

"The way forward for the Eurocodes implementation in the Balkans"

10-11 October 2018, Tirana

Designers' experience in using the Eurocodes

Pietro CROCE

Convenor of CEN/TC 250 HG Bridges

University of Pisa

EN 1991-2 – Traffic loads on bridges (background)

SAMPLES OF EUROPEAN TRAFFIC DATA

	<i>Cars</i>	<i>Lorries</i>	<i>% intervehicle distance<100 m</i>
<i>Brohltal (D)</i>	11126	4793	26.7
<i>Garonor (F-1982)</i>	--	2570	32.6
<i>Garonor (F-1984)</i>	--	3686	32.3
<i>Auxerre (slow lane) (F)</i>	8158	2630	18
<i>Auxerre (slow lane) (F)</i>	1664	153	8.5
<i>Fiano R. (I)</i>	8500	4000	26.1
<i>Piacenza (I)</i>	8500	5000	30.9
<i>Sasso M. (I)</i>	7500	3500	24.3

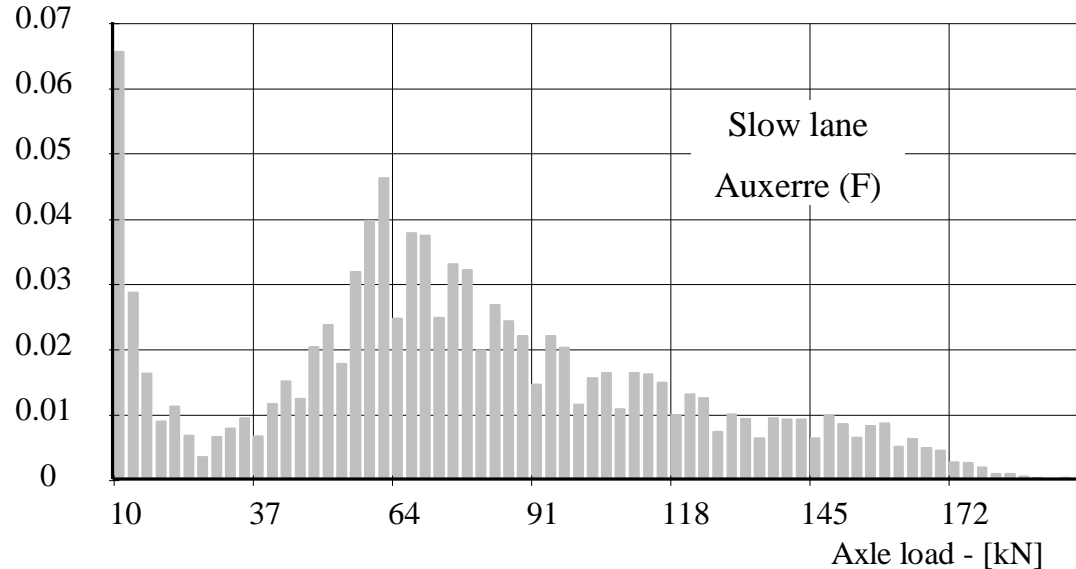
Table 1. Daily traffic flows per lane

	<i>Lorries (%) (2 Axles)</i>	<i>Lorries (%) (>2 Axles)</i>	<i>Articulated lorries (%)</i>	<i>Lorries with trailer (%)</i>
<i>Brohltal (D)</i>	16.6	1.6	40.2	41.6
<i>Garonor (F-1982)</i>	38.6	2.6	47.6	11.2
<i>Garonor (F-1984)</i>	47.5	2.2	44.3	6.0
<i>Auxerre (slow lane) (F)</i>	22.7	1.3	65.2	10.8
<i>Auxerre (fast lane) (F)</i>	27.6	3.5	58.4	10.5
<i>Fiano R. (I)</i>	41.4	7.0	29.0	22.6
<i>Piacenza (I)</i>	35.3	7.5	35.8	21.4
<i>Sasso M. (I)</i>	40.1	10.0	30.2	19.7

Table 2. Composition of the commercial traffic

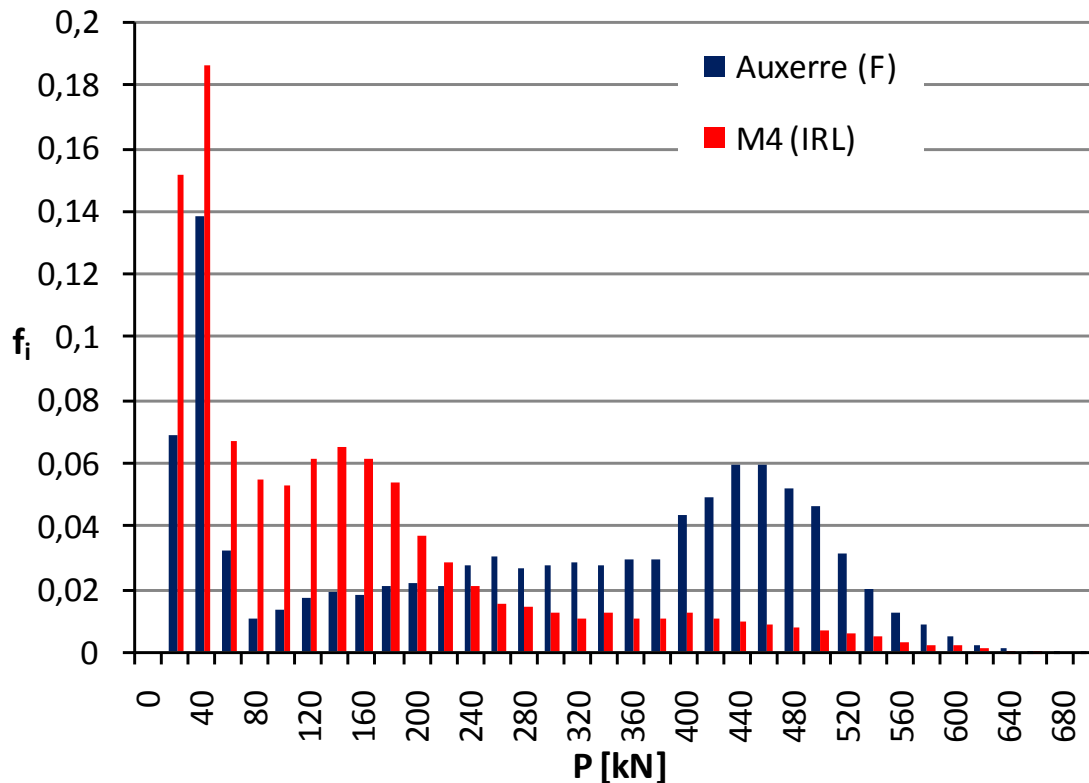
EN 1991-2: TRAFFIC LOADS ON BRIDGES

Traffic measurements:



Histogram of the axle load frequency – Auxerre slow lane – lorries

Traffic measurements:



Histograms of the truck gross weight – Auxerre slow lane and M4 motorway (Ireland)

Long and heavy vehicles - MOERDIJK

Speed	Length	Weight	1 st axle load	2 nd axle load
[m/s]	[m]	[kN]	[kN]	[kN]
30.6	21.02	707	398	309
3.3	13.05	613	208	405
3.3	62.02	684	318	366
27.2	11.32	689	353	336

Unreliable data

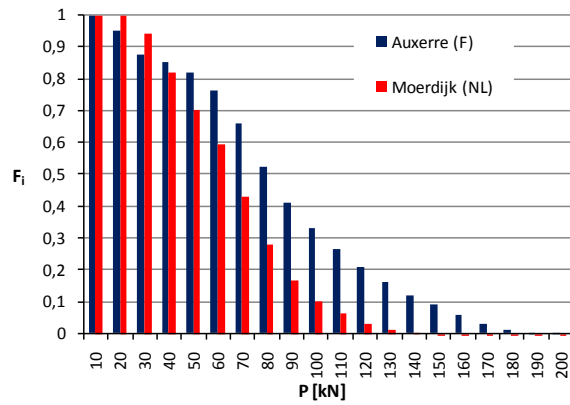
WIM-NL vehicle sub-classes

sub-class	symbol
B11	
B11A1	
B11A2	
B12	
B12A1	
B12A2	
B21	
R111111	
R1111111	
R11112	
R111121	
R11113	
R11211	
R112121	
R1122	
R11221	
R1123	
R121221	
R12211	
R1222	
R12221	
R1223	
T1101	
T1102	
T1103	
T1104	
T11011	
T11021	
T110111	
T1101111	

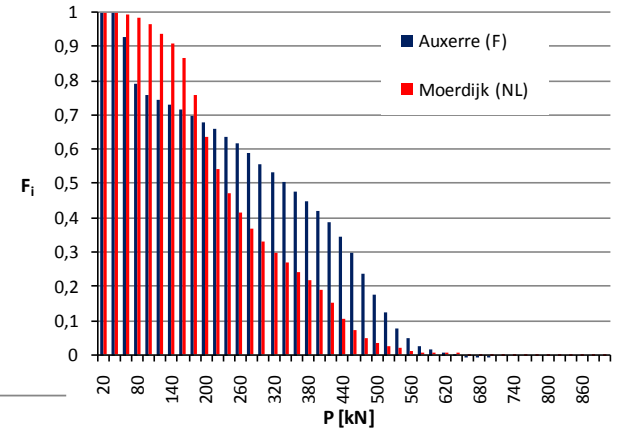
WIM-NL vehicle sub-classes

sub-class	symbol
T1201	
T1202	
T1203	
T1204	
T12011	
T12021	
T120111	
T1201111	
T21011	
V11	
V11A1	
V11A2	
V11A11	
V11A12	
V12	
V12A2	
V12A11	
V12A12	
V13	
V21	
V22	
V22A2	
V22A11	
V22A12	
V111	
V211	
V1111	
O (* others*)	

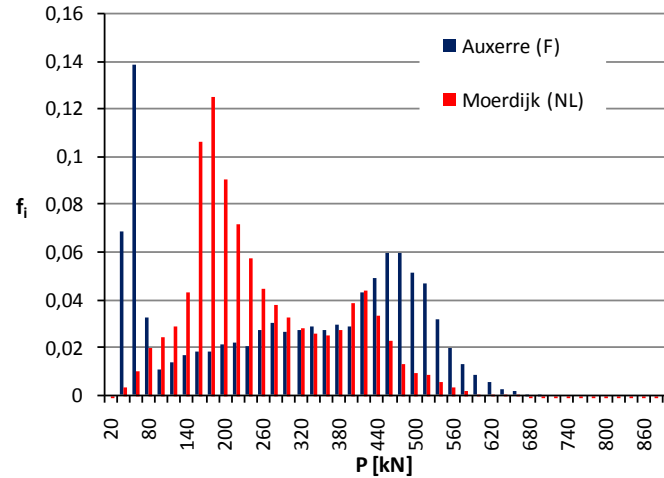
Modified: 24-08-2006



Axle loads spectra



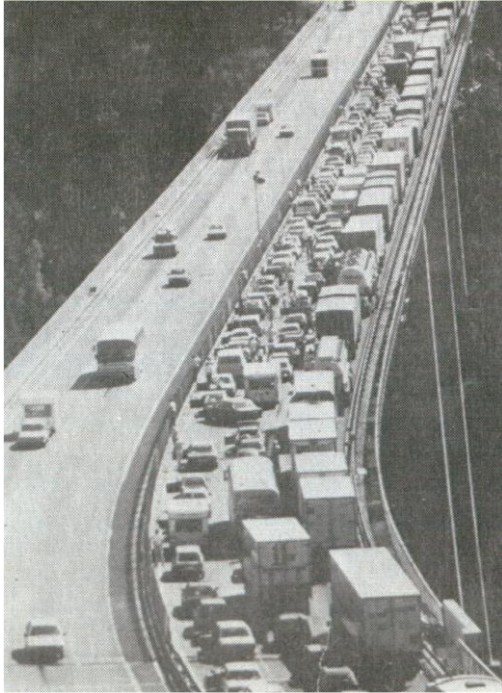
Total loads spectra



Total load distributions

EN 1991-2: TRAFFIC LOADS ON BRIDGES

Extreme traffic scenarios



**Traffic jam on the Europa Bridge
(from Tschermmenegg)**



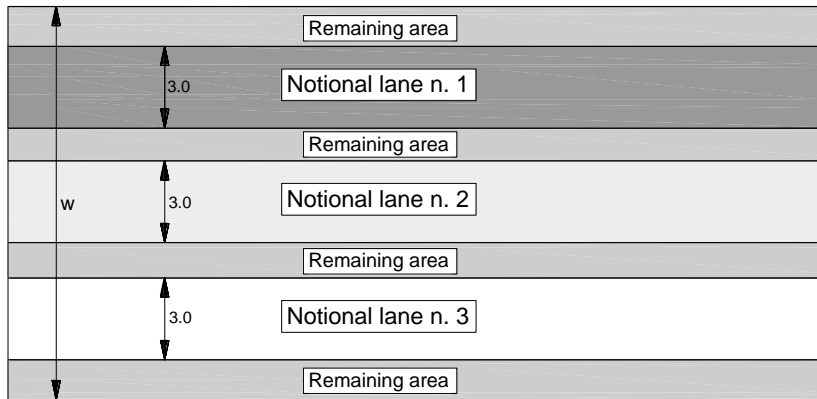
EN 1991-2: TRAFFIC LOADS ON BRIDGES

Load models should:

- **be easy to use**
- **produce main load effects correctly**
- **be the same for local and global verifications**
- **cover all possible situations (traffic scenarios)**
- **correspond to the target reliability levels**
- **include dynamic effects**

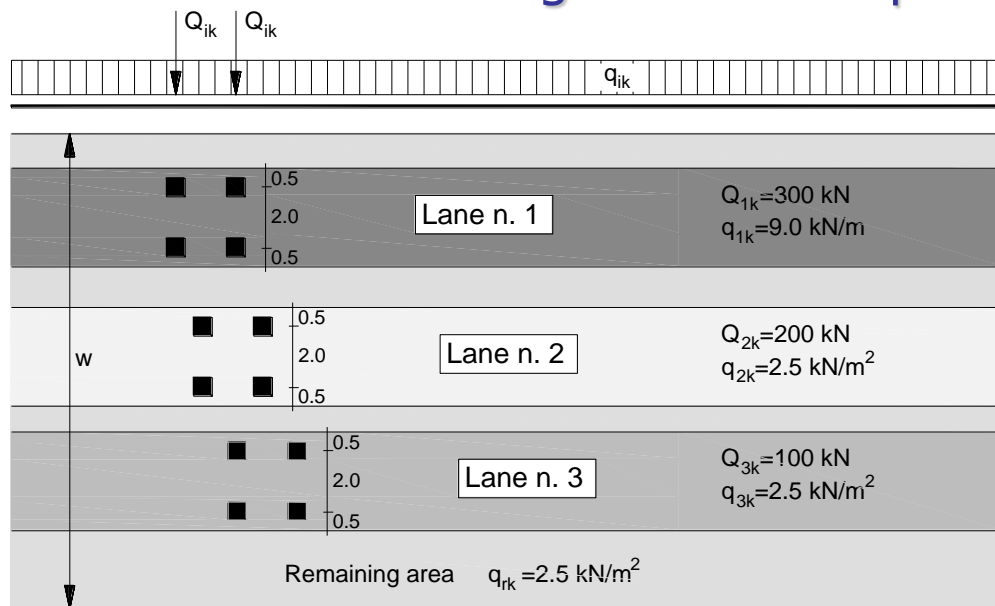
Division of the carriageway into notional lanes

Carriageway width	Number of notional lanes	Notional lane width	Width of the remaining area
$w < 5,4 \text{ m}$	$n_\ell = 1$	3 m	$w - 3 \text{ m}$
$5,4 \text{ m} \leq w < 6 \text{ m}$	$n_\ell = 2$	$w / 2$	0
$6 \text{ m} \leq w$	$n_\ell = \text{int}(w/3)$	3 m	$w - 3 \times n_\ell$



- 1 – Lane n° 1 (3m)
- 2 – Lane n° 2 (3m)
- 3 – Lane n° 3 (3m)
- 4 – Remaining area

The main load model for road bridges (LM1) diagrammatic representation



For the determination of general effects, the tandems travel along the axis of the notional lanes

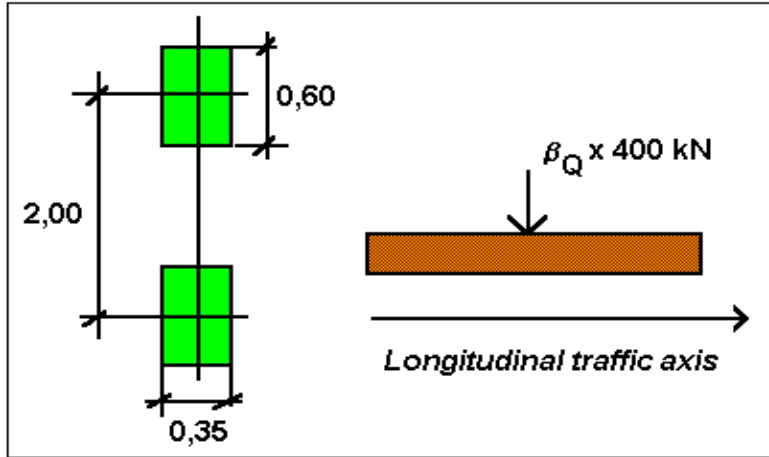
For local verifications, the heaviest tandem should be positioned to get the most unfavourable effect.

