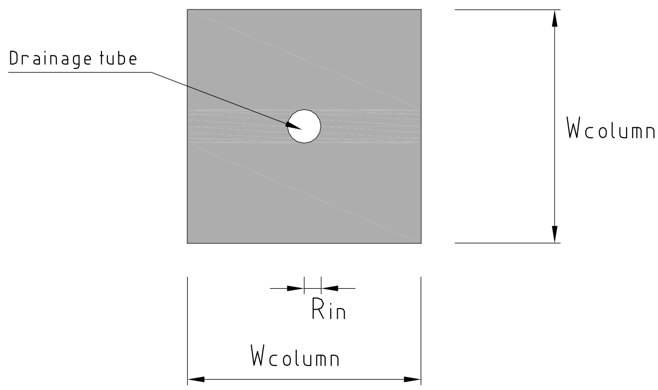


Figure 184: Cross-section column

Length span box girder $L = 45m$

Height column $H = 15m$

Width of the column $w_{column} = 2.06m$

Inner radius in drainage tube $r_{in} = 0.15m$

Moment of inertia of the column $I_{column} = \frac{1}{12} * w_{column}{}^4 - \frac{1}{4} * \pi * r_{in}{}^4 = 1.5m^4$

Cross-sectional area of the column $A_{column} = w_{column}{}^2 - \pi * r_{in}{}^2 = 4.173m^2$

Section modulus column $W_{column} = \frac{I_c}{w_{column} / 2} = 1.457m^3$

Dead load column:

$$F_{v,column} = A_c * H * \rho_c * g = 1535.11kN$$

Where:

$\rho_c = 2500kg / m^3$ Density of concrete

$g = 9.81m / s^2$ Acceleration due to gravity

G.1.3 Loads at the top of the column

Vertical force at the top of the column

$$F_{v,viaduct} = q_{tot} * L + Q_{v,tendons} + Q_{v,anchorage} + Q_{v,widening} = 9320kN$$

Where:

$q_{tot} = g_{dead} + g_{perm} + q_{var} = 194.73kN/m$	
$g_{dead} = 102.02kN/m$	Dead load box girder
$g_{perm} = 34.42kN/m$	Permanent load at the box girder
$q_{var} = 58.29kN/m$	Variable load of the metros and snow loading
$Q_{v,tendons} = L_{tendon} * n_{tendons} * A_p * \rho_p * g = 115.66kN$	Dead load tendons
$L_{tendon} = 2*\sqrt{f^2 + a^2} + L - 2*a = 45.101m$	Length tendon
$f = 1.23m$	Tendon eccentricity at mid-span
$a = 15m$	Distance of deviation blocks to supports
$n_{tendons} = 6$	Number of tendons
$A_p = 5550mm^2$	Cross-sectional area of one tendon
$\rho_p = 7850kg/m^3$	Density prestressing steel
$Q_{v,anchorage} = V_{anchorage} * \rho_c * g = 196.2kN$	Dead load extra concrete for anchorage and deviation blocks of the box girder
$V_{anchorage} = 8m^3$	Assumed volume of extra concrete for anchorage and deviation blocks of the box girder
$Q_{v,widening} = V_{widening} * \rho_c * g = 245.25kN$	Dead load extra concrete for widening of the column at the top
$V_{widening} = 10m^3$	Assumed volume of extra concrete for widening of the column at the top

Horizontal force at the top of the column in longitudinal direction of the viaduct

$$F_{h,long} = \frac{q_{mob}}{g} * \phi * a_a * L + \frac{q_{mob}}{g} * \phi * a_d * L = 326.25kN$$

Where:

$q_{mob} = 25.5kN/m \ per\ track$	Mobile load (metros)
$\phi = 1 + 4/(10 + L) = 1.07$	Dynamic factor
$a_a = 1.2m/s^2$	Maximum acceleration of the metros [8]
$a_d = 1.4m/s^2$	Maximum deceleration of the metros [8]

Horizontal force at the top of the column in transversal direction of the viaduct

$$F_{h,trans} = q_{wind} * H_{viaduct} * L + Q_{sidewf} = 485.63kN$$

Where:

$q_{wind} = 1.5kN/m^2$ Wind load

$H_{viaduct} = H_{boxgirder} + H_{usr} + H_{wind} = 6.75m$ Height superstructure subjected to wind forces

$H_{boxgirder} = 2.8m$ Depth box girder

$H_{usr} = 0.35m$ Height upper side rail, see Figure 90

$H_{wind} = 3.6m$ Range wind load on the superstructure, see Figure 90

$Q_{sidewf} = 30kN$ Sideward force due to the metro

G.1.4 Foundation

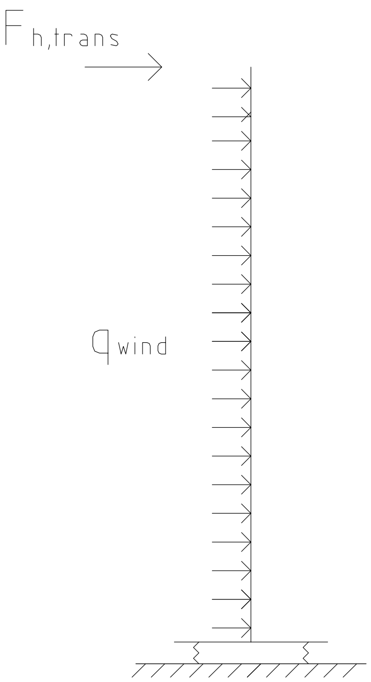


Figure 185: Load schematisation in transversal direction

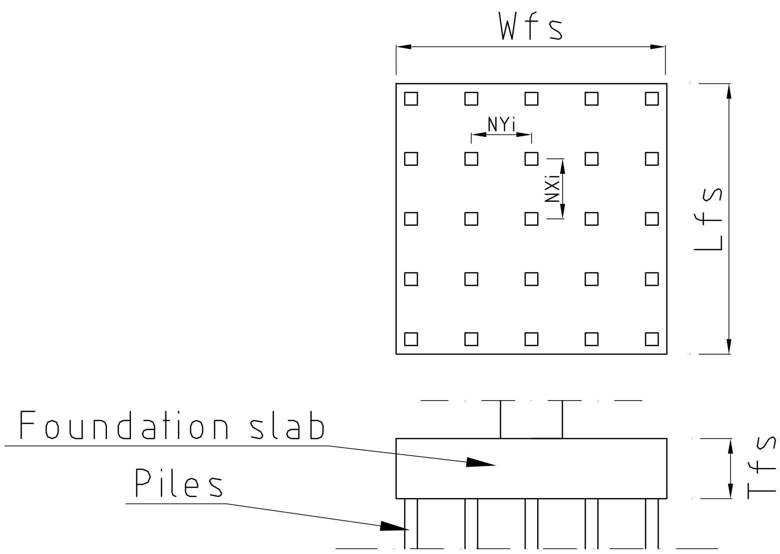


Figure 186: Pile foundation, top- and side-view

Moment at the top of the foundation slab in transversal direction

$$M_y = F_{h,trans} * H + 0.5 * q_{wind} * w_{column} * H^2 = 7632kNm$$

Foundation

The foundation consists of 25 piles underneath a foundation slab with the dimensions:

$L_{fs} = 9m$ Length foundation slab (assumption)

$W_{fs} = 9m$ Width foundation slab (assumption)

$T_{fs} = 2m$ Thickness foundation slab (assumption)

The foundation slab is considered as an infinite stiff slab.

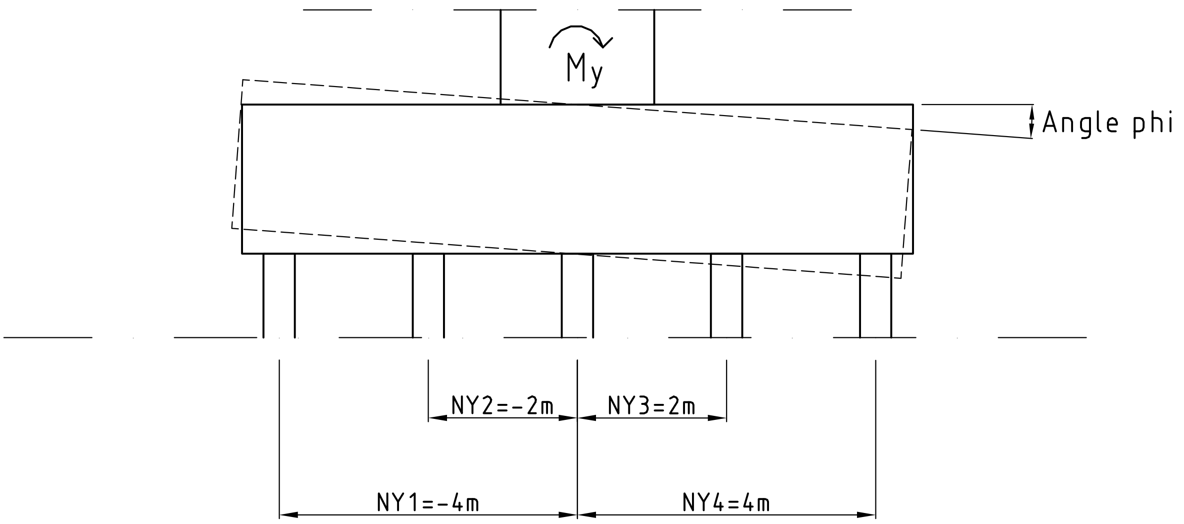


Figure 187: Rotation of the stiff foundation slab

Dead load of the foundation slab:

$$F_{v,fs} = L_{fs} * W_{fs} * T_{fs} * \rho_c * g = 3973.05kN$$

The spring stiffness of a pile is (assumption):

$$k = 100000kN/m$$

The force in a pile due to a moment is:

$$F_{pile} = k * n_i * \varphi$$

The moment at the foundation is:

$$M = \sum k * n_i^{\ 2} * \varphi = k * \varphi \sum n_i^{\ 2}$$

The rotation stiffness of the foundation is:

$$C = \frac{M}{\varphi} = \frac{k * \varphi * \sum n_i^{\ 2}}{\varphi} = k * \sum n_i^{\ 2}$$

The rotation stiffness of the foundation in transversal direction is:

$$C_y = k * \sum ny_i^{\ 2} = 20000000kNm/rad$$

The rotation stiffness of the foundation in longitudinal direction is:

$$C_x = k * \sum nx_i^{\ 2} = 20000000kNm/rad$$

G.1.5 Stability

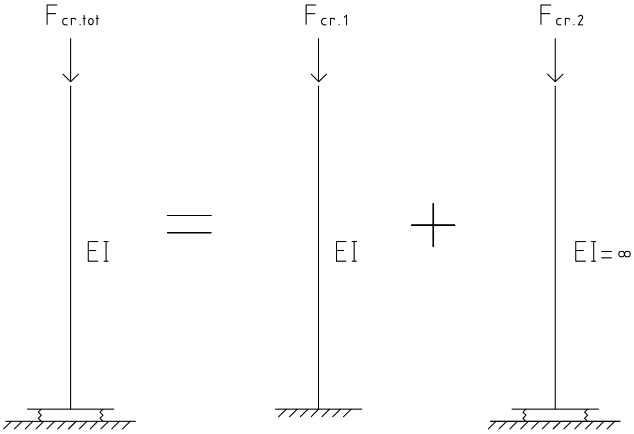


Figure 188: Structural model

$$F_{cr,1} = \frac{\pi^2 * E_{c,eff} * I_{column}}{l_c^{\ 2}} = 278777.87kN$$ Critical buckling force mode 1

Where:

$$E_{c,eff} = \frac{E_{cm}}{1+\varphi(\infty,t_0)} = 16944.49 N/mm^2$$ Effective modulus of elasticity of concrete

$$E_{cm} = 37277.87 N/mm^2$$ Secant modulus of elasticity of concrete

$$\varphi(\infty,t_0) = 1.2$$ Creep coefficient, see B.5.5: creep

$$l_c = 2*H = 30m$$ Effective length column

$$F_{cr,2} = \frac{C_y}{H} = 1333333.33kN$$ Critical buckling force mode 2

Total critical buckling force:

$$\frac{1}{F_{cr,tot}} = \frac{1}{F_{cr,1}} + \frac{1}{F_{cr,2}} \rightarrow F_{cr,tot} = 230569.59kN$$

Vertical force at the top of the column, ULS:

$$F_{v,viaduct,uls} = q_{tot,uls} * L + Q_{v,tendons} * \gamma_{G,unfav} + Q_{v,anchorage} * \gamma_{G,unfav} + Q_{v,widening} * \gamma_{G,unfav} = 12975.4kN$$

Where:

$$q_{tot,uls} = \gamma_{G,unfav} * g_{dead} + \gamma_{G,unfav} * g_{perm} + \gamma_{Q,unfav} * q_{var} = 271.63kN/m$$

$$\gamma_{G,unfav} = 1.35$$

$$\gamma_{Q,unfav} = 1.5$$

Factor n:

$$n = \frac{F_{cr,tot}}{F_{v,viaduct,uls}} = 17.77 \geq 10 \rightarrow Ok,\ structure\ is\ stable$$

2nd degree magnification factor:

$$\frac{n}{n-1} = 1.06$$

G.1.6 Stiffness

$$\delta_h = \frac{F_{h,trans} * H^3}{3 * E_{c,eff} * I_{column}} = 0.0215m$$

Deflection at the top due to the horizontal force at the top of the column

$$\delta_q = \frac{q_{wind} * w_{column} * H^4}{8 * E_{c,eff} * I_{column}} = 0.0008m$$

Deflection at the top due to wind load at the column

$$\delta_c = \frac{M_y}{C_y} * H = 0.0057m$$

Deflection at the top due to rotation of the foundation slab

Total 2nd order deflection at the top:

$$\delta_{tot} = (\delta_h + \delta_q + \delta_c) * \frac{n}{n-1} = 0.0297m$$

The maximum allowed deflection at the top:

$$\delta_{\max} = \frac{H}{500} = 0.03m$$

Deflection check:

$$\delta_{tot} = 0.0297m \leq \delta_{\max} = 0.03m \rightarrow Ok, \; structure \; is \; stiff \; enough$$

