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

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

Table 1. Daily traffic flows per lane

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

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

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

**Unreliable data**
**Axle loads spectra**

**Total loads spectra**

**Total load distributions**
EN 1991-2: TRAFFIC LOADS ON BRIDGES

Traffic jam on the Europa Bridge
(from Tschermmenegg)

Extreme traffic scenarios
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

<table>
<thead>
<tr>
<th>Carriageway width</th>
<th>Number of notional lanes</th>
<th>Notional lane width</th>
<th>Width of the remaining area</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w &lt; 5.4 \text{ m}$</td>
<td>$n_\ell = 1$</td>
<td>$3 \text{ m}$</td>
<td>$w - 3 \text{ m}$</td>
</tr>
<tr>
<td>$5.4 \text{ m} \leq w &lt; 6 \text{ m}$</td>
<td>$n_\ell = 2$</td>
<td>$w / 2$</td>
<td>$0$</td>
</tr>
<tr>
<td>$6 \text{ m} \leq w$</td>
<td>$n_\ell = \text{int}(w / 3)$</td>
<td>$3 \text{ m}$</td>
<td>$w - 3 \times n_\ell$</td>
</tr>
</tbody>
</table>

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.

Lane n. 1
\[ Q_{1k} = 300 \text{ kN} \]
\[ q_{1k} = 9.0 \text{ kN/m} \]

Lane n. 2
\[ Q_{2k} = 200 \text{ kN} \]
\[ q_{2k} = 2.5 \text{ kN/m}^2 \]

Lane n. 3
\[ Q_{3k} = 100 \text{ kN} \]
\[ q_{3k} = 2.5 \text{ kN/m}^2 \]

Remaining area
\[ q_{k} = 2.5 \text{ kN/m}^2 \]
<table>
<thead>
<tr>
<th>Location</th>
<th>Tandem system $TS$</th>
<th>UDL system</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Axle loads $Q_{ik}$ (kN)</td>
<td>$q_{ik}$ (or $q_{ik}$) (kN/m$^2$)</td>
</tr>
<tr>
<td>Lane Number 1</td>
<td>300</td>
<td>9</td>
</tr>
<tr>
<td>Lane Number 2</td>
<td>200</td>
<td>2.5</td>
</tr>
<tr>
<td>Lane Number 3</td>
<td>100</td>
<td>2.5</td>
</tr>
<tr>
<td>Other lanes</td>
<td>0</td>
<td>2.5</td>
</tr>
<tr>
<td>Remaining area ($q_{ik}$)</td>
<td>0</td>
<td>2.5</td>
</tr>
</tbody>
</table>
Load models for road bridges: LM2 – isolated single axle

**Recommended value:**

\[ \beta_Q = \alpha_Q \]

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):**

\[ Q_x = 0,60 \alpha Q_1 (2Q_{1k}) + 0,10 \alpha q_1 q_{1k} w_1 L \]

**Centrifugal loads**

<table>
<thead>
<tr>
<th>( Q_{tk} )</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>( Q_{tk} = 0,2Q_v ) (kN)</td>
<td>( r &lt; 200 \text{ m} )</td>
</tr>
<tr>
<td>( Q_{tk} = 40Q_v / r ) (kN)</td>
<td>( 200 \leq r \leq 1500 \text{ m} )</td>
</tr>
<tr>
<td>( Q_{tk} = 0 )</td>
<td>( r &gt; 1500 \text{ m} )</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Group of loads</th>
<th>Carriageway</th>
<th>Footways and cycle tracks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vertical loads</td>
<td>Horizontal loads</td>
</tr>
<tr>
<td></td>
<td>Carriageway</td>
<td>Footways and cycle tracks</td>
</tr>
<tr>
<td></td>
<td>Carriageway</td>
<td>Footways and cycle tracks</td>
</tr>
<tr>
<td></td>
<td>Carriageway</td>
<td>Footways and cycle tracks</td>
</tr>
<tr>
<td></td>
<td>Carriageway</td>
<td>Footways and cycle tracks</td>
</tr>
<tr>
<td></td>
<td>Carriageway</td>
<td>Footways and cycle tracks</td>
</tr>
<tr>
<td>1</td>
<td>Characteristic values</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Frequent values</td>
<td>Characteristic values</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>Characteristic values</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>Characteristic values</td>
</tr>
<tr>
<td>5</td>
<td>see 3.5 and figure 19</td>
<td>Characteristic values</td>
</tr>
</tbody>
</table>
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:**