Location	Tandem system <i>TS</i>	<i>UDL</i> system
	Axle loads Q_{ik} (kN)	q_{ik} (or q_{ik}) (kN/m ²)
Lane Number 1	300	9
Lane Number 2	200	2,5
Lane Number 3	100	2,5
Other lanes	0	2,5
Remaining area (q_{rk})	0	2,5

Load models for road bridges : LM2 – isolated single axle

**Recommended
value :**

$$\beta_Q = \alpha_{Q1}$$



In the vicinity of expansion joints, an additional dynamic amplification factor equal to the value defined in 4.6.1(6) should be applied.

when relevant, only one wheel of 200 (kN) may be taken into account



Horizontal loads

Braking / acceleration (limite to 900 kN):

$$Q_x = 0,60 \alpha_{Q1} (2Q_{1k}) + 0,10 \alpha_{q1} q_{1k} w_1 L$$

Centrifugal loads

$Q_{tk} = 0,2Q_v$ (kN)	$r < 200$ m
$Q_{tk} = 40Q_v / r$ (kN)	$200 \leq r \leq 1500$ m
$Q_{tk} = 0$	$r > 1500$ m

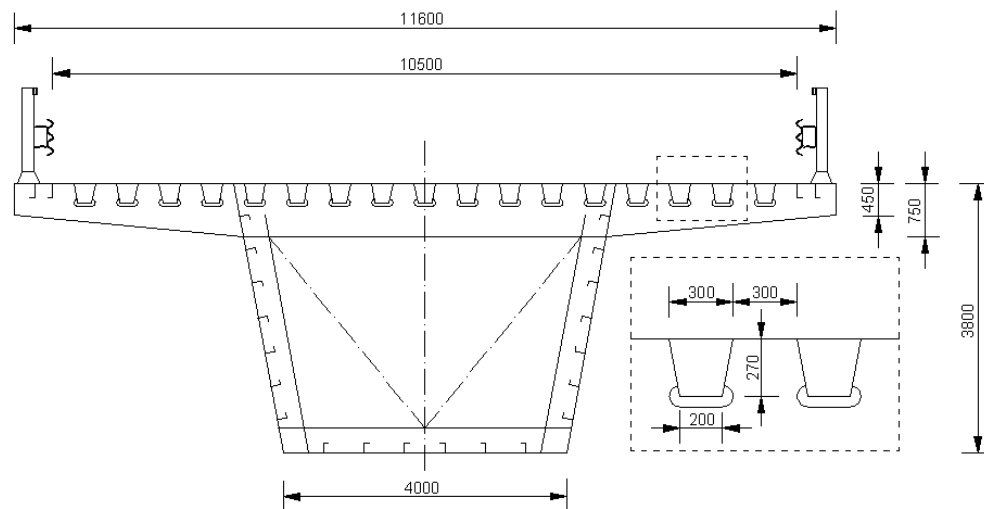
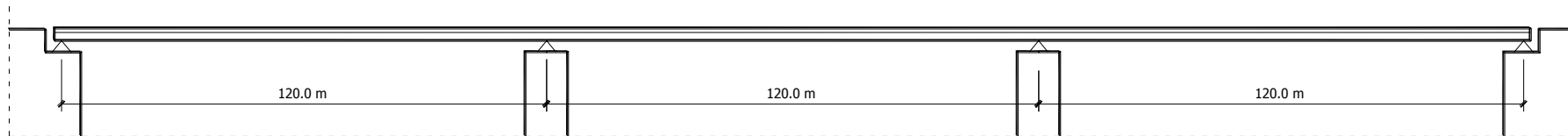
L = length of bridge part, w = lane width [m], r = arc radius [m]

Q and q correspond to LM 1 for lane 1

Characteristic values of multi-component traffic loads

<i>Carriageway</i>						<i>Footways and cycle tracks</i>
Vertical loads				Horizontal loads		Vertical loads only
Group of loads	Main load model	Special vehicles	Crowd loading	Braking force	Centrifugal force	Uniformly distributed loads
1	Characteristic values					Combination value
2	Frequent values			Characteristic values	Characteristic values	
3						Characteristic values
4			Characteristic values			Characteristic values
5	see 3.5 and figure 19	Characteristic values				

EXAMPLE OF AN ORTHOTROPIC STEEL DECK BRIDGE (THREE SPAN CONTINUOUS BRIDGE)



Effective length = 360 m.

The structure is made up of an orthotropic steel deck, with closed trapezoidal stiffeners, sustained by a box girder.

The bridge is 11.60 m wide.

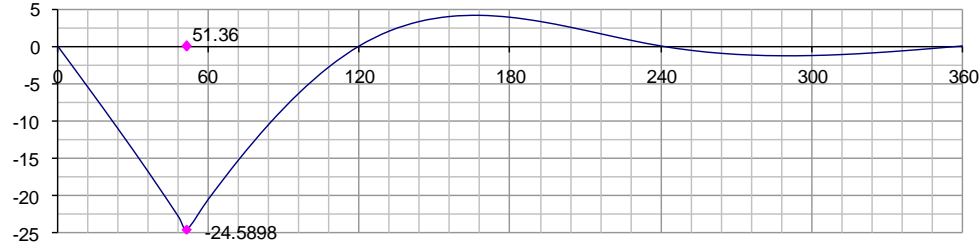
Carriageway is 10.50 m wide.

Two lateral walkways, each 150 cm in width, separated from the roadway only by road signs.

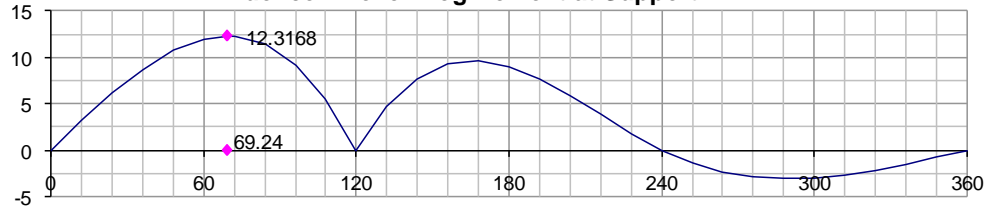
The bridge is to be located in an extra-urban area.

RELEVANT INFLUENCE LINES (Bending Moment M)

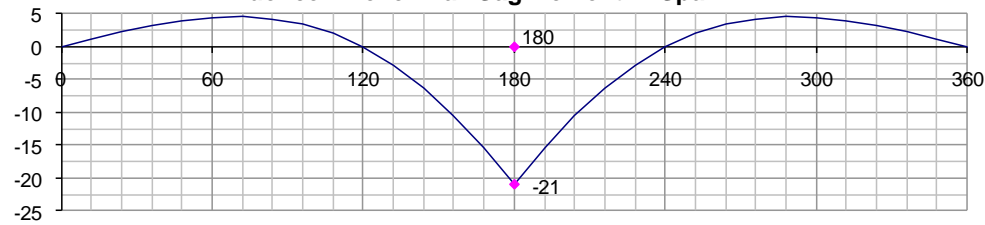
Influence Line for Max Sag Moment in Span 1



Influence Line for Hog Moment at Support 2



Influence Line for Max Sag Moment in Span 2



Definition of loads

Structural self-weights :

$$g_b = A_b \gamma = 0.584 \text{ m}^2 \times 78.5 \text{ kN/m}^3 = 45.84 \text{ kN/m}$$

$$\text{Increased by 6\%} \quad \quad \quad = 48.6 \text{ kN/m}$$

Other dead loads:

$$g_p = 2.2 \text{ kN/m}^2 \Rightarrow 25.52 \text{ kN/m}$$

Traffic loads:

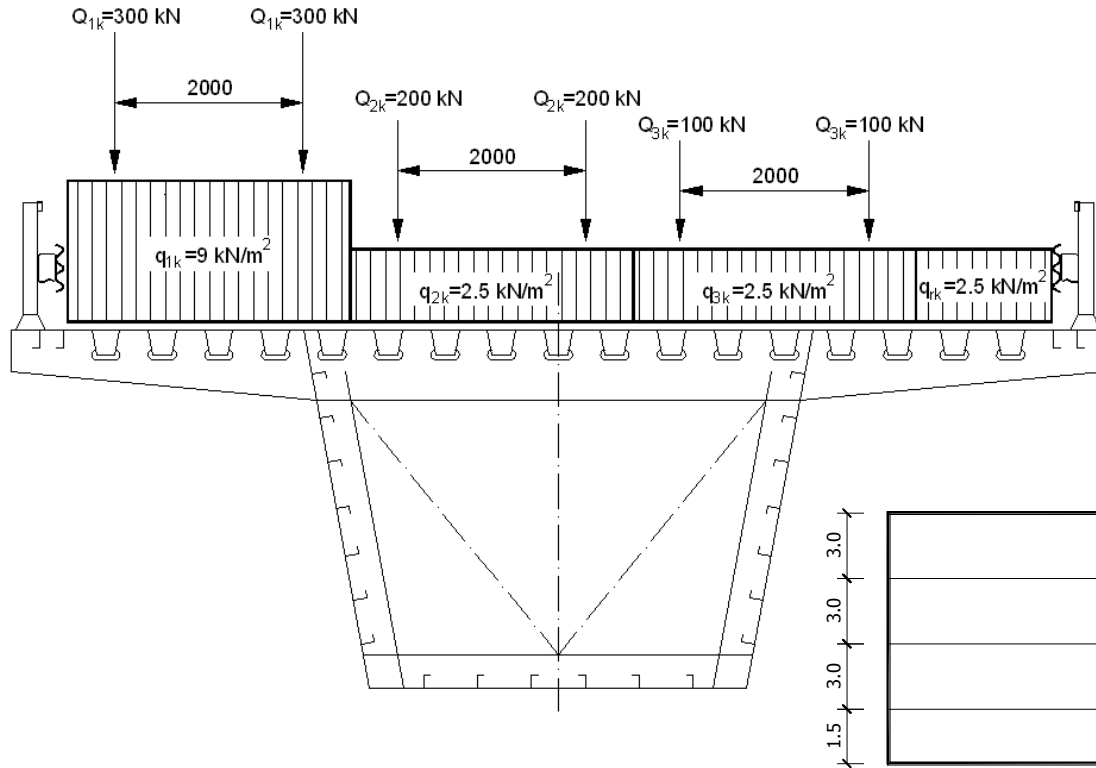
$w = 10.50 \text{ m}$. As $w > 6.0 \text{ m}$, the number of conventional lanes (each of which has a width, $w_l = 3.0 \text{ m}$) is given by:

$$n_l = \text{Int}\left[\frac{w}{3}\right] = \text{Int}\left[\frac{10.50}{3}\right] = 3$$

A residual area is left, its width given by:

$$w_r = w - n_l \cdot 3.0 \text{ m} = 10.50 \text{ m} - 3 \cdot 3.0 \text{ m} = 1.50 \text{ m}$$

Load Model 1

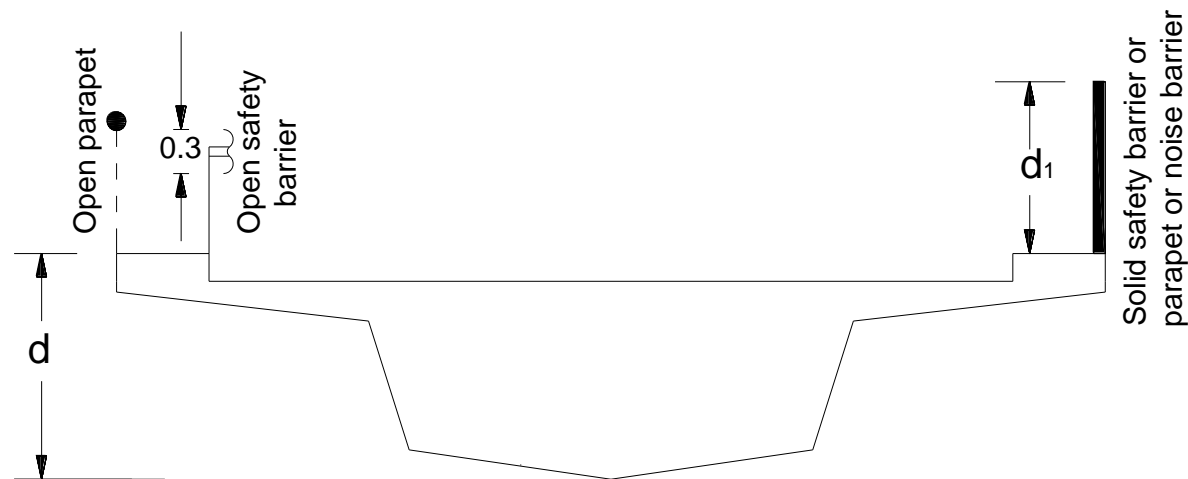


Conventional lane	Q_k [kN]	q_k [kN/m ²]
Lane 1	300	9.0
Lane 2	200	2.5
Lane 3	100	2.5
Residual area	0	2.5

notional lane 1	10.50
notional lane 1	
notional lane 2	
remaining area	

Wind actions

$$F_{wk} = \frac{1}{2} \rho v_b^2 C A_{ref}$$



<i>Road restraint systems and shields</i>	<i>On one side</i>	<i>On both sides</i>
Open parapet or open safety barrier	$d+0.3 \text{ m}$	$d+0.6 \text{ m}$
Solid parapet or solid safety barrier	$d+d_1$	$d+2 d_1$
Open parapet and open safety barrier	$d+0.6 \text{ m}$	$d+1.2 \text{ m}$

Wind actions

$$F_{wk} = \frac{1}{2} \rho v_b^2 C A_{ref}$$

C is the wind force coefficient

$$C = c_e c_{f,x}$$

$$C = c_e c_{f,z}$$

c_e is the exposure coefficient for kinetic pressure

$$c_e(z_e) = c_r^2(z_e) c_0^2(z_e) [1 + 7 \cdot I_v(z_e)]$$

$c_{f,x}$ is the force (drag) coefficient

Orography factor $c_0=1$

$$I_v(z_e) = \begin{cases} \frac{k_I}{c_0(z_e) \ln\left(\frac{z}{z_0}\right)} & \text{if } z_{min} < z \\ I_v(z_{min}) & \text{if } z_{min} \geq z \end{cases}$$

Turbulence intensity

Turbulence factor $k_I=1$

Terrain category II: *Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights*

roughness length $z_0=0.05$ m minimum height $z_{min}=2$ m

Terrain factor $k_r=0.19$

Wind actions

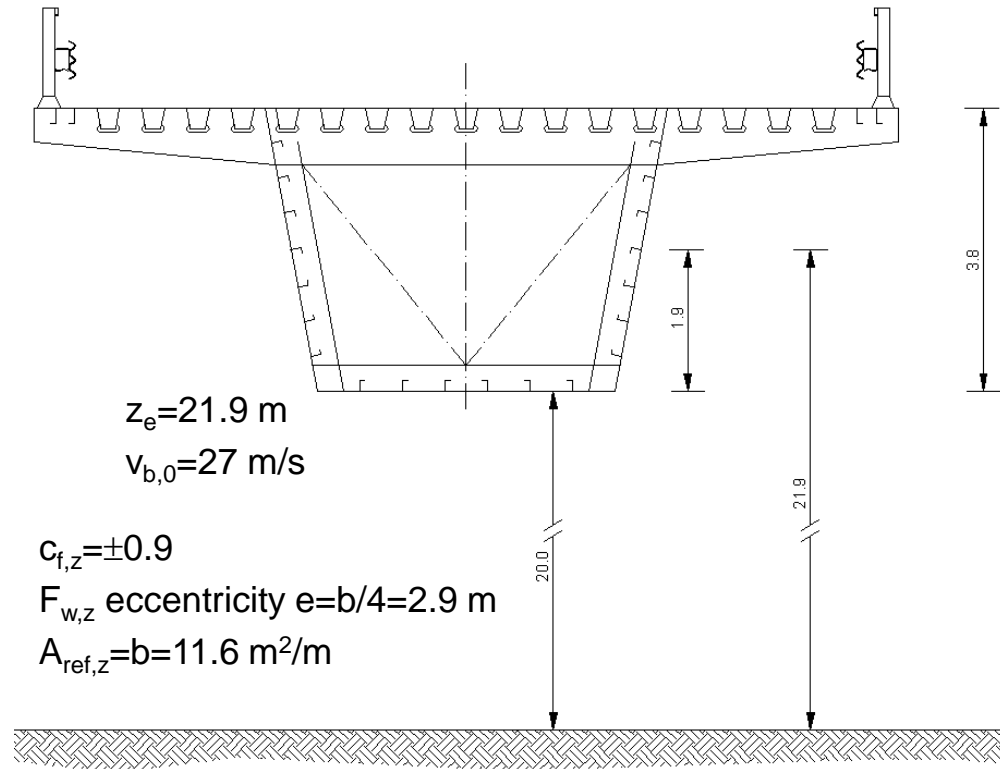
$$c_r(z_e) = \begin{cases} k_r(z_e) \ln\left(\frac{z}{z_0}\right) & \text{if } z_{\min} < z \\ c_r(z_{\min}) & \text{if } z_{\min} \geq z \end{cases}$$

$$c_r(z_e) = 0.19 \cdot \ln\left(\frac{21.9}{0.05}\right) = 1.156$$

$$I_v(z_e) = \frac{1}{1.0 \cdot \ln\left(\frac{21.90}{0.05}\right)} = 0.164$$

$$c_e(z_e) = 1.156^2 \cdot 1.0^2 \cdot [1 + 7 \cdot 0.164] = 2.869$$

$$q_p(z_e) = 2.869 \cdot \frac{1.25}{2} \cdot 27.0^2 = 1307 \text{ N/m}^2$$



$$F_{w,z} = q_p(z_e) \cdot c_{f,z} \cdot A_{ref,z} = 1.307 \text{ kN/m}^2 \cdot (\pm 0.9) \cdot 11.60 \text{ m} = \pm 13.64 \text{ kN/m}$$

Wind actions

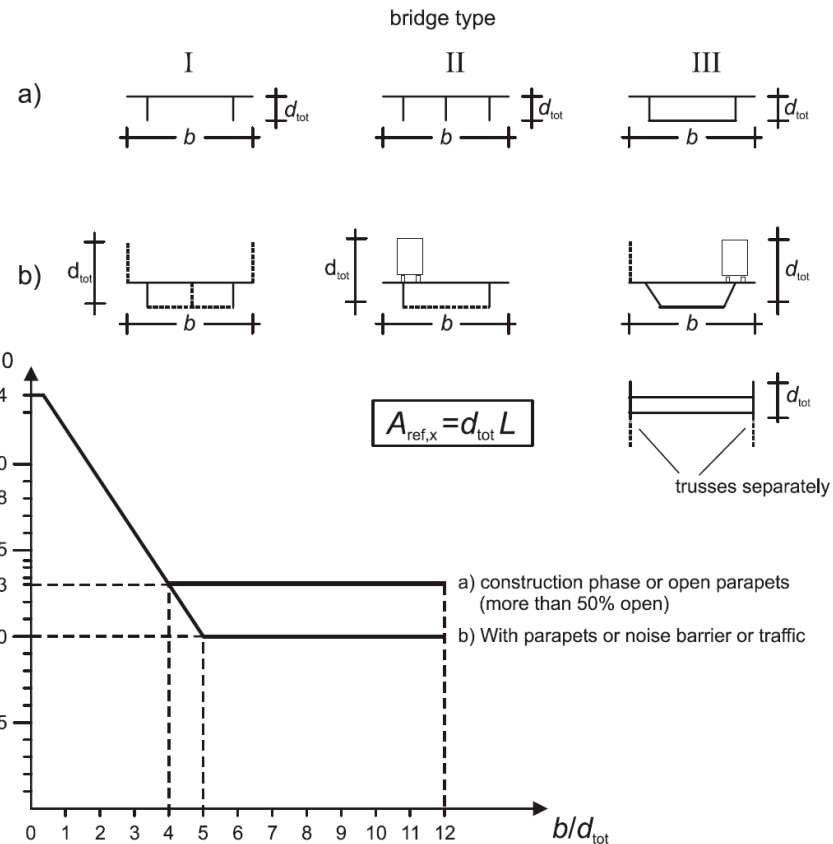
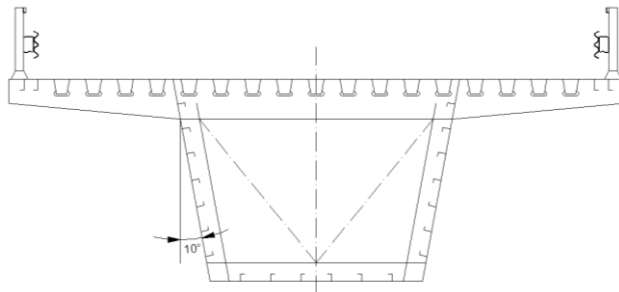
$$c_e(z_e) = 1.156^2 \cdot 1.0^2 \cdot [1 + 7 \cdot 0.164] = 2.869$$

$$q_p(z_e) = 2.869 \cdot \frac{1.25}{2} \cdot 27.0^2 = 1307 \text{ N/m}^2$$

Unloaded bridge Simplified method $c_{f,x}=1.3$

$$c_{f,x} = \min \left(2.4; \max \left(2.5 - 0.3 \frac{b}{d_{\text{tot}}}; 1.3 \right) \right) \quad c_{f,x} = 2.5 - 0.3 \frac{11.6}{4.4} = 1.709$$