The 2nd order rotation of the foundation slab:

$$\varphi = \frac{M_y}{C_y} * \frac{n}{n-1} = 0.0004rad$$

The maximum allowed rotation of the foundation slab [23]:

$$\varphi_{\max} = \frac{1}{300} = 0.0033rad$$

Rotation check:

$$\varphi = 0.0004rad \leq \varphi_{\max} = 0.0033rad \rightarrow Ok$$

G.1.7 Foundation piles

Pile force

Total vertical force at the piles, ULS:

$$F_{v,piles} = F_{v,viaduct,uls} + F_{v,column} * \gamma_{G,unfav} + F_{v,fs} * \gamma_{G,unfav} = 20411.41kN$$

Total horizontal force at the piles, ULS:

$$F_h = F_{h,trans} * \gamma_{Q,unfav} + q_{wind} * \gamma_{Q,unfav} * w_{column} * H = 797.96kN$$

Total moment at the foundation slab in transversal direction, ULS:

$$M_y = F_{h,trans} * \gamma_{Q,unfav} * H + 0.5 * q_{wind} * \gamma_{Q,unfav} * w_{column} * H^2 + \delta_{tot} * F_{v,viaduct,uls} = 11832.76kNm$$

Total moment at the foundation slab in longitudinal direction, ULS:

$$M_x = F_{h,long} * \gamma_{Q,unfav} * H = 7340.56kNm$$

The number of piles:

$$n_p = 25$$

The maximum allowed pile force (assumption):

$$P_{\max,allow} = 1200kN \ \ per \ \ pile$$

$$P_v = \frac{-F_{v,piles}}{n_p} = -816.46kN$$

Load on piles due to vertical load

$$P_{my} = \frac{M_y}{C_y} * ny_{\max} * k = {}^{+}\!/\!_{-} 236.66kN$$

Load on the outside piles in transversal direction due to the moment in the transversal direction

$$P_{mx} = \frac{M_x}{C_x} * nx_{\max} * k = {}^{+}\!/\!_{-} 146.81kN$$

Load on the outside piles in longitudinal direction due to the moment in the longitudinal direction

Where:

$$ny_{\max} = 4m$$

$$nx_{\max} = 4m$$

The maximum pile force in the corner piles of the foundation slab is:

$$P_{\max} = P_v - P_{my} - P_{mx} = -1199.92kN \leq P_{\max,allow} = 1200kN \rightarrow Ok$$

The minimum pile force in the corner piles of the foundation slab is:

$$P_{\min} = P_v + P_{my} + P_{mx} = -432.99kN$$

Most likely in all load phases the piles will not be in tension considering the large vertical force.

Pile head moment due to horizontal force

The pile is schematised as a beam of infinite length on one side and is fixed in the foundation slab on the other side. The pile which is supported by linear elastic springs (soil) is subjected to a concentrated horizontal force at the foundation slab. The structural model for a pile subjected to the horizontal force is shown in Figure 189.

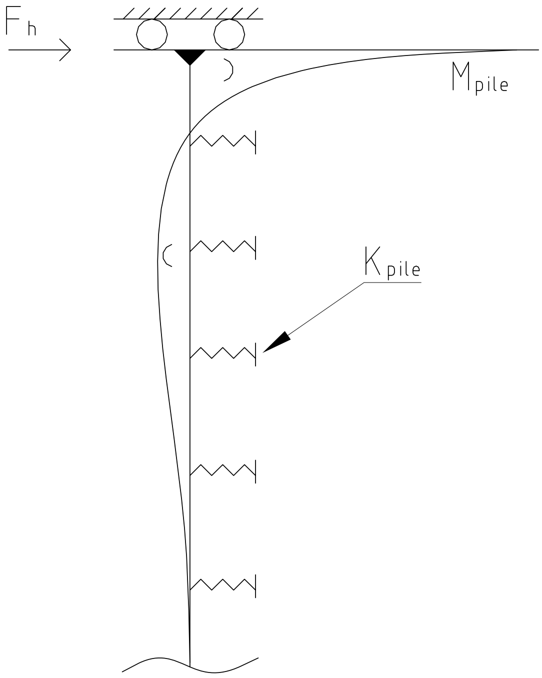


Figure 189: Structural model piles

Characteristics of the piles:

$w_{pile} = 0.42m$	Width of the pile
$h_{pile} = 0.42m$	Depth of the pile
$A_{pile} = w_{pile} * h_{pile} = 0.1764m^2$	Cross-sectional area of the pile
$I_{pile} = \frac{1}{12} * w_{pile} * h_{pile}^3 = 0.0026m^4$	Moment of inertia of the pile
$E_{pile} = E_{cm} = 37277.87 N/mm^2$	Modulus of elasticity of the pile, assumption of uncracked pile as the vertical pile force is large
$c_{effectivewidth} = 1.5$	Factor for determining the effective width of the pile
$k = 3000 kN/m^3$	Modulus of subgrade reaction
$k_{pile} = k * w_{pile} * c_{effectivewidth} = 1890 kN/m^2$	Modulus of subgrade reaction of pile

The pile head moment in one pile is [19]:

$$M_{pile} = \frac{\frac{1}{2} * \frac{F_h}{n_p}}{2 * \beta} = 30.18 kNm$$

Where:

$$\beta = \sqrt[4]{\frac{k_{pile}}{4 E_{pile} I_{pile}}} = 0.26 m^{-1}$$

The required reinforcement at one side of the pile is:

$$\frac{M_{pile}}{z} = A_s * f_{yd} \rightarrow A_s = 202.85 mm^2$$

Where:

$z = 0.9 * d = 0.342m$	Lever arm of internal forces
$d = h_{pile} - c = 0.38m$	Effective depth
$c = 40mm$	Concrete cover, see B.3.3
$f_{yd} = f_{yk} / \gamma_s = 435N/mm^2$	Design yield strength of reinforcement
$f_{yk} = 500N/mm^2$	Characteristic yield strength of reinforcement
$\gamma_s = 1.15$	Partial factor for reinforcing steel

The total required reinforcement in a pile is:

$$A_{s,tot} = 4 * A_s = 811.42mm^2$$

The reinforcement percentage in a pile then becomes:

$$\omega_0 = \frac{A_{s,tot}}{A_{pile}} * 100\% = 0.46\%$$

The maximum reinforcement percentage in a column/pile is:

$$\omega_{\max} = 4\%$$

Reinforcement percentage check

$$\omega_0 = 0.46\% \le \omega_{\max} = 4\% \rightarrow Ok$$

G.1.8 Stresses in column

Total vertical force at the bottom of the column, ULS:

$$F_{v,tot,bot,column} = F_{v,viaduct,uls} + F_{v,column} * \gamma_{G,unfav} = 15047.8kN$$

Total moment at the bottom of the column in transversal direction, ULS:

$$M_y = F_{h,trans} * \gamma_{Q,unfav} * H + 0.5 * q_{wind} * \gamma_{Q,unfav} * w_{column} * H^2 + \delta_{tot} * F_{v,viaduct,uls} = 11832.76kNm$$

The compressive stress in the column due to the vertical force is:

$$\sigma_n = \frac{-F_{v,tot,bot,column}}{A_{column}} = -3.61N/mm^2$$

The stress in the column due to the moment in transversal direction is:

$$\sigma_m = \pm \frac{M_y}{W_{column}} = \pm 8.12N/mm^2$$

The maximum compressive stress in the column is:

$$\sigma_{c\max} = \sigma_n - \sigma_m = -11.73N/mm^2$$

The minimum compressive stress in the column is:

$$\sigma_{c\min} = \sigma_n + \sigma_m = 4.52N/mm^2$$

There arises tension in the column which means that the section is cracked. The assumption that the effective modulus of elasticity of concrete should be taken into account is thus correct.

G.1.9 Overview weight contribution of elements

In the table below the contribution to the total vertical load at the piles is given for the different loads.

Loads		*Value*		*Percentage of the total vertical load at the piles*	
Dead load box girder	$g_{dead} * L$	4590.9	kN	30.96	%
Dead load box girder + tendons + anchorage and deviation blocks	$g_{dead} * L + Q_{v,tendons} + Q_{v,anchorage}$	4902.76	kN	33.06	%
Total load box girder fully loaded	$q_{tot} * L + Q_{v,tendons} + Q_{v,anchorage}$	9074.71	kN	61.2	%
Dead load column	$F_{v,column} + Q_{v,widening}$	1780.36	kN	12.01	%
Dead load foundation slab	$F_{v,fs}$	3973.05	kN	26.79	%
Total vertical load at the piles	$q_{tot} * L + Q_{v,tendons} + Q_{v,anchorage} + F_{v,column} + Q_{v,widening} + F_{v,fs}$	14828.12	kN	100	%

G.2 Column + foundation in combination with a UHPC box girder

G.2.1 General

This Appendix presents the calculations of the column + foundation of the elevated metro structure with a UHPC box girder. First the cross-sectional properties of the column are described. Paragraph 3 deals with the loads to which the column is subjected. The geometry and characteristics of the foundation are treated in paragraph 4. Furthermore this Appendix describes the calculations on stability and stiffness of the structure in respectively the paragraphs 5 and 6. The forces in the piles and stresses in the column are treated in paragraph 7 and 8. Finally this Appendix gives an overview of the weight contribution of the different elements of the structure.

The formulas and values used in the calculations are taken from [11] and other references, which are then stated in the text.

G.2.2 Column C50/60

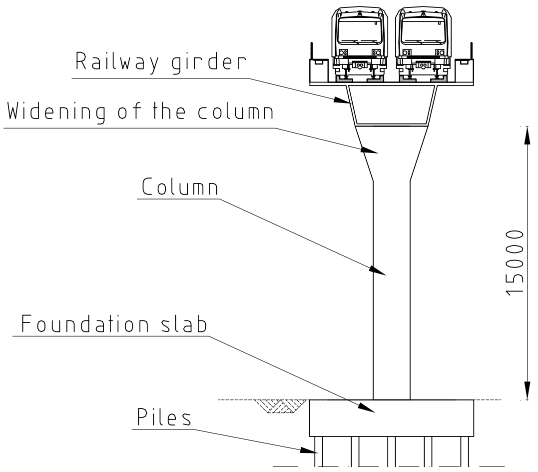


Figure 190: Schematisation of the elevated metro structure with a UHPC box girder

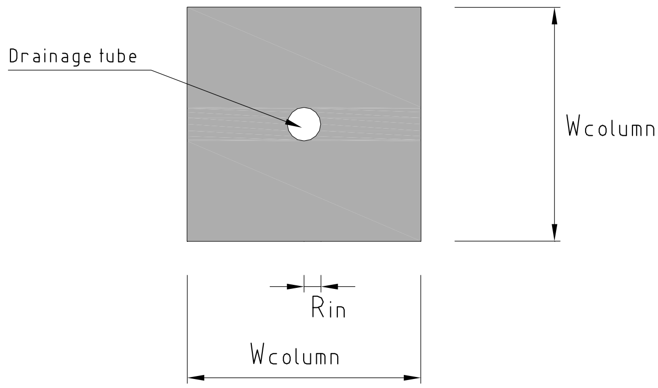


Figure 191: Cross-section column

Length span box girder $L = 45m$

Height column $H = 15m$

Width of the column $w_{column} = 2.02m$

Inner radius drainage tube $r_{in} = 0.15m$

Moment of inertia of the column $I_{column} = \frac{1}{12} * w_{column}{}^4 - \frac{1}{4} * \pi * r_{in}{}^4 = 1.387m^4$

Cross-sectional area of the column $A_{column} = w_{column}{}^2 - \pi * r_{in}{}^2 = 4.010m^2$

Section modulus column $W_{column} = \frac{I_c}{w_{column}/2} = 1.373m^3$

Dead load column:

$$F_{v,column} = A_c * H * \rho_c * g = 1475.07kN$$

Where:

$\rho_c = 2500kg/m^3$ Density of concrete

$g = 9.81m/s^2$ Acceleration due to gravity

G.2.3 Loads at the top of the column

Vertical force at the top of the column

$$F_{v,viaduct} = q_{tot} * L + Q_{v,tendons} + Q_{v,anchorage} + Q_{v,widening} = 7834.34kN$$

Where:

$q_{tot} = g_{dead} + g_{perm} + q_{var} = 162.11kN/m$

$g_{dead} = 69.4kN/m$ Dead load box girder

$g_{perm} = 34.42kN/m$ Permanent load at the box girder

$q_{var} = 58.29kN/m$ Variable load of the metros and snow loading

$Q_{v,tendons} = L_{tendon} * n_{tendons} * A_p * \rho_p * g = 115.59kN$ Dead load tendons

$L_{tendon} = 2 * \sqrt{f^2 + a^2} + L - 2 * a = 45.077m$ Length tendon

$f = 1.143m$ Tendon eccentricity at mid-span

$a = 17m$ Distance of deviation blocks to supports

$n_{tendons} = 6$ Number of tendons

$A_p = 5550mm^2$ Cross-sectional area of one tendon

$\rho_p = 7850kg/m^3$ Density prestressing steel

$Q_{v,anchorage} = V_{anchorage} * \rho_{UHPC} * g = 178.54kN$ Dead load extra UHPC for anchorage and deviation blocks of the box girder

$V_{anchorage} = 7m^3$ Assumed volume of extra UHPC for anchorage and deviation blocks of the box girder

$\rho_{UHPC} = 2600kg/m^3$ Density of UHPC

$Q_{v,widening} = V_{widening} * \rho_c * g = 245.25kN$ — Dead load extra concrete for widening of the column at the top

$V_{widening} = 10m^3$ — Assumed volume of extra concrete for widening of the column at the top

Horizontal force at the top of the column in longitudinal direction of the viaduct

$$F_{h,long} = \frac{q_{mob}}{g} * \phi * a_a * L + \frac{q_{mob}}{g} * \phi * a_d * L = 326.25kN$$

Where:

$q_{mob} = 25.5kN/m \ \ per \ track$ — Mobile load (metros)

