Geotechnical aspects of bridge design (EN 1997)

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Outline

1. General presentation of Eurocode 7
   Contents of Part 1 and 2
   3 ULS-Design Approaches (DAs)
   Allowable movements of foundations
   Spread foundations
   Retaining structures (mainly gravity walls)

2. Application to bridge design
   Geotechnical context
   Abutment C0
      ULS-bearing capacity
      ULS-sliding
   Squat pier P1
      ULS-bearing capacity
      SLS-settlement

3. Seismic design situations
General presentation of Eurocode 7

STRUCTURAL EUROCODES

EN 1990

EN 1991

EN 1992
EN 1993
EN 1994

EN 1995
EN 1996
EN 1999

Basis of Structural design

Actions on structures

«Material » resistance

Geotechnical and seismic design
Eurocode 7 – Geotechnical design


Contents of Part 1 (EN 1997-1)

Section 1 General
Section 2 Basis of geotechnical design
Section 3 Geotechnical data
Section 4 Supervision of construction, monitoring and maintenance
Section 5 Fill, dewatering, ground improvement and reinforcement
Section 6 Spread foundations
Section 7 Pile foundations
Section 8 Anchorages
Section 9 Retaining structures
Section 10 Hydraulic failure
Section 11 Site stability
Section 12 Embankments
Informative annexes

Annex C
Active earth pressure

Annexes D & E : Bearing capacity of foundations

\[ R/A' = c' \times N_c \times b_c \times s_c \times i_c + \]

\[ q' \times N_q \times b_q \times s_q \times i_q + \]

\[ 0,5 \times \gamma' \times B \times N_\gamma \times b_\gamma \times s_\gamma \times i_\gamma \]

\[ R/A' = \sigma_{v0} + k \times p^{*}_{le} \]

Annexe F : Settlement of foundations

\[ s = p \times b \times f / E_m \]
Contents of Part 2 (EN 1997-2)

Section 1 General
Section 2 Planning and reporting of ground investigations
Section 3 Drilling, sampling and gw measurements
Section 4 Field tests in soils and rocks
Section 5 Laboratory tests on soils and rocks
Section 6 Ground investigation report

Also a number of Informative annexes
EN 1997- 2  
Field tests in soils and rocks (Section 4)

Clauses on:

CPT(U), PMT, FDT, SPT, DP, WST, FVT, DMT, PLT

Objectives, specific requirements, evaluation of test results, use of test results and derived values

Annexes with examples on use of results and derived values for geotechnical design
EN 1997- 2
Laboratory tests on soils and rocks (Section 5)

- preparation of soil specimens for testing
- preparation of rock specimens for testing
- tests for classification, identification and description of soils
- chemical testing of soils and groundwater
- strength index testing of soils
- strength testing of soils
- compressibility and deformation testing of soils
- compaction testing of soils
- permeability testing of soils
- tests for classification of rocks
- swelling testing of rock material
- strength testing of rock material
## Results of test standards

EN 1997-2  Annex A

### Field test

<table>
<thead>
<tr>
<th>Field test</th>
<th>Test results</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT/CPTU</td>
<td>$q_c, f_s, R_f$ (CPT) / $q_t, f_s, u$ (CPTU)</td>
</tr>
<tr>
<td>Dynamic probing</td>
<td>$N_{10}$ (DPL, DPM, DPH); $N_{10}$ or $N_{20}$ (DPSH)</td>
</tr>
<tr>
<td>SPT</td>
<td>$N, E_r$ (SPT), soil description</td>
</tr>
<tr>
<td>Pressuremeters (PMT)</td>
<td>$E_M, p_f, p_{IM}$ (MPM); expansion curve (all)</td>
</tr>
<tr>
<td>Flexible dilatometer (FDT)</td>
<td>$E_{FDT}$, deformation curve</td>
</tr>
<tr>
<td>Field vane test (FVT)</td>
<td>$c_{fv}, c_{rv}$, torque-rotation curve</td>
</tr>
<tr>
<td>Weight sounding test (WST)</td>
<td>continuous record of penetration depth or $N_b$</td>
</tr>
<tr>
<td>Plate loading test</td>
<td>$p_u$</td>
</tr>
<tr>
<td>Fita dilatometer test</td>
<td>$P_0, P_1, E_{DMT}, I_{DMT}, K_{DMT}$ (DMT)</td>
</tr>
</tbody>
</table>

