4 Geotechnical calculations at the critical cross section (Relevant paragraph of the paper: GEOTECHNICAL CALCULATIONS ASSUMPTIONS)

The examined embankment has total length and maximum height equal to 1200m and 13.0m, respectively. It is founded on the alluvial deposits formation (AL), whereas the rock formation of gneiss constitutes the bedrock in the broader area of the embankment. The encountered alluvial soil formation is divided into sub-layers along the embankment, depending a) on grain size distribution of each intercalation (layer consisting of sandy-gravelly materials, ALsg, and layer of silty-clayey composition, ALcm) and b) on the density of the soil formation (loose or dense structure), as presented in the attached drawing of the geotechnical longitudinal section (filename: 3-8_GEOTECHNICAL LONGITUDINAL SECTION.pdf).

The critical cross section used in the geotechnical calculations was selected based on the geometrical characteristics of the embankment as well as on the geological and geotechnical conditions at the location of the cross section. More particularly, cross section GG1201 (CH. 93+560) (having height equal to 12.20m) is considered as the most unfavourable cross section in terms of geometrical characteristics, which is representative for the area where the embankment is founded on alluvial deposits of sandy-gravelly composition (ALsg) of medium density (Layer Ib) with 11.0m thickness. The underlying layer consists of alluvial deposits of silty-clayey composition (ALcm: Layer II) having thickness of 10.0m, approximately. According to the geotechnical longitudinal section, at greater depths Layer Ib is again encountered (sandy-gravelly alluvial deposits of medium density). The groundwater level is taken at 4.0m depth below the ground surface. In the geotechnical calculations, a sanitary layer of 1.00m thickness is taken into consideration.

The geotechnical calculations include a) the performance of global and internal slope stability analyses against circular failure under both static and seismic loading conditions and b) the determination of the soil settlements due to the construction of the embankment at the most critical cross section (Salgado, 2007). At the area of the examined embankment the groundwater level is located at 4.0m depth below the ground surface. The relatively high groundwater level in combination with the existence of loose sandy and silty-sandy soil layers, creates the conditions for possible initiation of liquefaction phenomena under seismic loading conditions. This mode of failure is not taken into consideration for further analysis in the presentation of the current case study.

Slope stability analyses are performed according to Eurocode 7.1 and Eurocode 8 for seismic loading conditions. For the stability analysis of this particular project the limit equilibrium method was applied (using Larix-4 software, v. 2.21-Cubus).

For the construction of the main part of the embankment, coarse materials, described in detail in a following section, are used. The slope stability analyses and the settlement calculations of the embankments were carried out assuming the following geotechnical parameters for the material of the embankment:

Unit weight: γ=20kN/m^3, Angle of internal friction: φ'=35^o, Cohesion c'=5kPa,
Elasticity modulus, E = 50MPa

The soil settlement analyses at the most critical cross section of the embankment includes the calculation of both immediate and consolidation settlements (in the silty-clayey soil layer). Settlement calculations due to the construction of the embankment are performed by implementing conventional methods, taking also into consideration the theory of one-dimensional consolidation. The above mentioned methodology is included in a Microsoft excel calculation sheet.

Methodology for overall stability analyses based on Eurocode 7.1 & Eurocode 8

According to the Greek National Annex of Eurocode 7.1, for the overall stability of slopes (including any existing or planned future structures on them) Design Approach 3 (DA-3) is adopted for the geotechnical (GEO) ultimate limit state (ULS) design. The stabilizing actions of the structural retaining elements (i.e. reaction forces and moments of the structural elements e.g. nails, anchors or piles) are considered as favourable actions with corresponding partial factor of actions equal to γ_F=1.0.

Design approach 3 (DA-3) is applied in combination with equation (1) (expression 2.6a-EN 1997-1) for the actions and equation (2) (expression 2.7a-EN 1997-1) for the reactions:

$$E_d = E\ (F_d, X_d\ ) = E\ (\gamma_F\ F_k, X_k\ /\ \gamma_M\ ) \qquad (4\text{-}1)$$

$$R_d = R\ (F_d, X_d\ ) = R\ (\gamma_F\ F_k, X_k\ /\ \gamma_M\ ) \qquad (4\text{-}2)$$

by applying equation (3) (expression 2.5-EN 1997-1):

$$E_d \le R_d \quad \text{therefore} \quad E\ (\gamma_F\ F_k, X_k\ /\ \gamma_M\ ) \le R\ (\gamma_F\ F_k, X_k\ /\ \gamma_M\ ) \qquad (4\text{-}3)$$

where:

