EN 1992-2

EUROCODE 2 – Design of concrete structures
Concrete bridges: design and detailing rules

Approved by CEN on 25 April 2005
Published on October 2005

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- EN 1992-2 contains principles and application rules for the design of bridges in addition to those stated in EN 1992-1-1

- Scope: basis for design of bridges in plain/reinforced/prestressed concrete made with normal/light weight aggregates
Section 3 ⇒ MATERIALS

- Recommended values for $C_{\text{min}}$ and $C_{\text{max}}$

  $C_{30/37}$ (Durability)  $C_{70/85}$ (Ductility)

- $\alpha_{cc}$ coefficient for long term effects and unfavourable effects resulting from the way the load is applied.

  Recommended value: 0.85 → high stress values during construction

- Recommended classes for reinforcement:
  “B” and “C”

  (Ductility reduction with corrosion / Ductility for bending and shear mechanisms)
Section 4 ⇒ Durability and cover to reinforcement

- XC3 class recommended for surface protected by waterproofing

- When de-icing salt is used

  Exposed concrete surfaces within (6 m) of the carriage way and supports under expansion joints: directly affected by de-icing salt

  Recommended classes for surfaces directly affected by de-icing salt: XD3 – XF2 – XF4, with covers given in tables 4.4N and 4.5N for XD classes
- Bare concrete decks without waterproofing or surfacing should be classified as abrasion class XM2

- When concrete surface is subject to abrasion by ice or solid transportation in running water → increase the cover by 10 mm, min
Section 5 ⇒ Structural analysis

- Geometrical imperfections

Piers
\[ \vartheta = \frac{1}{200} \text{ (recom.)} \]

Arches

Shape of imperfections based on the shape of first horizontal and vertical buckling mode, idealised by a sinusoidal profile having amplitude

\[ a = \vartheta \frac{l}{2} \text{ (l = half wavelength)} \]
- Linear elastic analysis with limited Redistributions

Limitation of $\delta$ due to uncertainties on size effect and bending-shear interaction

$\delta \geq 0.85$ (recommended value)
- Plastic analysis

Restrictions due to uncertainties on size effect and bending-shear interaction:

\[
\frac{x_u}{d} \leq \begin{cases} 
0.15 & \text{for concrete strength classes } \leq C50/60 \\
0.10 & \text{for concrete strength classes } \geq C55/67 
\end{cases}
\]
- Rotation capacity

Restrictions due to uncertainties on size effect and bending-shear interaction:

\[ \frac{x_u}{d} \leq 0.30 \text{ for concrete strength classes } \leq C50/60 \\
0.23 \text{ for concrete strength classes } \geq C55/67 \]
Numerical rotation capacity

\[ \gamma_{pL} \text{ [mrad]} \]

\[ x/d \]

\[ h = 0.2 \text{ m} \]

0
10
20
30
40
50
60
70
0.00 0.10 0.20 0.30 0.40 0.50

0.6
0.4
0.8
Nonlinear analysis \( \Rightarrow \) Safety format

- Reinforcing steel
  - Mean values
  - \( 1.1 \ f_{yk} \)

- Prestressing steel
  - Mean values
  - \( 1.1 \ k \ f_{yk} \)
  - \( 1.1 \ f_{pk} \)

- Concrete
  - Sargin modified mean values
  - \( \gamma_{cf} \ f_{ck} \)
  - \( \gamma_{cf} = 1.1 \ \gamma_{s} / \gamma_{c} \)
Design format

- Incremental analysis from SLS, so to reach $\gamma_G G_k + \gamma_Q Q$ in the same step

- Continuation of incremental procedure up to the peak strength of the structure, in correspondence of ultimate load $q_{ud}$

- Evaluation of structural strength by use of a global safety factor $\gamma_0$

$$R \left( \frac{q_{ud}}{\gamma_0} \right)$$
Verification of one of the following inequalities

\[ \gamma_{Rd} E\left( \gamma_G G + \gamma_Q Q \right) \leq R \left( \frac{q_{ud}}{\gamma_O} \right) \]

\[ E\left( \gamma_G G + \gamma_Q Q \right) \leq R \left( \frac{q_{ud}}{\gamma_{Rd} \cdot \gamma_O} \right) \]

(i.e.) \[ R \left( \frac{q_{ud}}{\gamma_{O'}} \right) \]

\[ \gamma_{Rd} \gamma_{Sd} E\left( \gamma_g G + \gamma_q Q \right) \leq R \left( \frac{q_{ud}}{\gamma_O} \right) \]
With
\[ \gamma_{Rd} = 1.06 \text{ partial factor for model uncertainties (resistance side)} \]
\[ \gamma_{Sd} = 1.15 \text{ partial factor for model uncertainties (actions side)} \]
\[ \gamma_0 = 1.20 \text{ structural safety factor} \]

If \( \gamma_{Rd} = 1.00 \) then \( \gamma_0' = 1.27 \) is the structural safety factor
Safety format

Application for scalar combination of internal actions and underproportional structural behaviour
Safety format

Application for scalar combination of internal actions and overproportional structural behaviour

\[
R \left( \frac{q_{ud}}{\gamma_o} \right)
\]

\[
R \left( \frac{q_{ud}}{\gamma_o} \right) / \gamma_{Rd}
\]

\[
R \left( \frac{q_{ud}}{\gamma_o} \right) / \gamma_{Rd} \gamma_{Sd}
\]

\[
\left( \gamma + \gamma \right) / \gamma_{Rd}
\]

\[
\left( \gamma + \gamma \right) / \gamma_{Sd}
\]
Safety format

Application for vectorial combination of internal actions and underproportional structural behaviour
Safety format

Application for vectorial combination of internal actions and overproportional structural behaviour
For vectorial combination and $\gamma_{Rd} = \gamma_{Sd} = 1.00$ the safety check is satisfied if:

\[
M_{ED} \leq M_{Rd} \left( \frac{q_{ud}}{\gamma_0'} \right)
\]

and

\[
N_{ED} \leq N_{Rd} \left( \frac{q_{ud}}{\gamma_0'} \right)
\]
Example 1:

- Two spans R. C. bridge (l = 20 + 20 m)
- Advance shoring (20+5 m / 15 m)
- Dead load at $t_0 = 28$ days and $t_1 = 90$ days
- $\xi (28, 90, \infty) = 0.51$

N. L. analyses at:
- $t_1$: (no redistribution due to creep)
- $t_{\infty}$: (full redistribution due to creep)
Load distribution for the design of the region close to the central support

Load distribution for the design of the midspan

\( q = 32.75 \text{ kN/m} \)

\( g = 101.4 \text{ kN/m} \)

\( 300 \text{ kN} \)

\( 10.80 \)

\( 12.00 \)

\( 8,00 \)

\( 9,20 \)
**Incremental loading process**

- Application of self weight in different statical schemes with $\gamma_G = 1$
- Modification of internal actions by creep by means of $\xi$ function ($\gamma_G = 1$) only for $t = t_\infty$
- Application of other permanent actions ($\gamma_G = 1$) on the final statical scheme
- Application of live loads with $\gamma_G = 1$
- Starting of incremental process so that $\gamma_G = 1.4$ and $\gamma_Q = 1.5$ is reached in the same step
- Continuation of incremental process up to attainment of peak load (Critical region: central support section)
Safety format: $\gamma_{Gi}$