### Laboratory tests

Soils: $w$; $\rho$; $\rho_s$; grain size distribution curve; $w_p$, $w_L$; $e_{\max}$, $e_{\min}$, $I_D$; $C_{OM}$; $C_{CaCO_3}$; $C_{SO_4^{2-}}$, $C_{SO_3^{2-}}$; $C_{cl}$; $pH$; compressibility, consolidation, creep curves, $E_{oed}$; $\sigma'_p$ or $C_s$, $C_c$, $\sigma'_p$, $C_a$, $c_u$ (lab vane); $c_u$ (fall cone); $q_u$; $c_u$ (UU); $\sigma$–$\varepsilon$ and $u$ curves, $\sigma$–paths, Mohr circles; $c'$, $\varphi'$ or $c_u$, $c_u=f(\sigma'c)$, $E'$ or $E_u$; $\sigma$–$u$ curve, $\tau$–$\sigma$ diagram, $c'$, $\varphi'$, residual parameters; $I_{CBR}$; $k$ (direct lab, field or oedometer)

Rocks: $w$; $\rho$ and $n$; swelling results; $\sigma_c$, $E$ and $\nu$; $I_{s50}$; $\sigma$–$u$ curve, Mohr diagram, $c'$, $\varphi'$, res par; $\sigma_T$; $\sigma$–$\varepsilon$ curve, $\sigma$–paths, Mohr circles; $c'$, $\varphi'$, $E$ and $\nu$
Geotechnical properties

F = field  L = laboratory

Correlations

Test results and derived values

EN 1997 -2
EN 1997 -1

Information from other sources on the site, the soils and rocks and the project

Cautious selection

Geotechnical model and characteristic value of geotechnical properties

Application of partial factors

Design values of geotechnical properties
Geotechnical properties

**Type of test**
- F = field
- L = laboratory

**Correlations**

**Test results and derived values**
- EN 1997 -2
- EN 1997 -1

**Cautious selection**

**Geotechnical model and characteristic value of geotechnical properties**

**Application of partial factors**

**Design values of geotechnical properties**

Information from other sources on the site, the soils and rocks and the project
Some aspects of Eurocode 7-1

Characteristic values
and design values

ULS Design Approaches

SLS and deformations of structures
The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state.

If statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%.
Design values of geotechnical parameters

Design value of a parameter: \( X_d = \frac{X_k}{\gamma_M} \)

Design values of actions and resistances fulfilling for STR/GEO ULS: \( E_d \leq R_d \)

\[
E_d = E \{ \gamma_F F_k \} \quad \text{and} \quad R_d = R \{ \frac{X_k}{\gamma_M} \} \\
(= \text{“at the source”})
\]

or \( E_d = \gamma_E E \{ F_k \} \quad \text{and} \quad R_d = R \{ X_k \} / \gamma_R \)
Ultimate limit states – Eurocode 7-1

- **EQU**: loss of equilibrium of the structure
- **STR**: internal failure or excessive deformation of the structure or structural elements
- **GEO**: failure or excessive deformation of the ground
- **UPL**: loss of equilibrium due to uplift by water pressure (buoyancy) or other vertical actions
- **HYD**: hydraulic heave, internal erosion and piping caused by hydraulic gradients
EN1990 - Ultimate limit states
EQU and STR/GEO

\[ E_d < R_d \]

J.A Calgaro
## ULS - STR/GEO: Persistent and Transient Situations

### The 3 Design Approaches – Format: \( E_d < R_d \)

<table>
<thead>
<tr>
<th>Approaches</th>
<th>Combinations</th>
<th>Action ( (\gamma_F) )</th>
<th>Symbol</th>
<th>Set A1</th>
<th>Set A2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A1 “+” M1 “+” R1 &amp; A2 “+” M2 “+” R1</td>
<td>Permanent</td>
<td>( \gamma_G )</td>
<td>1,35</td>
<td>1,00</td>
</tr>
<tr>
<td></td>
<td>Or A2 “+” M1 or M2 “+” R4</td>
<td>Unfavourable</td>
<td>( \gamma_G )</td>
<td>1,00</td>
<td>1,00</td>
</tr>
<tr>
<td>2</td>
<td>A1 “+” M1 “+” R2</td>
<td>Favourable</td>
<td>( \gamma_Q )</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>A1 or A2 “+” M2 “+” R3</td>
<td>Variable</td>
<td>( \gamma_Q )</td>
<td>1,50</td>
<td>1,30</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Soil parameter ( (\gamma_M) )</th>
<th>Symbol</th>
<th>Set M1</th>
<th>Set M2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle of shearing resistance</td>
<td>( \gamma_{\varphi}' )</td>
<td>1,00</td>
<td>1,25</td>
</tr>
<tr>
<td>Effective cohesion</td>
<td>( \gamma_{c}' )</td>
<td>1,00</td>
<td>1,25</td>
</tr>
<tr>
<td>Undrained shear strength</td>
<td>( \gamma_{cu} )</td>
<td>1,00</td>
<td>1,40</td>
</tr>
<tr>
<td>Unconfined strength</td>
<td>( \gamma_{qu} )</td>
<td>1,00</td>
<td>1,40</td>
</tr>
<tr>
<td>Weight density</td>
<td>( \gamma_\gamma )</td>
<td>1,00</td>
<td>1,00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Resistance ( (\gamma_R) )</th>
<th>Symbol</th>
<th>Set R1</th>
<th>Set R2</th>
<th>Set R3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing capacity</td>
<td>( \gamma_{Rv} )</td>
<td>1,00</td>
<td>1,4</td>
<td>1,00</td>
</tr>
<tr>
<td>Sliding</td>
<td>( \gamma_{Rh} )</td>
<td>1,00</td>
<td>1,1</td>
<td>1,00</td>
</tr>
</tbody>
</table>