$\phi = 1 + 4/(10 + L) = 1.07$ — Dynamic factor

$a_a = 1.2m/s^2$ — Maximum acceleration of the metros [8]

$a_d = 1.4m/s^2$ — Maximum deceleration of the metros [8]

Horizontal force at the top of the column in transversal direction of the viaduct

$$F_{h,trans} = q_{wind} * H_{viaduct} * L + Q_{sidewf} = 459.3kN$$

Where:

$q_{wind} = 1.5kN/m^2$ — Wind load

$H_{viaduct} = H_{boxgirder} + H_{usr} + H_{wind} = 6.36m$ — Height superstructure subjected to wind forces

$H_{boxgirder} = 2.41m$ — Depth box girder

$H_{usr} = 0.35m$ — Height upper side rail, see Figure 145

$H_{wind} = 3.6m$ — Range wind load on the superstructure, see Figure 145

$Q_{sidewf} = 30kN$ — Sideward force due to the metro

G.2.4 Foundation

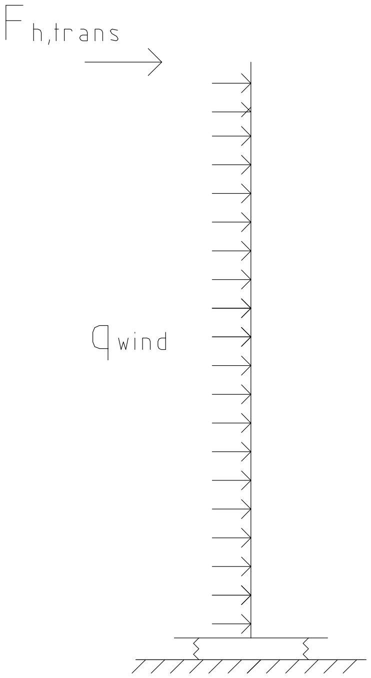


Figure 192: Load schematisation in transversal direction

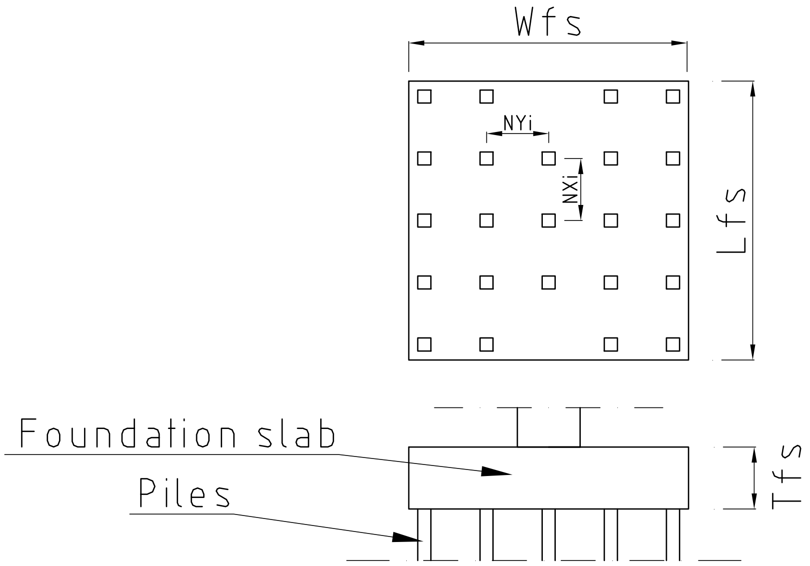


Figure 193: Pile foundation, top- and side-view

Moment at the top of the foundation slab in transversal direction

$$M_y = F_{h,trans} * H + 0.5 * q_{wind} * w_{column} * H^2 = 7230.38kNm$$

Foundation

The foundation consists of 23 piles underneath a foundation slab with the dimensions:

$L_{fs} = 9m$ Length foundation slab (assumption)

$W_{fs} = 9m$ Width foundation slab (assumption)

$T_{fs} = 2m$ Thickness foundation slab (assumption)

The foundation slab is considered as an infinite stiff slab.

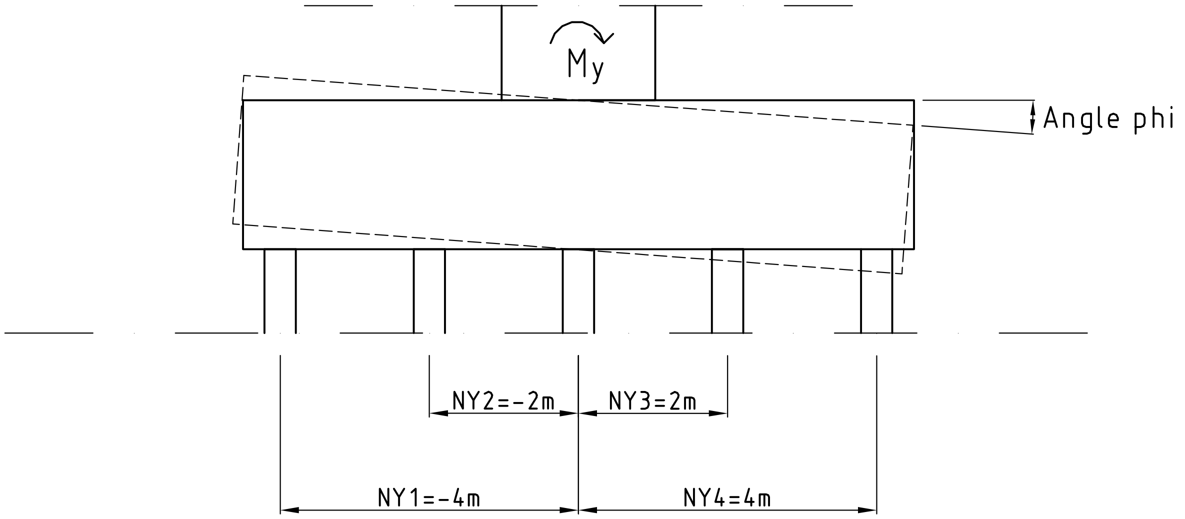


Figure 194: Rotation of the stiff foundation slab

Dead load of the foundation slab:

$$F_{v,fs} = L_{fs} * W_{fs} * T_{fs} * \rho_c * g = 3973.05kN$$

The spring stiffness of a pile is (assumption):

$$k = 100000kN/m$$

The force in a pile due to a moment is:

$$F_{pile} = k * n_i * \varphi$$

The moment at the foundation is:

$$M = \sum k * n_i^2 * \varphi = k * \varphi \sum n_i^2$$

The rotation stiffness of the foundation is:

$$C = \frac{M}{\varphi} = \frac{k * \varphi * \sum n_i^2}{\varphi} = k * \sum n_i^2$$

The rotation stiffness of the foundation in transversal direction is:

$$C_y = k * \sum ny_i^2 = 20000000kNm/rad$$

The rotation stiffness of the foundation in longitudinal direction is:

$$C_x = k * \sum nx_i^2 = 16800000kNm/rad$$

G.2.5 Stability

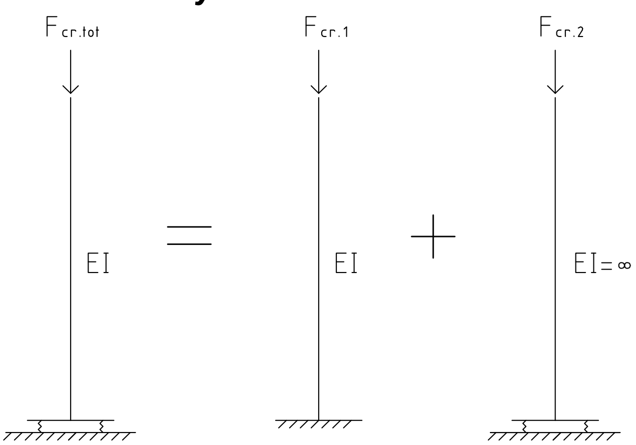


Figure 195: Structural model

$$F_{cr,1} = \frac{\pi^2 * E_{c,eff} * I_{column}}{l_c^2} = 257742.18kN$$ Critical buckling force mode 1

Where:

$$E_{c,eff} = \frac{E_{cm}}{1+\varphi(\infty,t_0)} = 16944.49N/mm^2$$ Effective modulus of elasticity of concrete

$$E_{cm} = 37277.87N/mm^2$$ Secant modulus of elasticity of concrete

$$\varphi(\infty,t_0) = 1.2$$ Creep coefficient, see B.5.5: creep

$$l_c = 2 * H = 30m$$ Effective length column

$$F_{cr,2} = \frac{C_y}{H} = 1333333.33kN$$

Critical buckling force mode 2

Total critical buckling force:

$$\frac{1}{F_{cr,tot}} = \frac{1}{F_{cr,1}} + \frac{1}{F_{cr,2}} \rightarrow F_{cr,tot} = 215989.90kN$$

Vertical force at the top of the column, ULS:

$$F_{v,viaduct,uls} = q_{tot,uls} * L + Q_{v,tendons} * \gamma_{G,unfav} + Q_{v,anchorage} * \gamma_{G,unfav} + Q_{v,widening} * \gamma_{G,unfav} = 10969.81kN$$

Where:

$$q_{tot,uls} = \gamma_{G,unfav} * g_{dead} + \gamma_{G,unfav} * g_{perm} + \gamma_{Q,unfav} * q_{var} = 227.59kN/m$$

$$\gamma_{G,unfav} = 1.35$$

$$\gamma_{Q,unfav} = 1.5$$

Factor n:

$$n = \frac{F_{cr,tot}}{F_{v,viaduct,uls}} = 19.69 \geq 10 \rightarrow Ok,\ structure\ is\ stable$$

2nd degree magnification factor:

$$\frac{n}{n-1} = 1.05$$

G.2.6 Stiffness

$$\delta_h = \frac{F_{h,trans} * H^3}{3 * E_{c,eff} * I_{column}} = 0.0220m$$

Deflection at the top due to the horizontal force at the top of the column

$$\delta_q = \frac{q_{wind} * w_{column} * H^4}{8 * E_{c,eff} * I_{column}} = 0.0008m$$

Deflection at the top due to wind load at the column

$$\delta_c = \frac{M_y}{C_y} * H = 0.0054m$$

Deflection at the top due to rotation of the foundation slab

Total 2nd order deflection at the top:

$$\delta_{tot} = (\delta_h + \delta_q + \delta_c) * \frac{n}{n-1} = 0.0297m$$

The maximum allowed deflection at the top:

$$\delta_{max} = \frac{H}{500} = 0.03m$$

Deflection check:

$$\delta_{tot} = 0.0297m \leq \delta_{\max} = 0.03m \rightarrow Ok,\ structure\ is\ stiff\ \ enough$$

The 2nd order rotation of the foundation slab:

$$\varphi = \frac{M_y}{C_y} * \frac{n}{n-1} = 0.0004rad$$

The maximum allowed rotation of the foundation slab [23]:

$$\varphi_{\max} = \frac{1}{300} = 0.0033rad$$

Rotation check:

$$\varphi = 0.0004rad \leq \varphi_{\max} = 0.0033rad \rightarrow Ok$$

G.2.7 Foundation piles

Pile force

Total vertical force at the piles, ULS:

$$F_{v,piles} = F_{v,viaduct,uls} + F_{v,column} * \gamma_{G,unfav} + F_{v,fs} * \gamma_{G,unfav} = 18324.78kN$$

Total horizontal force at the piles, ULS:

$$F_h = F_{h,trans} * \gamma_{Q,unfav} + q_{wind} * \gamma_{Q,unfav} * w_{column} * H = 757.13kN$$

Total moment at the foundation slab in transversal direction, ULS:

$$M_y = F_{h,trans} * \gamma_{Q,unfav} * H + 0.5 * q_{wind} * \gamma_{Q,unfav} * w_{column} * H^2 + \delta_{tot} * F_{v,viaduct,uls} = 11171.73kNm$$

Total moment at the foundation slab in longitudinal direction, ULS:

$$M_x = F_{h,long} * \gamma_{Q,unfav} * H = 7340.56kNm$$

The number of piles:

$n_p = 23$

The maximum allowed pile force (assumption):

$P_{\max,allow} = 1200kN\ \ per\ \ pile$

$$P_v = \frac{-F_{v,piles}}{n_p} = -796.73kN$$ Load on piles due to vertical load

$$P_{my} = \frac{M_y}{C_y} * ny_{\max} * k = +/-\,223.43kN$$ Load on the outside piles in transversal direction due to the moment in the transversal direction

$$P_{mx} = \frac{M_x}{C_x} * nx_{\max} * k = +/-\,174.78kN$$ Load on the outside piles in longitudinal direction due to the moment in the longitudinal direction

Where:

$ny_{\max} = 4m$

$nx_{\max} = 4m$

The maximum pile force in the corner piles of the foundation slab is:

$$P_{\max} = P_v - P_{my} - P_{mx} = -1194.94kN \leq P_{\max,allow} = 1200kN \rightarrow Ok$$

The minimum pile force in the corner piles of the foundation slab is:

$$P_{\min} = P_v + P_{my} + P_{mx} = -398.52kN$$

Most likely in all load phases the piles will not be in tension considering the large vertical force.

Pile head moment due to horizontal force

The pile is schematised as a beam of infinite length on one side and is fixed in the foundation slab on the other side. The pile which is supported by linear elastic springs (soil) is subjected to a concentrated horizontal force at the foundation slab. The structural model for a pile subjected to the horizontal force is shown in Figure 196.