<table>
<thead>
<tr>
<th>Load case (bending moments)</th>
<th>Time</th>
<th>$\gamma_{Gu}$</th>
<th>$\gamma_{Qu}$</th>
<th>$\gamma_{Gu} / \gamma_{Gl}$</th>
<th>$\gamma_{Qu} / \gamma_{Gl}$</th>
<th>$M(\gamma_{Gi})^<em>$ [kN</em>m*10^3]</th>
<th>Gain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum negative (X)</td>
<td>$t_1$</td>
<td>2.03</td>
<td>2.175</td>
<td>1.60</td>
<td>1.71</td>
<td>-12.2</td>
<td>14%</td>
</tr>
<tr>
<td>Maximum negative (Y)</td>
<td>$\infty$</td>
<td>2.03</td>
<td>2.175</td>
<td>1.60</td>
<td>1.71</td>
<td>-12.2</td>
<td>14%</td>
</tr>
<tr>
<td>Maximum positive (W)</td>
<td>$t_1$</td>
<td>1.97</td>
<td>2.11</td>
<td>1.55</td>
<td>1.66</td>
<td>-10.4</td>
<td>11%</td>
</tr>
<tr>
<td>Maximum positive (Z)</td>
<td>$\infty$</td>
<td>1.92</td>
<td>1.07</td>
<td>1.51</td>
<td>1.62</td>
<td>-10.1</td>
<td>7.8%</td>
</tr>
</tbody>
</table>

Where:

$$M(\gamma_{Gi}) = M \left( \frac{\gamma_{Gu} \cdot G + \gamma_{Qu} \cdot Q}{\gamma_{Gi}} \right)$$

$G$ and $Q$ are the design values for gravity and accidental loads, respectively.
### Safety format: $\gamma_{gl}$

<table>
<thead>
<tr>
<th>Load case (bending moments)</th>
<th>Time</th>
<th>$\gamma_{Gu}/\gamma_{gl}$</th>
<th>$M(\gamma_{gl})^<em>$ [kNm</em>10³]</th>
<th>$M(\gamma_{gl})/\gamma_{Rd}$ [kNm*10³]</th>
<th>$\gamma_G(M(\gamma_{gl})/\gamma_{Rd})$</th>
<th>Gain $\Gamma$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum negative (X)</td>
<td>$t_1$</td>
<td>1.69</td>
<td>-12.0</td>
<td>-11.3</td>
<td>1.60</td>
<td>14%</td>
</tr>
<tr>
<td>Maximum negative (Y)</td>
<td>$\infty$</td>
<td>1.69</td>
<td>-12.2</td>
<td>-11.5</td>
<td>1.53</td>
<td>7.1%</td>
</tr>
<tr>
<td>Maximum positive (W)</td>
<td>$t_1$</td>
<td>1.64</td>
<td>-9.96</td>
<td>-9.40</td>
<td>1.55</td>
<td>11%</td>
</tr>
<tr>
<td>Maximum positive (Z)</td>
<td>$\infty$</td>
<td>1.60</td>
<td>-10.6</td>
<td>-10.0</td>
<td>1.51</td>
<td>7.8%</td>
</tr>
</tbody>
</table>

Where:

$$^* M(\gamma_{gl}) = M \left( \frac{\gamma_{Gu} \cdot G + \gamma_{Qu} \cdot Q}{\gamma_{gl}} \right)$$
Gain = \frac{\gamma_{Qu} - 1.4}{1.4} = \frac{\gamma_{Qu} - 1.5}{1.5}

Critical section: number 22

Reduction of gain by application of model uncertainties only in case Y due to the increase of negative bending moment by creep and consequent translation of N.L. behaviour
Bending moment for load case X (max. negative $t = t_1$)

Bending moment for load case W (max. positive $t = t_1$)
Bending moment for load case Y (max. negative $t = \infty$)

Bending moment for load case Z (max. positive $t = \infty$)
Example 2: Set of slender piers with variable section

- Depth: 82 / 87 / 92 / 97 m
- Unforeseen eccentricity: 5/1000 x depth
- $\gamma_G = \gamma_Q = 1.5$ (for simplification)
- Critical section at 53.30 m from plinth in which both thickness and reinforcement undergo a change

Safety format applied to that section
Pier geometry and reinforcement arrangement
### Safety format: $\gamma_{Gl}$

<table>
<thead>
<tr>
<th>Pier depth [m]</th>
<th>$\gamma_{Gu}$</th>
<th>$N(\gamma_{Gl})$ $^{*}$ [kN x 10^3]</th>
<th>$M(\gamma_{Gl})$ $^{**}$ [kN m x 10^3]</th>
<th>$N$ Safety [kN m x 10^3]</th>
<th>$M$ Safety [kN m x 10^3]</th>
<th>$\gamma_G \left( \frac{E(\gamma_{gl})}{\gamma_{Rd}} \right)$</th>
<th>Gain $\Gamma$</th>
</tr>
</thead>
<tbody>
<tr>
<td>82</td>
<td>2.45</td>
<td>151</td>
<td>242</td>
<td>134</td>
<td>74.9</td>
<td>2.42</td>
<td>62%</td>
</tr>
<tr>
<td>87</td>
<td>2.15</td>
<td>137</td>
<td>243</td>
<td>119</td>
<td>66.5</td>
<td>2.15</td>
<td>43%</td>
</tr>
<tr>
<td>92</td>
<td>1.85</td>
<td>122</td>
<td>233</td>
<td>103</td>
<td>57.6</td>
<td>1.86</td>
<td>24%</td>
</tr>
<tr>
<td>97</td>
<td>1.58</td>
<td>103</td>
<td>218</td>
<td>86</td>
<td>48.1</td>
<td>1.56</td>
<td>4%</td>
</tr>
</tbody>
</table>

Where:

\[ N(\gamma_{Gl}) = N \left( \frac{\gamma_{Gu} \cdot G + \gamma_{Qu} \cdot Q}{\gamma_{Gl}} \right) = N \left( \frac{\gamma_{Gu} \cdot (G + Q)}{\gamma_{Gl}} \right) \]

\[ M(\gamma_{Gl}) = M \left( \frac{\gamma_{Gu} \cdot G + \gamma_{Qu} \cdot Q}{\gamma_{Gl}} \right) = M \left( \frac{\gamma_{Gu} \cdot (G + Q)}{\gamma_{Gl}} \right) \]
### Safety format: $\gamma_{gl}$