\( \gamma_R \) for Spread foundations
Verifications:

\[ E_d \leq C_d \]

\( C_d \) = limiting design value of the relevant serviceability criterion

\( E_d \) = design value of the effects of actions specified in the serviceability criterion, determined on the basis of the relevant combination

\( \gamma_F \) and \( \gamma_M = 1.0 \)
EN 1997-1 annex H
Movements and deformations of structures

settlement $s$, differential settlement $\delta s$, rotation $\theta$ and angular strain $\alpha$

relative deflection $\Delta$ and deflection ratio $\Delta/L$

$\omega$ and relative rotation (angular distortion) $\beta$

(after Burland and Wroth, 1975)
Allowable movements of foundations

Foundations of buildings (Eurocode 7, 1994)
* Serviceability limit states (SLS) : $\beta_{\text{max}} \approx 1/500$
* Ultimate limit states (ULS) : $\beta_{\text{max}} \approx 1/150$
• $s_{\text{max}} \approx 50 \text{ mm}$
• $\delta_{s_{\text{max}}} \approx 20 \text{ mm}$

Foundations of bridges
Moulton (1986) for 314 bridges in the US and Canada : 
* $\beta_{\text{max}} \approx 1/250$ (continuous deck bridges)
and $\beta_{\text{max}} \approx 1/200$ (simply supported spans)
* $s_{H_{\text{max}}} \approx 40 \text{ mm}$

In France, in practice :
ULS : $\beta_{\text{max}} \approx 1/250$
SLS : $\beta_{\text{max}} \approx 1/1000 \text{ à } 1/500$
Spread foundations
STR/GEO Ultimate limit states (ULS)

Bearing resistance:

\[ V_d \leq R_d = R_k \gamma_{Rv} \]

\( R_k \) : analytical, semi-empirical or prescriptive

Sliding resistance:

\[ H_d \leq R_d + R_{pd} \]
\[ + R_d \leq 0.4 V_d \]

Design approach 2:

\[ R_d = \left( V'_d \tan \delta_k \right) \gamma_{Rh} \quad \text{or} \quad R_d = \left( A_c c_{uk} \right) \gamma_{Rh} \]
Overall stability

Large eccentricities: special precautions if:
\[ \frac{e}{B} > \frac{1}{3} \text{ (or 0.6 f)} \]

Structural failure due to foundation movement

Structural design of spread foundation:
see EN 1992
## STR/GEO persistent and transient design situations (spread foundations without geotechnical actions)

<table>
<thead>
<tr>
<th>Design approach</th>
<th>Actions on/from the structure $\gamma_F$</th>
<th>Geotechnical resistance $\gamma_R$ or $\gamma_M$ at the source)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1,35 and 1,5</td>
<td>$\gamma_{Rv} = 1,0$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\gamma_{Rh} = 1,0$</td>
</tr>
<tr>
<td></td>
<td>1,0 and 1,3</td>
<td>$\gamma_M = 1,25$ or 1,4</td>
</tr>
<tr>
<td>2</td>
<td>1,35 and 1,5</td>
<td>$\gamma_{Rv} = 1,4$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\gamma_{Rh} = 1,1$</td>
</tr>
<tr>
<td>3</td>
<td>1,35 and 1,5</td>
<td>$\gamma_M = 1,25$ or 1,4</td>
</tr>
</tbody>
</table>
Serviceability limit states (SLS)

Include both immediate and delayed settlements

Assess differential settlements and relative rotations

Check that limit values for the structure are not reached
Verifications to carry out for spread foundations

Direct method:
- check each limit states (ULS and SLS)
- check the settlement for the SLSs

Indirect method:
- only a SLS calculation based on experience

Prescriptive method:
- example of the presumed bearing resistance on rocks (Annex G)
Annexes relevant to spread foundations in EN 1997-1

Annex A  (normative) Safety factors for ultimate limit states

Informative annexes :