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

Increased by 6% \( = 48.6 \text{ kN/m} \)

**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, \( w_i = 3.0 \text{ m} \)) 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**

<table>
<thead>
<tr>
<th>Lane</th>
<th>$Q_k$ [kN]</th>
<th>$q_k$ [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane 1</td>
<td>300</td>
<td>9.0</td>
</tr>
<tr>
<td>Lane 2</td>
<td>200</td>
<td>2.5</td>
</tr>
<tr>
<td>Lane 3</td>
<td>100</td>
<td>2.5</td>
</tr>
<tr>
<td>Residual area</td>
<td>0</td>
<td>2.5</td>
</tr>
</tbody>
</table>

---

**Diagram**

- $Q_{1k} = 300$ kN
- $Q_{2k} = 200$ kN
- $Q_{3k} = 100$ kN
- $q_1 = 9$ kN/m²
- $q_2 = 2.5$ kN/m²
- $q_3 = 2.5$ kN/m²
- Remaining area

---

**Notional lanes**

- Notional lane 1
- Notional lane 1
- Notional lane 2
- Remaining area
Wind actions

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

### Road restraint systems and shields

<table>
<thead>
<tr>
<th></th>
<th>On one side</th>
<th>On both sides</th>
</tr>
</thead>
<tbody>
<tr>
<td>Open parapet or open safety barrier</td>
<td>(d+0.3) m</td>
<td>(d+0.6) m</td>
</tr>
<tr>
<td>Solid parapet or solid safety barrier</td>
<td>(d+d_1)</td>
<td>(d+2) (d_1)</td>
</tr>
<tr>
<td>Open parapet and open safety barrier</td>
<td>(d+0.6) m</td>
<td>(d+1.2) m</td>
</tr>
</tbody>
</table>
Wind actions

\[ F_{wk} = \frac{1}{2} \rho v_b^2 C A_{\text{ref}} \]

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 \( c_e(z_e) = c_r(z_e) c_0(z_e) \left[ 1 + 7 \cdot I_v(z_e) \right] \)
c\(_{f,x}\) is the force (drag) coefficient

Orography factor \( c_0=1 \)
\[ I_v(z_e) = \begin{cases} \frac{k_1}{c_0(z_e) \ln \left( \frac{z}{z_0} \right)} & \text{if } z_{\text{min}} < z \\ I_v(z_{\text{min}}) & \text{if } z_{\text{min}} \geq z \end{cases} \]

Turbulence intensity
Turbulence factor \( k_1=1 \)

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 \( z_0=0.05 \text{ m} \)  minimum height \( z_{\text{min}}=2 \text{ m} \)
Terrain factor \( k_r=0.19 \)
Wind actions

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

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

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

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

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

\[ F_{w,z} = q_p(z_e) \cdot c_{f,z} \cdot A_{\text{ref},z} = 1.307 \text{ kN/m}^2 \cdot (\pm 0.9) \cdot 11.60 \text{ m} = \pm 13.64 \text{ kN/m} \]
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  \( c_{f,x} = 1.3 \)

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

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

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

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

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

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

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

$F_{w,x} = 0.6 \cdot 1.307 \cdot 1.99 \cdot 0.95 \cdot 6.8 = 10.07 \text{kN/m} \quad \text{NAD(I)}$
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 $T_{\text{max}}=40 \, ^\circ \text{C}$ and $T_{\text{min}}=-10 \, ^\circ \text{C}$, respectively, the uniform bridge temperature components for a steel bridge result $T_{e,\text{max}}=56.6 \, ^\circ \text{C}$ and $T_{e,\text{min}}=-13.3 \, ^\circ \text{C}$