<table>
<thead>
<tr>
<th>Pier depth [m]</th>
<th>$\gamma_{Gu}/\gamma_{gl}$</th>
<th>$N(\gamma_{gl})$ (* [\text{kN}\times 10^3]$</th>
<th>$M(\gamma_{gl})$ (** [\text{kNm}\times 10^3]$</th>
<th>$N(\gamma_{gl})/\gamma_{Ra}$ ([\text{kNm}\times 10^3]$</th>
<th>$M(\gamma_{gl})/\gamma_{Ra}$ ([\text{kNm}\times 10^3]$</th>
<th>$N\ Safety$ ([\text{kNm}\times 10^3]$</th>
<th>$M\ Safety$ ([\text{kNm}\times 10^3]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>82</td>
<td>2.59</td>
<td>159</td>
<td>260</td>
<td>150</td>
<td>245</td>
<td>152</td>
<td>243</td>
</tr>
<tr>
<td>87</td>
<td>2.28</td>
<td>146</td>
<td>262</td>
<td>138</td>
<td>247</td>
<td>139</td>
<td>246</td>
</tr>
<tr>
<td>92</td>
<td>1.96</td>
<td>129</td>
<td>251</td>
<td>122</td>
<td>237</td>
<td>123</td>
<td>236</td>
</tr>
<tr>
<td>97</td>
<td>1.67</td>
<td>110</td>
<td>234</td>
<td>104</td>
<td>221</td>
<td>104</td>
<td>220</td>
</tr>
</tbody>
</table>

Where:

\[
* N(\gamma_{gl}) = N\left(\frac{\gamma_{Gu} \cdot G + \gamma_{Qu} \cdot Q}{\gamma_{gl}}\right) = N\left(\frac{\gamma_{Gu} \cdot (G + Q)}{\gamma_{gl}}\right)
\]

\[
** M(\gamma_{gl}) = M\left(\frac{\gamma_{Gu} \cdot G + \gamma_{Qu} \cdot Q}{\gamma_{gl}}\right) = M\left(\frac{\gamma_{Gu} \cdot (G + Q)}{\gamma_{gl}}\right)
\]
<table>
<thead>
<tr>
<th>Pier depth [m]</th>
<th>Top section</th>
<th>Gain</th>
<th>( \frac{E(\gamma_{\text{gl}}) / \gamma_{\text{Rd}}}{\gamma_{\text{G}}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>82</td>
<td>135</td>
<td>75.4</td>
<td>2.44</td>
</tr>
<tr>
<td>87</td>
<td>121</td>
<td>67.6</td>
<td>2.19</td>
</tr>
<tr>
<td>92</td>
<td>104</td>
<td>58.1</td>
<td>1.88</td>
</tr>
<tr>
<td>97</td>
<td>87</td>
<td>48.6</td>
<td>1.57</td>
</tr>
</tbody>
</table>

**Table:**

- **Pier depth [m]:** The depth of the pier.
- **Top section:** The top section of the pier.
- **\( N_{\text{Safety}} [\text{kNm} \times 10^3] \):** The safety load capacity.
- **\( M_{\text{Safety}} [\text{kNm} \times 10^3] \):** The safety moment capacity.
- **\( \gamma_{\text{G}} \)\( \frac{E(\gamma_{\text{gl}}) / \gamma_{\text{Rd}}}{\gamma_{\text{G}}} \):** The safety coefficient.
- **Gain \( \Gamma \):** The gain of the system.
Safety format: $\gamma_{gl}$

![Graph showing the relationship between axial force and bending moment for different sections of a pier.](image-url)

**Legend:**
- **Collapse surface for section 11**
- **82 m Pier**
- **87 m Pier**
- **92 m Pier**
- **97 m Pier**

**Axes:**
- **Axial force N [kN]**
- **Bending moment [kNm]**

**Note:**
- The graph illustrates the safety format $\gamma_{gl}$ for different pier sections, showing how the axial force and bending moment interact to determine the collapse surface.
Safety format: $\gamma_{gl}$ - enlargement of the most interesting region of the omothetic curves
Example 3: Continuous deep beam experimentally tested (Rogowsky, Mac Gregor, Ong)

- Adina N.L. code
- Concrete strength criterion by Carbone, Giordano, Mancini
- Peak load reached at the crushing of second element (model unable to reach the equilibrium for further load increments)
R/C deep beam: FE half mesh (right) load-displacement curve of point A (left)
Resisting interaction surface
\( \sigma_x, \sigma_y, \tau_{xy} \)

Application of safety format in the vectorial space of internal actions
Set of external and internal actions

- \( q_{ud} = 404.0 \text{kN}, \quad \sigma_x = -8.47 \text{MPa}, \quad \sigma_y = -5.77 \text{MPa}, \quad \tau = 6.99 \text{MPa} \)
- \( q_{\text{max}} = 319.5 \text{kN}, \quad \sigma_x = -6.82 \text{MPa}, \quad \sigma_y = -4.75 \text{MPa}, \quad \tau = 5.69 \text{MPa} \)

Behaviour with limited non-linearity

Very limited effect of model uncertainties
Section 6 ⇒ Ultimate limit state (ULS)

- Robustness criteria for prestressed structures

3 different approaches
a) Verification of load capacity with a reduced area of prestressing

- Evaluation of bending moment in frequent combination of actions: $M_{freq}$
- Reduction of prestressing up to the reaching of $f_{ctm}$ at the extreme tensed fibre, in presence of $M_{freq}$
- Evaluation of resisting bending moment $M_{Rd}$ with reduced prestressing and check that:

$$M_{Rd} > M_{freq}$$

Redistributions can be applied

Material partial safety factors as for accidental combinations
b) Verification with nil residual prestressing

Provide a minimum reinforcement so that

$$A_{s,\text{min}} = \frac{M_{\text{rep}}}{z_s f_{yk}} \left( -\frac{A_p \cdot \Delta\sigma_p}{f_{yk}} \right)$$

where $M_{\text{rep}}$ is the cracking bending moment evaluated with $f_{ctx}$

(f$_{ctm}$ recommended)

$\Delta\sigma_p < 0.4 f_{ptk}$ and 500 MPa

c) Establish an appropriate inspection regime
(External tendons!)
Simple supported span

Transverse section
Materials:

- Concrete
  \[ f_{ck} = 30.0 \text{ Mpa} \]
  \[ f_{ctm} = 2.0 \text{ Mpa} \]
  \[ f_{cd} = 20 \text{ Mpa} \ (\gamma_m=1.5) \]

- Ordinary reinforcement
  \[ f_{syk} = 430 \text{ Mpa} \]
  \[ f_{syd} = 374 \text{ Mpa} \ (\gamma_m=1.15) \]

- Prestressing steel
  \[ f_{ptk} = 1800 \text{ Mpa} \]

Geometrical parameters:

\[ A = 6.055 \text{ m}^2 \]
\[ J = 3.59 \text{ m}^4 \]
\[ y_G = 1.21 \text{ m} \]
Tendon layout
Brittle failure