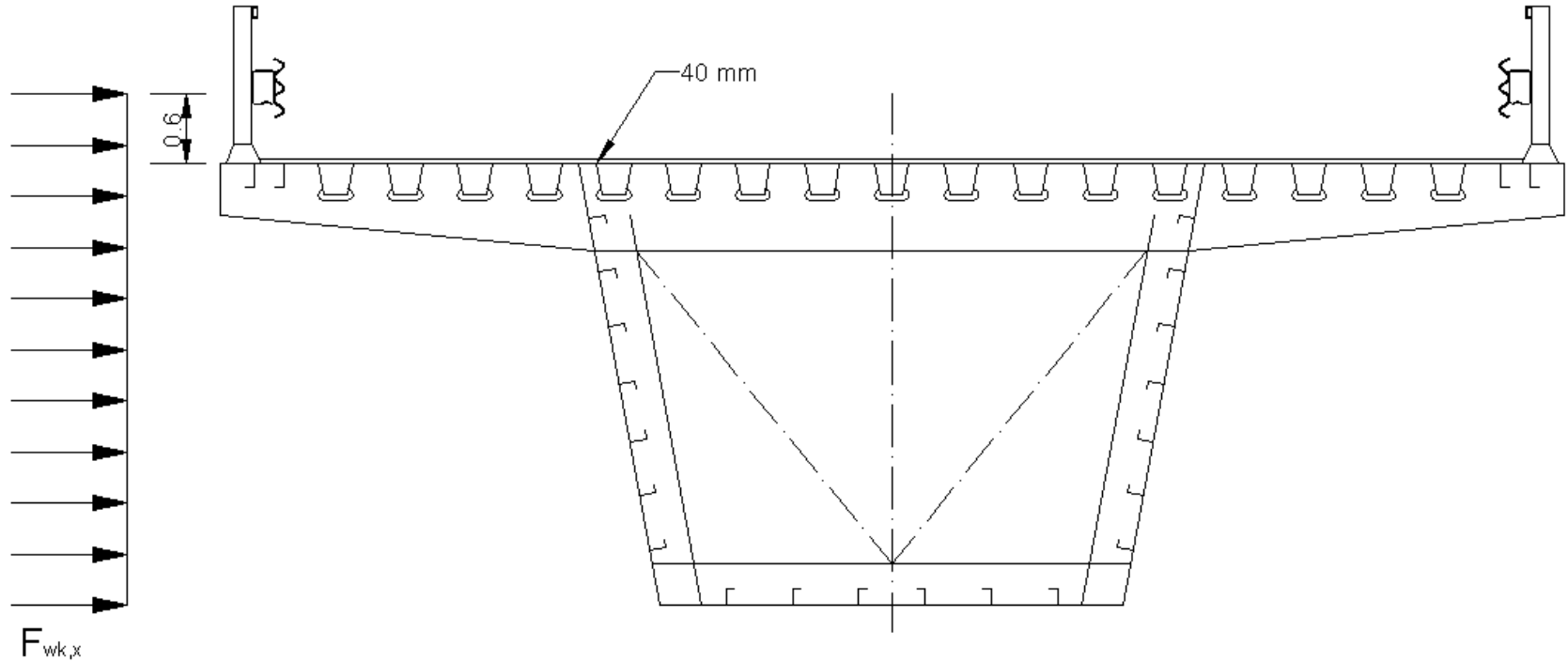
$$\eta_1 = \max(1 - 0.005 \alpha_1; 0.7) = 0.95$$



$$F_{w,x} = q_p(z_e) c_{f,x} A_{\text{ref},x} = 1.307 \cdot 1.709 \cdot 0.95 \cdot 4.4 = 9.34 \text{ kN/m}$$

Unloaded bridge

$$F_{w,x} = q_p(z_e) c_{f,x} A_{ref,x} = 1.307 \cdot 1.709 \cdot 0.95 \cdot 4.4 = 9.34 \text{ kN/m}$$



Wind actions

Loaded bridge

$$\eta_1 = \max(1 - 0.005 \alpha_1; 0.7) = 0.95$$

Simplified method $c_{f,x} = 1.3$

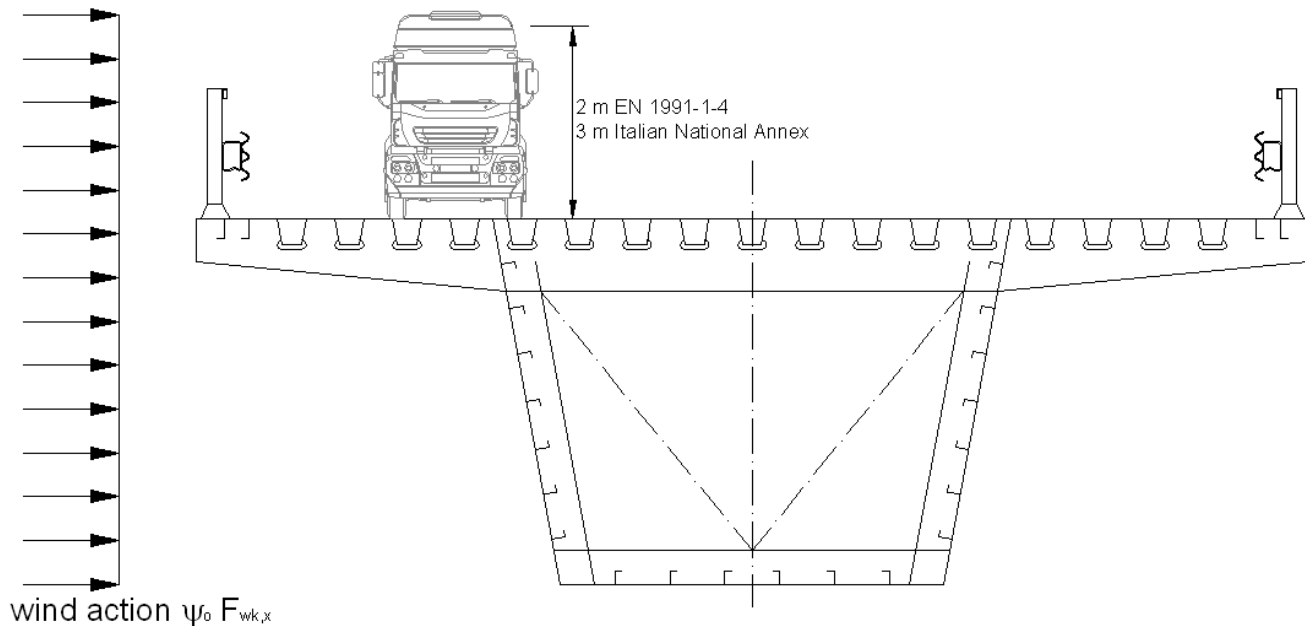
$$c_{f,x} = \min\left(2.4; \max\left(2.5 - 0.3 \frac{b}{d_{\text{tot}}}; 1.0\right)\right)$$

$$c_{f,x} = 2.5 - 0.3 \frac{11.6}{5.8} = 1.9 \quad \text{EN1991-1-4}$$

$$F_{w,x} = 0.6 \cdot 1.307 \cdot 1.9 \cdot 0.95 \cdot 5.8 = 8.21 \text{ kN/m} \quad \text{EN1991-1-4}$$

$$c_{f,x} = 2.5 - 0.3 \frac{11.6}{6.8} = 2.023 \quad \text{NAD(I)}$$

$$F_{w,x} = 0.6 \cdot 1.307 \cdot 1.99 \cdot 0.95 \cdot 6.8 = 10.07 \text{ kN/m} \quad \text{NAD(I)}$$

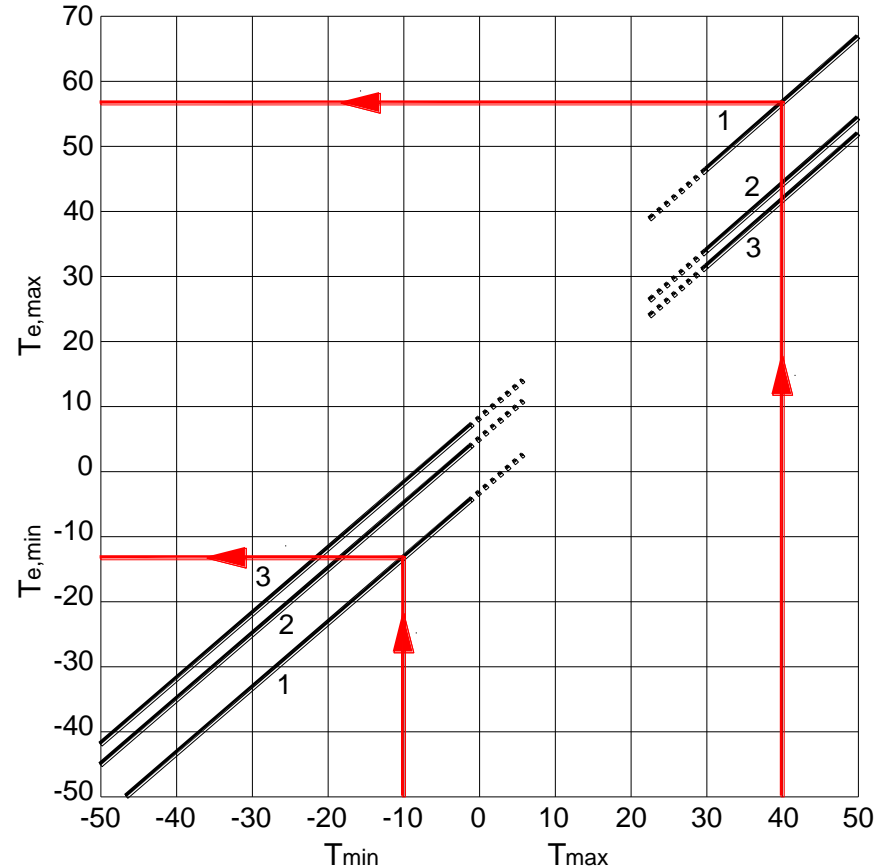


Thermal actions (Uniform)

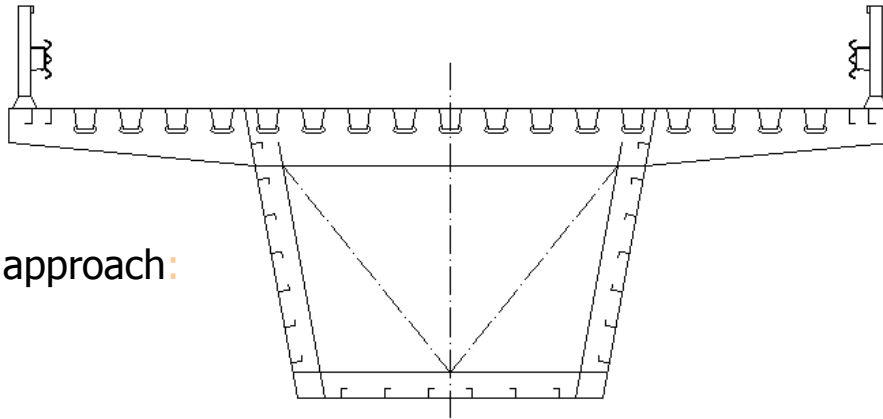
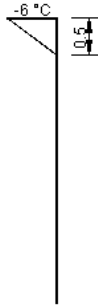
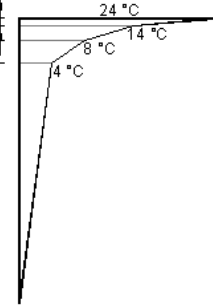
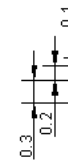
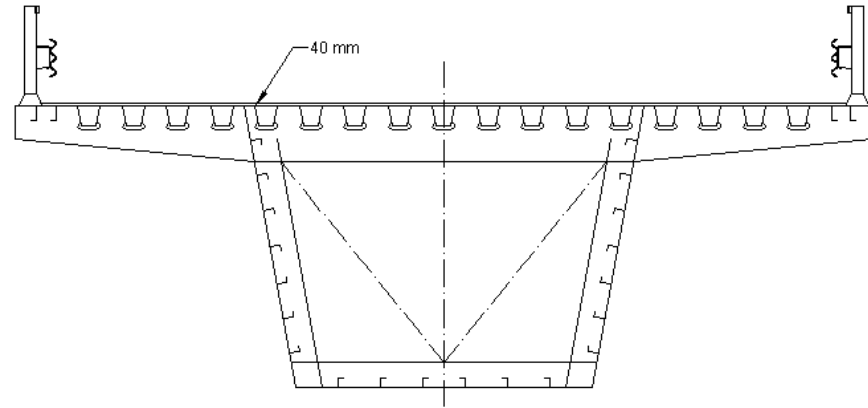
Assuming that in the site under consideration the maximum and minimum air shade temperatures with an annual probability of being exceeded of 0.02 are

$T_{\max}=40\text{ }^{\circ}\text{C}$ and $T_{\min}=-10\text{ }^{\circ}\text{C}$, respectively, the uniform bridge temperature components for a steel bridge result $T_{e,\max}=56.6\text{ }^{\circ}\text{C}$ and $T_{e,\min}=-13.3\text{ }^{\circ}\text{C}$

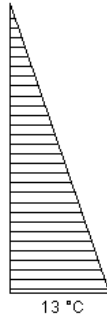
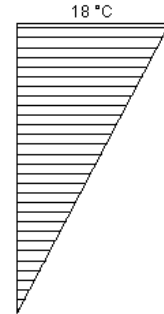
$$\Delta T = \pm 20\text{ }^{\circ}\text{C} - \text{NAD (I)}$$



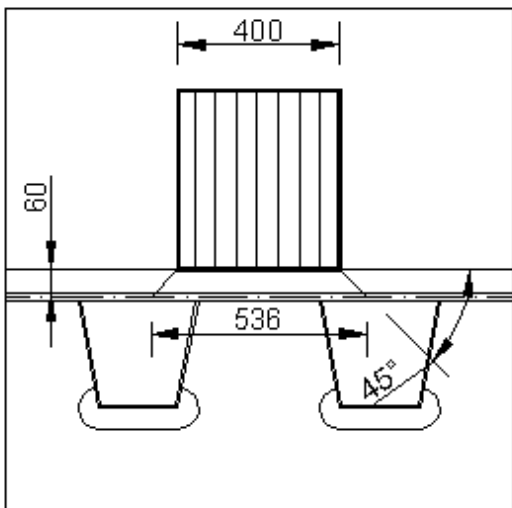
Thermal actions - Non uniform variations



Simplified approach:



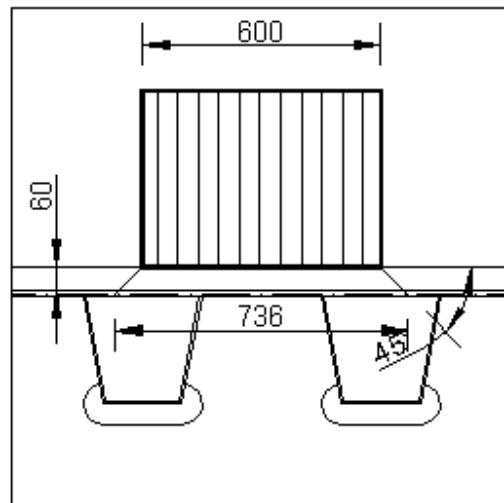
Stress calculation



LM1 - TS

$$p_{Q1} = \frac{150}{0.536 \cdot 0.536} = 522.1 \text{ kN/m}^2$$

Local effects on the deck plate

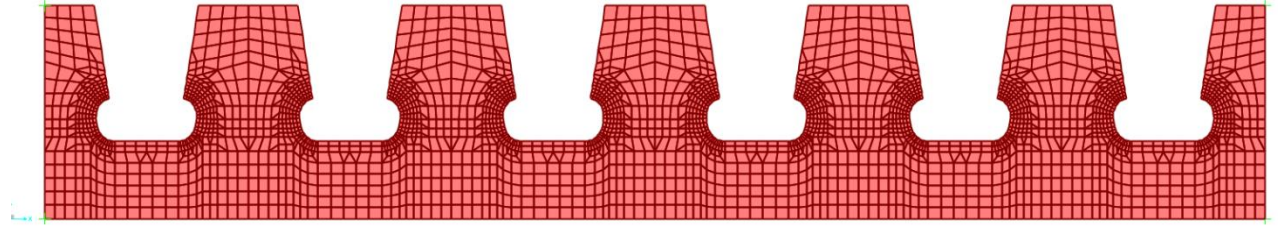
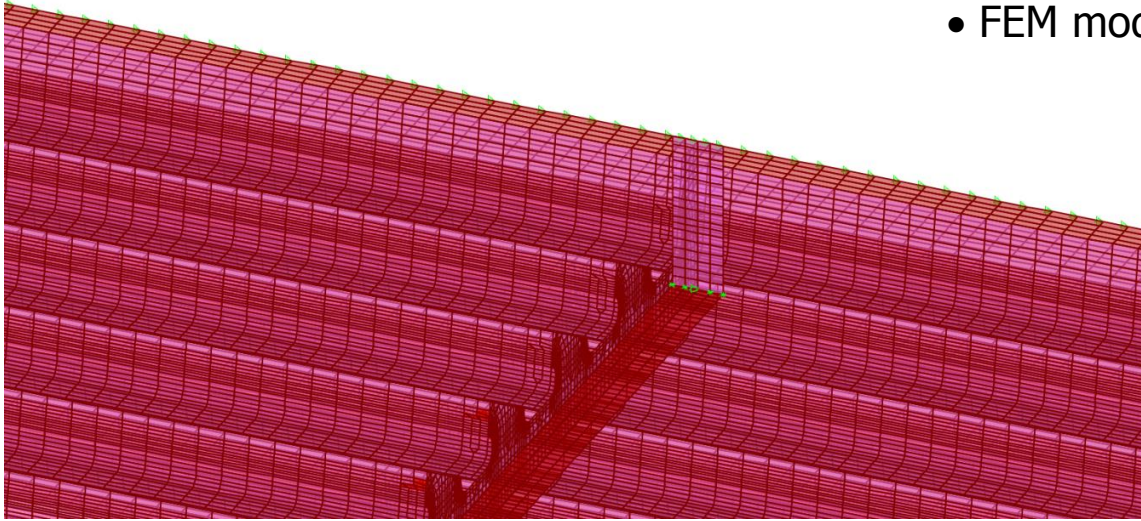