Annex D  A sample analytical method for bearing resistance calculation

Annex E  A sample semi-empirical method for bearing resistance estimation

Annex F  Sample methods for settlement evaluation

Annex G A sample method for deriving presumed bearing resistance for spread foundations on rock

Annex H Limiting foundation movements and structural deformation
EN 1997-1 annexes D, E, F
Bearings capacity and settlement of foundations

“c-φ” model (annex D)

\[
\frac{R}{A'} = \left( c' \times N_c \times b_c \times s_c \times i_c \right) + q' \times N_q \times b_q \times s_q \times i_q + 0,5 \times \gamma' \times B' \times N_\gamma \times b_\gamma \times s_\gamma \times i_\gamma
\]

Pressuremeter model (annexe E)

\[
\frac{R}{A'} = \sigma_{v0} + k \times p_{le}^*
\]

Settlement of foundations (Annex F)

\[
s = p \times b \times f / E_m
\]
### EN 1997-1 annex G

#### Bearing resistance on rocks

<table>
<thead>
<tr>
<th>Group</th>
<th>Type of rock</th>
</tr>
</thead>
</table>
| 1     | Pure limestones and dolomites  
Carbonate sandstones of low porosity |
| 2     | Igneous  
Oolitic and marly limestones  
Well cemented sandstones  
Indurated carbonate mudstones  
Metamorphic rocks, including slates and schist  
(flat cleavage/foliation) |
| 3     | Very marly limestones  
Poorly cemented sandstones  
Slates and schists (steep cleavage/foliation) |
| 4     | Uncemented mudstones and shales |

5. Allowable bearing pressure not to exceed uniaxial compressive strength of rock if joints are tight or 50% of this value if joints are open.

6. Allowable bearing pressures: a) very weak rock, b) weak rock  
c) moderately weak rock  
d) moderately strong rock,  
e) strong rock  
Spacings: f) closely spaced discontinuities  
g) medium spaced discontinuities  
h) widely spaced discontinuities  
For types of rock in each of four groups, see Table G.1. Presumed bearing resistance in hatched areas to be assessed after inspection and/or making tests on rock. (from BS 8004)
Annexes relevant to spread foundations in EN 1997-2

Informative annexes:

D.3 Example of a method to determine the settlement for spread foundations from CPT
D.4 Example of a correlation between the oedometer modulus and the cone penetration resistance from CPT
D.5 Examples of establishing the stress-dependent oedometer modulus from CPT results
E.1 Example of a method to calculate the bearing resistance of spread foundations from PMT
E.2 Example of a method to calculate the settlements for spread foundations from PMT
F.3 Example of a method to calculate the settlement of spread foundations from SPT
G.3 Example of establishing the stress-dependent oedometer modulus from DP results
J Flat dilatometer test (DMT)
K.4 Example of a method to calculate the settlement of spread foundations in sand from (PLT)
Retaining structures
Scope of Eurocode 7 (Section 9)

Gravity walls (in stone, concrete, reinforced concrete)

Embedded walls (sheet pile walls, slurry trench walls; cantilever or supported walls)

Composite retaining structures (walls composed of elements, double wall cofferdams, reinforced earth structures)
Ultimate limit states of gravity walls

9.7.2 Overall stability (principles of section 11)

9.7.3 Foundation failure of gravity walls (principles of section 6)

9.7.6 Structural design (in accordance with EC 2, EC 3, EC5 and EC6)
Geometrical data – clause 9.3.2

Ground surface

ULS with passive pressure (ie rotational failure): the level of the resisting soil depends on the degree of site control over the level of the surface

($\Delta a = 0$, if surface controlled, otherwise $\Delta a > 0$)

Recommended values $\Delta a$:

equal to 10 % of the wall height above excavation level, limited to a maximum of 0.5 m
Water levels

The water levels to be selected shall be based on the data for the hydraulic and hydrogeological conditions at the site.

Nota: The variability of water levels is taken into account through the various design situations considered.
Determination of earth pressures (clause 9.5)

Magnitudes and directions of forces resulting from earth pressures shall take account of:

- the amount and direction of the relative ground-wall movement
- the horizontal as well as vertical equilibrium for the entire retaining structure

Range of inclinations recommended

$< \frac{2}{3} \varphi$ (steel sheet piles) ; $< \varphi$ (concrete cast against soil)

Allowed or recommended models:

At rest values: $K_0 = (1-\sin\varphi')(R_{oc})^{0.5}$

Limiting values: Caquot-Kérisel-Absi (Annex C)

Intermediate values (subgrade reaction, FEM)
Water pressures – clause 9.6

For structures retaining earth of medium or low permeability (silts and clays), water pressures shall correspond to a water table at the surface of the retained material, unless:

- a reliable drainage system is installed or infiltration is prevented

Where sudden changes in a free water level may occur, both the non-steady condition occurring immediately after the change and the steady condition shall be examined.
STR/GEO: persistent and transient situations