1. Reduction of prestressing up to reaching of $f_{ctm}$ at the extreme tensed fibre in presence of $M_{freq}$

<table>
<thead>
<tr>
<th>Active tendons</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{brec}$ [kN]</td>
<td>0.00</td>
<td>-2852.28</td>
<td>-5704.56</td>
<td>-8556.84</td>
<td>-11409.12</td>
<td>-14261.40</td>
<td>-17113.68</td>
<td>-19965.96</td>
<td>-22818.24</td>
</tr>
<tr>
<td>$M_{brec}$ [kN m]</td>
<td>0.00</td>
<td>-3166.03</td>
<td>-6332.06</td>
<td>-9498.09</td>
<td>-12664.12</td>
<td>-15830.15</td>
<td>-18996.18</td>
<td>-22162.22</td>
<td>-25328.25</td>
</tr>
<tr>
<td>$M_{fess}$ [kN m]</td>
<td>5933.88</td>
<td>10497.53</td>
<td>15061.17</td>
<td>19624.82</td>
<td>24188.46</td>
<td>28752.11</td>
<td>33315.75</td>
<td>37879.39</td>
<td>42443.04</td>
</tr>
<tr>
<td>$\sigma_{inf}$ [MPa]</td>
<td>2.00</td>
<td>2.00</td>
<td>2.00</td>
<td>2.00</td>
<td>2.00</td>
<td>2.00</td>
<td>2.00</td>
<td>2.00</td>
<td>2.00</td>
</tr>
<tr>
<td>$M_{frea}$ [kN m]</td>
<td>51933.5</td>
<td>48767.47</td>
<td>45601.438</td>
<td>42435.41</td>
<td>39269.38</td>
<td>36103.35</td>
<td>32937.32</td>
<td>29771.28</td>
<td>26605.25</td>
</tr>
</tbody>
</table>

In such condition add ordinary reinforcement so that $M_{Rd} \geq M_{freq}$, with $\gamma_C = 1.3$ and $\gamma_S = 1.0$
Take care of preelongation for the contribution of tendons to the evaluation of $M_{Rd}$

Such condition is reached for an addition of $1\phi 14 / 150$ mm in the bottom slab and $1\phi 12 / 150$ mm in the webs and top slab
2. Provide a minimum reinforcement evaluated as

\[ A_{s,\text{min}} = \frac{M_{\text{rep}}}{z_s f_{yk}} \]

\( M_{\text{rep}} \) = cracking moment evaluated with \( f_{\text{ctm}} \) and nil prestressing

\( z_s \) = lever arm at USL = 1.62 m

The required ordinary reinforcement results
1\#12 / 150 mm in the bottom slab
- Shear design of precast prestressed beams

- High level of prestress $\rightarrow \sigma_{cp}/f_{cd} > 0.5$
- Thin webs
- End blocks
- Redundancy in compressed and tensed chords

Web verification only for compression field due to shear ($\alpha_{cw} = 1$)
- Superimposition of different truss models
- Bending–shear behaviour of segmental precast bridges with external prestressing (only)

\[ h_{\text{red}} = \frac{V_{Ed}}{b_w \nu f_{cd}} \left( \cot \theta + \tan \theta \right) \]

\[ A_{sw} = \frac{V_{Ed}}{h_{\text{red}} f_{ywd} \cot \theta} \]

\[ h_{\text{red,min}} = 0.5 \, h \]
(recommended value)

Field A : arrangement of stirrups with \( \theta_{\text{max}} \) (cot \( \theta = 1.0 \))

Field B : arrangement of stirrups with \( \theta_{\text{min}} \) (cot \( \theta = 2.5 \))
- Shear and transverse bending interaction

When

\[
\frac{V_{Ed}}{V_{Rd,\text{max}}} < 0.20
\]

or

\[
\frac{M_{Ed}}{M_{Rd,\text{max}}} < 0.10
\]

The interaction can be disregarded
- Combination of shear and torsion for box sections

Each wall should be designed separately
- Bending–shear-torsion behaviour of segmental precast bridges with external prestressing (only)

Design the shear keys so that circulatory torsion can be maintained!
- Fatigue

- Verification of concrete under compression or shear
  - Traffic data
  - S-N curves
  - Load models

- $\lambda$ values simplified approach (Annex NN, from ENV 1992-2)
Application of Miner rule

\[ \sum_{i=1}^{m} \frac{n_i}{N_i} \leq 1 \]

Given by national authorities (S-N curves)

\[ N_i = 10 \exp \left( 14 \cdot \frac{1-E_{cd,max,i}}{\sqrt{1-R_i}} \right) \]

where:

\[ R_i = \frac{E_{cd,\text{min},i}}{E_{cd,\text{max},i}} \]
 \[ E_{cd,\text{min},i} = \frac{\sigma_{cd,\text{min},i}}{f_{cd,\text{fat}}} \]
 \[ E_{cd,\text{max},i} = \frac{\sigma_{cd,\text{max},i}}{f_{cd,\text{fat}}} \]

\[ f_{cd,\text{fat}} = k_1 \beta_{cc} (t_0) f_{cd} \left( 1 - \frac{f_{ck}}{250} \right) \]

\[ K_1 = 0.85 \] (Recommended value)
- Membrane elements

- Compressive stress field strength defined as a function of principal stresses

- If both principal stresses are compressive

\[ \sigma_{cd_{\text{max}}} = 0.85 f_{cd} \frac{1 + 3.80\alpha}{(1 + \alpha)^2} \]

\( \alpha \leq 1 \)

is the ratio between the two principal stresses
Where a plastic analysis has been carried out with $\theta = \theta_{el}$ and at least one principal stress is in tension and no reinforcement yields

$$\sigma_{cd\,max} = f_{cd} \left[ 0.85 - \frac{\sigma_s}{f_{yd}} (0.85 - \nu) \right]$$

is the maximum tensile stress value in the reinforcement

Where a plastic analysis is carried out with yielding of any reinforcement

$$\sigma_{cd\,max} = \nu f_{cd} (1 - 0.032 |\theta - \theta_{el}|)$$

is the angle to the X axis of plastic compression field at ULS (principal compressive stress)

$|\theta - \theta_{el}| \leq 15$ degrees

is the inclination to the X axis of principal compressive stress in the elastic analysis
Assumption: strength of concrete subjected to biaxial stresses is correlated to the angular deviation between angle $\vartheta_{el}$ which identifies the principal compressive stresses in incipient cracking and angle $\vartheta_u$ which identifies the inclination of compression stress field in concrete at ULS.

With increasing $\Delta \vartheta$ concrete damage increases progressively and strength is reduced accordingly.
Plastic equilibrium condition

\[ \sigma_x + \tau \cot \theta_{pl} - \sigma_{sx} \rho_x = 0 \]

\[ \tau + \sigma_x \cot \theta_{pl} - \sigma_{sy} \rho_y \cot \theta_{pl} = 0 \]

\[ \tau \tan \theta_{pl} - \sigma_x + \sigma_{sx} \rho_x - \sigma_c = 0 \]

\[ \tau - \sigma_y \tan \theta_{pl} + \sigma_{sy} \rho_y \tan \theta_{pl} - \sigma_c \tan \theta_{pl} = 0 \]
Graphical solution of inequalities system

\[ v \geq - (\omega_x + n_x) \tan \vartheta_{pl} \] (69)

\[ v \leq (\omega_x - n_x) \tan \vartheta_{pl} \] (70)

\[ v \geq (-\omega_y + n_y) \cot \vartheta_{pl} \] (71)

\[ v \leq \omega_y \cot \vartheta_{pl} \] (72)

\[ v \leq v \sin \vartheta_{pl} \cos \vartheta_{pl} \] (73)
Resisting domain for $v_{\text{max}}$ (a) and $v_{\text{min}}$ (b) with $\theta_{el}=45^\circ$, $\omega_x=\omega_y=0.3$
Experimental versus calculated panel strength by Marti and Kaufmann (a) and by Carbone, Giordano and Mancini (b)
Skew reinforcement