LM2

$$p_{Qak} = \frac{200}{0.736 \cdot 0.486} = 523.6 \text{ kN/m}^2$$

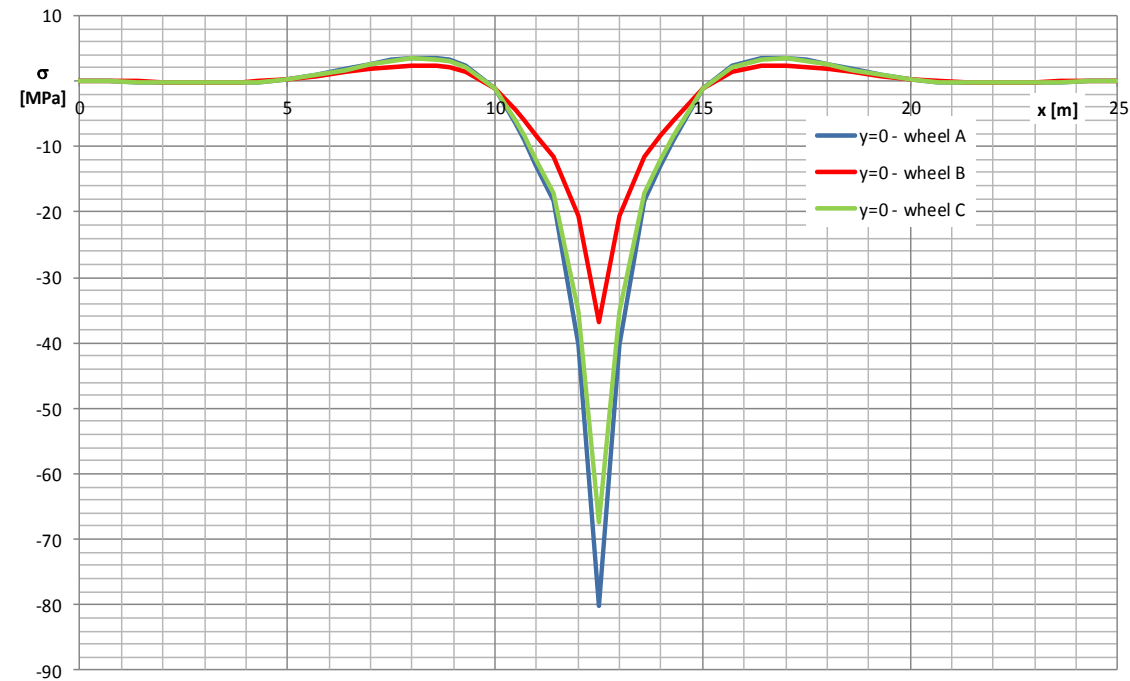
Calculation methods for the orthotropic deck

- simplified calculation models: e.g., Cornelius, ...
- Analytical models: Pelikan-Esslinger
- FEM models



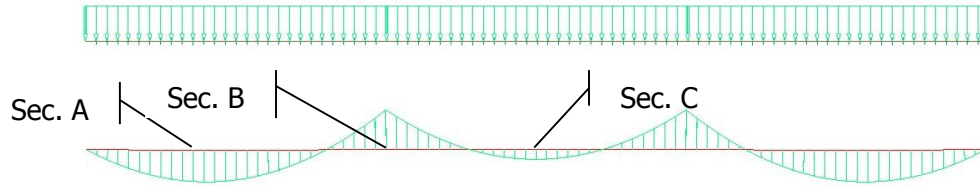
Orthotropic deck

«Influence lines» for different wheel contact areas



WHEEL/ AXLE TYPE	GEOMETRICAL DEFINITION
A	
B	
C	

Calculating the main structure



$$M_g(A) = \frac{2}{25} \cdot (g_b + g_p) \cdot L^2 = \frac{2}{25} \cdot 74.12 \cdot 120.0^2 = 85381.8 \text{ kNm}$$

The effects of self-weight and dead loads

$\gamma_G = 1.35$ unfavourable - $\gamma_G = 1.00$ favourable

Single source principle

$$M_g(A)_d = 1.35 \cdot 85381.8 = 115265.4 \text{ kNm}$$

$$M_g(B) = -\frac{1}{10} \cdot (g_b + g_p) \cdot L^2 = -\frac{1}{10} \cdot 74.12 \cdot 120.0^2 = -106727.4 \text{ kNm}$$

$$M_g(B)_d = 1.35 \cdot -106727.4 = -144082.0 \text{ kNm}$$

$$M_g(C) = \frac{1}{40} \cdot (g_b + g_p) \cdot L^2 = \frac{1}{40} \cdot 74.12 \cdot 120.0^2 = 26681.8 \text{ kNm}$$


$$M_g(C)_d = 1.35 \cdot 26681.8 = 36020.4 \text{ kNm}$$

Effects of traffic loads

$\gamma_Q = 1.35$ unfavourable - $\gamma_Q = 0$ favourable

$$q = (7.2 + 2.5 + 2.5) \cdot 3.0 + 2.5 \cdot 1.5 = 40.35 \text{ kN/m}$$

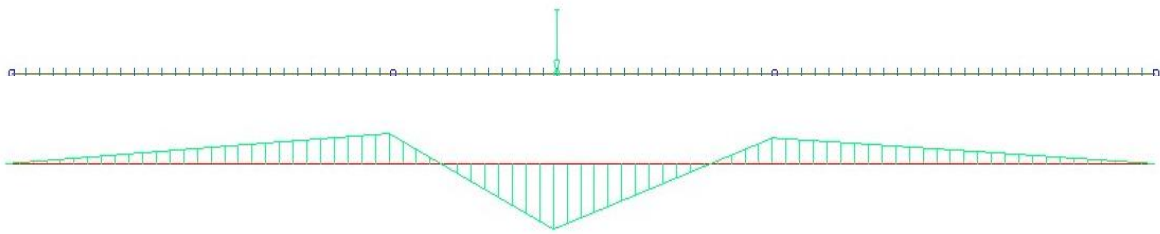
$$q_d = 1.35 \cdot 40.35 = 54.47 \text{ kN/m}$$


$$Q = 2 \cdot 2(Q_1 + Q_2 + Q_3) = 2 \cdot (300 + 200 + 100) = 1200 \text{ kN}$$

(involves the implicit assumption that a single concentrated force represents the 12 forces corresponding to the wheels of the two pairs of tandem axles on each lane).

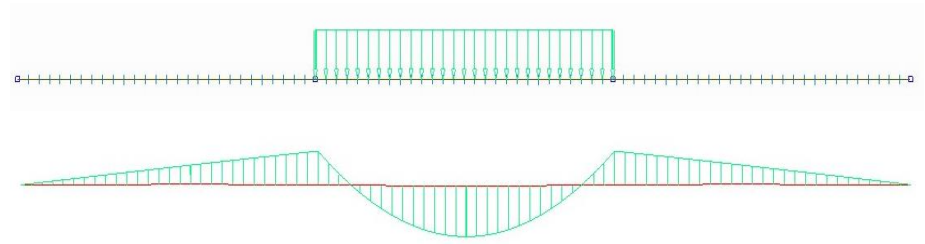
$$Q_d = 1.35 \cdot 1200 = 1620 \text{ kN}$$

Concentrated load :



Uniformly distributed load

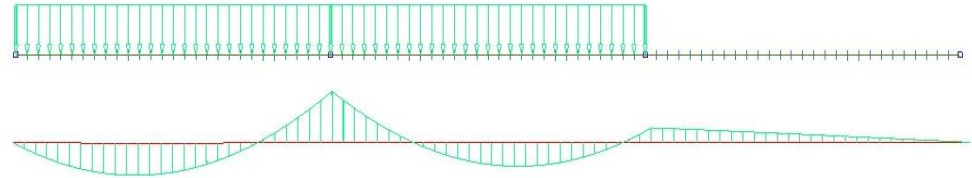
- Maximum positive moment in the central span



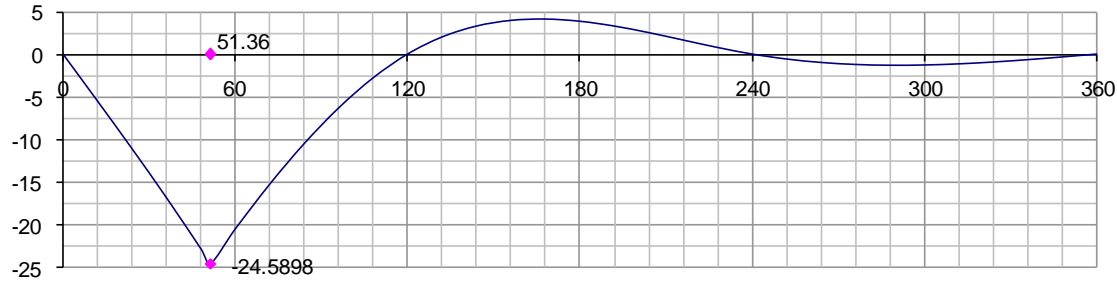
- Maximum positive moment in the two lateral spans:



- Maximum negative moment on the support.



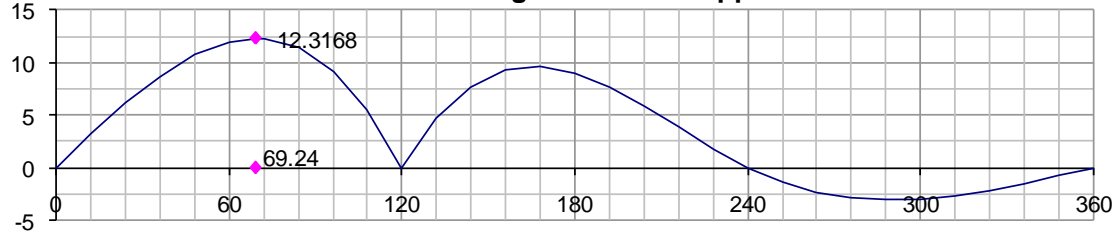
Influence Line for Max Sag Moment in Span 1



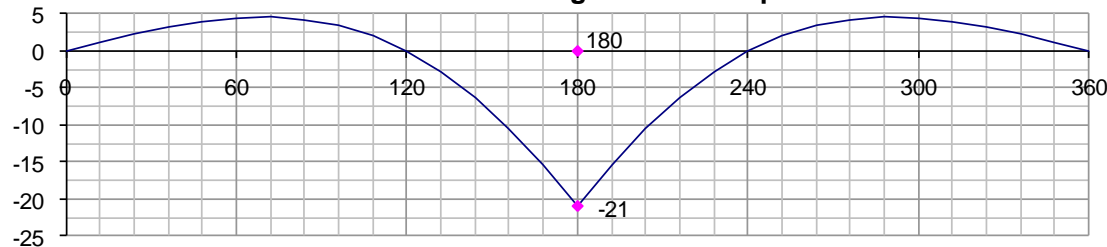
$$Q_d = 1620 \text{ kN}$$

$$q_d = 54.47 \text{ kN/m}$$

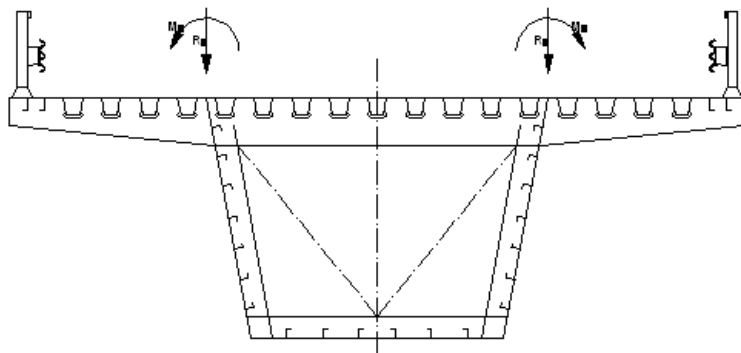
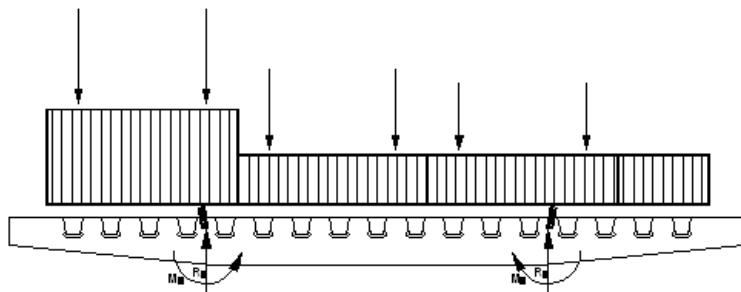
Influence Line for Hog Moment at Support 2



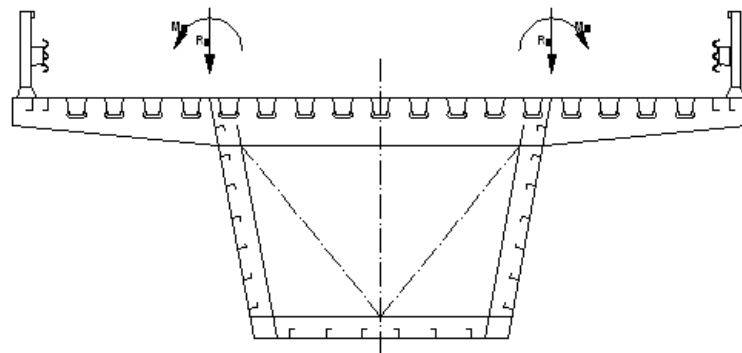
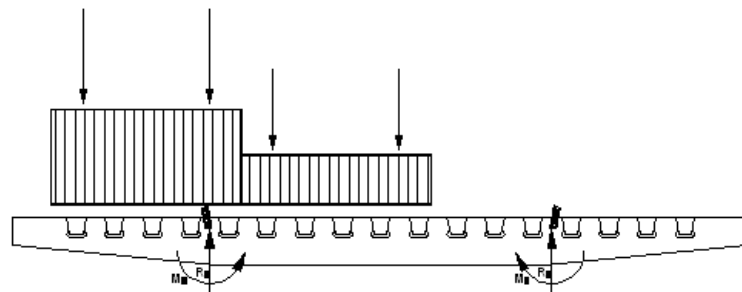
Influence Line for Max Sag Moment in Span 2



Calculation of the transverse section



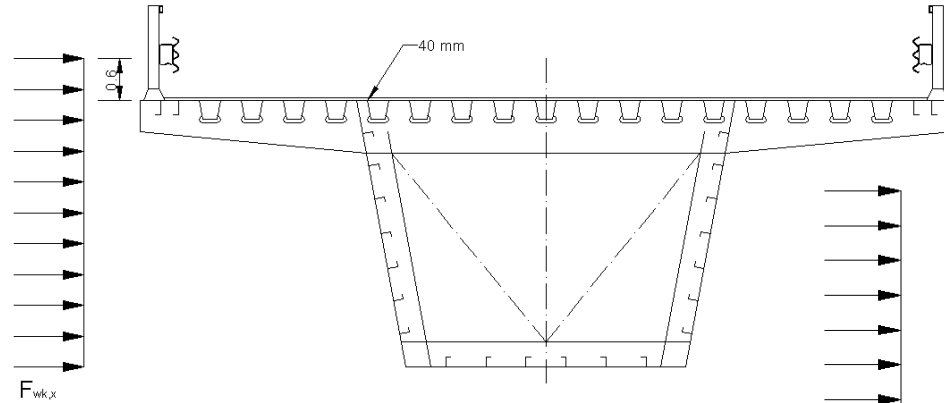
maximum bending moment in the box girder



maximum torque in the box girder

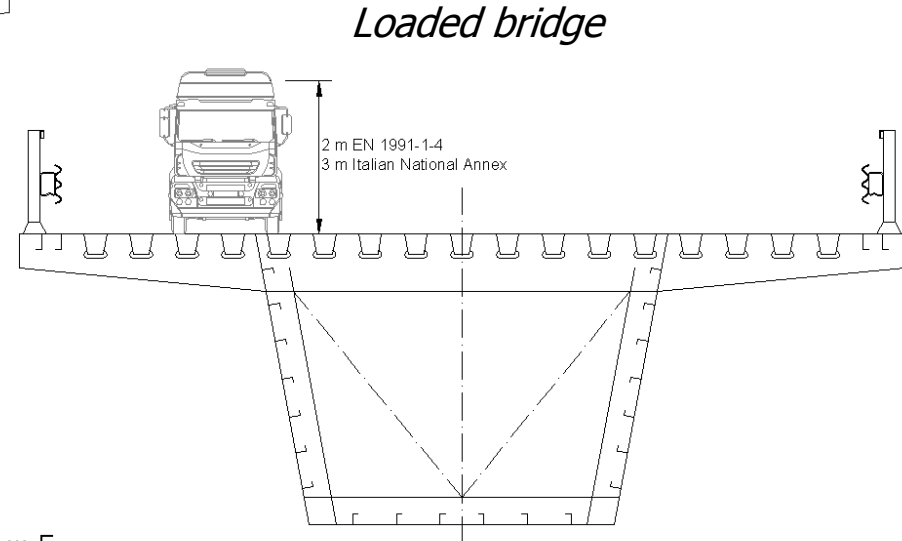
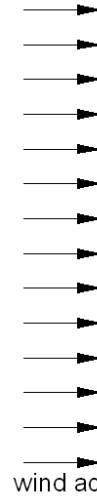
Application of transverse wind load