The 3 Design Approaches – Format: $E_d < R_d$

<table>
<thead>
<tr>
<th>Approaches</th>
<th>Combinations</th>
<th>Action ($\gamma_F$)</th>
<th>Symbol</th>
<th>Set A1</th>
<th>Set A2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A1 “+” M1 “+” R1 &amp; A2 “+” M2 “+” R1</td>
<td>Permanent</td>
<td>$\gamma_G$</td>
<td>1,35</td>
<td>1,00</td>
</tr>
<tr>
<td></td>
<td>Or A2 “+” M1 or M2 “+” R4</td>
<td>Unfavourable</td>
<td>$\gamma_G$</td>
<td>1,00</td>
<td>1,00</td>
</tr>
<tr>
<td>2</td>
<td>A1 “+” M1 “+” R2</td>
<td>Variable</td>
<td>$\gamma_Q$</td>
<td>1,50</td>
<td>1,30</td>
</tr>
<tr>
<td>3</td>
<td>A1 or A2 “+” M2 “+” R3</td>
<td>Favourable</td>
<td>$\gamma_Q$</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Soil parameter ($\gamma_M$)</th>
<th>Symbol</th>
<th>Set M1</th>
<th>Set M2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle of shearing resistance</td>
<td>$\gamma_{\varphi'}$</td>
<td>1,00</td>
<td>1,25</td>
</tr>
<tr>
<td>Effective cohesion</td>
<td>$\gamma_{c'}$</td>
<td>1,00</td>
<td>1,25</td>
</tr>
<tr>
<td>Undrained shear strength</td>
<td>$\gamma_{cu}$</td>
<td>1,00</td>
<td>1,40</td>
</tr>
<tr>
<td>Unconfined strength</td>
<td>$\gamma_{qu}$</td>
<td>1,00</td>
<td>1,40</td>
</tr>
<tr>
<td>Weight density</td>
<td>$\gamma_\gamma$</td>
<td>1,00</td>
<td>1,00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Resistance ($\gamma_R$)</th>
<th>Symbol</th>
<th>Set R1</th>
<th>Set R2</th>
<th>Set R3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing capacity</td>
<td>$\gamma_{Rv}$</td>
<td>-1,0</td>
<td>1,4</td>
<td>-1,0</td>
</tr>
<tr>
<td>Sliding resistance</td>
<td>$\gamma_{Rh}$</td>
<td>1,0</td>
<td>1,1</td>
<td>1,0</td>
</tr>
<tr>
<td>Earth resistance</td>
<td>$\gamma_{Rh}$</td>
<td>1,0</td>
<td>1,4</td>
<td>1,0</td>
</tr>
</tbody>
</table>

$\gamma_R$ for Retaining structures
Serviceability limit states - SLS

**Principle** : P Design values of earth pressures shall be derived using characteristic values of all soil parameters

**Displacement** : The design shall be justified by a more detailed investigation including displacement calculations where:

- the initial estimate exceeds the limiting values,
- where nearby structures and services are unusually sensitive to displacement;
- where comparable experience is not well established.
Annexes relevant to retaining structures in EN 1997-1

Annex A (normative) Safety factors for ultimate limit states

Informative annexes:
Annex C Limit values of earth pressures on vertical walls
Annex H Limiting foundation movements and structural deformation
Active / Passive earth pressures - annex C

Active/Passive earth pressures

\[ \beta = - \phi \delta + \phi \]

\[ \delta = 0 \text{ or } 2/3 \phi \text{ or } \phi \]
Bridge design
Geotechnical data

Identification of soils: core sampling results between abutment C0 and pier P1
Geotechnical data

Results of pressuremeter tests between abutment C0 and pier P1
Geotechnical data for C0 and P1

Normally fractured calcareous marl (at 2.5 m depth and 3 m depth):
- $c'_{kg} = 0$
- $\varphi'_{kg} = 30^\circ$
- $\gamma_{kg} = 20 \text{ kN/m}^3$

From ground level to base of foundation: $\gamma = 20 \text{kN/m}^3$.

Water level is assumed to be one metre below the foundation level in both cases.