$\Delta T=\pm 20 \, ^\circ \text{C} \quad \text{– NAD (I))}$
Thermalaactions - Non uniform variations

Simplified approach:
Stress calculation

Local effects on the deck plate

**LM1 - TS**

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

**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
Calculating the main structure

The effects of self-weight and dead loads
$\gamma_G=1.35$ unfavourable - $\gamma_G=1.00$ favourable

Single source principle

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

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

\[ M_g(C) = \frac{1}{40} \cdot (g_b + g_p) \cdot L^2 = \frac{1}{40} \cdot 74.12 \cdot 120.0^2 = 26681.8 \text{ kNm} \]
Effects of traffic loads

$$\gamma_Q = 1.35 \text{ unfavourable} - \gamma_Q = 0 \text{ favourable}$$

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

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

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

(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).

$$Q_{cd} = 1.35 \cdot 1200 = 1620 \text{ kN}$$

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.
$Q_d = 1620$ kN

$q_d = 54.47$ kN/m
Calculation of the transverse section

maximum bending moment in the box girder

maximum torque in the box girder
Application of transverse wind load

\[ \gamma_Q = 1.5 \text{ unfavourable} - \gamma_Q = 0 \text{ favourable} \]

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

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

\[ F_{w,xd} = 1.5 \cdot 10.07 = 15.10 \text{ kN/m} \quad \text{NAD(I)} \]
Thermal actions - Non uniform variations

Simplified approach:

\[
M_{\Delta T_{18}} = \frac{\alpha \cdot \Delta T}{h} \cdot \frac{6 \cdot EJ}{5} = 13530.9 \text{ kNm}
\]

\[
M_{\Delta T_{13}} = \frac{\alpha \cdot \Delta T}{h} \cdot \frac{6 \cdot EJ}{5} = -9772.3 \text{ kNm}
\]
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**

\[
\begin{align*}
  n_l &= \text{Int} \left[ \frac{w}{3} \right] = \text{Int} \left[ \frac{7.50}{3} \right] = 2 \\
  w_r &= w - n_l \cdot 3.0 \text{m} = 7.50 - 2 \cdot 3.0 = 1.50 \text{m}
\end{align*}
\]
Load Model 1

Conventional lane

<table>
<thead>
<tr>
<th>Lane</th>
<th>$Q_k$ [kN]</th>
<th>$q_k$ [kN/m$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lane 1</td>
<td>300</td>
<td>9.0</td>
</tr>
<tr>
<td>Lane 2</td>
<td>200</td>
<td>2.5</td>
</tr>
<tr>
<td>Residual area</td>
<td>0</td>
<td>2.5</td>
</tr>
</tbody>
</table>
Load Model 4 – Crowd loading

$q_{nk} = 5.0 \text{kN/m}^2$
Wind action (unloaded)

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

$$c_{f,x} = 0.9 \left( 2.4; \max \left( 2.5 - 0.3 \frac{b}{d_{tot}} ; 1.3 \right) \right) = 2.5 - 0.3 \frac{11.8}{4.26} = 1.669.$$

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

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

\[
c_{fx} = \min \left( 2.4; \max \left( 2.5 - 0.3 \frac{b}{d_{tot}}; 1.0 \right) \right) = 2.5 - 0.3 \frac{11.8}{5.06} = 1.80
\]

or, according to the Italian National Annex,

\[
c_{fx}' = \min \left( 2.4; \max \left( 2.5 - 0.3 \frac{b}{d_{tot}}; 1.0 \right) \right) = 2.5 - 0.3 \frac{11.8}{6.06} = 1.916
\]

and also for loaded bridge the simplification \( c_{fx}' = 1.3 \) results unsafe-sided.