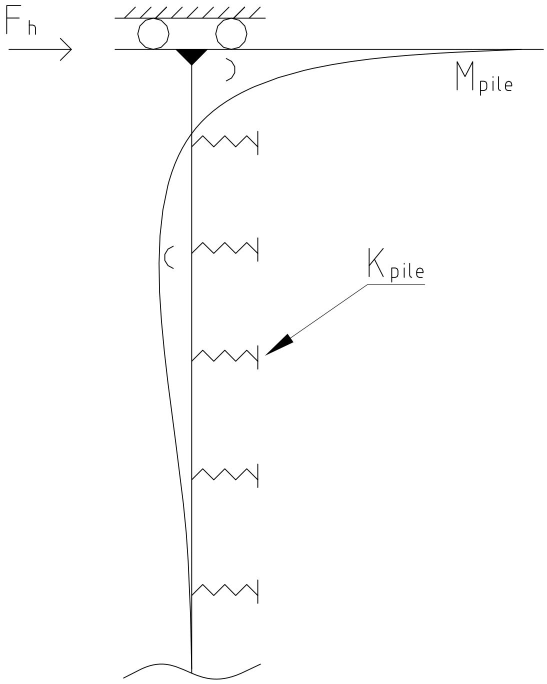


Figure 196: Structural model piles

Characteristics of the piles:

$w_{pile} = 0.42m$	Width of the pile
$h_{pile} = 0.42m$	Depth of the pile
$A_{pile} = w_{pile} * h_{pile} = 0.1764m^2$	Cross-sectional area of the pile
$I_{pile} = \frac{1}{12} * w_{pile} * h_{pile}{}^3 = 0.0026m^4$	Moment of inertia of the pile
$E_{pile} = E_{cm} = 37277.87N/mm^2$	Modulus of elasticity of the pile, assumption of uncracked pile as the vertical pile force is large
$c_{effectivewidth} = 1.5$	Factor for determining the effective width of the pile
$k = 3000kN/m^3$	Modulus of subgrade reaction
$k_{pile} = k * w_{pile} * c_{effectivewidth} = 1890kN/m^2$	Modulus of subgrade reaction of pile

The pile head moment in one pile is [19]:

$$M_{pile} = \frac{\frac{1}{2} * \frac{F_h}{n_p}}{2 * \beta} = 31.12kNm$$

Where:

$$\beta = \sqrt[4]{\frac{k_{pile}}{4E_{pile} I_{pile}}} = 0.26m^{-1}$$

The required reinforcement at one side of the pile is:

$$\frac{M_{pile}}{z} = A_s * f_{yd} \rightarrow A_s = 209.21mm^2$$

Where:

$z = 0.9 * d = 0.342m$	Lever arm of internal forces
$d = h_{pile} - c = 0.38m$	Effective depth
$c = 40mm$	Concrete cover, see B.3.3
$f_{yd} = f_{yk} / \gamma_s = 435N / mm^2$	Design yield strength of reinforcement
$f_{yk} = 500N / mm^2$	Characteristic yield strength of reinforcement
$\gamma_s = 1.15$	Partial factor for reinforcing steel

The total required reinforcement in a pile is:

$$A_{s,tot} = 4 * A_s = 836.84mm^2$$

The reinforcement percentage in a pile then becomes:

$$\omega_0 = \frac{A_{s,tot}}{A_{pile}} * 100\% = 0.47\%$$

The maximum reinforcement percentage in a column/pile is:

$$\omega_{\max} = 4\%$$

Reinforcement percentage check

$$\omega_0 = 0.47\% \leq \omega_{\max} = 4\% \rightarrow Ok$$

G.2.8 Stresses in column

Total vertical force at the bottom of the column, ULS:

$$F_{v,tot,bot,column} = F_{v,viaduct,uls} + F_{v,column} * \gamma_{G,unfav} = 12961.16kN$$

Total moment at the bottom of the column in transversal direction, ULS:

$$M_y = F_{h,trans} * \gamma_{Q,unfav} * H + 0.5 * q_{wind} * \gamma_{Q,unfav} * w_{column} * H^2 + \delta_{tot} * F_{v,viaduct,uls} = 11171.73kNm$$

The compressive stress in the column due to the vertical force is:

$$\sigma_n = \frac{-F_{v,tot,bot,column}}{A_{column}} = -3.23N / mm^2$$

The stress in the column due to the moment in transversal direction is:

$$\sigma_m = {}^{+}\!/\!_{-} \frac{M_y}{W_{column}} = {}^{+}\!/\!_{-} 8.13 N/mm^2$$

The maximum compressive stress in the column is:

$$\sigma_{c\max} = \sigma_n - \sigma_m = -11.37 N/mm^2$$

The minimum compressive stress in the column is:

$$\sigma_{c\min} = \sigma_n + \sigma_m = 4.90 N/mm^2$$

There arises tension in the column which means that the section is cracked. The assumption that the effective modulus of elasticity of concrete should be taken into account is thus correct.

G.2.9 Overview weight contribution of elements

In the table below the contribution to the total vertical load at the piles is given for the different loads.

Loads		*Value*		*Percentage of the total vertical load at the piles*	
Dead load box girder	$g_{dead} * L$	3123	kN	23.51	%
Dead load box girder + tendons + anchorage and deviation blocks	$g_{dead} * L + Q_{v,tendons} + Q_{v,anchorage}$	3417.14	kN	25.73	%
Total load box girder fully loaded	$q_{tot} * L + Q_{v,tendons} + Q_{v,anchorage}$	7589.09	kN	57.14	%
Dead load column	$F_{v,column} + Q_{v,widening}$	1720.32	kN	12.95	%
Dead load foundation slab	$F_{v,fs}$	3973.05	kN	29.91	%
Total vartical load at the piles	$q_{tot} * L + Q_{v,tendons} + Q_{v,anchorage} + F_{v,column} + Q_{v,widening} + F_{v,fs}$	13282.46	kN	100	%

G.3 Column + foundation in combination with a FRP sandwich girder

G.3.1 General

This Appendix presents the calculations of the column + foundation of the elevated metro structure with a FRP sandwich girder. First the cross-sectional properties of the column are described. Paragraph 3 deals with the loads to which the column is subjected. The geometry and characteristics of the foundation are treated in paragraph 4. Furthermore this Appendix describes the calculations on stability and stiffness of the structure in respectively the paragraphs 5 and 6. The forces in the piles and stresses in the column are treated in paragraph 7 and 8. Finally this Appendix gives an overview of the weight contribution of the different elements of the structure.

The formulas and values used in the calculations are taken from [11] and other references, which are then stated in the text.

G.3.2 Column C50/60

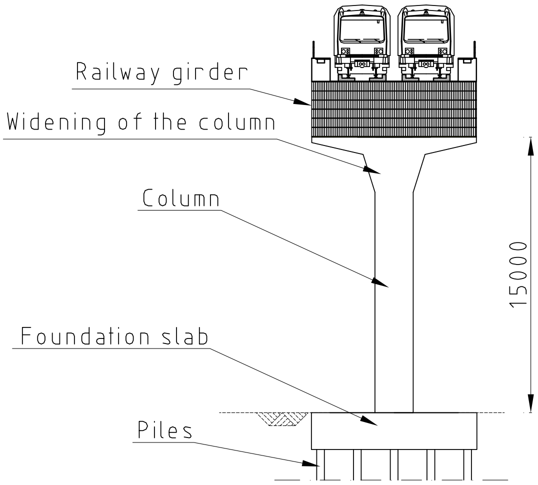


Figure 197: Schematisation of the elevated metro structure with a FRP sandwich girder

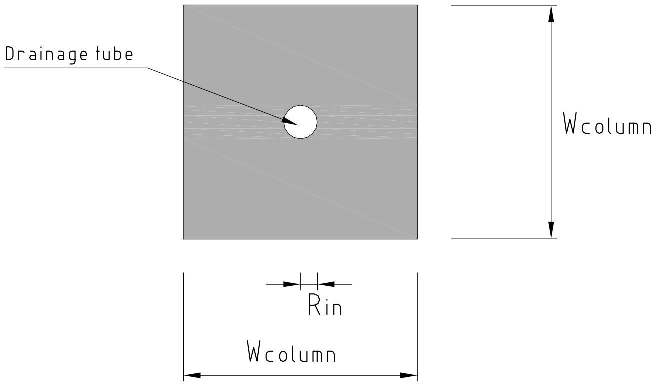


Figure 198: Cross-section column

Length span sandwich girder $L = 45m$

Height column $H = 15m$

Width of the column $w_{column} = 2.08m$

Inner radius drainage tube $r_{in} = 0.15m$

Moment of inertia of the column $I_{column} = \frac{1}{12} * w_{column}^4 - \frac{1}{4} * \pi * r_{in}^4 = 1.559m^4$

Cross-sectional area of the column $A_{column} = w_{column}^2 - \pi * r_{in}^2 = 4.256m^2$

Section modulus column $W_{column} = \frac{I_c}{w_{column}/2} = 1.499m^3$

Dead load column:

$$F_{v,column} = A_c * H * \rho_c * g = 1565.57kN$$

Where:

$\rho_c = 2500kg/m^3$ Density of concrete

$g = 9.81m/s^2$ Acceleration due to gravity

G.3.3 Loads at the top of the column

Vertical force at the top of the column

$$F_{v,viaduct} = q_{tot} * L + Q_{v,widening} = 6165kN$$

Where:

$q_{tot} = g_{dead} + g_{perm} + q_{var} = 127.19kN/m$

$g_{dead} = 34.48kN/m$ Dead load sandwich girder

$g_{perm} = 34.42kN/m$ Permanent load at the sandwich girder

$q_{var} = 58.29kN/m$ Variable load of the metros and snow loading

$Q_{v,widening} = V_{widening} * \rho_c * g = 441.45kN$ — Dead load extra concrete for widening of the column at the top

$V_{widening} = 18m^3$ — Assumed volume of extra concrete for widening of the column at the top

Horizontal force at the top of the column in longitudinal direction of the viaduct

$$F_{h,long} = \frac{q_{mob}}{g} * \phi * a_a * L + \frac{q_{mob}}{g} * \phi * a_d * L = 326.25kN$$

Where:

$q_{mob} = 25.5kN/m\ per\ track$ — Mobile load (metros)

$\phi = 1 + 4/(10 + L) = 1.07$ — Dynamic factor

$a_a = 1.2m/s^2$ — Maximum acceleration of the metros [8]

$a_d = 1.4m/s^2$ — Maximum deceleration of the metros [8]

Horizontal force at the top of the column in transversal direction of the viaduct

$$F_{h,trans} = q_{wind} * H_{viaduct} * L + Q_{sidewf} = 505.88kN$$

Where:

$q_{wind} = 1.5kN/m^2$ — Wind load

$H_{viaduct} = H_{sandwich} + H_{usr} + H_{wind} = 7.05m$ — Height superstructure subjected to wind forces

$H_{sandwich} = 3.04m$ — Depth sandwich girder

$H_{usr} = 0.35m$ — Height upper side rail, see Figure 90

$H_{wind} = 3.6m$ — Range wind load on the superstructure, see Figure 90

$Q_{sidewf} = 30kN$ — Sideward force due to the metro

G.3.4 Foundation

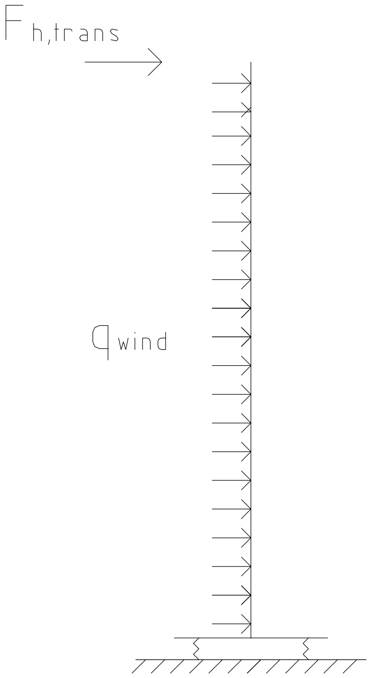


Figure 199: Load schematisation in transversal direction

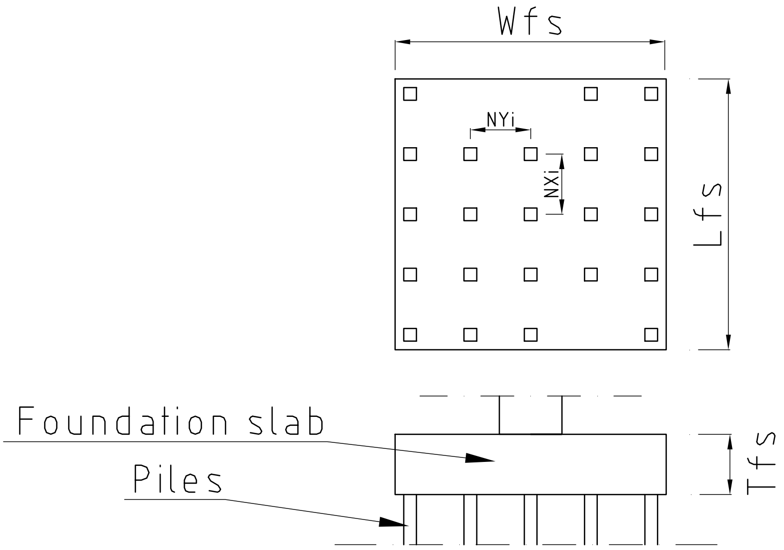


Figure 200: Pile foundation, top- and side-view

Moment at the top of the foundation slab in transversal direction

$$M_y = F_{h,trans} * H + 0.5 * q_{wind} * w_{column} * H^2 = 7939.13kNm$$

Foundation

The foundation consists of 22 piles underneath a foundation slab with the dimensions:

$L_{fs} = 9m$	Length foundation slab (assumption)
$W_{fs} = 9m$	Width foundation slab (assumption)
$T_{fs} = 2m$	Thickness foundation slab (assumption)

The foundation slab is considered as an infinite stiff slab.