$\gamma_Q = 1.5$ unfavourable - $\gamma_Q = 0$ favourable



$$F_{w,x} = 1.5 \cdot 9.34 = 14.01 \text{ kN/m}$$

Unloaded bridge



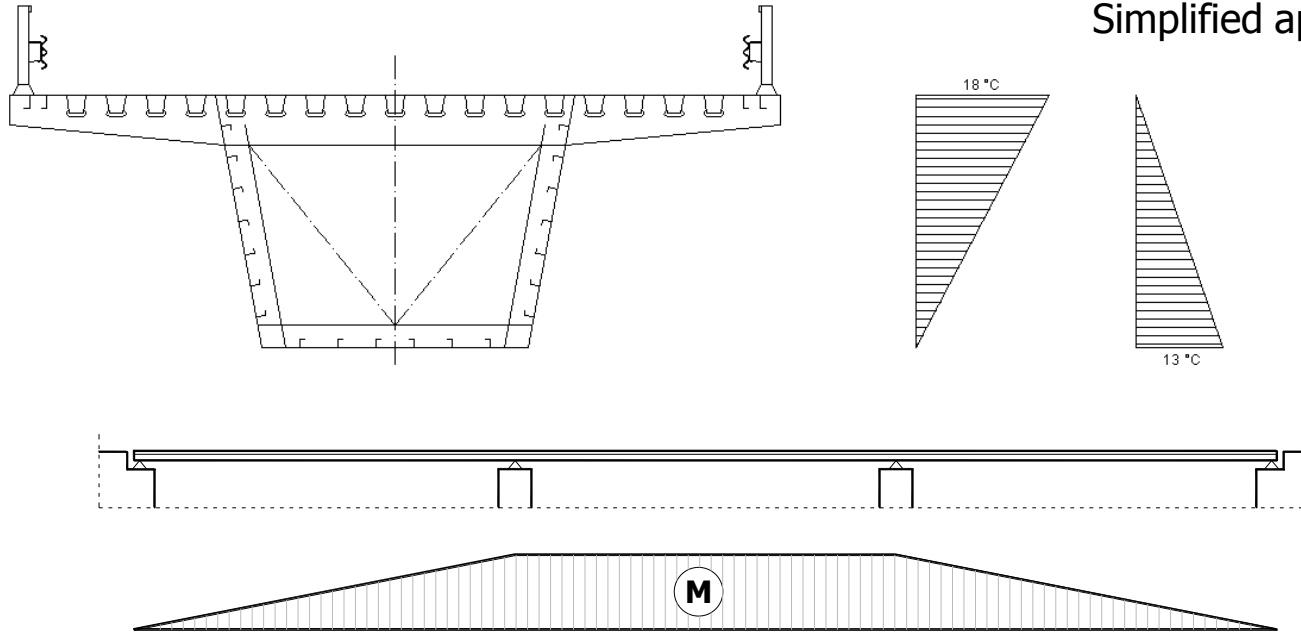
Loaded bridge

$$F_{w,xd} = 1.5 \cdot 8.21 = 12.31 \text{ kN/m} \quad \text{EN1991-1-4}$$

$$F_{w,xd} = 1.5 \cdot 10.07 = 15.10 \text{ kN/m} \quad \text{NAD(I)}$$

Thermal actions - Non uniform variations

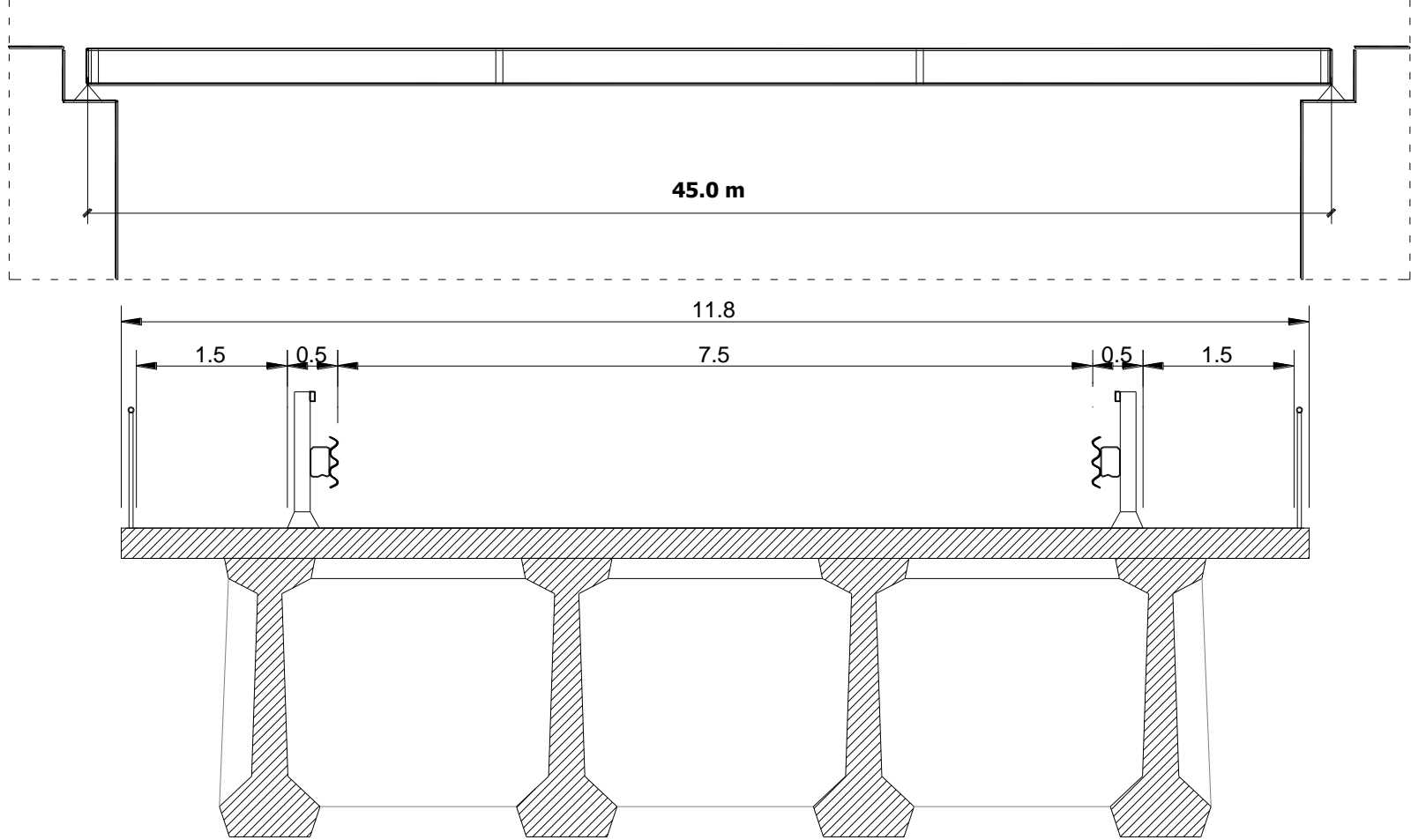
Simplified approach:

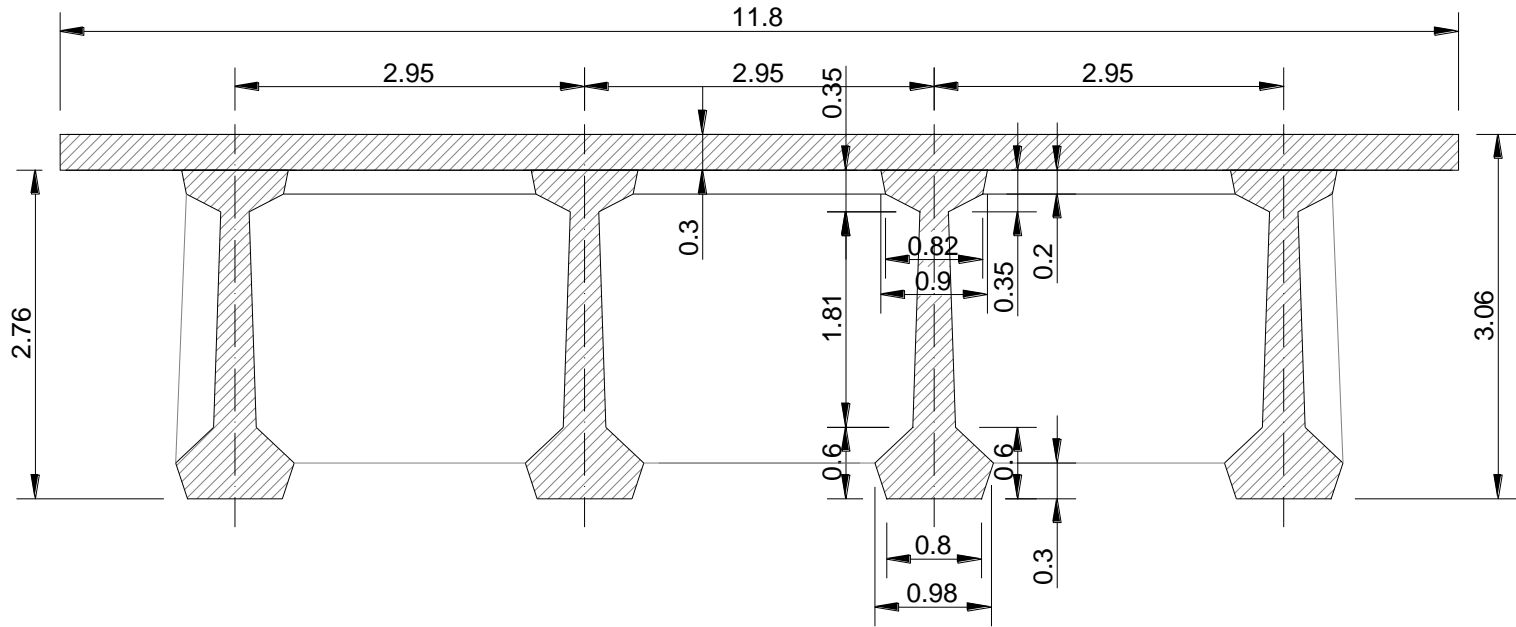


$$M_{\Delta T18} = \frac{\alpha \cdot \Delta T}{h} \cdot \frac{6 \cdot EJ}{5} = 13530.9 \text{ kNm}$$

$$M_{\Delta T13} = \frac{\alpha \cdot \Delta T}{h} \cdot \frac{6 \cdot EJ}{5} = -9772.3 \text{ kNm}$$

EXAMPLE OF A CONCRETE BRIDGE (SIMPLY SUPPORTED PRESTRESSED CONCRETE BRIDGE IN AN URBAN AREA)





Load definition

Structural self-weights :

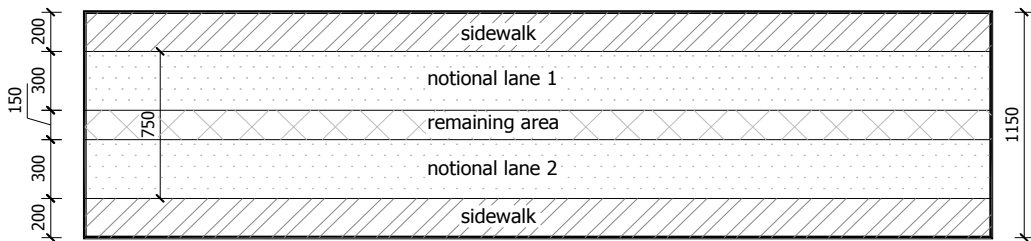
The weight of each beam is: $g_b = A_b \gamma = 31.0 \text{ kN/m}$

The thickness of the concrete slab is 30.0 cm; therefore, its weight is: $g_s = 7.5 \text{ kN/m}^2$.

Dead loads: $g_{\text{add}} = 2.2 \text{ kN/m}^2$

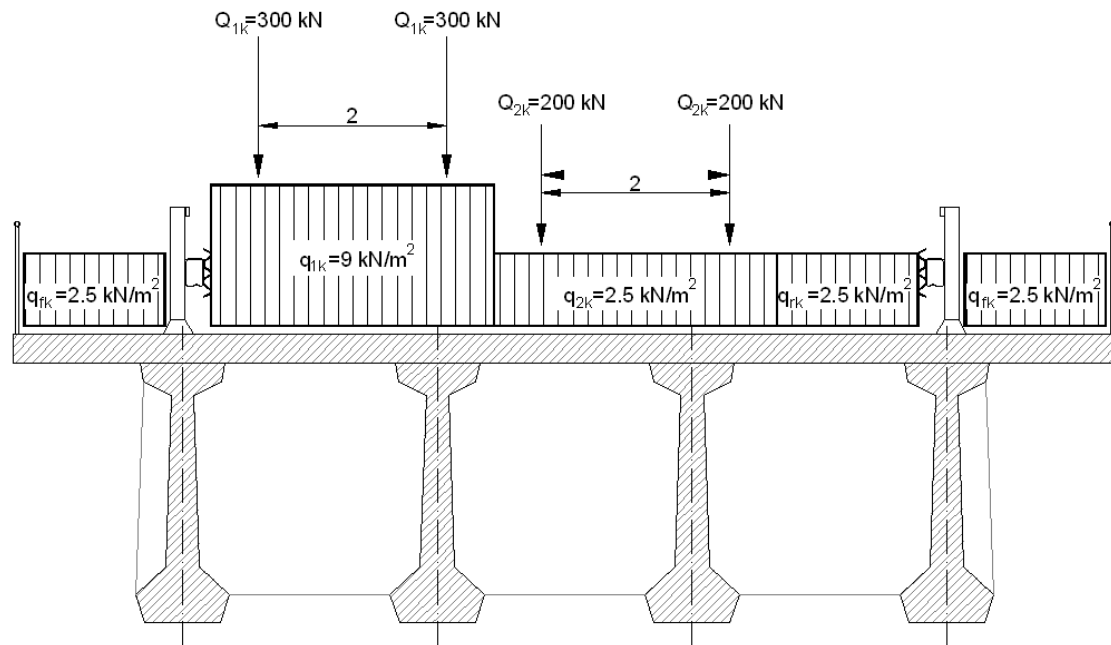
Notional lanes for traffic loads

$$n_l = \text{Int}\left[\frac{w}{3}\right] = \text{Int}\left[\frac{7.50}{3}\right] = 2 \qquad w_r = w - n_l \cdot 3.0 \text{ m} = 7.50 - 2 \cdot 3.0 = 1.50 \text{ m}$$

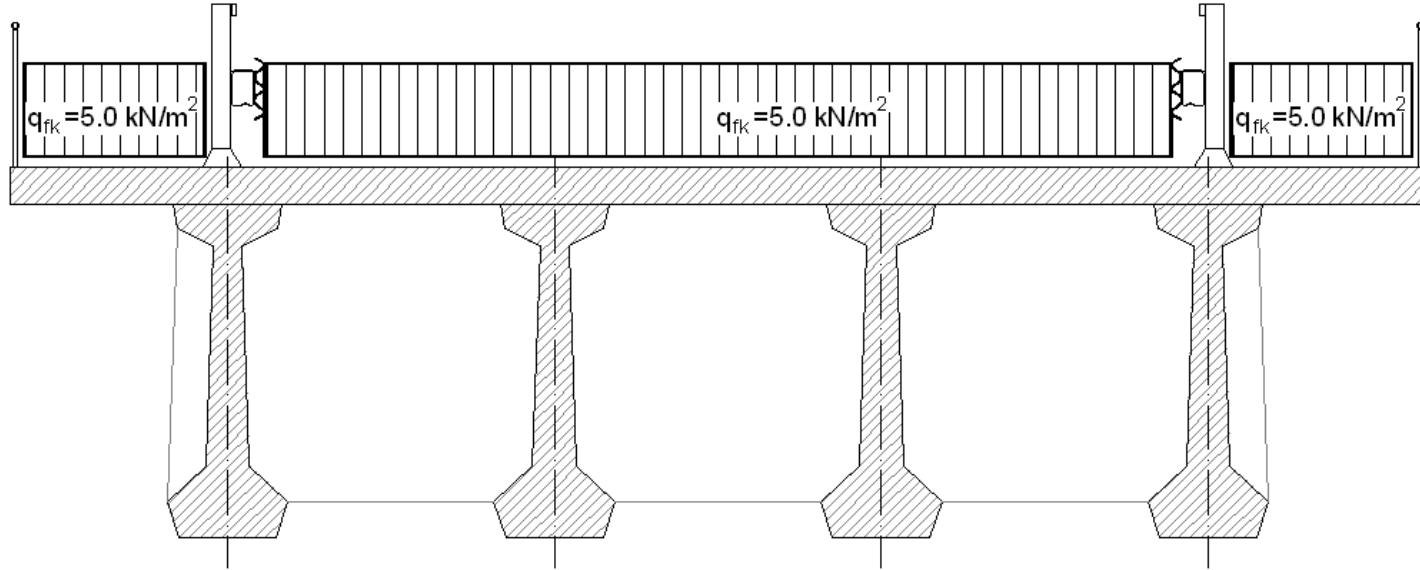


Conventional lane	Q_k [kN]	q_k [kN/m ²]
Lane 1	300	9.0
Lane 2	200	2.5
Residual area	0	2.5

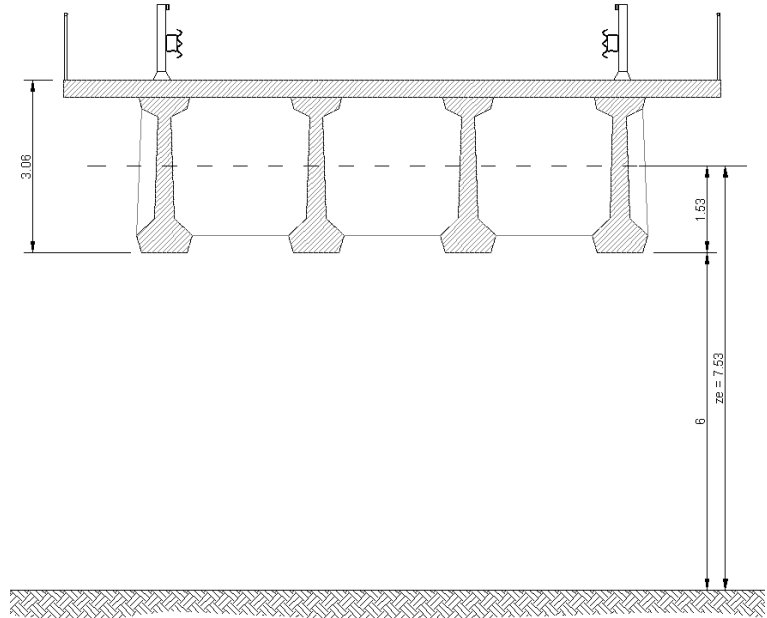
Load Model 1



Load Model 4 – Crowd loading



Wind action (unloaded)



For unloaded bridge, the coefficient $c_{f,x}$ is

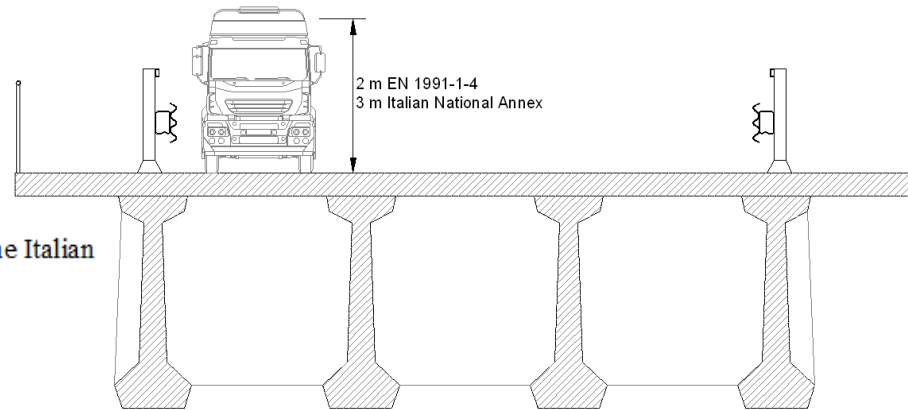
$$c_{f,x} = \min \left(2.4; \max \left(2.5 - 0.3 \frac{b}{d_{tot}}; 1.3 \right) \right) = 2.5 - 0.3 \frac{11.8}{4.26} = 1.669 .$$

If must be noted that the simplified approach proposed in EN1991-1-4 allowing to set $c_{f,x}=1.3$ is generally unsafe-sided.

the force $F_{wk,x}$ for unloaded bridge is then

$$F_{wk,x} = q_p(z_e) c_{f,x} A_{ref,x} = 0.54 \cdot 1.669 \cdot 4.26 = 3.84 \text{ kN/m} .$$

Wind action (loaded)



When the bridge is loaded, the exposed height increases by 2.0 m (3.0 m in the Italian National Annex), so it becomes 5.06 m (6.06 m).
In that case the coefficient $c_{f,x}$ is then

$$c_{f,x} = \min \left(2.4; \max \left(2.5 - 0.3 \frac{b}{d_{tot}}; 1.0 \right) \right) = 2.5 - 0.3 \frac{11.8}{5.06} = 1.80$$

or, according to the Italian National Annex,

$$c_{f,x} = \min \left(2.4; \max \left(2.5 - 0.3 \frac{b}{d_{tot}}; 1.0 \right) \right) = 2.5 - 0.3 \frac{11.8}{6.06} = 1.916$$

and also for loaded bridge the simplification $c_{f,x}=1.3$ results unsafe-sided.