Plates conventions

Thickness = \( t_r \)
Equilibrium of the section parallel to the compression field

\[ \rho_{\alpha r} \sigma_{s \alpha r} = \frac{\sigma_{xy} \sin \theta_r \cos \beta - \sigma_{yy} \cos \theta_r \sin \beta + \tau_{xy} \cos(\theta_y + \beta)}{\sin(\theta_r - \alpha) \cos(\alpha - \beta)} \]

\[ \rho_{\beta r} \sigma_{s \beta r} = \frac{\sigma_{xy} \sin \theta_r \sin \alpha + \sigma_{yy} \cos \theta_r \cos \alpha + \tau_{xy} \sin(\theta_y + \alpha)}{\cos(\theta_y - \beta) \cos(\alpha - \beta)} \]
Equilibrium of the section orthogonal to the compression field

\[-\sigma_{xy} \cos \theta_r + \tau_{xy} \sin \theta_r + \rho_{ar} \sigma_{s,ar} a'_r \cos \alpha - \rho_{br} \sigma_{s,br} b'_r \sin \beta + \sigma_{cr} \cos \theta_r = 0\]

\[-\sigma_{yy} \sin \theta_r + \tau_{xy} \cos \theta_r + \rho_{ar} \sigma_{s,ar} a'_r \sin \alpha - \rho_{br} \sigma_{s,br} b'_r \cos \beta + \sigma_{cr} \sin \theta_r = 0\]
Use of genetic algorithms (Genecop III) for the optimization of reinforcement and concrete verification

**Objective:** minimization of global reinforcement

**Stability:** find correct results also if the starting point is very far from the actual solution
Section 7 ⇒ Serviceability limit state (SLS)

- Compressive stresses limited to $k_1 f_{ck}$ with exposure classes XD, XF, XS (Microcracking)

$k_1 = 0.6$  (recommended value)

$k_1 = 0.66$  in confined concrete (recommended value)
### - Crack control

<table>
<thead>
<tr>
<th>Exposure Class</th>
<th>Reinforced members and prestressed members with unbonded tendons</th>
<th>Prestressed members with bonded tendons</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Quasi-permanent load combination</td>
<td>Frequent load combination</td>
</tr>
<tr>
<td>X0, XC1</td>
<td>0.3&lt;sup&gt;1&lt;/sup&gt;</td>
<td>0.2</td>
</tr>
<tr>
<td>XC2, XC3, XC4</td>
<td>0.3</td>
<td>0.2&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>XD1, XD2, XD3</td>
<td>0.3</td>
<td>Decompression</td>
</tr>
<tr>
<td>XS1, XS2, XS3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Note 1:** For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed.

**Note 2:** For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.

*Decompression requires that concrete is in compression within a distance of 100 mm (recommended value) from bondend tendons*
- Minimum reinforcement areas

Clarification about T and Box beams

Component section “flange”
Component section “web”
Component section “web”

Component section “flange”

“Web” “Flange”

\[ f_{ct,eff} \]

\[ \sigma_{c,flange} \]

\[ \sigma_{c,web} \]
- Control of shear cracks within the webs

Concrete tensile strength $f_{ctb}$ is:

$$f_{ctb} = \left( 1 - 0.8 \frac{\sigma_3}{f_{ck}} \right) f_{ck,0.05}$$

$\sigma_3$ is the larger compressive principal stress ($\sigma_3 > 0$ and $\sigma_3 < 0.6 f_{ck}$)

- The larger tensile principal stress $\sigma_1$ is compared with $f_{ctb}$

$$\frac{\sigma_1}{f_{ctb}} \begin{cases} < 1 \Rightarrow \text{minimum longitudinal reinforcement} \\ \geq 1 \Rightarrow \text{crack width controlled or calculated considering the skewness of reinforcement} \end{cases}$$
Section 8 ⇒ Detailing of reinforcement and prestressing tendons

- Couplers for prestressing tendons
- In the same section maximum 67% of coupled tendons
- For more than 50% of coupled tendons:

  - Continuous minimum reinforcement
  - or
  - Residual stress > 3 MPa in characteristic combination
- Minimum distance of sections in which couplers are used

<table>
<thead>
<tr>
<th>Construction depth h</th>
<th>Distance a</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \leq 1.5 \text{ m} )</td>
<td>1.5 m</td>
</tr>
<tr>
<td>( 1.5 \text{ m} &lt; h &lt; 3.0 \text{ m} )</td>
<td>( a = h )</td>
</tr>
<tr>
<td>( \geq 3.0 \text{ m} )</td>
<td>3.0 m</td>
</tr>
</tbody>
</table>

- For tendons anchored at a construction joint a minimum residual compressive stress of 3 MPa is required under the frequent combination of actions, otherwise reinforcement should be provided to carter for the local tension behind the anchor.
Baricentric prestressing, two coupled tendons over two

t = 14 gg

deformation

σ_x

σ_y
Baricentric prestressing
two coupled tendons over two

\( t = 41 \, \text{gg} \)

\[ \sigma_x \]

\[ \sigma_y \]
Baricentric prestressing

two coupled tendons over two

t = 42 gg

\[ \sigma_x \]

\[ \sigma_y \]
**Baricentric prestressing**

**two coupled tendons over two**

t = 70 gg

![Graph showing deformation](image-url)
Baricentric prestressing
two coupled tendons over two

\[ \sigma_x, \sigma_y \]
Baricentric prestressing, one coupled tendon over two

t = 14 gg

deformation

\sigma_x

\sigma_y
Baricentric prestressing
one coupled tendons over two

\[ t = 41 \text{ gg} \]

\[ \sigma_x \quad \sigma_y \]
Baricentric prestressing

one coupled tendons over two

t = 42 gg

\sigma_x

\sigma_y

deformation

EUROCODES - Background and Applications - Brussels 18-20 February 2008
Prof. Ing. Giuseppe Mancini - DISTR - Politecnico di Torino
Baricentric prestressing
one coupled tendons over two

t = 70 gg

$\sigma_x$

$\sigma_y$
Baricentric prestressing
one coupled tendons over two
Baricentric prestressing, two anchored tendons over two

t = 14 gg

deformation

$\sigma_x$, $\sigma_y$
Baricentric prestressing
two anchored tendons over two
t = 41 gg

\( \sigma_x \)

\( \sigma_y \)
Baricentric prestressing
two anchored tendons over two
t = 70 gg
**Baricentric prestressing**

two anchored tendons over two

deformation

\[ \sigma_x \]

\[ \sigma_y \]
Baricentric prestressing
two anchored tendons over two

$t = \infty$

Zoomed areas near anchorages
Section 113 ⇒ Design for the execution stages

Take account of construction procedure

- Construction stages
- Redistribution by creep in the section
- Redistribution by creep for variation of statical scheme
- Actions during execution

- Statically equilibrium of cantilever bridge → unbalanced wind pressure of 200 N/m² (recommended value)

- For cantilever construction
  - Fall of formwork
  - Fall of one segment

- For incremental launching → Imposed deformations!