Fill material: - $c'_{kf} = 0$; $\varphi'_{kf} = 30^\circ$; $\gamma_{kf} = 20 \text{ kN/m}^3$
Abutment C0 and pier P1 (squat pier)
Forces and notations
Support reactions for static analysis (Davaine, Malakatas)

Table 1. Vertical ‘structural’ actions for half of the bridge deck (Davaine, 2010b et c)

<table>
<thead>
<tr>
<th>Load cases</th>
<th>Designation</th>
<th>C0 (MN)</th>
<th>P1 (MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Self weight (structural steel + concrete)</td>
<td>G_{k,1}</td>
<td>1.1683</td>
<td>5.2867</td>
</tr>
<tr>
<td>Nominal non structural equipments</td>
<td>G_{k,2}</td>
<td>0.39769</td>
<td>1.4665</td>
</tr>
<tr>
<td>3 cm settlement on support P1</td>
<td>S_k</td>
<td>0.060</td>
<td>-0.137</td>
</tr>
<tr>
<td>Traffic UDL</td>
<td>Q_{vk,1} max/min</td>
<td>0.97612/-0.21869</td>
<td>2.693/-0.15637</td>
</tr>
<tr>
<td>Traffic TS</td>
<td>Q_{vk,2} max/min</td>
<td>0.92718/-0.11741</td>
<td>0.94458/-0.1057</td>
</tr>
</tbody>
</table>

Horizontal traffic action effects

The horizontal longitudinal reactions $Q_{xk,1} + Q_{xk,2}$ on abutments and piers due to traffic loads UDL and TS are, for half of the bridge deck (Davaine, 2010b):

- **Braking**:
  - min: -0.90658
  - max: 0 MN

- **Acceleration**:
  - min: 0
  - max: 0.90658 MN
Support reactions for static analysis (Davaine, Malakatas)

Transverse horizontal wind action effects (Malakatas, 2010 and Davaine 2010c)

Fig. 7. Displacement conditions of the bridge (Davaine, 2010b and 2010c)

Table 2. Transverse horizontal variable actions $H_{ykw}$ due to wind (Davaine, 2010c)

<table>
<thead>
<tr>
<th>Transverse horizontal force $H_y$ due to:</th>
<th>C0</th>
<th>P1</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{wk,1}$ without traffic load</td>
<td>164 kN</td>
<td>596 kN</td>
</tr>
<tr>
<td>$F_{wk,2}$ with traffic load</td>
<td>206.7 kN</td>
<td>751.3 kN</td>
</tr>
</tbody>
</table>
Abutment C0

- ULS - Bearing capacity
- ULS – Sliding resistance
Geotechnical actions

Weight of the wall : \( G_{\text{wall,k}} = 26.4 \text{ MN} \)

Active earth pressure:
\[
P_{\text{ad}} = \gamma_{G,\text{sup}} \times 0.5 \cdot K_{\text{ad}} \cdot \gamma_{kf} \cdot h^2 \cdot L_a
\]
\[
K_{\text{ad}} = \tan \left( \frac{\pi}{4} - \frac{\varphi_{df}}{2} \right)^2
\]
- for DA1-1 and DA2 : \( \varphi_{df} = \varphi_{kf} = 30^\circ \); \( K_{\text{ad}} = 0.333 \)
\[
\gamma_{kd} = \gamma_{kf} = 20 \text{ kN/m}^3 \text{ and } P_{\text{ad}} = 1.35 \times 3.84 = 5.18 \text{ MN}
\]
- for DA1-2 and DA3 : \( \tan \varphi_{df} = (\tan \varphi_{kf})/1.25 = \tan 30^\circ/1.25 \text{ and } \varphi_{df} = 24.8^\circ; \)
\[
K_{\text{ad}} = 0.409 \text{ and } P_{\text{ad}} = 1.00 \times 4.71 = 4.71 \text{ MN}
C0 – ULS Bearing capacity

Resultant actions

\[ F_v = V + G_{\text{wall}} \]
\[ F_x = H_x + P_a \]
\[ F_y = H_y \]
\[ M_y = P_a(h_2/3) + H_x h_1 - G_{\text{wall}} d_1 + V d_2 \]
\[ M_x = H_y h_1 \]

Resistance

\[ R = (B-2e_B)(L-2e_L) \{ q'N_q(\varphi')s_q i_q + 0.5\gamma'(B-2e_B)N_\gamma(\varphi')s_\gamma i_\gamma \} \]

and \[ R_d = R / \gamma_{R;v} \]
C0 – ULS Bearing capacity

For DA1-1: \( \phi'_{dg} = \phi'_{kg} = 30^\circ \)

\[
F_{vd} = 9.88 + 35.64 = 45.52 \text{ MN}
\]

\[
F_{xd} = 2.43 + 5.18 = 7.61 \text{ MN}
\]

\[
F_{yd} = 0.19 \text{ MN}
\]

\( \gamma_{Rv} = 1.0 \)

Thus, \( e_B = 1.04 \text{ m}, e_L = 0.03 \text{ m} \) and \( R_d = 150.2/1.0 = 150.2 \text{ MN} \)