E_d: design value of the effect of actions
R_d: design value of the resistance to an action
F_d: design value of an action
X_d: design value of a material property
F_k: characteristic value of an action
X_K: characteristic value of a material property
γ_F: partial factor for an action
γ_M: partial factor for a soil parameter (material property)
γ_R: partial factor for a resistance

According to Design approach 3, it shall be verified that a limit state of rupture or excessive deformation will not occur with the following combination of sets of partial factors (applied either on actions or on the effects of actions from the structure and to ground strength parameters):

Combination: (A1*or A2**) + (M2) + (R3)

- (A1): the characteristic values of actions coming from the structure (structural actions), e.g. loads from buildings and traffic loads at ground surface are multiplied by the factor of set A1 (Table VI-1-Table A3 from EN 1997-1)
- (A2): for actions arising from the ground or transferred through it (geotechnical actions) including the weight of the soil. For slope stability analyses, the actions on the ground (e.g. loading from structures, traffic loads) are considered as geotechnical actions using the factor of set A2 (Table A3 from EN 1997-1)
- (M2): for the soil parameters (Table A4 from EN 1997-1)
- (R3): for the resistance (γ_R=1.0 from Table A14 EN 1997-1)

Table A3: Partial factors on actions (γ_F) or on the effects of actions (γ_E) (Table A3, EN 1997-1)

Action		Symbol	Set	
			A1	A2
Permanent	Unfavourable	γ_G	1.35	1.00
	Favourable		1.00	1.00
Variable	Unfavourable	γ_Q	1.50	1.30
	Favourable		0	0

Table A4: Partial factors for soil parameters (γ_M) (Table A4, EN 1997-1)

Soil parameter	Symbol	Set	
		M1	M2
Angle of shearing resistance (tanφ')	$\gamma_{\varphi'}$	1.0	1.25
Effective cohesion (c')	$\gamma_{c'}$	1.0	1.25
Undrained shear strength (c_u)	γ_{cu}	1.0	1.40
Unconfined strength (q_u)	γ_{qu}	1.0	1.40 1.60 for rock
Weight density (γ)	γ_γ	1.0	1.00

For the geotechnical projects which are analyzed according to Eurocode EN1997-1, the corresponding analyses under seismic loading conditions are performed according to Eurocode 8 – Part 5 (EN 1998-5) of the Greek National Annex. In the above analyses Design Approach 2 is implemented (Variation DA-2*), in which the characteristic values of soil parameters are used (set M1 with γ_M=1.0, see Table A4) and the total resulting resistance of the soil is divided by a resistance factor equal to γ_R=1.0. For the case of seismic design a common value for the partial factor on the effect of actions is adopted i.e. γ_E=1.0.

The stability verification under seismic conditions may be carried out by means of simplified pseudo-static methods, in areas where the surface topography and soil stratigraphy do not present very abrupt irregularities. The design seismic inertia forces F_H and F_V acting on the ground mass, for the horizontal and vertical directions respectively, in pseudo-static analyses shall be taken as:

$$F_H = 0.5\ \alpha\ S\ W \qquad (4\text{-}4)$$

$$F_v = \pm 0.5\ F_H \text{ (for } \alpha_{vg}/\alpha_g > 0.6) \qquad (4\text{-}5)$$

where:

- α: the ratio of the design ground acceleration on type A ground, α_g, to the acceleration of gravity, g
- α_{vg}: the design ground acceleration in the vertical direction
- α_g: the design ground acceleration for type A ground
- α_{vg}/α_g: equal to 0.90 (from Table 3.4, EC-8, Part 1 (EN 1998-1), Type of spectrum 1)
- S: the soil parameter from Table 3 of the Greek National Annex of EC-8-Part 1 (EN 1998-1) (see Table 3)
- W: the weight of the sliding mass

Table 3: Values of parameters which determine the horizontal spectrum of elastic response (Type 1) (Table 3, Greek National Annex EC8-Part 1, EN 1998-1)

Soil Category	S
A	1.00
B	1.20
C	1.15
D	1.35
E	1.40

Slope stability analyses for the examined embankment according to Eurocode 7.1 & Eurocode 8

The methodology applied for the slope stability analyses of the embankment according to Eurocode 7.1 and Eurocode 8 is described in detail in the above paragraphs.

The traffic load was taken into account in the slope stability calculations of the embankment by applying a distributed load on the crest of the embankment (over 3.0m width) equal to P=69.27kPa for both the case of static and seismic loading. It is noted that the above mentioned load is further increased in case of static loading by multiplying it with a partial factor of actions equal to γ_F=1.30.