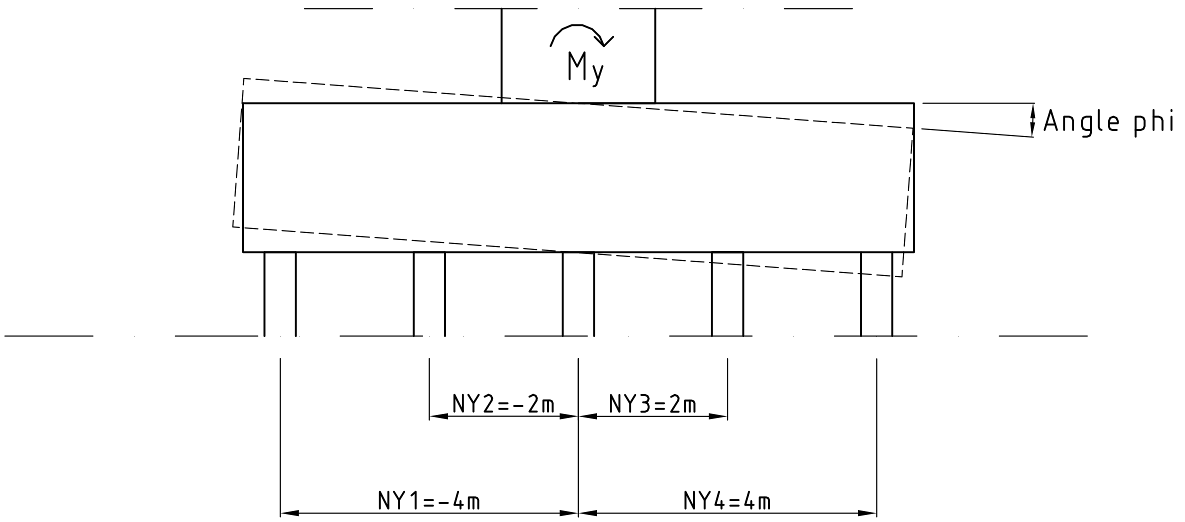


Figure 201: Rotation of the stiff foundation slab

Dead load of the foundation slab:

$$F_{v,fs} = L_{fs} * W_{fs} * T_{fs} * \rho_c * g = 3973.05kN$$

The spring stiffness of a pile is (assumption):

$$k = 100000kN/m$$

The force in a pile due to a moment is:

$$F_{pile} = k * n_i * \varphi$$

The moment at the foundation is:

$$M = \sum k * n_i^2 * \varphi = k * \varphi \sum n_i^2$$

The rotation stiffness of the foundation is:

$$C = \frac{M}{\varphi} = \frac{k * \varphi * \sum n_i^2}{\varphi} = k * \sum n_i^2$$

The rotation stiffness of the foundation in transversal direction is:

$$C_y = k * \sum ny_i^2 = 19200000kNm/rad$$

The rotation stiffness of the foundation in longitudinal direction is:

$$C_x = k * \sum nx_i^2 = 15200000kNm/rad$$

G.3.5 Stability

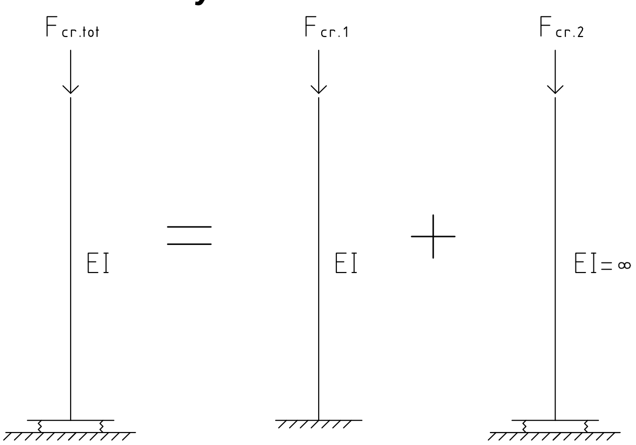


Figure 202: Structural model

$$F_{cr,1} = \frac{\pi^2 * E_{c,eff} * I_{column}}{l_c^2} = 289765.79kN$$ Critical buckling force mode 1

Where:

$$E_{c,eff} = \frac{E_{cm}}{1+\varphi(\infty,t_0)} = 16944.49N/mm^2$$ Effective modulus of elasticity of concrete

$$E_{cm} = 37277.87N/mm^2$$ Secant modulus of elasticity of concrete

$$\varphi(\infty,t_0) = 1.2$$ Creep coefficient, see B.5.5: creep

$$l_c = 2 * H = 30m$$ Effective length column

$$F_{cr,2} = \frac{C_y}{H} = 1280000kN$$ Critical buckling force mode 2

Total critical buckling force:

$$\frac{1}{F_{cr,tot}} = \frac{1}{F_{cr,1}} + \frac{1}{F_{cr,2}} \rightarrow F_{cr,tot} = 236277.42kN$$

Vertical force at the top of the column, ULS:

$$F_{v,viaduct,uls} = q_{tot,uls} * L + Q_{v,widening} * \gamma_{G,unfav} = 8716.21kN$$

Where:

$$q_{tot,uls} = \gamma_{G,unfav} * g_{dead} + \gamma_{G,unfav} * g_{perm} + \gamma_{Q,unfav} * q_{var} = 180.45kN/m$$
$$\gamma_{G,unfav} = 1.35$$
$$\gamma_{Q,unfav} = 1.5$$

Factor n:

$$n = \frac{F_{cr,tot}}{F_{v,viaduct,uls}} = 27.11 \geq 10 \rightarrow Ok,\ structure\ is\ stable$$

2nd degree magnification factor:

$$\frac{n}{n-1} = 1.04$$

G.3.6 Stiffness

$$\delta_h = \frac{F_{h,trans} * H^3}{3 * E_{c,eff} * I_{column}} = 0.0215m$$ Deflection at the top due to the horizontal force at the top of the column

$$\delta_q = \frac{q_{wind} * w_{column} * H^4}{8 * E_{c,eff} * I_{column}} = 0.0007m$$ Deflection at the top due to wind load at the column

$$\delta_c = \frac{M_y}{C_y} * H = 0.0062m$$ Deflection at the top due to rotation of the foundation slab

Total 2nd order deflection at the top:

$$\delta_{tot} = (\delta_h + \delta_q + \delta_c) * \frac{n}{n-1} = 0.0296m$$

The maximum allowed deflection at the top:

$$\delta_{max} = \frac{H}{500} = 0.03m$$

Deflection check:

$$\delta_{tot} = 0.0296m \leq \delta_{\max} = 0.03m \rightarrow Ok, \ structure \ is \ stiff \ \ enough$$

The 2nd order rotation of the foundation slab:

$$\varphi = \frac{M_y}{C_y} * \frac{n}{n-1} = 0.0004rad$$

The maximum allowed rotation of the foundation slab [23]:

$$\varphi_{\max} = \frac{1}{300} = 0.0033rad$$

Rotation check:

$$\varphi = 0.0004rad \leq \varphi_{\max} = 0.0033rad \rightarrow Ok$$

G.3.7 Foundation piles

Pile force

Total vertical force at the piles, ULS:

$$F_{v,piles} = F_{v,viaduct,uls} + F_{v,column} * \gamma_{G,unfav} + F_{v,fs} * \gamma_{G,unfav} = 16193.35kN$$

Total horizontal force at the piles, ULS:

$$F_h = F_{h,trans} * \gamma_{Q,unfav} + q_{wind} * \gamma_{Q,unfav} * w_{column} * H = 829.01kN$$

Total moment at the foundation slab in transversal direction, ULS:

$$M_y = F_{h,trans} * \gamma_{Q,unfav} * H + 0.5 * q_{wind} * \gamma_{Q,unfav} * w_{column} * H^2 + \delta_{tot} * F_{v,viaduct,uls} = 12166.5kNm$$

Total moment at the foundation slab in longitudinal direction, ULS:

$$M_x = F_{h,long} * \gamma_{Q,unfav} * H = 7340.56kNm$$

The number of piles:

$$n_p = 22$$

The maximum allowed pile force (assumption):

$$P_{\max,allow} = 1200kN \ \ per \ \ pile$$

$$P_v = \frac{-F_{v,piles}}{n_p} = -736.06kN$$ Load on piles due to vertical load

$$P_{my} = \frac{M_y}{C_y} * ny_{\max} * k = +/- 253.47kN$$ Load on the outside piles in transversal direction due to the moment in the transversal direction

$$P_{mx} = \frac{M_x}{C_x} * nx_{\max} * k = +/- 193.17kN$$ Load on the outside piles in longitudinal direction due to the moment in the longitudinal direction

Where:

$$ny_{\max} = 4m$$

$$nx_{\max} = 4m$$

The maximum pile force in the corner piles of the foundation slab is:

$$P_{\max} = P_v - P_{my} - P_{mx} = -1182.7kN \leq P_{\max,allow} = 1200kN \rightarrow Ok$$

The minimum pile force in the corner piles of the foundation slab is:

$$P_{\min} = P_v + P_{my} + P_{mx} = -289.42kN$$

Most likely in all load phases the piles will not be in tension considering the large vertical force.

Pile head moment due to horizontal force

The pile is schematised as a beam of infinite length on one side and is fixed in the foundation slab on the other side. The pile which is supported by linear elastic springs (soil) is subjected to a concentrated horizontal force at the foundation slab. The structural model for a pile subjected to the horizontal force is shown in Figure 203.

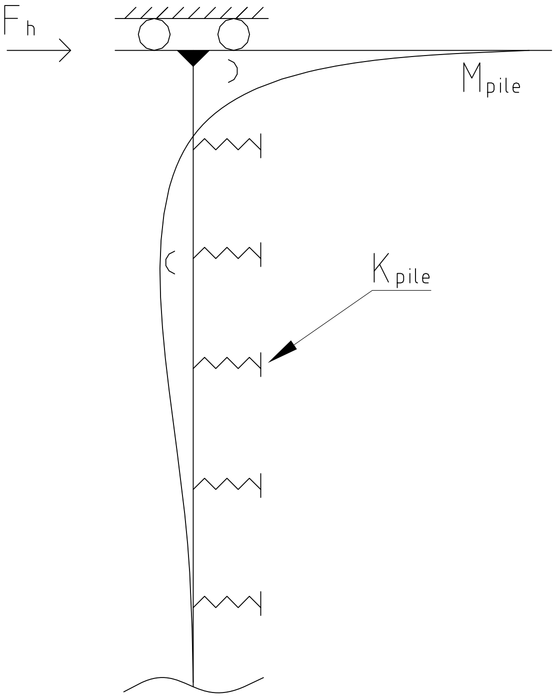


Figure 203: Structural model piles

Characteristics of the piles:

$w_{pile} = 0.42m$	Width of the pile
$h_{pile} = 0.42m$	Depth of the pile
$A_{pile} = w_{pile} * h_{pile} = 0.1764m^2$	Cross-sectional area of the pile
$I_{pile} = \frac{1}{12} * w_{pile} * h_{pile}{}^3 = 0.0026m^4$	Moment of inertia of the pile
$E_{pile} = E_{cm} = 37277.87N/mm^2$	Modulus of elasticity of the pile, assumption of uncracked pile as the vertical pile force is large
$c_{effectivewidth} = 1.5$	Factor for determining the effective width of the pile
$k = 3000kN/m^3$	Modulus of subgrade reaction
$k_{pile} = k * w_{pile} * c_{effectivewidth} = 1890kN/m^2$	Modulus of subgrade reaction of pile

The pile head moment in one pile is [19]:

$$M_{pile} = \frac{\frac{1}{2} * \frac{F_h}{n_p}}{2 * \beta} = 35.63 kNm$$

Where:

$$\beta = \sqrt[4]{\frac{k_{pile}}{4 E_{pile} I_{pile}}} = 0.26 m^{-1}$$

The required reinforcement at one side of the pile is:

$$\frac{M_{pile}}{z} = A_s * f_{yd} \rightarrow A_s = 239.49 mm^2$$

Where:

$z = 0.9 * d = 0.342 m$	Lever arm of internal forces
$d = h_{pile} - c = 0.38 m$	Effective depth
$c = 40 mm$	Concrete cover, see B.3.3
$f_{yd} = f_{yk} / \gamma_s = 435 N/mm^2$	Design yield strength of reinforcement
$f_{yk} = 500 N/mm^2$	Characteristic yield strength of reinforcement
$\gamma_s = 1.15$	Partial factor for reinforcing steel

The total required reinforcement in a pile is:

$$A_{s,tot} = 4 * A_s = 957.94 mm^2$$

The reinforcement percentage in a pile then becomes:

$$\omega_0 = \frac{A_{s,tot}}{A_{pile}} * 100\% = 0.54\%$$

The maximum reinforcement percentage in a column/pile is:

$$\omega_{max} = 4\%$$

Reinforcement percentage check

$$\omega_0 = 0.54\% \leq \omega_{max} = 4\% \rightarrow Ok$$

G.3.8 Stresses in column

Total vertical force at the bottom of the column, ULS:

$$F_{v,tot,bot,column} = F_{v,viaduct,uls} + F_{v,column} * \gamma_{G,unfav} = 10829.73 kN$$

Total moment at the bottom of the column in transversal direction, ULS:

$$M_y = F_{h,trans} * \gamma_{Q,unfav} * H + 0.5 * q_{wind} * \gamma_{Q,unfav} * w_{column} * H^2 + \delta_{tot} * F_{v,viaduct,uls} = 12166.5 kNm$$

The compressive stress in the column due to the vertical force is:

$$\sigma_n = \frac{-F_{v,tot,bot,column}}{A_{column}} = -2.54 N/mm^2$$

The stress in the column due to the moment in transversal direction is:

$$\sigma_m = \pm \frac{M_y}{W_{column}} = \pm 8.11 N/mm^2$$

The maximum compressive stress in the column is:

$$\sigma_{c\max} = \sigma_n - \sigma_m = -10.66 N/mm^2$$

The minimum compressive stress in the column is:

$$\sigma_{c\min} = \sigma_n + \sigma_m = 5.57 N/mm^2$$

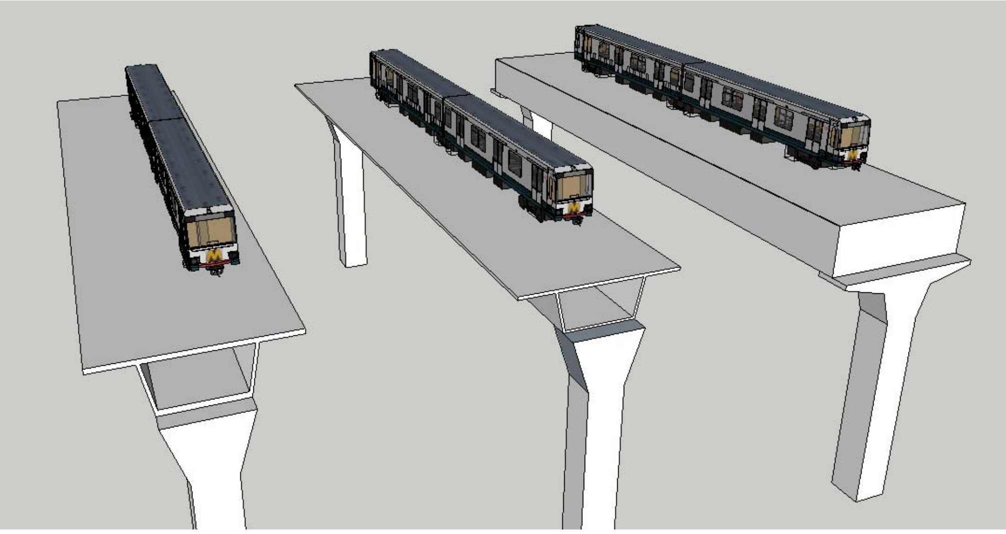
There arises tension in the column which means that the section is cracked. The assumption that the effective modulus of elasticity of concrete should be taken into account is thus correct.