For DA1-2: \( \tan \phi'_{dg} = (\tan \phi'_{kg}) / 1.25, \) thus \( \phi'_{dg} = 24.8^\circ \)

\[
F_{vd} = 7.86 + 26.4 = 34.26 \text{ MN}
\]

\[
F_{xd} = 2.07 + 4.71 = 6.78 \text{ MN}
\]

\[
F_{yd} = 0.16 \text{ MN}
\]

\( \gamma_{Rv} = 1.0 \)

Thus, \( e_B = 1.21 \text{ m}, e_L = 0.03 \text{ m} \) and \( R_d = 67.3/1.0 = 67.3 \text{ MN} \)

For DA2: \( \phi'_{dg} = \phi'_{kg} = 30^\circ \)

\[
F_{vd} = 9.88 + 35.64 = 45.52 \text{ MN}
\]

\[
F_{xd} = 2.43 + 5.18 = 7.61 \text{ MN}
\]

\[
F_{yd} = 0.19 \text{ MN}
\]

\( \gamma_{Rv} = 1.4 \)

Thus, \( e_B = 1.05 \text{ m}, e_L = 0.03 \text{ m} \) and \( R_d = 150.2/1.4 = 107.3 \text{ MN} \)

For DA3: \( \tan \phi'_{dg} = (\tan \phi'_{kg}) / 1.25, \) thus \( \phi'_{dg} = 24.8^\circ \)

\[
F_{vd} = 9.88 + 35.64 = 45.52 \text{ MN}
\]

\[
F_{xd} = 2.43 + 4.71 = 7.14 \text{ MN}
\]

\[
F_{yd} = 0.19 \text{ MN}
\]

\( \gamma_{Rv} = 1.0 \)

Thus, \( e_B = 1.01 \text{ m}, e_L = 0.03 \text{ m} \) and \( R_d = 79.6/1.0 = 79.6 \text{ MN} \)
C0 – ULS Bearing capacity

\[ F_{vd} \leq R_d \]

- fulfilled for all Design Approaches
- for DA1, combination 2 is governing
- DA3 the most conservative approach

All eccentricities are small: the maximum is
\[ e_B = 1.21 \text{ m} \]
C0 – ULS Sliding resistance

\[ F_{xd} \leq R_d + R_{p;d} \]

where

- \( F_{xd} \) is the horizontal component in the longitudinal direction
- \( R_d \) is the sliding resistance
- \( R_{p;d} \) is the passive earth force in front of the spread foundation.

\[ R_d = \{F'_{vd} (\tan \delta_k)/\gamma_M\}/\gamma_{R:h} \]

where

- \( F'_{vd} \) is the favourable effective vertical force
- \( \delta_k \) is the concrete-ground friction angle, assumed \( \delta_k = 2/3 \phi_{kg} \)

R\_d = \{F\'_{vd} (\tan \delta_k)/\gamma_M\}/\gamma_{R:h}
C0 – ULS Sliding resistance

Actions

\[ F'_{vd} = V_{d,\text{min}} + G_{\text{wall,d}} \]

- for DA1-1, DA2 and DA3: \( V_{d,\text{min}} = G_{k,1} + 0.8364 \ G_{k,2} + 1.35(Q_{vk,1} + Q_{vk,2}) = 1.047 \times 2 = 2.09 \text{ MN} \)
- for DA1-2: \( V_{d,\text{min}} = G_{k,1} + 0.8364 \ G_{k,2} + 1.15 (Q_{vk,1} + Q_{vk,2}) = 1.114 \times 2 = 2.23 \text{ MN} \)
- and for all DAs: \( G_{\text{wall,d}} = 1.0 \ G_{\text{wall,k}} = 26.4 \text{ MN} \)

DA1-1: \( F_{xd} = 7.61 \text{ MN} \) and \( F'_{vd} = 2.09 + 26.4 = 28.49 \text{ MN} \)
DA1-2: \( F_{xd} = 6.78 \text{ MN} \) and \( F'_{vd} = 2.23 + 26.4 = 28.63 \text{ MN} \)
DA2: \( F_{xd} = 7.61 \text{ MN} \) and \( F'_{vd} = 2.09 + 26.4 = 28.49 \text{ MN} \)
DA3: \( F_{xd} = 7.14 \text{ MN} \) and \( F'_{vd} = 2.09 + 26.4 = 28.49 \text{ MN} \)

Sliding resistances

DA1-1: \( \gamma_M = 1.0 \) and \( \gamma_{R;h} = 1.0 \), thus \( R_d = \{28.49 \times 0.364/1.0\} /1.0 = 10.37 \text{ MN} \)
DA1-2: \( \gamma_M = 1.25 \) and \( \gamma_{R;h} = 1.0 \), thus \( R_d = \{28.63 \times 0.364/1.25\}/1.0 = 8.33 \text{ MN} \)
DA2: \( \gamma_M = 1.0 \) and \( \gamma_{R;h} = 1.1 \), thus \( R_d = \{28.49 \times 0.364/1.0\} /1.1 = 9.42 \text{ MN} \)
DA3: \( \gamma_M = 1.25 \) and \( \gamma_{R;h} = 1.0 \), thus \( R_d = \{28.49 \times 0.364/1.25\}/1.0 = 8.29 \text{ MN} \)
Pier P1 (squat pier)