The design values of the seismic inertia forces used in slope stability analyses of the examined embankment under seimic loading according to Eurocode 8, are summarized in Table 4-1.

Note: in case the embankment is founded surficially on soil layers with significant thickness classified in Soil Category C or D, which are encountered above soil formations classified in Soil Category A, the slope stability analyses are performed by taking into account the most unfavourable values of acceleration (applicable for soil categories C or D).

For the determination of seismic inertia forces, the reference value of the maximum acceleration corresponding to soil of Type A, is increased and taken equal to $\mathbf{\alpha_{gR}/g}$=0.20, due to the fact that the examined embankment is in the vicinity of a fault.

Table 4-1: Values of seismic inertia forces, F_H and F_v, for the embankment (EC-8)

Embankment / Cross section	Formation / Soil category (EC-8)	Design values of seismic inertia forces	
		F_H	F_v
GG1201 (CH. 93+560)	ALsg (formation I) / B to D & ALcm (formation II) / D	0.135 W	0.068 W

In addition, for the embankments that are founded on coarse grained soil materials, analyses under undrained conditions are not required and therefore were not performed.

Internal slope stability analyses of the embankment under static and seismic loading conditions using limit equilibrium method

Slope stability analyses were carried out according to Krey's method (similar to Bishop method) by subdividing the slope into slices of constant width and by assuming circular slip surface. The safety factor is defined as:

$$F = \frac{\tau_d}{\tau_{existing}} = \frac{\sum_i resisting\ moments}{\sum_i driving\ moments} \qquad (4\text{-}6)$$

and it is determined through iterative procedure using the following equation:

$$F^{iter+1} = \frac{\sum_i \left[c_i \Delta x_i + \left(G_i + V_i - u\,\Delta x_i\right)\tan\varphi_i\right]\frac{1}{m_{ai}} + \sum_j S_j \cos\left(\alpha_j - \omega_j\right) + \sum_k \left[A_{Vk}\tan\varphi_i \frac{1}{m_{ai}}\right]}{\sum\left(G_i + V_i\right)\sin\alpha_i + \sum_l H_l\left(\frac{y_m - y_{H_l}}{R}\right) + \sum_k \left[A_{Vk}\sin\alpha_k - A_{Hk}\cos\alpha_k\right]} \qquad (4\text{-}7)$$

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$$m_{ai} = \cos\alpha_i\left[1 + \frac{\tan\varphi_i\ \tan\alpha_i}{F^{iter}}\right] \qquad (4\text{-}8)$$

where: G: weight of slice

- H: sum of the horizontal forces acting on the slice
- V: sum of the external vertical forces acting on the slice
- A_H, A_V: horizontal and vertical component of forces due to anchors or soil reinforcement elements
- S: shear resistance (from structural elements)
- x_m, y_m: coordinates of the centre of the slip circle
- y_H: y coordinate of the resulting horizontal force H
- Δx: slice width
- R: radius of slip circle
- α_i: slope of the bottom of slice at point P
- β: slope of the surface of the water table
- α_k: slope of the slip surface at the intersection with the anchor
- ω: slope of the shear resistance from structural element
- α_j: slope of the slip surface at the intersection with the shear resistance
- φ: soil friction angle at point P
- c: soil cohesion at point P
- u: absolute pore water pressure at point P

For the explanation of the parameters included in the above safety factor equation the following Figure 4-1 is used.