- In case in SLS decompression is required, tensile stresses less than \( f_{ctm} \) (recommended value) are permitted during the construction in quasi-permanent combination of actions
Annex B ⇒ Creep and shrinkage strain

- HPC, class R cement, strength $\geq 50/60$ MPa with or without silica fume
- Thick members $\rightarrow$ kinetic of basic creep and drying creep is different
- Distinction between
  - Autogenous shrinkage: related to process of hydration
  - Drying shrinkage: related to humidity exchanges
- Specific formulae for SFC (content $> 5\%$ of cement by weight)
- Autogenous shrinkage

For $t < 28$ days, $f_{ctm}(t) / f_{ck}$ is the main variable

$$\frac{f_{cm}(t)}{f_{ck}} < 0.1 \quad \varepsilon_{ca}(t, f_{ck}) = 0$$

$$\frac{f_{cm}(t)}{f_{ck}} \geq 0.1 \quad \varepsilon_{ca}(t, f_{ck}) = (f_{ck} - 20) \left(2.2 \frac{f_{cm}(t)}{f_{ck}} - 0.2\right) \times 10^{-6}$$

For $t \geq 28$ days

$$\varepsilon_{ca}(t, f_{ck}) = (f_{ck} - 20) \left[2.8 - 1.1 \exp\left(-t / 96\right)\right] \times 10^{-6}$$

97% of total autogenous shrinkage occurs within 3 months
- Drying shrinkage (RH ≤ 80%)

\[
\varepsilon_{cd}(t, t_s, f_{ck}, h_0, RH) = \frac{K(f_{ck}) \left[ 72 \exp(-0.046 f_{ck}) + 75 - RH \right] (t - t_s) 10^{-6}}{(t - t_s) + \beta_{cd} h_0^2}
\]

with:

\[K(f_{ck}) = \begin{cases} 
18 & \text{if } f_{ck} \leq 55 \text{ MPa} \\
30 - 0.21 f_{ck} & \text{if } f_{ck} > 55 \text{ MPa}
\end{cases}\]

\[\beta_{cd} = \begin{cases} 
0.007 & \text{for silica – fume concrete} \\
0.021 & \text{for non silica – fume concrete}
\end{cases}\]
- Creep

\[ \varepsilon_{cc}(t, t_0) = \frac{\sigma(t_0)}{E_{c28}} \left[ \Phi_b(t, t_0) + \Phi_d(t, t_0) \right] \]

Basic creep  Drying creep
- Basic creep

\[
\Phi_b(t, t_0, f_{ck}, f_{cm}(t_0)) = \phi_{b0} \frac{\sqrt{t - t_0}}{\sqrt{t - t_0} + \beta_{bc}}
\]

with:

\[
\phi_{b0} = \begin{cases} 
3.6 & \text{for silica–fume concrete} \\
1.4 & \text{for non silica–fume concrete} 
\end{cases}
\]

\[
\beta_{bc} = \begin{cases} 
0.37 \exp \left(2.8 \frac{f_{cm}(t_0)}{f_{ck}}\right) & \text{for silica–fume concrete} \\
0.4 \exp \left(3.1 \frac{f_{cm}(t_0)}{f_{ck}}\right) & \text{for non silica–fume concrete} 
\end{cases}
\]
- Drying creep

\[ \Phi_d(t,t_s,t_0,f_{ck},RH,h_0) = \phi_{d0} \left[ \varepsilon_{cd}(t,t_s) - \varepsilon_{cd}(t_0,t_s) \right] \]

with:

\[ \phi_{d0} = \begin{cases} 
  1000 & \text{for silica - fume concrete} \\
  3200 & \text{for non silica - fume concrete} 
\end{cases} \]

\( f_{ck} \) is the characteristic concrete strength.
- Experimental identification procedure

At least 6 months

- Long term delayed strain estimation

Formulae  Experimental determination
### Safety factor for long term extrapolation $\gamma_{lt}$

<table>
<thead>
<tr>
<th>$t$ (age of concrete for estimating the delayed strains)</th>
<th>$\gamma_{lt}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t &lt; 1$ year</td>
<td>1</td>
</tr>
<tr>
<td>$t = 5$ years</td>
<td>1.07</td>
</tr>
<tr>
<td>$t = 10$ years</td>
<td>1.1</td>
</tr>
<tr>
<td>$t = 50$ years</td>
<td>1.17</td>
</tr>
<tr>
<td>$t = 100$ years</td>
<td>1.20</td>
</tr>
<tr>
<td>$t = 300$ years</td>
<td>1.25</td>
</tr>
</tbody>
</table>
Annex J ⇒ Detailing rules for particular situations

- Consideration of brittleness of HSC with a factor to be applied to $f_{cd}$

$$\frac{0,46 \cdot f_{ck}^{2/3}}{1 + 0,1 \cdot f_{ck}} \cdot f_{cd} \leq 1$$

- Edge sliding

$$A_S \cdot f_{yd} \geq \frac{F_{Rdu}}{2}$$

$\theta = 30^\circ$
- Anchorage zones of postensioned members

Bursting and spalling in anchorage zones controlled by reinforcement evaluated in relation to the primary regularisation prism

\[
\frac{P_{\text{max}}}{c \cdot c'} \leq 0,6 \cdot f_{ck}(t)
\]

where \( c, c' \) are the dimensions of the associate rectangle similar to anchorage plate

\[
\left\{ \begin{array}{l}
c/a \\
c'/a'
\end{array} \right\} \leq 1,25 \sqrt{\frac{c \cdot c'}{a \cdot a'}}
\]

being \( a, a' \) the dimensions of smallest rectangle including anchorage plate
Primary regularisation prism represents the volume in which the stresses reduce from very high values to acceptable values under uniaxial compression.

The depth of the prism is $1.2 \max(c,c')$

Reinforcement for bursting and spalling (distributed in each direction within the prism)

$$A_S = 0.15 \frac{P_{\text{max}}}{f_{yd}} \gamma_{P,\text{unf}}$$

(with $\gamma_{P,\text{unf}} = 1.20$)

Surface reinforcement at the loaded face

$$A_{\text{surf}} \geq 0.03 \frac{P_{\text{max}}}{f_{yd}} \gamma_{P,\text{unf}}$$

(in each direction)
**Annex KK** ⇒ **Structural effects of time dependent behaviour of concrete**

**Assumptions**

- Creep and shrinkage independent of each other
- Average values for creep and shrinkage within the section
- Validity of principle of superposition (Mc-Henry)
<table>
<thead>
<tr>
<th>Type of analysis</th>
<th>Comment and typical application</th>
</tr>
</thead>
<tbody>
<tr>
<td>General and incremental step-by-step method</td>
<td>These are general methods and are applicable to all structures. Particularly useful for verification at intermediate stages of construction in structures in which properties vary along the length (e.g.) cantilever construction.</td>
</tr>
<tr>
<td>Methods based on the theorems of linear viscoelasticity</td>
<td>Applicable to homogeneous structures with rigid restraints.</td>
</tr>
<tr>
<td>The ageing coefficient method</td>
<td>This method will be useful when only the long-term distribution of forces and stresses are required. Applicable to bridges with composite sections (precast beams and in-situ concrete slabs).</td>
</tr>
<tr>
<td>Simplified ageing coefficient method</td>
<td>Applicable to structures that undergo changes in support conditions (e.g.) span-to-span or free cantilever construction.</td>
</tr>
</tbody>
</table>
- General method

\[ \varepsilon_c(t) = \frac{\sigma_0}{E_c(t_0)} + \varphi(t, t_0) \frac{\sigma_0}{E_c(28)} + \sum_{i=1}^{n} \left( \frac{1}{E_c(t_i)} + \frac{\varphi(t, t_i)}{E_c(28)} \right) \Delta \sigma(t_i) + \varepsilon_{cs}(t, t_s) \]