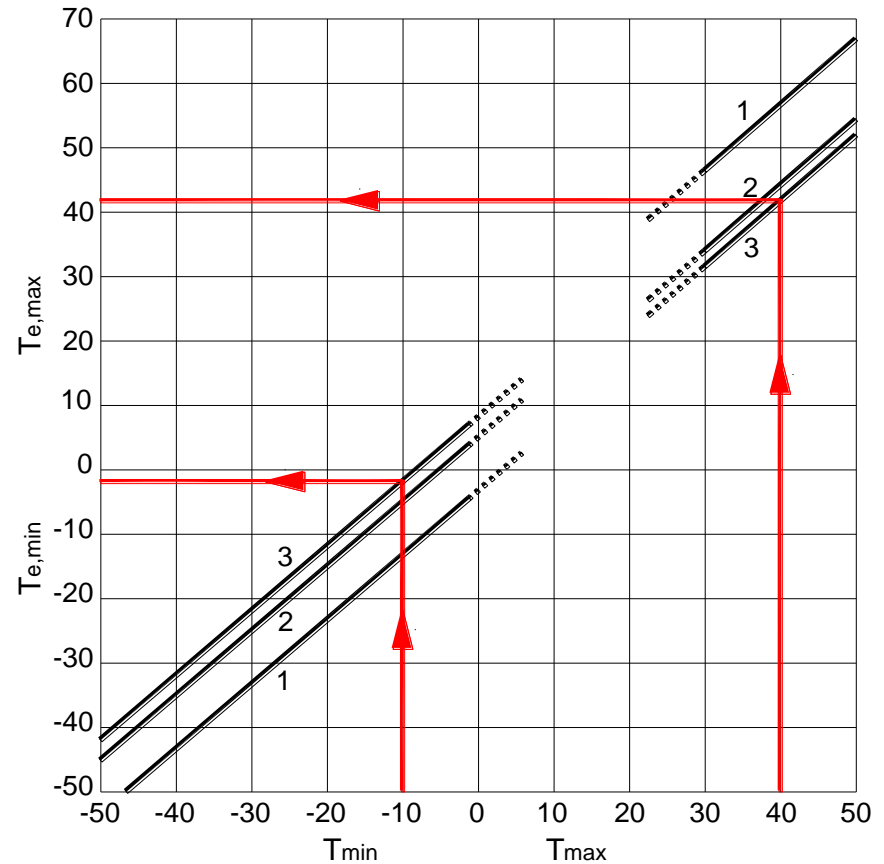
Since $\psi_{0w}=0.6$, the combination values $\psi_{0w}F_{wk,x}$ for the equivalent wind force for loaded bridge result

$$\psi_{0w}F_{wk,x} = \psi_{0w}q_p(z_e)c_{f,x}A_{ref,x} = 0.6 \cdot 0.54 \cdot 1.8 \cdot 5.06 = 2.95 \text{ kN/m}$$

$$\psi_{0w}F_{wk,x} = \psi_{0w}q_p(z_e)c_{f,x}A_{ref,x} = 0.6 \cdot 0.54 \cdot 1.916 \cdot 6.06 = 3.76 \text{ kN/m}$$

Thermal action (uniform)

Assuming that in the site under consideration the maximum and minimum air shade temperatures with an annual probability of being exceeded of 0.02 are $T_{\max}=40\text{ }^{\circ}\text{C}$ and $T_{\min}=-10\text{ }^{\circ}\text{C}$, respectively, the uniform bridge temperature components for a concrete bridge result $T_{e,\max}=41.7\text{ }^{\circ}\text{C}$ and $T_{e,\min}=-1.8\text{ }^{\circ}\text{C}$

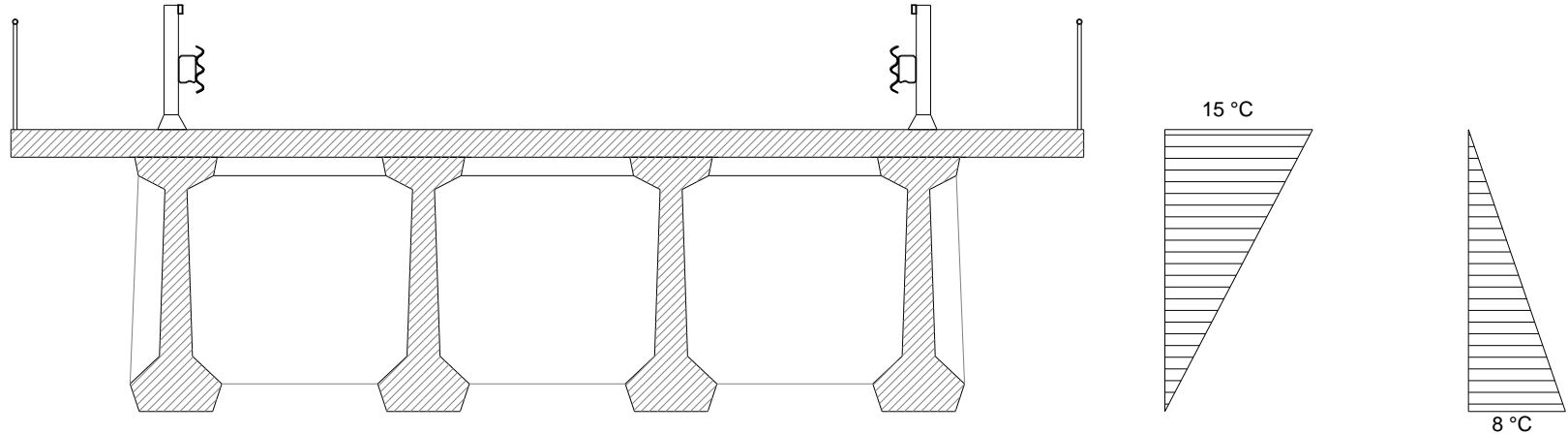


Thermal action (non uniform)

Linearly varying vertical temperature difference components can be set to

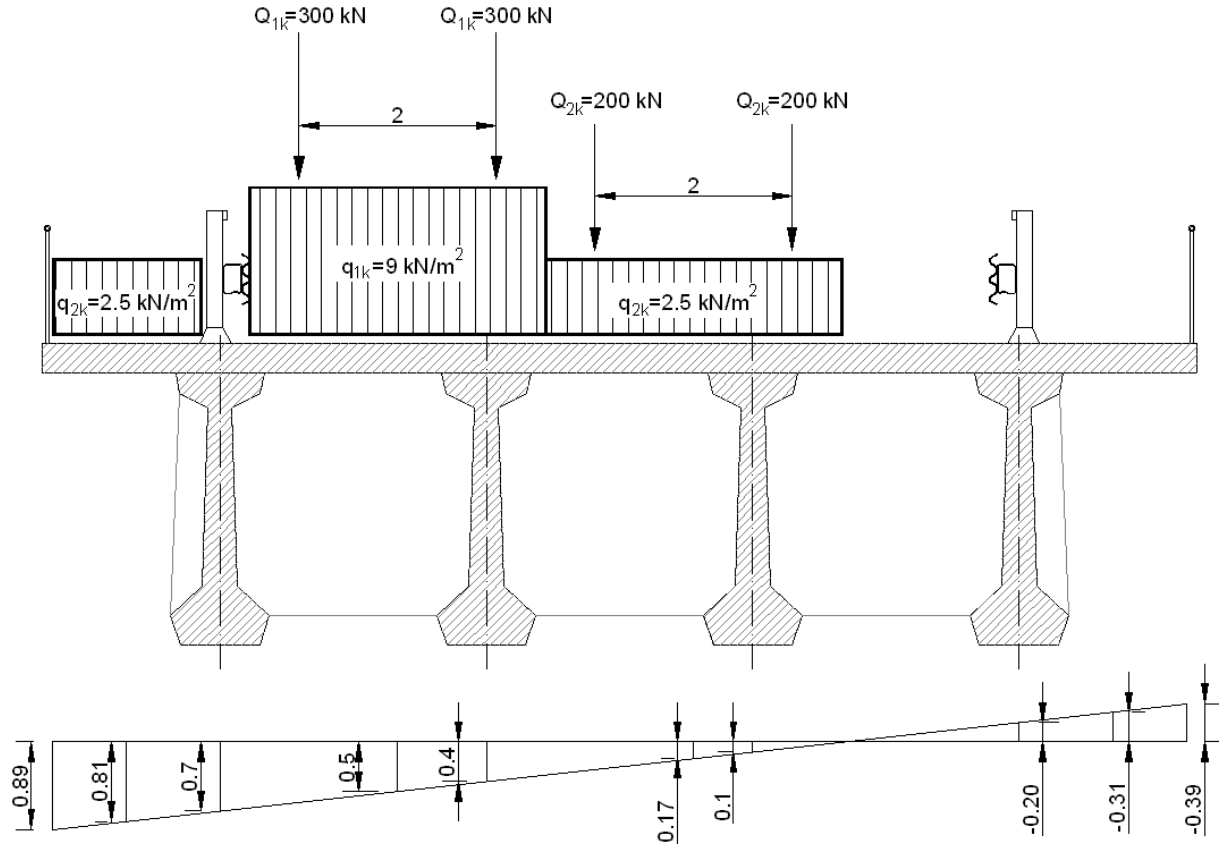
$\Delta T_{M,heat} = +15\text{ }^{\circ}\text{C}$ for top warmer than bottom and to

$\Delta T_{M,cool} = +8\text{ }^{\circ}\text{C}$ for bottom warmer than top



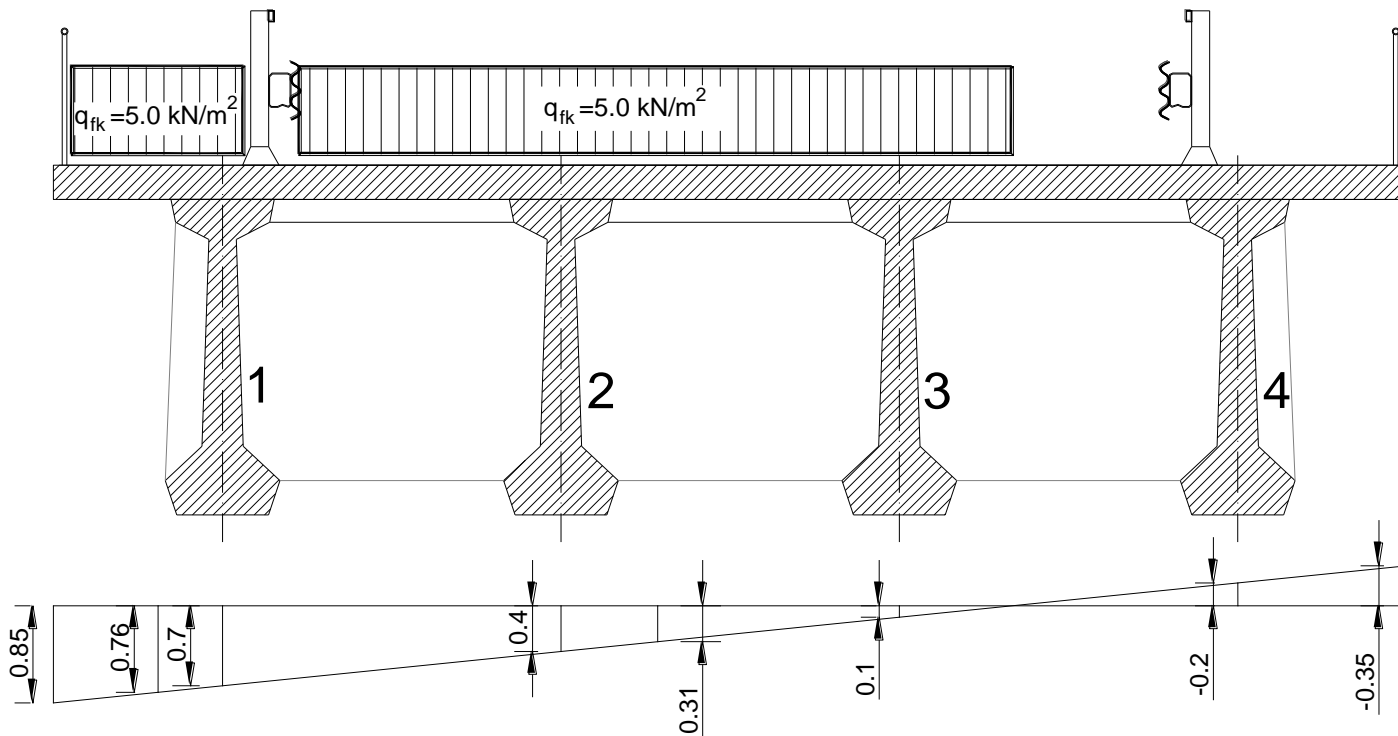
Load effects – Transversal distribution – Traffic load

$$R_j = Q \cdot \left(\frac{1}{n} + \frac{d_j \cdot e}{\sum_{i=1}^n d_i^2} \right)$$



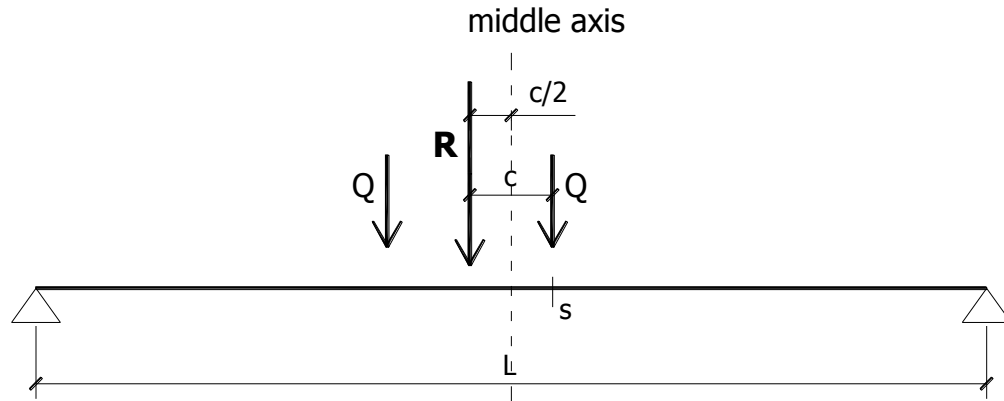
Load effects – Transversal distribution – Crowd loading

$$R_j = Q \cdot \left(\frac{1}{n} + \frac{d_j \cdot e}{\sum_{i=1}^n d_i^2} \right)$$



Calculating the maximum effects on the main beams

Distribution of a moving train of loads that yields the maximum absolute bending moment in the simply supported beam (Asimont's theorem).



Asimont's theorem

$$M_{\max} = M(s) = \frac{R}{4} \cdot \left(L - 2 \cdot c + \frac{c^2}{L} \right)$$

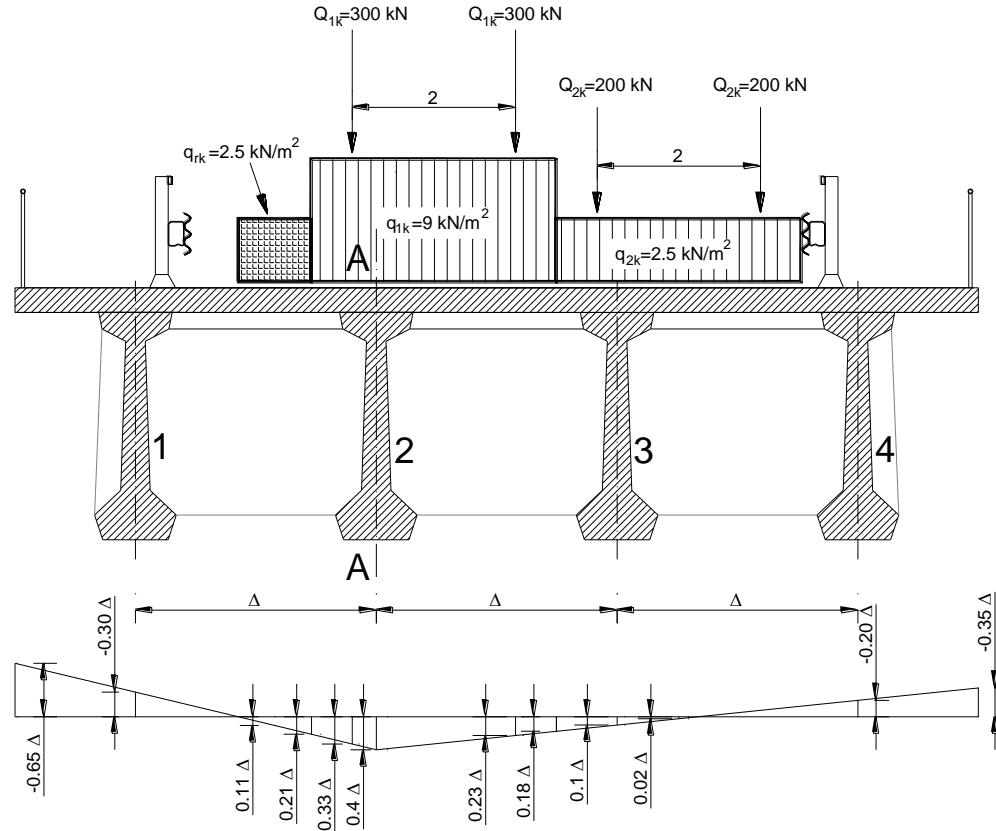
Simplified: R on midspan

$$M'_{\max} = \frac{R}{4} \cdot (L - 2 \cdot c)$$

$$\frac{M_{\max}}{M'_{\max}} \cong 0.018 \%$$

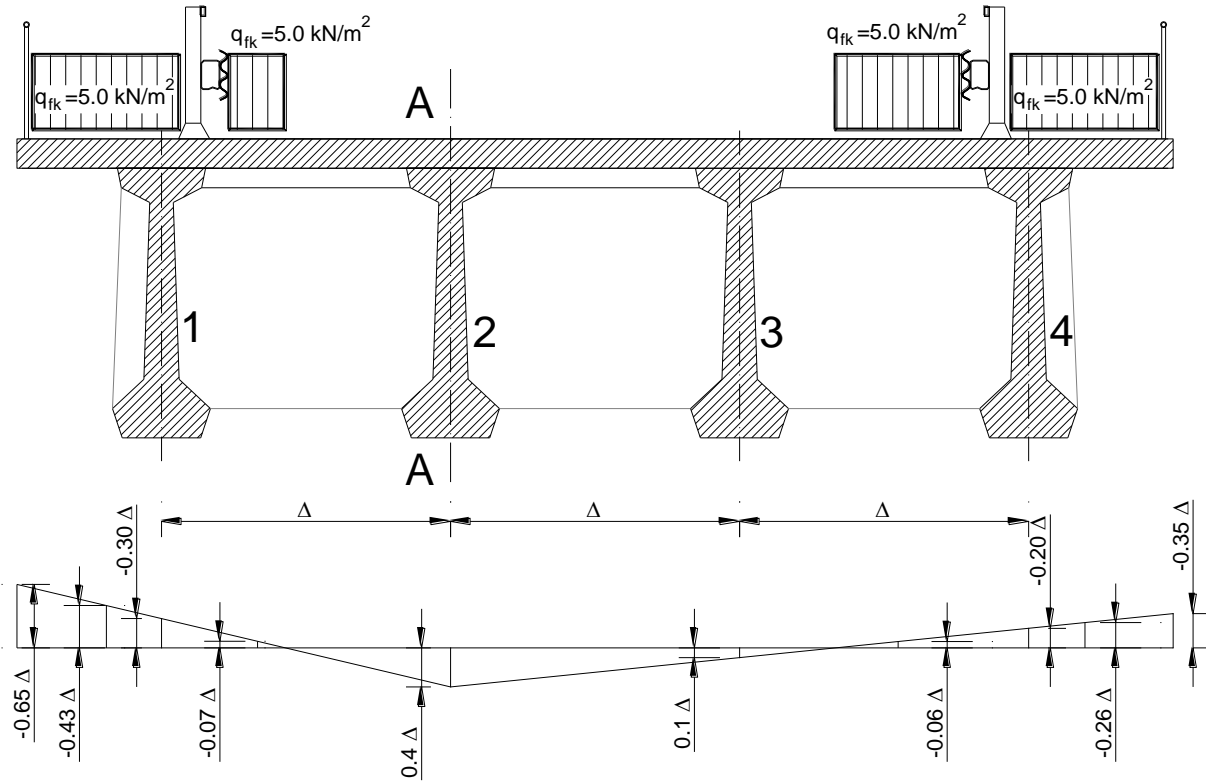
Load effects - Maximum bending moment in the transverse beam

$$R_j = Q \cdot \left(\frac{1}{n} + \frac{d_j \cdot e}{\sum_{i=1}^n d_i^2} \right)$$