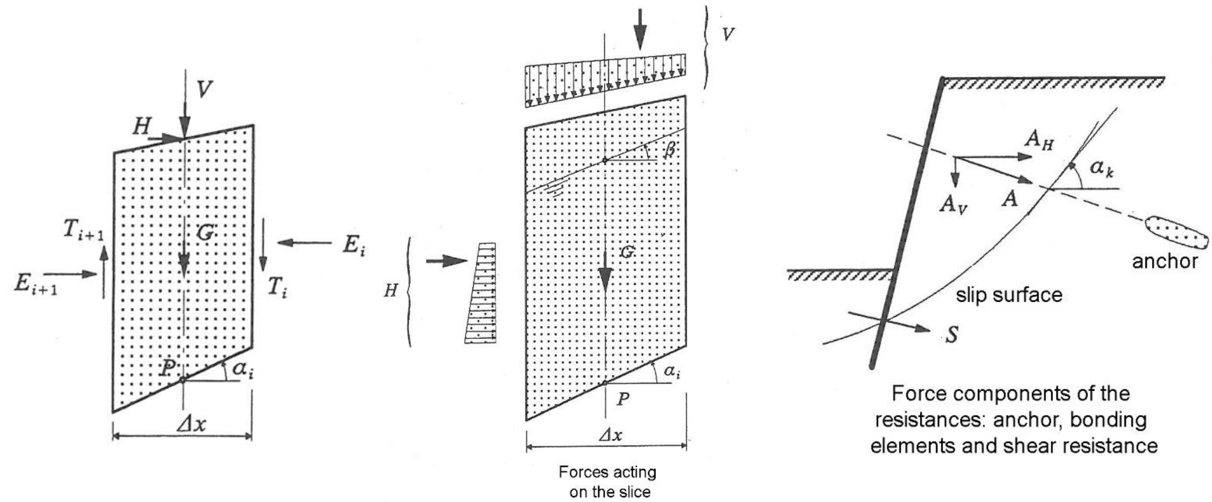

Figure 4-1. Schematic representation of a slice with forces acting on it

The above expression for the determination of safety factor is simplified in case no anchors and shear elements are used in the analyses. This is also the case for the examined embankment, for which in the safety factor equation the last two components of the numerator and the last component of the denominator are omitted.

Calculation of settlements due to the construction of the embankment

A. Calculation of immediate settlements

For the calculation of immediate settlements the following equation is applied:

$$S = H\frac{\Delta\sigma'_v}{E_s} \qquad (4\text{-}9)$$

where: H = the thickness of the examined soil layer

$\Delta\sigma'_v$ = applied stress at the examined depth due to the embankment's construction (for embankment of infinite length)

E_s = oedometer Young's modulus

B. Calculation of consolidation settlements

For the calculation of consolidation settlements it is assumed that the deformations of the compressible layer will occur in only one dimension. Therefore the theory of one-dimensional consolidation is implemented. The examined layer is divided into sub-layers of small thickness and the following equation, corresponding to normally consolidated clays, is applied. The laboratory oedometer test results for the clayey layer (Layer II), encountered in the examined embankment, indicate low values of preconsolidation stress, p'_c. Thus it considered that the clayey layer is normally consolidated.

For normally consolidated clays ($\sigma'_{vo}>p'_c$ και $\sigma'_{vo}+\Delta\sigma'_v>p'_c$):

$$S = H\frac{C_c}{1+e_o}\log\left(\frac{\sigma'_{vo}+\Delta\sigma'_v}{\sigma'_{vo}}\right) \qquad (4\text{-}10)$$

where: p'_c = preconsolidation stress (from laboratory test results)

σ'_{vo} = vertical effective stress at the depth of interest

$\Delta\sigma'_v$ = applied stress at the examined depth due to embankment's construction

C_c = compressibility index (from consolidation test results)

e_o = initial void ratio

H = thickness of each examined sub-layer

The variation of the applied stress due to the construction of the embankment with depth is given in the following diagram. For reasons of simplification in the current section the calculation of the applied stress from the embankment is given only at the embankment's axis. Therefore $\Delta\sigma'_v$=I*P (where P=embankment loading=$\gamma_{embank.}$*$H_{embank.}$).