A step by step analysis is required

- Incremental method

\[ \text{At the time } t \text{ of application of } \sigma \text{ the creep strain } \varepsilon_{cc}(t), \]
\[ \text{the potential creep strain } \varepsilon_{\infty cc}(t) \text{ and the creep rate are derived from the whole load history} \]
The potential creep strain at time $t$ is:

$$\frac{d\varepsilon_{\infty cc}(t)}{dt} = \frac{d\sigma}{dt} \varphi(\infty, t)$$

$t \Rightarrow t_e$

under constant stress from $t_e$, the same $\varepsilon_{cc}(t)$ and $\varepsilon_{\infty cc}(t)$ are obtained

$$\varepsilon_{\infty cc}(t) \cdot \beta_c(t, t_e) = \varepsilon_{cc}(t)$$

Creep rate at time $t$ may be evaluated using the creep curve for $t_e$

$$\frac{d\varepsilon_{cc}(t)}{dt} = \varepsilon_{\infty cc}(t) \frac{\partial \beta_c(t, t_e)}{\partial t}$$
For unloading procedures

\[ |\varepsilon_{cc}(t)| > |\varepsilon_{\infty cc}(t)| \]

and \( t_e \) accounts for the sign change

\[
\varepsilon_{ccMax}(t) - \varepsilon_{cc}(t) = (\varepsilon_{ccMax}(t) - \varepsilon_{\infty cc}(t)) \cdot \beta_c(t, t_e)
\]

\[
\frac{d}{dt} \left( \varepsilon_{ccMax}(t) - \varepsilon_{cc}(t) \right) = (\varepsilon_{ccMax}(t) - \varepsilon_{\infty cc}(t)) \cdot \frac{\partial \beta_c(t, t_e)}{\partial t}
\]

where \( \varepsilon_{ccMax}(t) \) is the last extreme creep strain reached before \( t \).
- Application of theorems of linear viscoelasticity

- \( J(t, t_0) \) and \( R(t, t_0) \) fully characterize the dependent properties of concrete

- Structures homogeneous, elastic, with rigid restraints

- Direct actions effect

\[
S(t) = S_{el}(t) \\
D(t) = E_C \int_0^t J(t, \tau) dD_{el}(\tau)
\]
Indirect action effect

\[ D(t) = D_{el}(t) \]

\[ S(t) = \frac{1}{E_C} \int_0^t R(t, \tau) \, dS_{el}(\tau) \]

Structure subjected to imposed constant loads whose initial statical scheme (1) is modified into the final scheme (2) by introduction of additional restraints at time \( t_1 \geq t_0 \)

\[ S_2(t) = S_{el,1} + \xi(t, t_0, t_1) \Delta S_{el,1} \]

\[ \xi(t, t_0, t_1) = \int_{t_1}^t R(t, \tau) \, dJ(\tau, t_0) \]

\[ \xi(t, t_0, t_0^+) = 1 - \frac{R(t, t_0)}{E_C(t_0)} \]
When additional restraints are introduced at different times $t_i \geq t_0$, the stress variation by effect of restraint $j$ introduced at $t_j$ is independent of the history of restraints added at $t_i < t_j$

$$S_{j+1} = S_{el,1} + \sum_{i=1}^{j} \xi(t, t_0, t_i) \Delta S_{el,i}$$

- **Ageing coefficient method**

Integration in a single step and correction by means of $\chi$ ($\chi \approx 0.8$)

$$\int_{\tau=t_0}^{t} \left[ \frac{E_c(28)}{E_c(\tau)} + \varphi_{28}(t, \tau) \right] d\sigma(\tau) = \left[ \frac{E_c(28)}{E_c(t_0)} + \chi(t, t_0) \varphi_{28}(t, t_0) \right] \Delta \sigma_{t_0 \to t}$$
- Simplified formulae

\[ S_\infty = S_0 + (S_1 - S_0) \frac{\phi(\infty, t_0) - \phi(t_1, t_0)}{1 + \chi \phi(\infty, t_1)} \frac{E_c(t_1)}{E_c(t_0)} \]

where: \( S_0 \) and \( S_1 \) refer respectively to construction and final statical scheme

\( t_1 \) is the age at the restraints variation
Annex LL ⇒ Concrete shell elements

A powerful tool to design 2D elements
Axial actions and bending moments in the outer layer

Membrane shear actions and twisting moments in the outer layer
RANTIVA BRIDGE

Sandwich model:
Numerical example
Mesh

2215 shell elements
2285 nodes
6 D.o.F. per node
13710 D.o.F. in total
Element chosen: n°682

X = 22
Y = 33
**Symbols, conventions and general data**

\[ \alpha = 0 \Rightarrow \text{transverse reinforcement, } A_{sx}, \text{ direction 22} \]

\[ \beta = 0 \Rightarrow \text{longitudinal reinforcement, } A_{sy}, \text{ direction 33} \]

**Concrete properties**

- \( f_{cd} = 20.75 \text{ MPa} \)
- \( f_{ctm} = 3.16 \text{ MPa} \)
- \( f_{ctd} = 1.38 \text{ MPa} \)

**Steel properties**

- \( f_{yd} = 373.9 \text{ MPa} \)
Dimensioning of $\alpha$ reinforcement (transverse) in the inferior layer

Distance of reinforcement from the outer surface = 6 cm

<table>
<thead>
<tr>
<th>Combination type</th>
<th>Nsd22 (KN/m)</th>
<th>Nsd33 (KN/m)</th>
<th>Nsd23 (KN/m)</th>
<th>Msd22 (KNm/m)</th>
<th>Msd33 (KNm/m)</th>
<th>Msd23 (KNm/m)</th>
<th>Vsd12 (KN/m)</th>
<th>Vsd13 (KN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max M33</td>
<td>277</td>
<td>-5134</td>
<td>-230</td>
<td>616</td>
<td>1121</td>
<td>-476</td>
<td>95</td>
<td>-212</td>
</tr>
</tbody>
</table>
### Layers thicknesses

<table>
<thead>
<tr>
<th>H sez. (m)</th>
<th>$t_{sup}$ (m)</th>
<th>$t_{inf}$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0000</td>
<td>0.23</td>
<td>0.18</td>
</tr>
</tbody>
</table>

### Increment of internal actions due to shear (for the single layer)

<table>
<thead>
<tr>
<th>nsd22 (KN/m)</th>
<th>nsd33 (KN/m)</th>
<th>nsd23 (KN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
### Upper layer verification