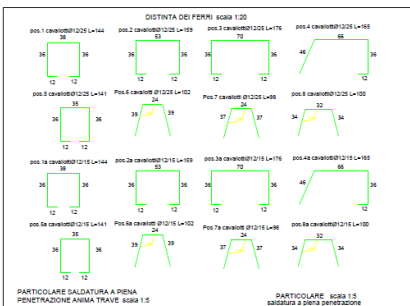


Load effects - Minimum bending moment in the transverse beam

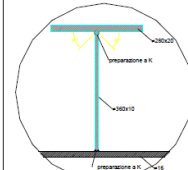
$$R_j = Q \cdot \left(\frac{1}{n} + \frac{d_j \cdot e}{\sum_{i=1}^n d_i^2} \right)$$



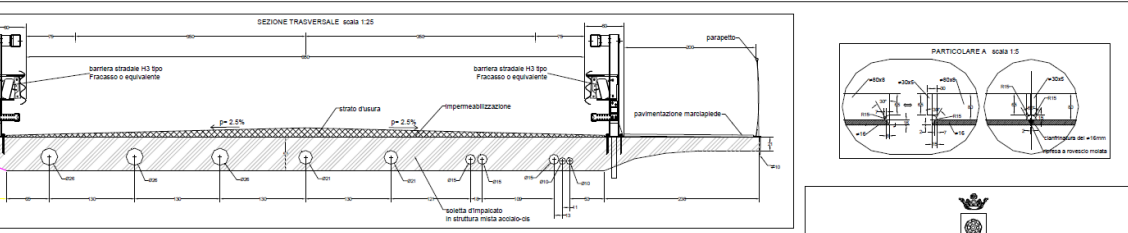
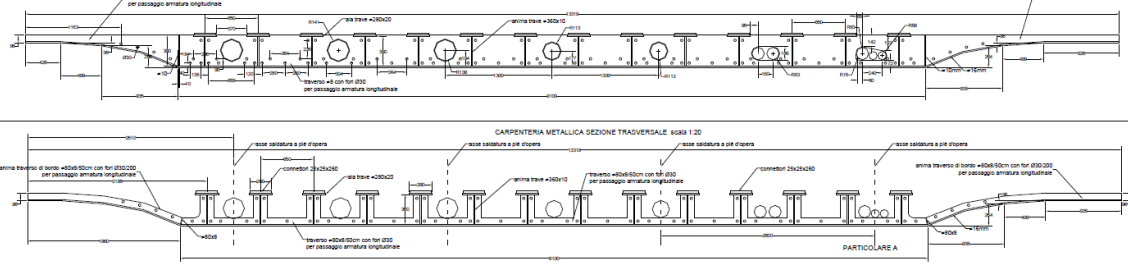
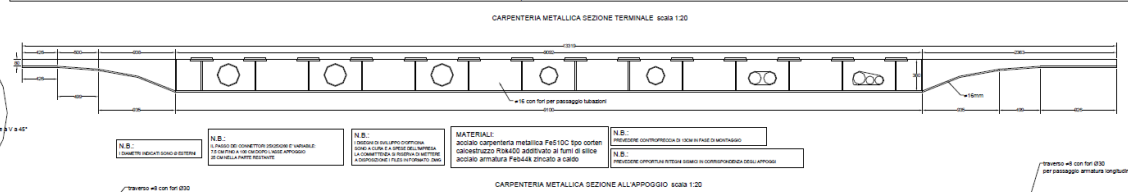
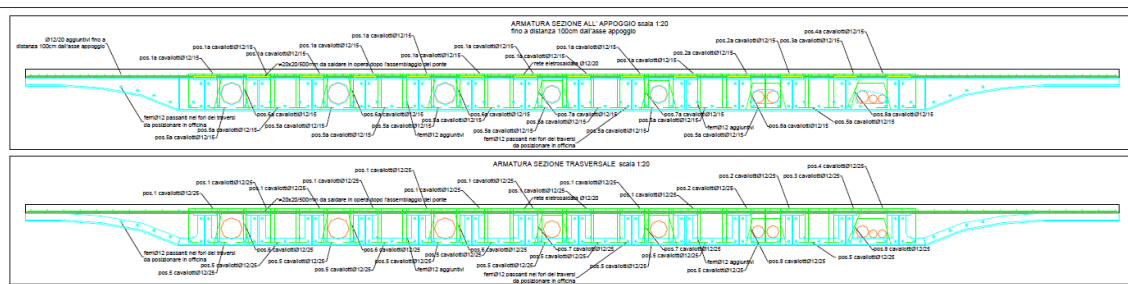
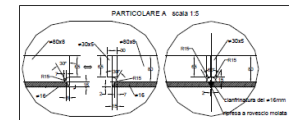
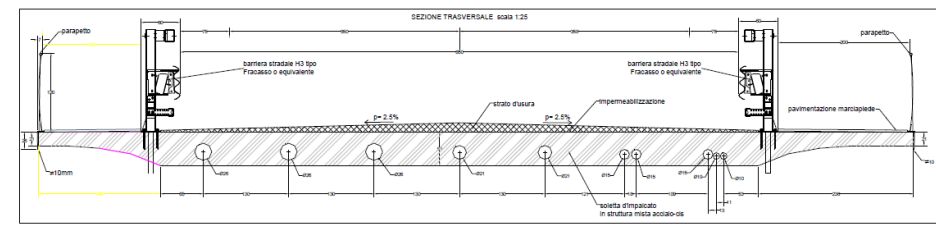
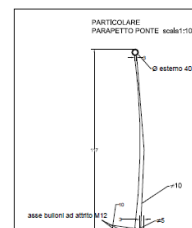
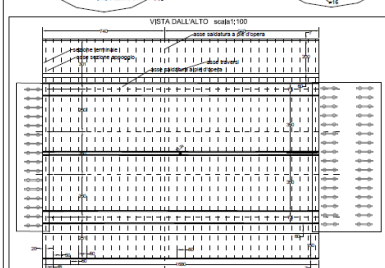
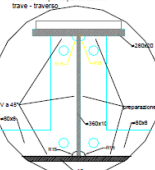
Some less common bridge examples

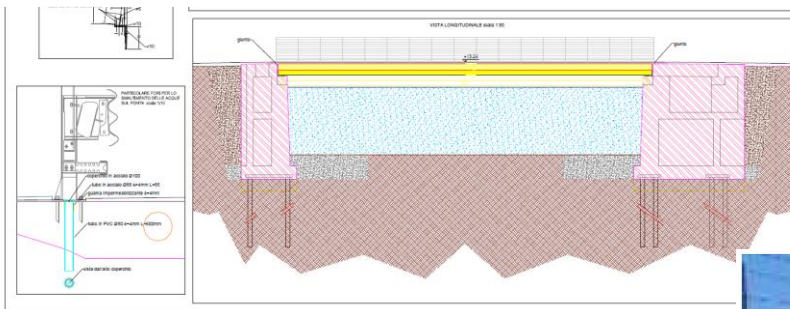


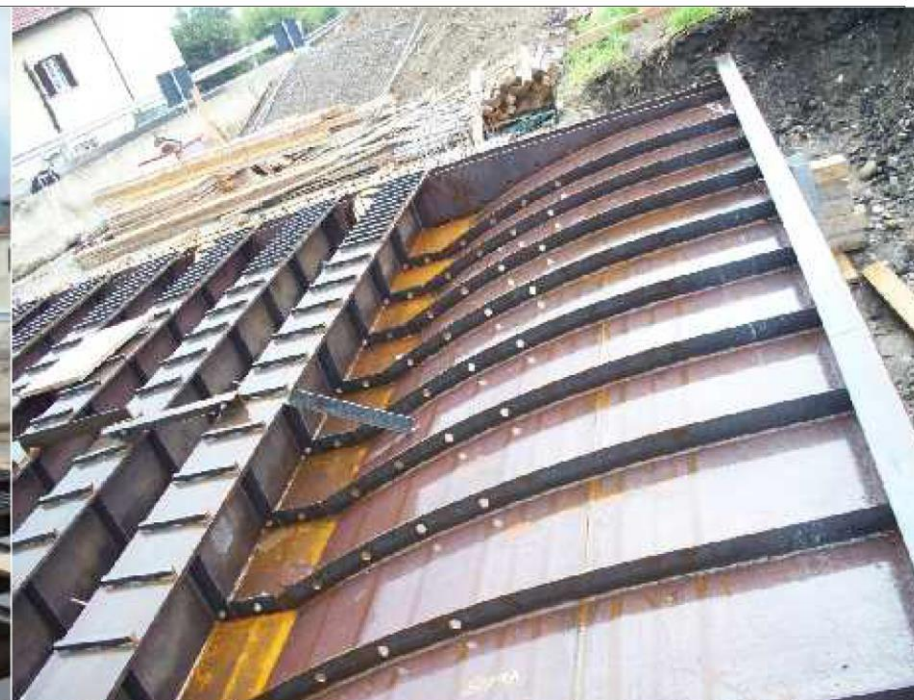
PARTICOLARE SALDATURA A PIENA PENETRAZIONE ANIMA TRAVE SCAIA 15

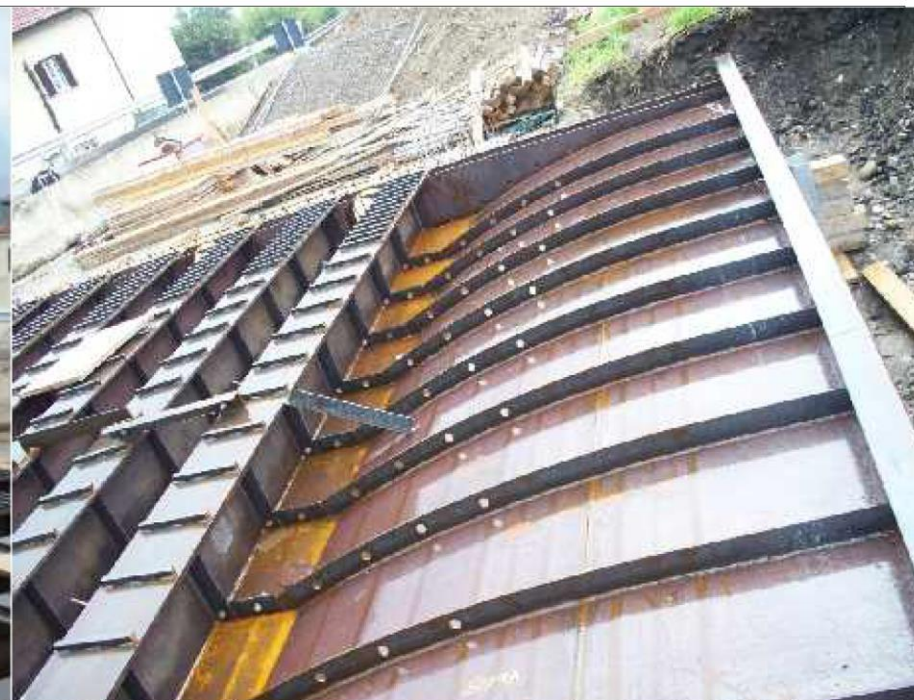


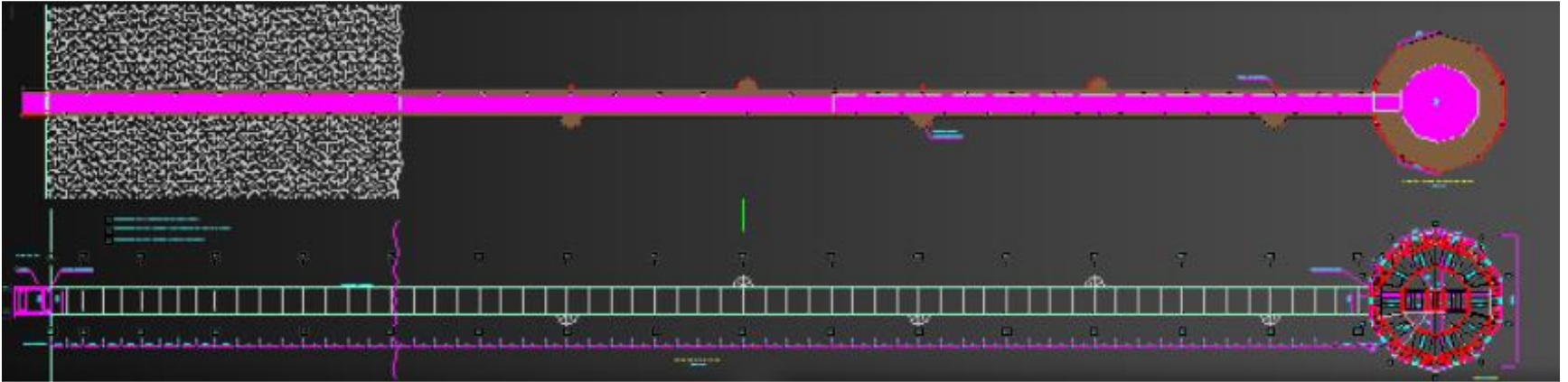
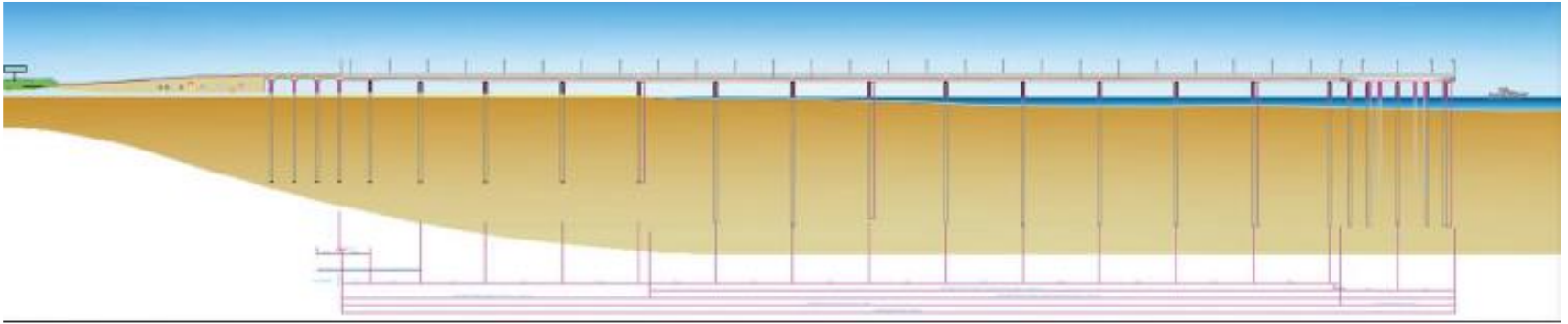
PARTICOLARE SCAIA 15



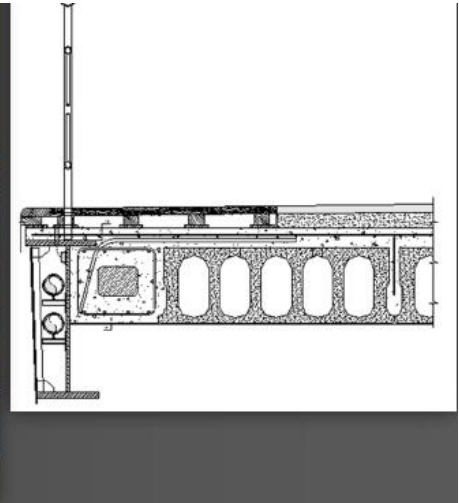
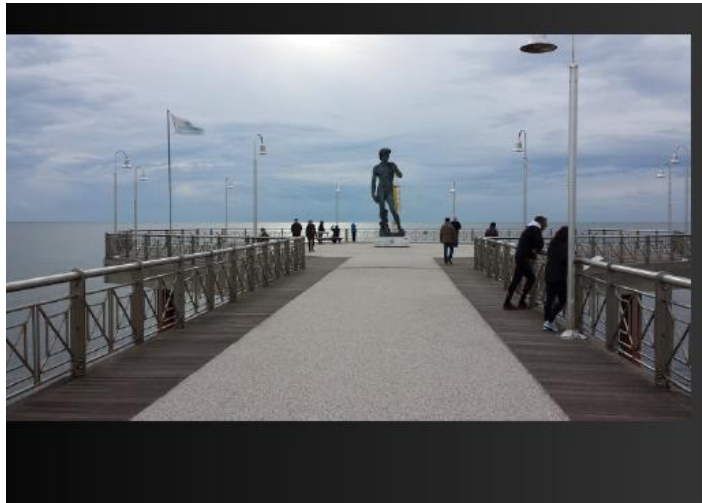
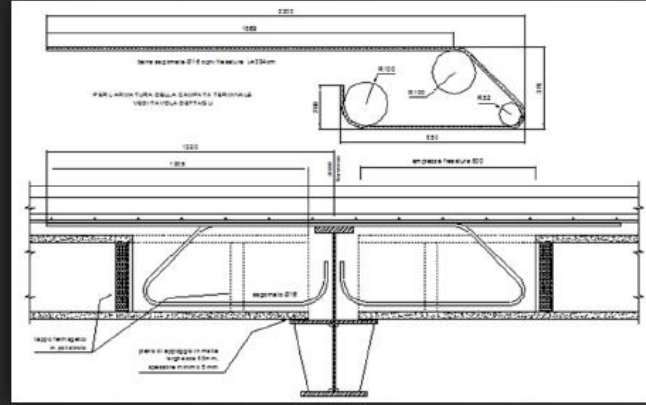