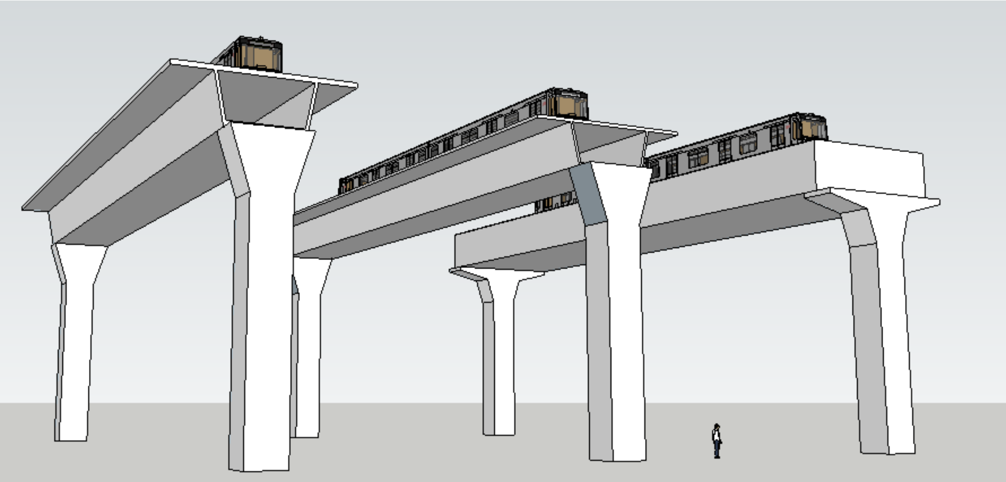
G.3.9 Overview weight contribution of elements

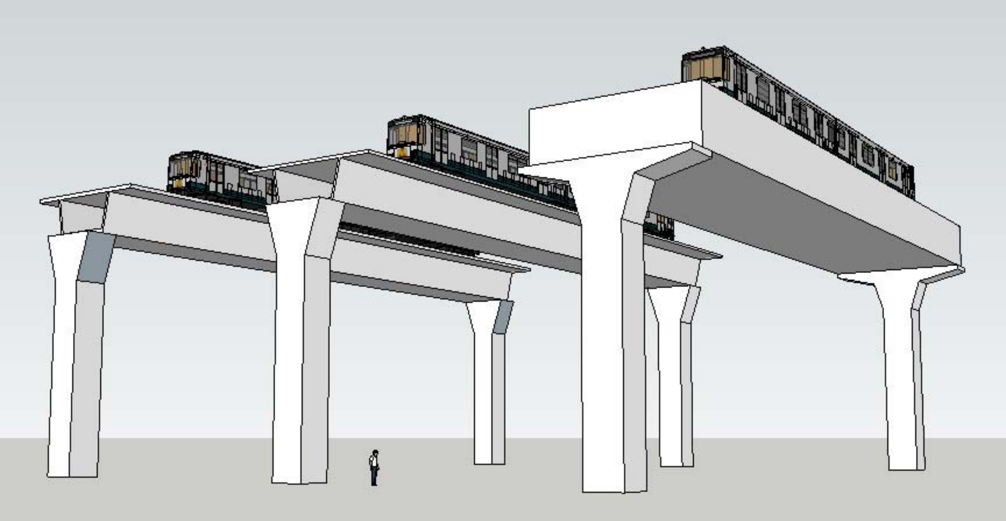
In the table below the contribution to the total vertical load at the piles is given for the different loads.

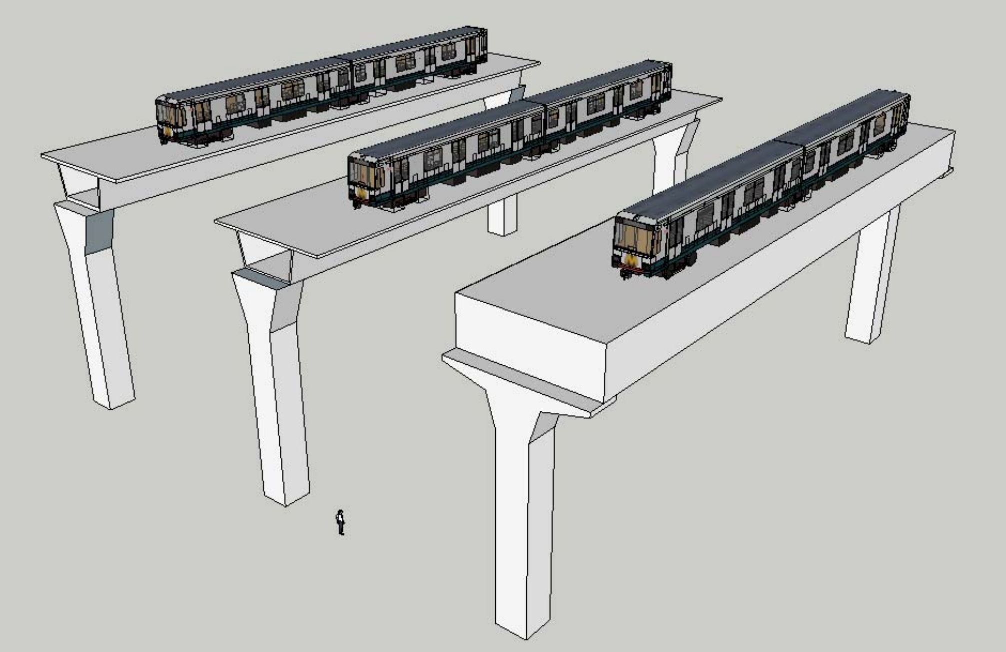
Loads		*Value*		*Percentage of the total vertical load at the piles*	
Dead load FRP girder	$g_{dead} * L$	1551.6	kN	13.26	%
Total load sandwich girder fully loaded	$q_{tot} * L$	5723.55	kN	48.9	%
Dead load column	$F_{v,column} + Q_{v,widening}$	2007.02	kN	17.15	%
Dead load foundation slab	$F_{v,fs}$	3973.05	kN	33.95	%
Total vertical load at the piles	$q_{tot} * L + F_{v,column} + Q_{v,widening} + F_{v,fs}$	11703.62	kN	100	%

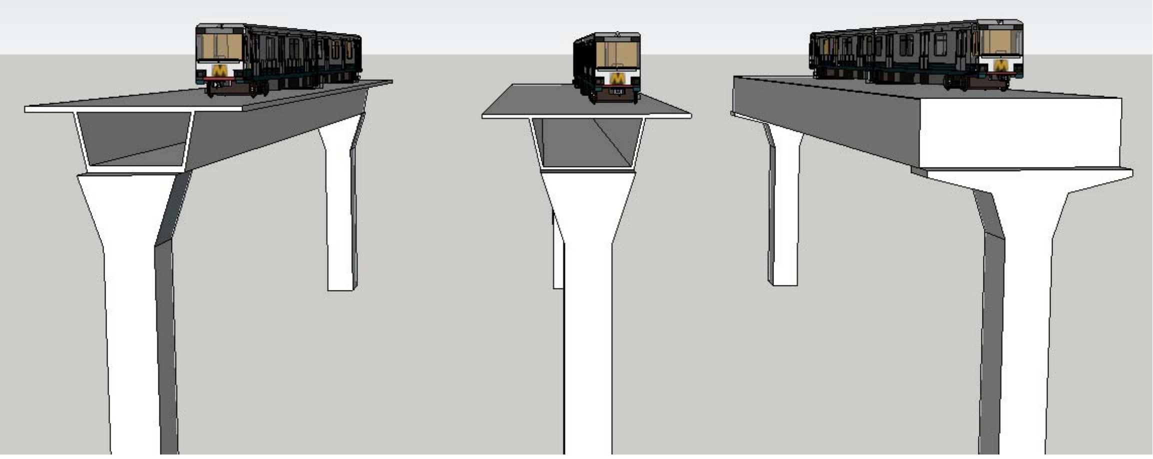
Appendix H: 3D-impressions three designs





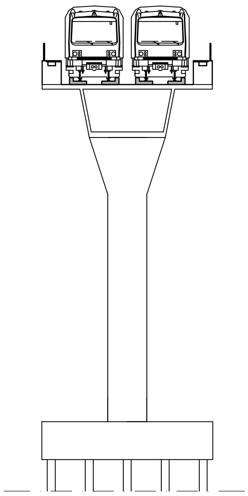
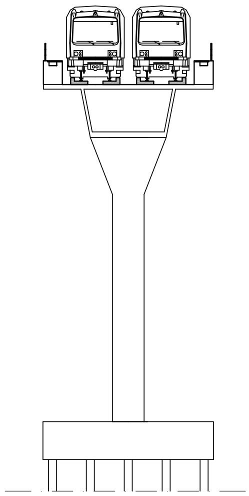
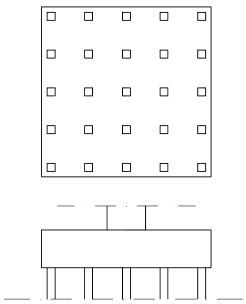
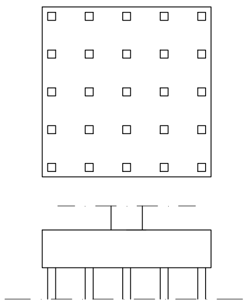






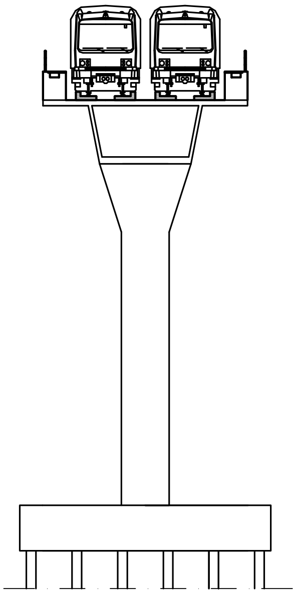
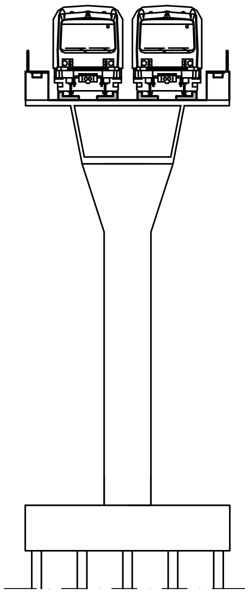
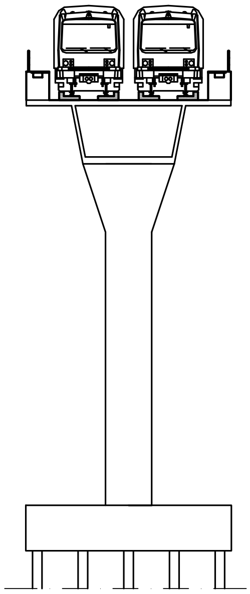
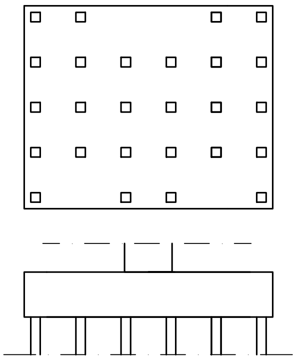
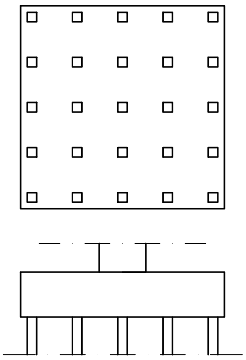
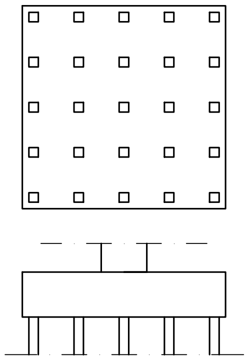
Appendix I: Alternative solutions and assumptions for the substructure

I.1 The application of columns made of UHPC instead of concrete

	Columns made of concrete C50/60	**Columns made of UHPC C180**
Cross-section of the elevated metro structure in transversal direction		
Pile foundation, top- and side-view		