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

\[
\psi_{0w} F_{w,k,x} = \psi_{0w} q_p (z_e) c_{fx}' A_{ref,x} = 0.6 \cdot 0.54 \cdot 1.8 \cdot 5.06 = 2.95 \text{ kN/m}
\]

\[
\psi_{0w} F_{w,k,x} = \psi_{0w} q_p (z_e) c_{fx}' A_{ref,x} = 0.6 \cdot 0.54 \cdot 1.916 \cdot 6.06 = 3.76 \text{ kN/m}
\]
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 $T_{\text{max}} = 40 \, ^{\circ}\text{C}$ and $T_{\text{min}} = -10 \, ^{\circ}\text{C}$, respectively, the uniform bridge temperature components for a concrete bridge result $T_{e,\text{max}} = 41.7 \, ^{\circ}\text{C}$ and $T_{e,\text{min}} = -1.8 \, ^{\circ}\text{C}$
Thermal action (non uniform)

Linearly varying vertical temperature difference components can be set to
\[ \Delta T_{M,\text{heat}} = +15 \, ^\circ\text{C} \] for top warmer than bottom and to
\[ \Delta T_{M,\text{cool}} = +8 \, ^\circ\text{C} \] 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

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

\[ q_{ik} = 5.0 \text{ kN/m}^2 \]

q = 5.0 kN/m

0.76

fk

0.31

0.76

0.7

0.4

0.31

0.1

-0.2

-0.35
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

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

Simplified: R on midspan

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

\[
\frac{M_{\text{max}}}{M'_{\text{max}}} \approx 0.018\%
\]
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

\[ R_j = Q \left( \frac{1}{n} + \frac{d_j \cdot e}{\sum_{i=1}^{n} d_i^2} \right) \]
Some less common bridge examples
“The way forward for the Eurocodes implementation in the Balkans”, 10-11 October 2018, Tirana
"The way forward for the Eurocodes implementation in the Balkans", 10-11 October 2018, Tirana
"The way forward for the Eurocodes implementation in the Balkans", 10-11 October 2018, Tirana
“The way forward for the Eurocodes implementation in the Balkans”, 10-11 October 2018, Tirana
“The way forward for the Eurocodes implementation in the Balkans”, 10-11 October 2018, Tirana
“The way forward for the Eurocodes implementation in the Balkans”, 10-11 October 2018, Tirana
Rehabilitation, repair and seismic upgrading of an historical amsonry 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;
depth 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

“The way forward for the Eurocodes implementation in the Balkans”, 10-11 October 2018, Tirana
During the 2011 Magra flooding two arches collapsed. Light traffic was allowed by a Bailey bridge.
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

- Ø24 injected steel rebars
- Polystyrene sheet
- 10 TUBFIX MICROPILES Ø 300 L=17m
- 3 TUBFIX MICROPILES Ø 300 L=13m
- 20 TUBFIX MICROPILES Ø 300 L=12m
- 25 TUBFIX MICROPILES Ø 300 L=12m
- 9 TUBFIX MICROPILES Ø 300 L=12m

RC CONTINUOUS SLAB
RC DIAPHRAGM s=0.30 m
RC deck s=0.30 m
EMBANKMENT
Details of the strengthening intervention

Ø24 injected steel rebars

10 TUBFIX MICROPILES
Ø 300 L=17m
Erection of the new concrete arches
The rc continuous slab
Strengthening of existing piers

- Polystyrene of 30 mm thickness
- Existing arch
- Existing pier
- Strengthening of the existing pier by means of steel bars Ø24 spaced 1 m horizontally and 0.8 m vertically
- Protective stone barrier
- Anchors between existing foundation and micropiles
- 20 MICROPILES TUBFIX Ø 300 L=12m
- 6 MICROPILES

“The way forward for the Eurocodes implementation in the Balkans”, 10-11 October 2018, Tirana
• 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