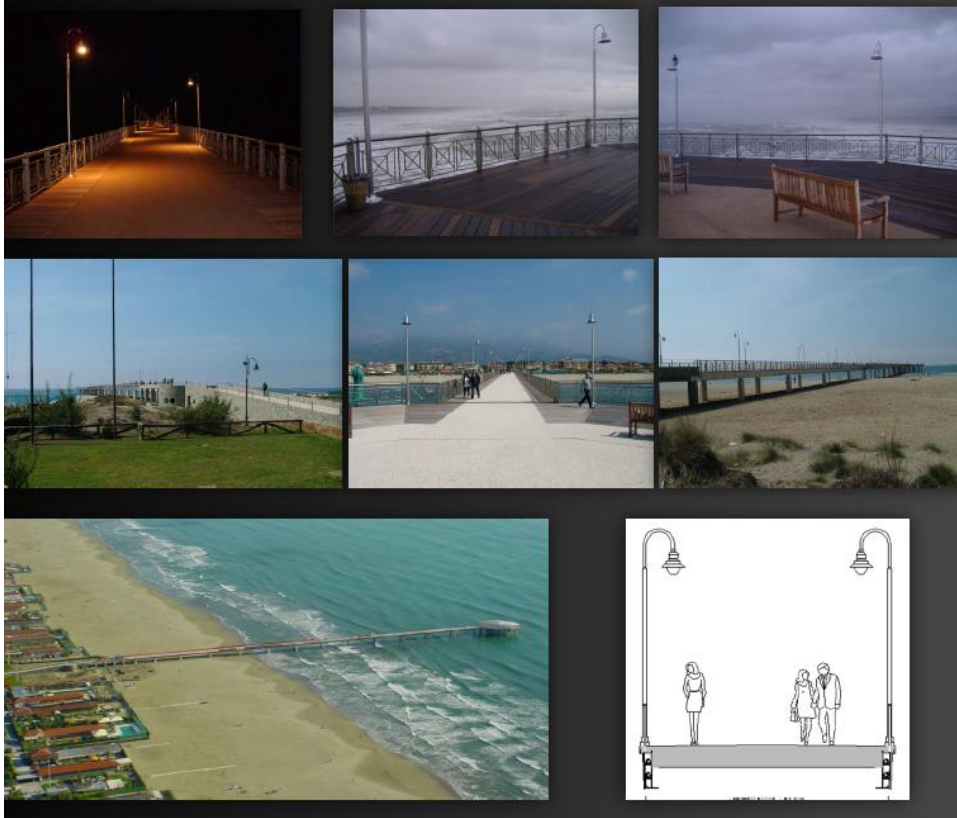












Rehabilitation, repair and seismic upgrading of an historical masonry bridge

The historical masonry bridge, crossing the Magra river, is connecting the small towns of Mulazzo and Villafranca in the northern part of Tuscany (I)



*Single lane carriageway masonry
arch bridge built in 1874:*

*8 arches spanning 19 m around
each;*

*12 m height intermediate masonry
piers on shallow foundations;*

*dept of the original pier
foundations diminishing from
Villafranca toward Mulazzo*

The bridge during the erection phase

In 1874 the course of the river was slightly different; the bridge was on a right bend (velocity of the current higher on Villafranca side (left bank))



To carry two lanes, in 1961 carriageway was widened by means of two lateral prestressed concrete beams, hiding the arches and modifying severely the bridge original aspect.



In 2005-2010 a severe crack pattern appeared in two arches in Mulazzo side, due to scour of the piers



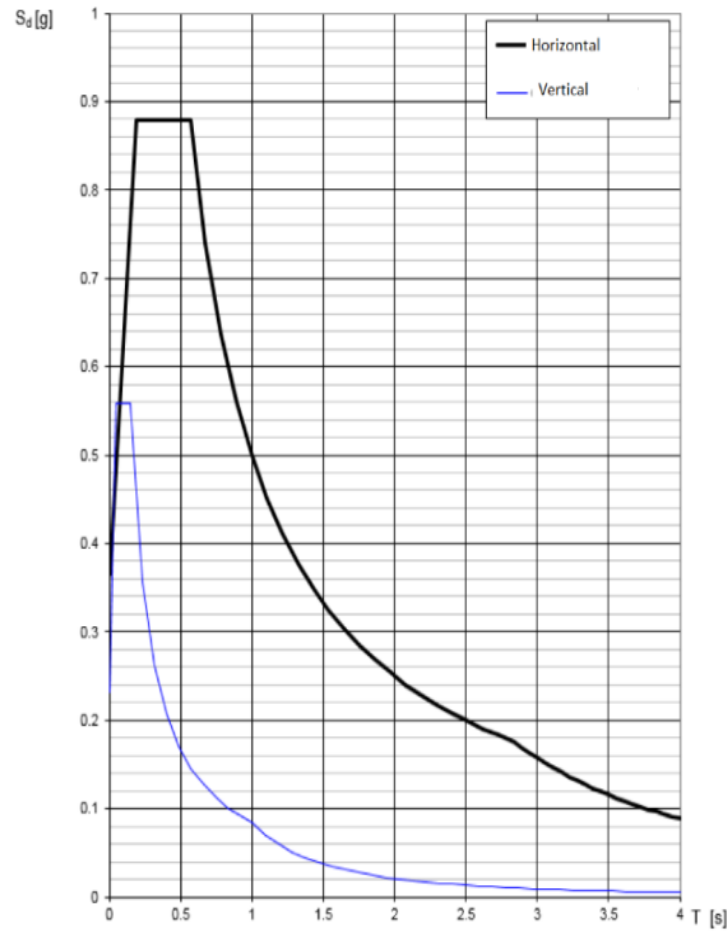
Monitoring and inspection programme



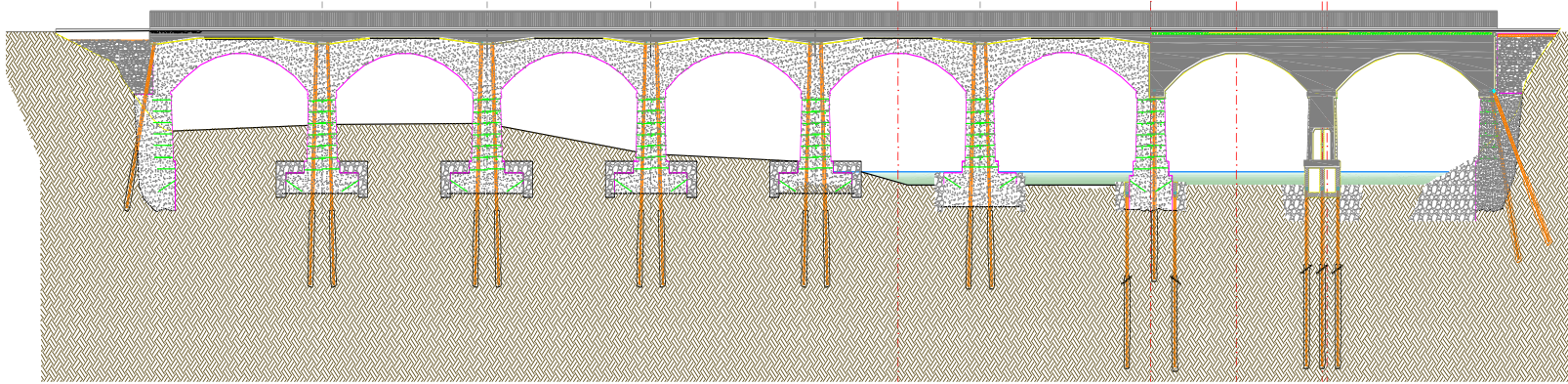
*During the 2011 Magra flooding two arches collapsed
Light traffic was allowed by a Bailey bridges*



*Rehabilitation,
restoration and
seismic
upgrading
(Class IV
structure – 200
years reference
period)*



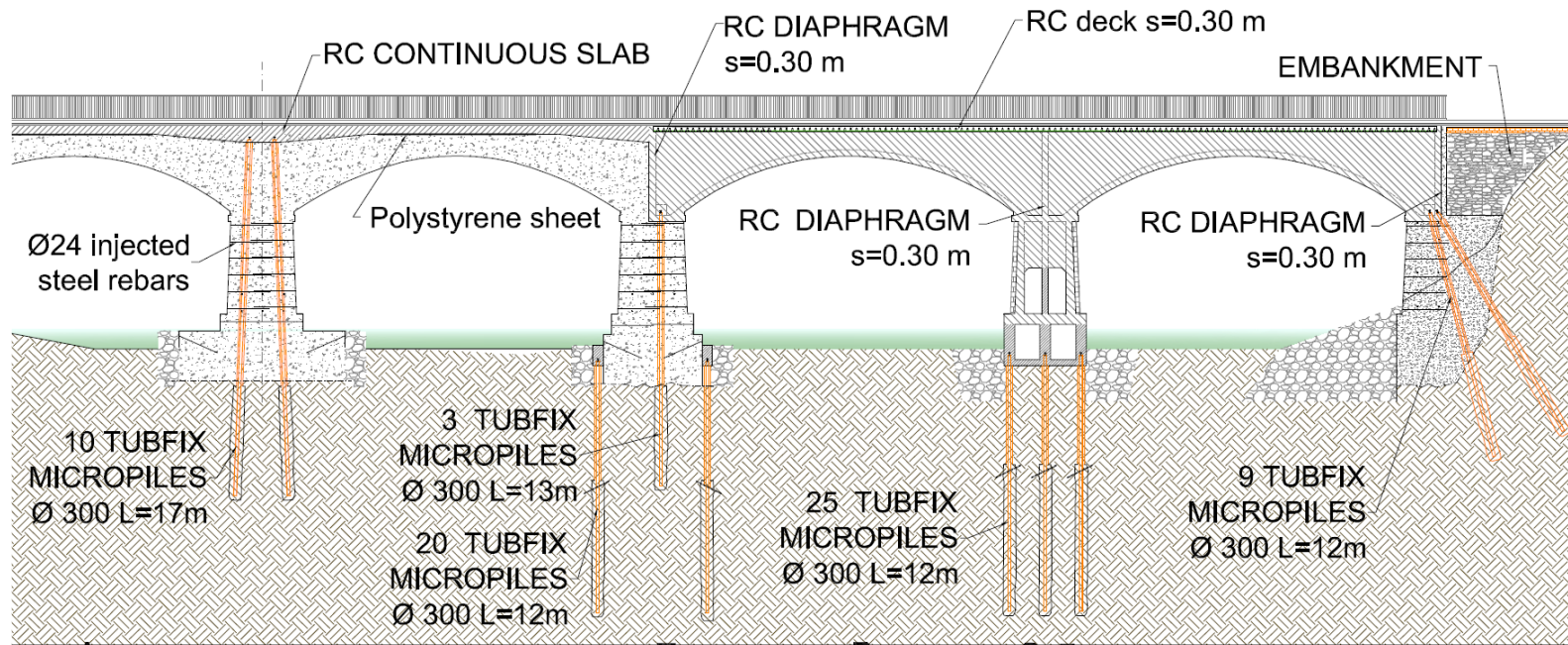
Rationale of the interventions



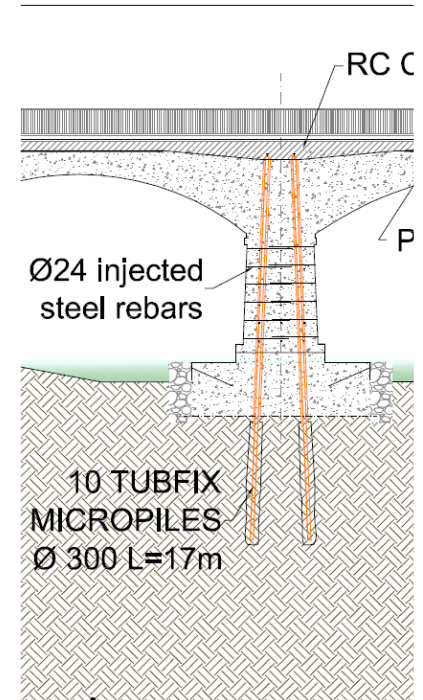
Recovering the original aspect



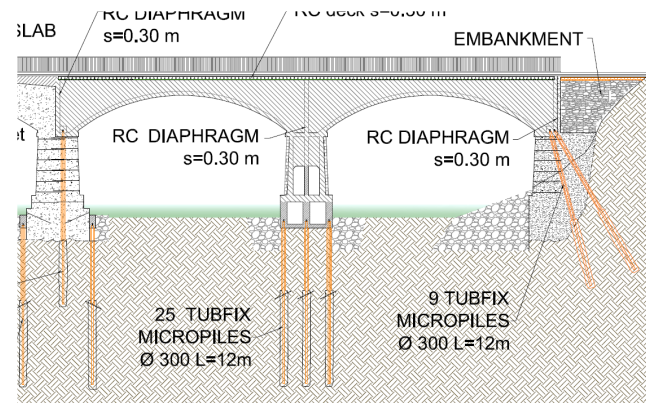
Details of the strengthening intervention



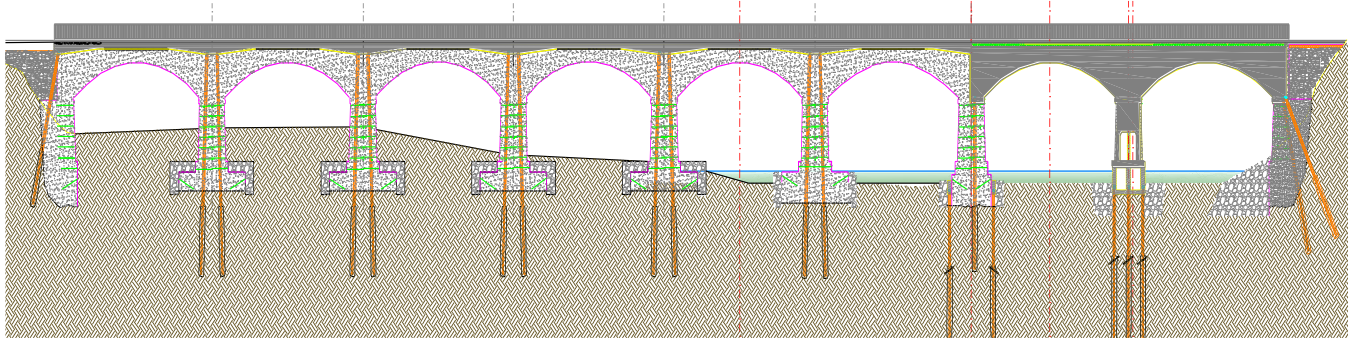
Details of the strengthening intervention



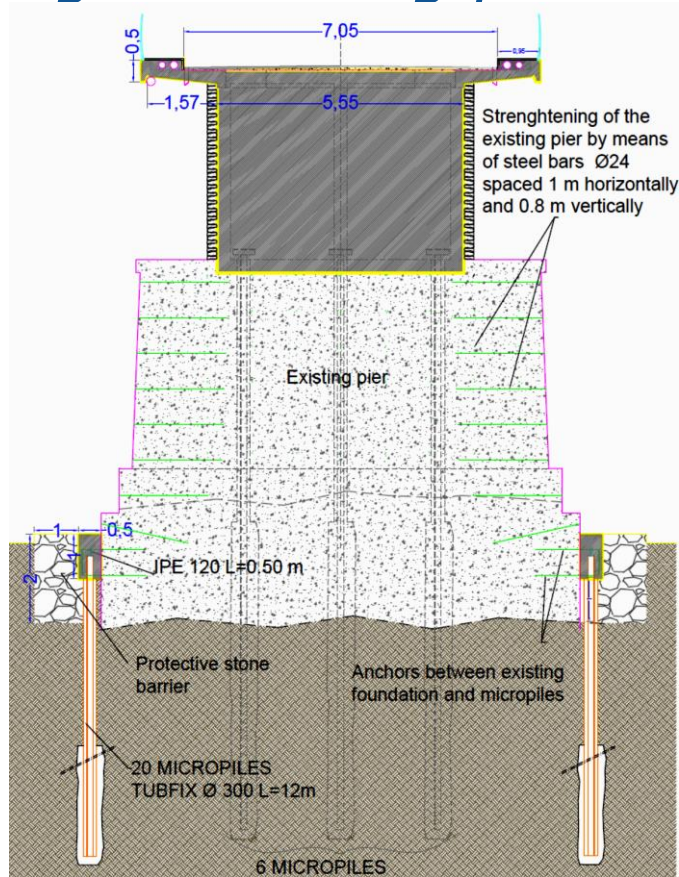
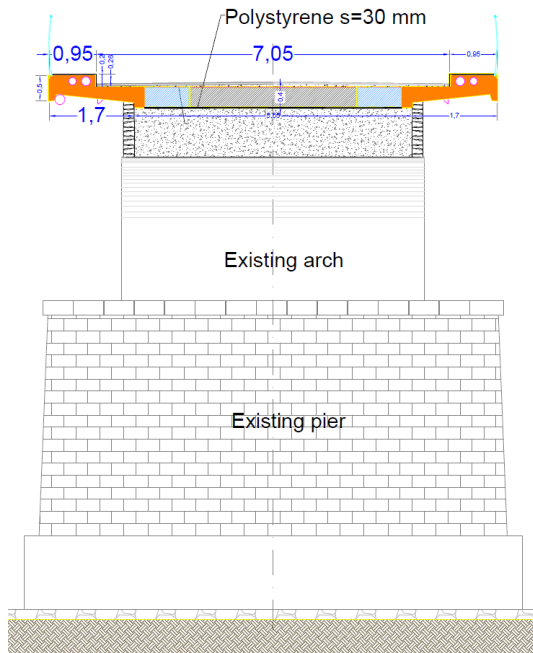
Erection of the new concrete arches



The rc continuous slab



Strengthening of existing piers



- Phase 1: strengthening of the external body of existing piers (steel bars $\phi 24$, spaced 0.8 m vertically and 1.0 m horizontally, duly injected) and foundations of the existing piers (external micropiles connected to the existing foundations and protected from scour by means of stone barrier (light traffic permitted));

- Phase 2: erection of foundation and body of the new concrete pier (light traffic permitted on the bridge);



- Phase 3: setup of alternative routes to mitigate the effects of the closure of the bridge;
- Phase 4: dismantling of prefabricated beams in c.a.p. put in place in 1961 to widen the carriageway and restoration of the external surfaces
- Phase 5: disassembly of the Bailey bridge;
- Phase 6: erection of the two new spans in c.a. cast in place (Fig. 19);
- Phase 7: strengthening of the remaining part of the bridge (6 arches);
- Phase 8: erection of the new r.c. concrete deck slab (Fig. 19);
- Phase 9: finishes.

- Phase 3: setup of alternative routes to mitigate the effects of the closure of the bridge;
- Phase 4: dismantling of prefabricated beams in c.a.p. put in place in 1961 to widen the carriageway and restoration of the external surfaces



- Phase 7: strengthening of the remaining part of the bridge (6 arches);
- Phase 8: erection of the new r.c. concrete deck slab
- Phase 9: finishes.





Thank you for your attention