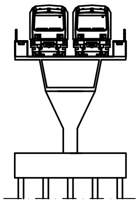
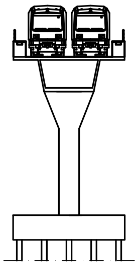
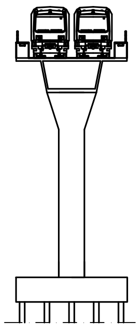
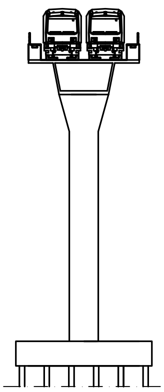
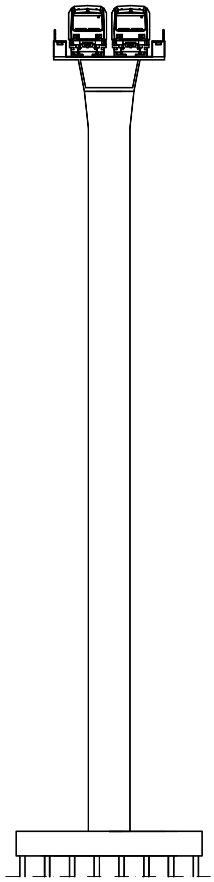
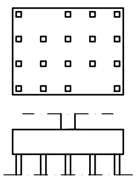
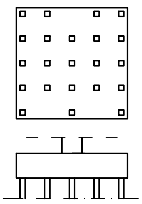
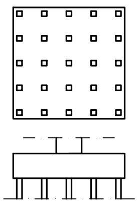
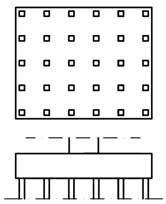
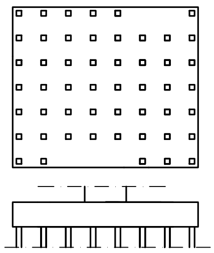
		Columns made of concrete C50/60	**Columns made of UHPC C180**	Unity
Width of the column	w_{column}	2.06	1.67	m
Number of piles	n_p	25	25	
Length foundation slab	L_{fs}	9	9	m
Width foundation slab	W_{fs}	9	9	m
Thickness foundation slab	T_{fs}	2	2	m

I.2 Changing the spring stiffness of the piles

	k = 50000 kN/m	**k = 100000 kN/m**	**k = 150000 kN/m**
Cross-section of the elevated metro structure in transversal direction			
Pile foundation, top- and side-view			

		k = 50000 kN/m	**k = 100000 kN/m**	**k = 150000 kN/m**	Unity
Width of the column	w_{column}	2.09	2.06	2.02	m
Number of piles	n_p	26	25	25	
Length foundation slab	L_{fs}	11	9	9	m
Width foundation slab	W_{fs}	9	9	9	m
Thickness foundation slab	T_{fs}	2	2	2	m

I.3 Changing the height of the columns

	H = 5 m	H = 10 m	H = 15 m	H = 20 m	H = 40 m
Cross-section of the elevated metro structure in transversal direction					
Pile foundation, top- and side-view					

		H = 5 m	H = 10 m	H = 15 m	H = 20 m	H = 40 m	Unity
Width of the column	w_{column}	1.14	1.64	2.06	2.35	3.35	m
Number of piles	n_p	18	22	25	30	51	
Length foundation slab	L_{fs}	9	9	9	11	15	m
Width foundation slab	W_{fs}	7	9	9	9	13	m
Thickness foundation slab	T_{fs}	2	2	2	2	2	m

Appendix J: Costs

J.1 Costs elevated metro structure C50/60

Viaduct C50	45.00	m										
Omschrijving	aantal	hoogte in m	lengte in m	dikte in m	breedte in m	diverse		totaal	ehd	prijs per eenheid	prijs per stuk	Prijs per per onderdeel
Poeren												€ 148,438
grondwerk	162.00					1.500		243.0	st	€ 25.00	€ 6,075	
palen	25.00							25.0	st	€ 1,250.00	€ 31,250	
poer	162.00							162.0	m3	€ 350.00	€ 56,700	
kolom	72.55							72.6	m3	€ 750.00	€ 54,413	
Viaduct								195.2	kg	€ 1,511.86		€ 295,115
bekisting	1.00		45.00		8.96		403.20	1,087.2	m2	€ 100.00	€ 108,720	
	1.00		45.00		4.00		180.000					
	4.00		45.00		2.80		504.000					
beton (€ 150/m3 koop)	195.20							195.2	m3	€ 175.00	€ 34,160	
wapening	195.20					75.000		14,640.0	kg	€ 1.20	€ 17,568	
voorspanwapening	222.00		45.00			1.520		15,181.8	kg	€ 5.00	€ 75,909	
ontkisten	1.0							1.0	pst	€ 3,488.00	€ 3,488	
transport	15.0							15.0	pst	€ 350.00	€ 5,250	
oplegblokken	2.0							2.0	st	€ 250.00	€ 500	
montage (reservering)	1.0							1.0	pst	€ 30,000.00	€ 30,000	
voegen	4.34			0.30		1500		1,952.0	dm	€ 10.00	€ 19,520	
												€ 443,553
										controle	€ 443,553	
										Kosten veld		€ 443,553

kosten op investeringsniveau		
direct		€ 443,553
nader te detailleren	15.0%	€ 66,533
indirect	27.5%	€ 121,977
nader te detailleren	15.0%	€ 18,297
direct + indirect		€ 650,359
object onvoorzien	15.0%	€ 97,554
bouwkosten		€ 747,913
engineering	16.0%	€ 119,666
overige bijkomende kosten	6.0%	€ 44,875
basisraming		€ 912,454
project onvoorzien	15.0%	€ 136,868
investeringskosten		**€ 1,049,322**

J.2 Costs elevated metro structure C180

Viaduct C180	45.00	m											
Omschrijving	aantal	hoogte in m	lengte in m	dikte in m	breedte in m	diverse			totaal	ehd	prijs per eenheid	prijs per stuk	Prijs per per onderdeel
Poeren													€ 144,138
grondwerk	162.00					1.500			243.0	st	€ 25.00	€ 6,075	
palen	23.00								23.0	st	€ 1,250.00	€ 28,750	
poer	162.00								162.0	m3	€ 350.00	€ 56,700	
kolom	70.15								70.2	m3	€ 750.00	€ 52,613	
Viaduct									129.4	kg	€ 2,313.88		€ 299,416
bekisting	1.00		45.00		8.96			403.20	1,017.0	m2	€ 100.00	€ 101,700	
	1.00		45.00		4.00			180.000					
	4.00		45.00		2.41			433.800					
beton (€ 500/m3 koop)	129.40								129.4	m3	€ 448.09	€ 57,982	
wapening	129.40					75.000			9,705.0	kg	€ 1.20	€ 11,646	
voorspanwapening	222.00		45.00			1.520			15,181.8	kg	€ 5.00	€ 75,909	
ontkisten	1.0								1.0	pst	€ 3,488.00	€ 3,488	
transport	15.0								15.0	pst	€ 350.00	€ 5,250	
oplegblokken	2.0								2.0	st	€ 250.00	€ 500	
montage (reservering)	1.0								1.0	pst	€ 30,000.00	€ 30,000	
voegen	2.88			0.30		1500			1,294.0	dm	€ 10.00	€ 12,940	
													€ 443,553
											controle	€ 443,553	
											Kosten veld		€ 443,553

kosten op investeringsniveau		
direct		€ 443,553
nader te detailleren	15.0%	€ 66,533
indirect	27.5%	€ 121,977
nader te detailleren	15.0%	€ 18,297
direct + indirect		€ 650,360
object onvoorzien	15.0%	€ 97,554
bouwkosten		€ 747,914
engineering	16.0%	€ 119,666
overige bijkomende kosten	6.0%	€ 44,875
basisraming		€ 912,454
project onvoorzien	15.0%	€ 136,868
investeringskosten		**€ 1,049,323**

J.3 Costs elevated metro structure FRP

Viaduct FRP	45.00	m											
Omschrijving	aantal	hoogte in m	lengte in m	dikte in m	breedte in m	diverse			totaal	ehd	prijs per eenheid	prijs per stuk	Prijs per per onderdeel
Poeren													€ 151,700
grondwerk	162.00					1.500			243.0	st	€ 25.00	€ 6,075	
palen	22.00								22.0	st	€ 1,250.00	€ 27,500	
poer	162.00								162.0	m3	€ 350.00	€ 56,700	
kolom	81.90								81.9	m3	€ 750.00	€ 61,425	
Viaduct													€ 291,853
VVK sandwich ligger	1.00		45.00		9.00			405.00	405.0	m2	€ 719.39	€ 291,353	
oplegblokken	2.0								2.0	st	€ 250.00	€ 500	
													€ 443,553
											controle	€ 443,553	
											Kosten veld		€ 443,553

kosten op investeringsniveau		
direct		€ 443,553
nader te detailleren	15.0%	€ 66,533
indirect	27.5%	€ 121,977
nader te detailleren	15.0%	€ 18,297
direct + indirect		€ 650,360
object onvoorzien	15.0%	€ 97,554
bouwkosten		€ 747,914
engineering	16.0%	€ 119,666
overige bijkomende kosten	6.0%	€ 44,875
basisraming		€ 912,454
project onvoorzien	15.0%	€ 136,868
investeringskosten		**€ 1,049,323**

Appendix K: Comparison fatigue verifications for concrete

K.1 General

The optimisation process of the concrete box girder showed that fatigue of the concrete is normative for the optimal design. In this design study the fatigue verification according to Annex NN.3.2 NN.112 [12] (Eurocode) is applied. The normative section of the box girder is at the deviation blocks at the bottom side. Here: the permanent as well as the maximum compressive stress at t=∞ is $\sigma_{\max} = 13.12 N/mm^2$ and the minimum compressive stress is $\sigma_{\min} = 8.90 N/mm^2$. As these compressive stresses are not very large and the fluctuation is small it is however quite remarkable that fatigue of the concrete is normative for the design. The more as the box girder is always fully prestressed and no tension stresses arise. For this reason, this appendix deals with comparison of different fatigue verifications for concrete in order to check whether fatigue is indeed that normative. First, the applied verification for the box girder design is described. The second verification is another fatigue verification according the Eurocode. The last two fatigue verifications are according NEN 6723 [14]. The conclusions of the comparison between the four verifications are treated in the last paragraph. The considered section of the box girder is at the deviation blocks at the bottom side.

K.2 Verification according to Annex NN.3.2 [12] (Eurocode)

The fatigue verification for concrete is calculated according to Equation NN.112 [12]:

$$14 * \frac{1 - E_{cd,\max,equ}}{\sqrt{1 - R_{equ}}} \geq 6 \rightarrow \frac{6}{14} * \sqrt{1 - R_{equ}} + E_{cd,\max,equ} = 0.998 \leq 1.0 \rightarrow Ok$$

Where:

$R_{equ} = \frac{E_{cd,\min,equ}}{E_{cd,\max,equ}} = 0.75$ Stress ratio

$E_{cd,\min,equ} = \gamma_{sd} \frac{\sigma_{cd,\min,equ}}{f_{cd,fat}} = 0.59$ Minimum compressive stress level

$E_{cd,\max,equ} = \gamma_{sd} \frac{\sigma_{cd,\max,equ}}{f_{cd,fat}} = 0.78$ Maximum compressive stress level

$f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd} \left(1 - \frac{f_{ck}}{250}\right) = 19.27 N/mm^2$ Design fatigue strength of concrete

$\beta_{cc}(t) = \exp\left\{ s \left[1 - \left(\frac{28}{t} \right)^{1/2} \right] \right\} \rightarrow \beta_{cc}(t28) = 1.0$ Coefficient for concrete strength at first load application

$k_1 = 0.85$ Recommended value for $N = 10^6$ cycles

$\gamma_{sd} = 1.15$ Is the partial factor for model uncertainty for action/action effort

$\sigma_{cd,\max,equ} = \sigma_{c,perm} - \lambda_c(\sigma_{c,\max} - \sigma_{c,perm}) = 13.12 N/mm^2$ Upper stress of the ultimate amplitude for N cycles

$\sigma_{cd,\min,equ} = \sigma_{c,perm} - \lambda_c(\sigma_{c,perm} - \sigma_{c,\min}) = 9.80 N/mm^2$ Lower stress of the ultimate amplitude for N cycles

$\sigma_{c,perm} = 13.12 N/mm^2$ Permanent stress, without variable load, see Eq. (14)

$\sigma_{c,\max} = 13.12 N/mm^2$ — Maximum compressive stress, without variable load, see Eq. (14)

$\sigma_{c,\min} = 8.90 N/mm^2$ — Minimum compressive stress, with variable load, see Eq. (10)

$\lambda_c = \lambda_{c,0} * \lambda_{c,1} * \lambda_{c,2,3} * \lambda_{c,4} = 0.79$ — Correction factor to calculate the upper and lower stresses of the damage equivalent stress

$\lambda_{c,0} = 0.94 + 0.2 \frac{\sigma_{c,perm}}{f_{cd,fat}} \geq 1 = 1.08$ — Is a factor to take account of the permanent stress

$\lambda_{c,1} = 0.75$ — Is a factor accounting for element type, see Table NN.3 [12]: (1) compression zone, s* standard traffic mix and simply supported beam

$\lambda_{c,2,3} = 1 + \frac{1}{8}\log\left[\frac{Vol}{25*10^6}\right] + \frac{1}{8}\log\left[\frac{N_{years}}{100}\right] = 0.98$ — Is a factor to take account of the traffic volume and the design life of the bridge

$Vol = Q_{metro} / g * 6 * 24 * 365 = 15848367 tonnes / year / track$ — Assumption of 6 metros per hour

$Q_{metro} = q_{mob} * 116m = 2958kN$ — 116 metres is the length of a metro

$N_{years} = 100 years$ — Is the design life of the viaduct

$\lambda_{c,4} = 1.0$ — Is a factor to be applied when the structure is loaded by more than one track, is the most conservative value

K.3 Verification according to the National Annex [13] (Eurocode)

The fatigue verification for concrete is calculated according to Miner's rule taking into account Equation 6.106.b [13]:

$$\sum_{i=1}^{m} \frac{n_i}{N_i} = 0.13 \leq 1 \rightarrow Ok$$

Where:

$m = 1$ — Number of intervals with constant amplitude

$n = 2 * 6 * 24 * 365 * 100 = 10{,}512{,}000$ — Actual number of constant amplitude cycles in interval "i": Assumption of 6 metros per hour per track

$N = 10\wedge\left(14\left(\frac{1 - E_{cd,\max}}{\sqrt{1-R}}\right)\right) = 83{,}099{,}029$ — Ultimate number of constant amplitude cycles in interval "i" that can be carried before failure