<table>
<thead>
<tr>
<th>Internal actions on the layer</th>
<th>Cracked?</th>
<th>Concrete parameters</th>
<th>Actions in reinforcement at $t_{sup}/2$</th>
<th>Reinforcement calculated at $c+\phi/2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>nsd22</td>
<td>nsd33</td>
<td>nsd23</td>
<td>case</td>
<td>$\theta$</td>
</tr>
<tr>
<td>(KN/m)</td>
<td>(KN/m)</td>
<td>(KN/m)</td>
<td>(-)</td>
<td>(°)</td>
</tr>
<tr>
<td>-633</td>
<td>-4070</td>
<td>481</td>
<td>no.</td>
<td>65.0</td>
</tr>
</tbody>
</table>

**Minimum reinforcement** $\phi 20/20 = 15.7 \text{ cm}^2/\text{m}$

### Lower layer verification

<table>
<thead>
<tr>
<th>Internal actions on the layer</th>
<th>Cracked?</th>
<th>Concrete parameters</th>
<th>Actions in reinforcement at $t_{sup}/2$</th>
<th>Reinforcement calculated at $c+\phi/2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>nsd22</td>
<td>nsd33</td>
<td>nsd23</td>
<td>case</td>
<td>$\theta$</td>
</tr>
<tr>
<td>(KN/m)</td>
<td>(KN/m)</td>
<td>(KN/m)</td>
<td>(-)</td>
<td>(°)</td>
</tr>
<tr>
<td>909</td>
<td>-1064</td>
<td>-711</td>
<td>yes</td>
<td>23.1</td>
</tr>
</tbody>
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**Minimum reinforcement** $\phi 20/20 = 15.7 \text{ cm}^2/\text{m}$

EUROCODES - Background and Applications - Brussels 18-20 February 2008

Prof. Ing. Giuseppe Mancini - DISTR - Politecnico di Torino
Dimensioning of \( \beta \) reinforcement (longitudinal) in the inferior layer

Distance of reinforcement from the outer surface = 6 cm

<table>
<thead>
<tr>
<th>Combination type</th>
<th>Nsd22 (KN/m)</th>
<th>Nsd33 (KN/m)</th>
<th>Nsd23 (KN/m)</th>
<th>Msd22 (KNm/m)</th>
<th>Msd33 (KNm/m)</th>
<th>Msd23 (KNm/m)</th>
<th>Vsd12 (KN)</th>
<th>Vsd13 (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max M22</td>
<td>261</td>
<td>-5134</td>
<td>-219</td>
<td>657</td>
<td>1014</td>
<td>-464</td>
<td>79</td>
<td>-197</td>
</tr>
</tbody>
</table>
## Layers thicknesses

<table>
<thead>
<tr>
<th>H sez. (m)</th>
<th>tsup (m)</th>
<th>tinf (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0000</td>
<td>0.23</td>
<td>0.19</td>
</tr>
</tbody>
</table>

## Increment of internal actions due to shear (for the single layer)

<table>
<thead>
<tr>
<th>nsd22 (KN/m)</th>
<th>nsd33 (KN/m)</th>
<th>nsd23 (KN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
### Upper layer verification

<table>
<thead>
<tr>
<th>Internal actions on the layer</th>
<th>Cracked?</th>
<th>Concrete parameters</th>
<th>Actions in reinforcement at ( t_{\text{sup}/2} )</th>
<th>Reinforcement calculated at ( c+\phi/2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>nsd22 (KN/m)</td>
<td>nsd33 (KN/m)</td>
<td>nsd23 (KN/m)</td>
<td>case (-)</td>
<td>( \theta )</td>
</tr>
<tr>
<td>-695</td>
<td>-3904</td>
<td>474</td>
<td>no</td>
<td>45.0</td>
</tr>
</tbody>
</table>

Minimum reinforcement \( \phi 20/20 = 15.7 \text{ cm}^2/\text{m} \)

### Lower layer verification

<table>
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</tr>
<tr>
<td>956</td>
<td>-1229</td>
<td>-693</td>
<td>yes</td>
<td>20.6</td>
</tr>
</tbody>
</table>

Minimum reinforcement \( \phi 20/20 = 15.7 \text{ cm}^2/\text{m} \)

EUROCODES - Background and Applications - Brussels 18-20 February 2008
Prof. Ing. Giuseppe Mancini - DISTR - Politecnico di Torino
Dimensioning of \( \alpha \) reinforcement (transverse) in the superior layer

Distance of reinforcement from the outer surface = 6 cm

<table>
<thead>
<tr>
<th>Combination type</th>
<th>Nsd22 (KN/m)</th>
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<td>H sez. (m)</td>
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<td></td>
<td></td>
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<tr>
<th>Internal actions on the layer</th>
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<th>Actions in reinforcement at $t_{\text{sup}}/2$</th>
<th>Reinforcement calculated at $c+\phi/2$</th>
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<tbody>
<tr>
<td>nsd22 (KN/m)</td>
<td>nsd33</td>
<td>nsd23</td>
<td>$\theta$ (°) $\nu f_{cd}$ $\sigma_c(f)$ $n_{R1(x)}$ $n_{R2(y)}$ $A_s(x)<em>{\text{nec}}$ $A_s(y)</em>{\text{nec}}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
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<td>-695</td>
<td>-3904</td>
<td>474</td>
<td>no</td>
<td>45.0 17.6 17.6 0.0 0.0 15.7 15.7</td>
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**Minimum reinforcement** $\phi 20/20 = 15.7 \text{ cm}^2/\text{m}$

### Lower layer verification

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<td>20.6 11.1 11.1 1216.7 611.5 31.3 15.7</td>
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**Minimum reinforcement** $\phi 20/20 = 15.7 \text{ cm}^2/\text{m}$
Dimensioning of \( \beta \) reinforcement (longitudinal) in the superior layer

Distance of reinforcement from the outer surface = 6 cm

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Load combination that maximizes this reinforcement
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<td>(m)</td>
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Minimum reinforcement $\phi 20/20 = 15.7 \text{ cm}^2/\text{m}$

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Minimum reinforcement $\phi 20/20 = 15.7 \text{ cm}^2/\text{m}$

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Annex MM ⇒ Shear and transverse bending

Webs of box girder bridges
Modified sandwich model
Annex NN ⇒ Damage equivalent stresses for fatigue verification

Unchanged with respect to ENV 1992-2

To be used only for simple cases
Annex OO ⇒ Typical bridge discontinuity regions

Strut and tie model for a solid type diaphragm without manhole
Strut and tie model for a solid type diaphragm with manhole
Diaphragms with indirect support. Strut and tie model.

Diaphragms with indirect support. Anchorage of the suspension reinforcement.
Diaphragms with indirect support. Links as suspension reinforcement.

Diaphragm in monolithic joint with double diaphragm: Equivalent system of struts and ties.

- Diaphragm
- Pier
- Longitudinal section
Torsion in the deck slab and reactions in the supports

Model of struts and ties for a typical diaphragm of a slab
**EN 1992-2** ⇒ A new design code to help in conceiving more and more enhanced concrete bridges
Thank you for the kind attention