- ULS - Bearing capacity (DA2 only)
- SLS – Settlement
P1 – ULS Bearing capacity

\[ G_{\text{pier,}k} = 8.3 \text{ MN} \]

for DA2:
\[ G_{\text{pier,}d} = 1.35 \times 8.3 = 11.2 \text{ MN} \]

At base of foundation:
\[ F_v = V + G_{\text{pier}} \]
\[ F_x = H_x \]
\[ F_y = H_y \]
\[ M_y = H_x h_p \]
\[ M_x = H_y h_p \]
P1 – ULS Bearing capacity

For DA2: \[ F_{vd} = 28.9 + 11.2 = 40.1 \text{ MN} \]
\[ F_{xd} = 2.45 \text{ MN} \]
\[ F_{yd} = 0.68 \text{ MN} \]

one obtains, for DA 2:
\[ e_B = 0.70 \text{ m}, \; e_L = 0.20 \text{ m} \text{ and } R_k = 101.2 \text{ MN} \text{ and} \]
\[ R_d = R/\gamma_{R;v} = 101.2/1.4 = 72.3 \text{ MN} \]

The ULS condition in permanent and transient design situation \( F_{vd} \leq R_d \) is fulfilled, as \( 40.1 \text{ MN} < 72.3 \text{ MN} \).
SLS-QP combination:

\[ Q = G_{k,1} + G_{k,2} = (5.2867 + 1.4665) \times 2 = 6.75 \times 2 = 13.5 \text{ MN} \]

Ménard pressuremeter (MPM) method is used (Annex D2 of EN 1997-2)

The settlement is expressed as:

\[
 s = (q - \sigma_{\nu_0}) \times \left[ \frac{2B_0}{9E_d} \times \left( \frac{\lambda_d B}{B_0} \right)^a + \frac{\alpha \lambda_c B}{9E_c} \right]
\]
Selection of moduli $E_C$ and $E_D$

$$E_c = E_1$$

$$\frac{4}{E_d} = \frac{1}{E_1} + \frac{1}{0.85 E_2} + \frac{1}{E_{3,5}} + \frac{1}{2.5 E_{6,8}} + \frac{1}{2.5 E_{9,16}}$$

$$\frac{3.0}{E_{3,5}} = \frac{1}{E_3} + \frac{1}{E_4} + \frac{1}{E_5}$$

Or

$$\frac{3.6}{E_d} = \frac{1}{E_1} + \frac{1}{0.85 E_2} + \frac{1}{E_{3,5}} + \frac{1}{2.5 E_{6,8}}$$

Or

$$\frac{3.2}{E_d} = \frac{1}{E_1} + \frac{1}{0.85 E_2} + \frac{1}{E_{3,5}}$$
\[ s = (0.18 - 0.06) [1.2 \times \frac{1.26 \times 7.5}{0.6}^{0.5} \times \frac{14.65}{9} + \frac{0.5 \times 1.13 \times 7.5}{9 \times 7.3}] \]
\[ = 0.12 [0.036 + 0.065] = 0.012 \text{ m} = 12 \text{ mm}, \]

( preliminary rough estimate, with \( E_c = E_d = 6 \text{ MPa} \), \( \sigma_{vo} = 0 : s = 0.030 \text{ m} = 3 \text{ cm!} \) )
Seismic design situations (EN 1998-5)

- no liquefiable layer – see Figs. 2 and 3

Annexes in Eurocode 8 – Part 5:
- Annex E (Normative) ‘Simplified analysis for retaining structures’,
- Annex F (Informative) ‘Seismic bearing capacity of shallow foundations’

$A_{ED}$ seismic action effects come from the capacity design of the superstructure (see Kolias 2010a and 2010b)

The recommended values of $\gamma_M$ seem very conservative:
$\gamma_{cu} = 1.4, \gamma_{\tau cu} = 1.25, \gamma_{qu} = 1.4, \text{ and } \gamma_{\phi'} = 1.25$.

The NA for Greece, for instance, requires : all $\gamma = 1.0$ !
and to conclude:

It should be considered that knowledge of the ground conditions depends on the extent and quality of the geotechnical investigations. Such knowledge and the control of workmanship are usually more significant to fulfilling the fundamental requirements than is precision in the calculation models and partial factors.
Thank you for your kind and patient attention!