$R = \frac{E_{cd,\min}}{E_{cd,\max}} = 0.68$ — Stress ratio

$E_{cd,\min} = \frac{\sigma_{cd,\min}}{f_{cd,fat}} = 0.46$ — Minimum compressive stress level

$E_{cd,\max} = \frac{\sigma_{cd,\max}}{f_{cd,fat}} = 0.68$ — Maximum compressive stress level

$f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd} \left(1 - \frac{f_{ck}}{250}\right) = 19.27 N/mm^2$ Design fatigue strength of concrete

$\sigma_{cd,\max} = 13.12 N/mm^2$ Upper stress in a cycle, see Eq. (14)

$\sigma_{cd,\min} = 8.90 N/mm^2$ Lower stress in a cycle, see Eq. (10)

K.4 Verification according to 9.6.2.2.a.1 [14] (NEN 6723)

The fatigue verification for concrete is calculated according to 9.6.2.2.a.1 [14]. This verification holds for road traffic:

$$S_{b;d;\max} = 13.12 N/mm^2 \leq S_u(n) = 19.22 N/mm^2 \rightarrow Ok$$

Where:

$S_{b;d;\max} = \sigma'_{b;d;\max} = 13.12 N/mm^2$

$\sigma'_{b;d;\max} = 13.12 N/mm^2$ Upper stress in a cycle, see Eq. (14)

$S_u(n) = f'_{b;u;v}(n) = 18.14 N/mm^2$

$f'_{b;u;v}(n) = (1 - 0.1 * \sqrt{1-R} * \log n) * f'_{b;v} = 19.22 N/mm^2$ Design value of concrete compressive strength at n cycles

$R = \frac{\sigma'_{b;d;\min}}{\sigma'_{b;d;\max}} = 0.68$ Stress ratio

$\sigma'_{b;d;\min} = 8.90 N/mm^2$ Lower stress in a cycle, see Eq. (10)

$n = 2*6*24*365*100 = 10{,}512{,}000$ Actual number of constant amplitude cycles in interval "i": Assumption of 6 metros per hour per track

$f'_{b;v} = f'_{b;rep;v} / \gamma_m = 31.88 N/mm^2$ Design fatigue strength of concrete

$f'_{b;rep;v} = 0.5*\left(f'_{b;rep;k} - 0.85*30\right) + 0.85*30 = 38.25 N/mm^2$ Representative fatigue compressive strength of concrete

$f'_{b;rep;k} = 0.85 * f'_{c;k} = 51.0 N/mm^2$ Representative short term concrete compressive strength

$\gamma_m = 1.2$ Partial factor for concrete

K.5 Verification according to 9.6.2.2.a.2 [14] (NEN 6723)

The fatigue verification for concrete is calculated according to Miner's rule in 9.6.2.2.a.2 [14]. This verification holds for road as well as railway traffic:

$$\sum_{i=1}^{m} \frac{n_i}{N_i} = 0.00039 \leq 1 \rightarrow Ok$$

Where:

$m = 1$ Number of intervals with constant amplitude

$n = 2*6*24*365*100 = 10{,}512{,}000$ Actual number of constant amplitude cycles in interval "i": Assumption of 6 metros per hour per track

$N = 10 \wedge \left(\frac{10}{\sqrt{1-R}} \left(1 - \frac{\sigma'_{b;d;\max}}{f'_{b;v}} \right) \right) = 10 \wedge \left(10 \left(\frac{1 - E_{cd,\max}}{\sqrt{1-R}} \right) \right) = 2.69 * 10^{10}$ Ultimate number of constant amplitude cycles in interval "i" that can be carried before failure

$R = \frac{\sigma'_{b;d;\min}}{\sigma'_{b;d;\max}} = 0.68$ Stress ratio

$\sigma'_{b;d;\max} = 13.12 N / mm^2$ Upper stress in a cycle, see Eq. (14)

$\sigma'_{b;d;\min} = 8.90 N / mm^2$ Lower stress in a cycle, see Eq. (10)

$E_{cd,\max} = \frac{\sigma'_{b;d;\max}}{f'_{b;v}} = 0.41$ Maximum compressive stress level

$f'_{b;v} = f'_{b;rep;v} / \gamma_m = 31.88 N / mm^2$ Design fatigue strength of concrete

$f'_{b;rep;v} = 0.5 * \left(f'_{b;rep;k} - 0.85 * 30 \right) + 0.85 * 30 = 38.25 N / mm^2$ Representative fatigue compressive strength of concrete

$f'_{b;rep;k} = 0.85 * f'_{c;k} = 51.0 N / mm^2$ Representative short term concrete compressive strength

$\gamma_m = 1.2$ Partial factor for concrete

K.6 Comparison different verifications

The applied fatigue verification according to Annex NN.3.2 [12] is a simplified approach based on λ values, which may be used for railway bridges. This simplified approach results in a conservative fatigue verification, which is even normative for the design. Another fatigue verification according to the National Annex of the Eurocode (see K.3) shows that fatigue is not that critical. The fatigue verification according to 9.6.2.2.a.2 [14] (NEN 6723) for railway traffic also uses Miner's rule, just as in K.3, but results in a unity check which is much easier satisfied than the one according the Eurocode (compare K.3 and K.5). The reason for this difference is twofold:

- The factor 14 instead of 10 in the formula of the ultimate number of constant amplitude cycles in interval "i" that can be carried before failure
- The difference in the design fatigue strength of the concrete

This is shown in the overview below:

$N_{Eurocode} = 10 \wedge \left(14 \left(\frac{1 - E_{cd,\max}}{\sqrt{1-R}} \right) \right) = 83{,}099{,}029$ VS $N_{NEN6723} = 10 \wedge \left(10 \left(\frac{1 - E_{cd,\max}}{\sqrt{1-R}} \right) \right) = 2.69 * 10^{10}$

$E_{cd,\max,Eurocode} = \frac{\sigma_{cd,\max}}{f_{cd,fat}} = 0.68$ VS $E_{cd,\max,NEN6723} = \frac{\sigma'_{b;d;\max}}{f'_{b;v}} = 0.41$

$f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd} \left(1 - \frac{f_{ck}}{250} \right) = 19.27 N / mm^2$ VS $f'_{b;v} = f'_{b;rep;v} / \gamma_m = 31.88 N / mm^2$

Especially the difference in the design fatigue strength of the concrete according the two codes has a large contribution to the difference in the ultimate number of constant amplitude cycles in interval "i" that can be carried before failure. Notice that the design fatigue strength of concrete according the Eurocode: $f_{cd,fat} = 19.27 N / mm^2$ looks like the design value of the concrete compressive strength at n cycles according NEN6723: $f'_{b;u;v}(n) = 19.22 N / mm^2$ (see K.4). It is however contradictory if $f'_{b;u;v}(n)$ is meant with $f_{cd,fat}$ in the Eurocode verification as $f'_{b;u;v}(n)$ already includes n cycles of loading. Therefore, applying Miner's rule with $f'_{b;u;v}(n)$ is not correct as then the fatigue loading (n cycles) is taken into account two times. Because the difference in the design fatigue strength of the concrete according the two codes is large, which has a large impact on the fatigue verification, it is recommended to verify the formula for the design fatigue strength of the concrete: $f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd}\left(1 - \frac{f_{ck}}{250}\right)$ according the Eurocode.

Nonetheless, the other fatigue verifications for concrete show that fatigue is not normative for the box girder design. In the optimisation process of the concrete box girder the conservative fatigue verification according to Annex NN.3.2 [12] is taken into account. In reality the fatigue verification for concrete is thus not normative. When the fatigue verification is not normative for the box girder the verification of the ultimate resistance moment of the box girder at t=0 becomes normative. The result of the optimal concrete box girder taking into account the new normative verification is shown below. Notice that the difference with the optimal design presented in Chapter 4 is small and the difference in substructure between the two designs is even marginal (see Chapter 9). The optimal design of the concrete box girder shown below is however not taken along in this design study. This is because this box girder design does not results in a radically different design of the elevated metro structure except that the normative verification for the box girder is not fatigue but the ultimate resistance moment of the box girder at t=0.

The cross-section of the box girder is shown in Figure 204, where:

Length span	L	45	m
Depth box girder	H	2.75	m
Width top flange	b_{tf}	8.96	m
Thickness top flange	t_{tf}	0.25	m
Width web	b_w	0.16	m
Width bottom flange	b_{bf}	4	m
Thickness bottom flange	t_{bf}	0.25	m
Width box top side	b_{boxts}	5	m
Cantilever length top flange	L_{cant}	1.98	m
Depth webs	H_{box}	2.25	m

Angle of webs with vertical axis $\alpha_w = \tan^{-1}\left(\frac{(b_{boxts} - b_{bf})/2}{H_{box} + t_{bf}}\right) = 11.31°$

Furthermore:

Dead load of the concrete box girder:	$g_{dead} = 97.12 kN/m$
Distance of deviation blocks to supports	$a = 16m$
Width of the column	$w_{column} = 2.05m$
Number of piles	$n_p = 25$

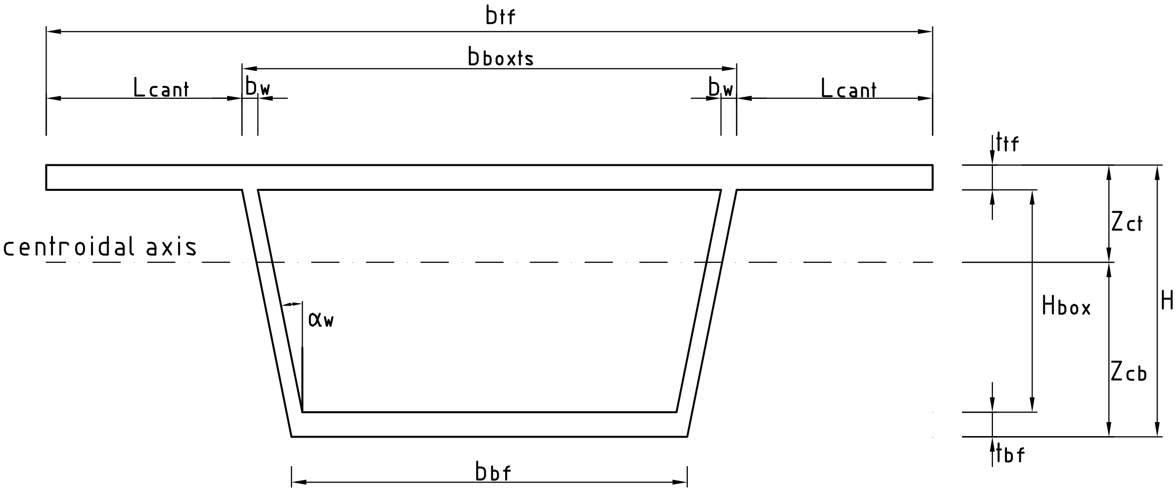


Figure 204: Cross-section of the box girder

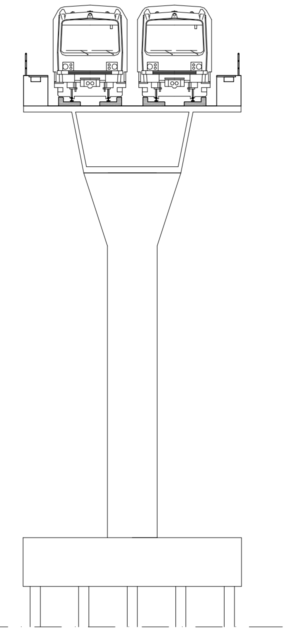


Figure 205: Cross-section of the elevated metro structure in transversal direction

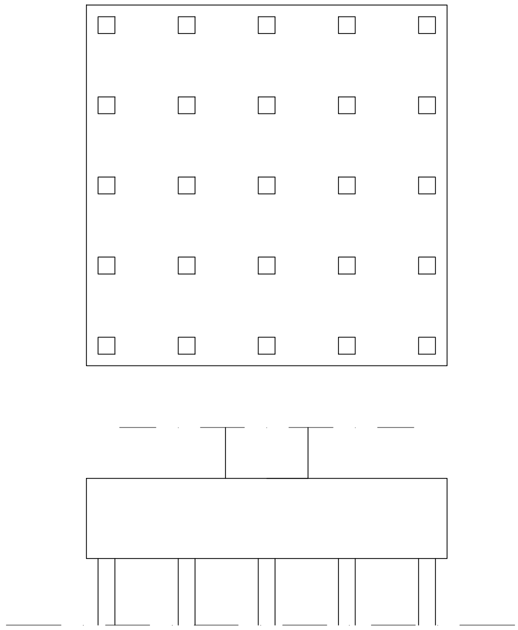


Figure 206: Pile foundation, top- and side-view

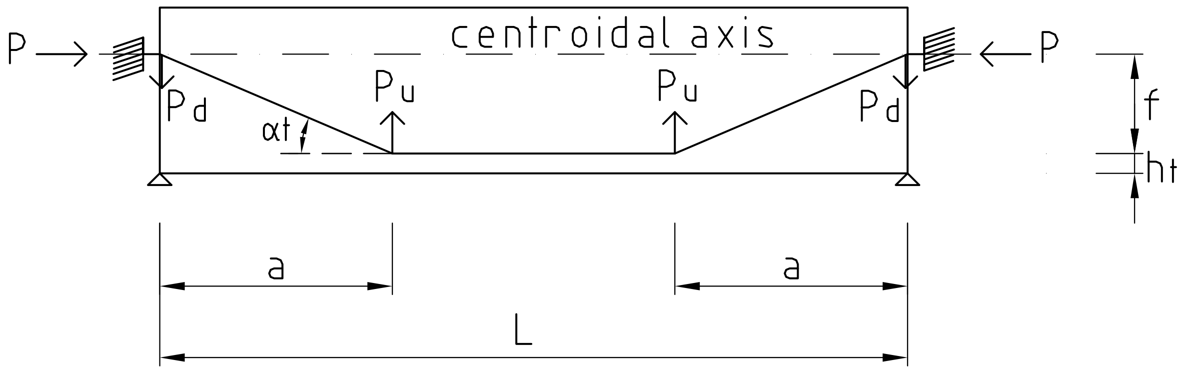


Figure 207: Layout external prestressing tendons