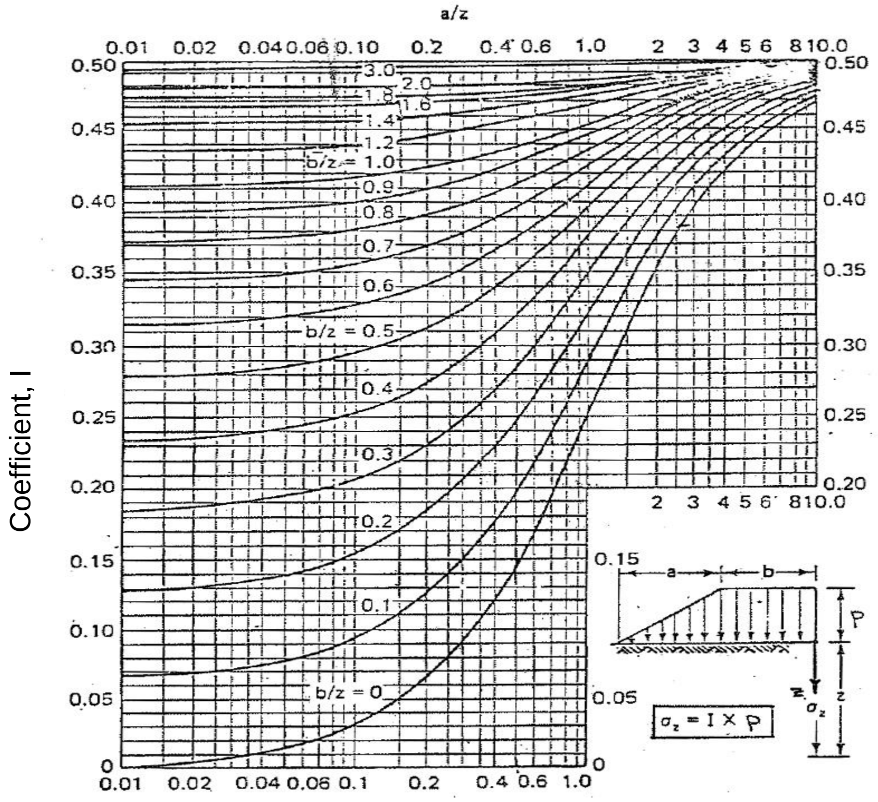

Figure 4-2. Applied load from the embankment (below the axis)

C. Calculation of consolidation rate

For practical purposes apart from the total magnitude of consolidation settlement (ultimate consolidation settlement at time t=∞), the rate of consolidation settlements is also of great significance. For the determination of consolidation settlement after time "t", the average degree of consolidation, U_t, is used.

$$U_t = \text{settlement after time "t" / ultimate consolidation settlement} \qquad (4\text{-}11)$$

C1. Without vertical drains

The average degree of consolidation, U_t, is determined from the diagram of Figure 4-3, in relation to the non-dimensional time factor, T_v for different cases of linear variation of Δ_{uo} (excess pore water pressure) with depth and corresponding drainage conditions.

The consolidation settlement after time "t", S_t, is calculated as:

$$S_t = U_t * S_c \qquad (4\text{-}12)$$

where: T_v = non-dimensional time factor = C_v*t/H^2

H = length of the longest drainage path

C_v = coefficient of consolidation

S_c=total magnitude of consolidation settlement

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In case of two-way drainage condition (top and bottom of the clayey layer), the thickness of the layer is taken equal to 2H, whereas for one-way drainage condition the layer thickness is taken equal to H.

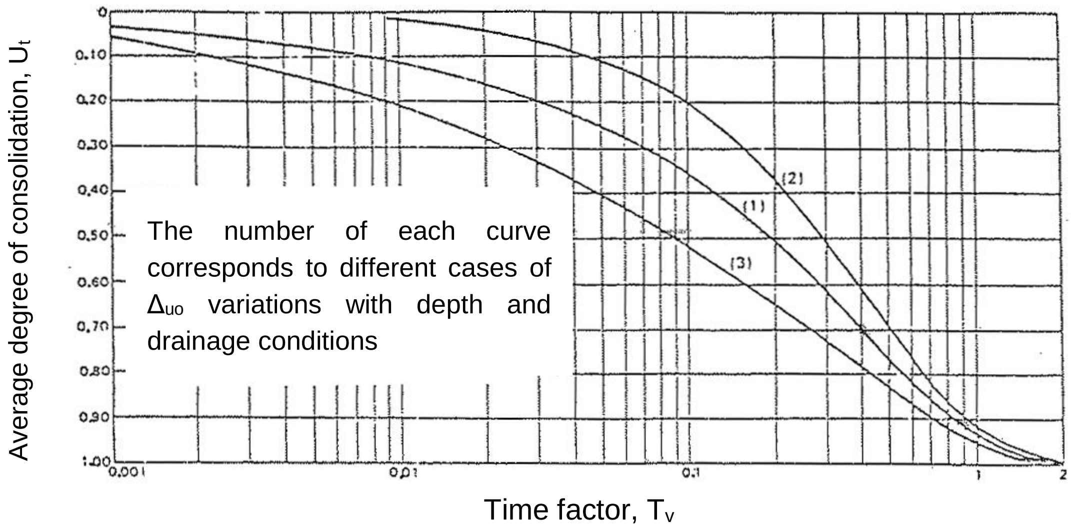

Figure 4-3. Average degree of consolidation (for drainage in the vertical direction)

Alternatively, the average degree of consolidation is calculated using the following expressions (drainage in the vertical direction):

$$U_{tv}=2\left(\frac{T_v}{\pi}\right)^{1/2} \qquad \text{for } 0<T_v<0.2827 \qquad (4\text{-}13)$$

$$U_{tv}=1-10^{\frac{-0.085-T_v}{0.933}} \qquad \text{for } Tv>0.2827 \qquad (4\text{-}14)$$

C2. With vertical drains

In this case the vertical drainage as well as the radial drainage are taken into consideration.

The time factor for radial drainage is given by the following expression:

$$T_r=\frac{t*C_h}{(1.05*S)^2} \qquad (4\text{-}15)$$

where: C_h = coefficient of consolidation in the horizontal direction (taken equal to 2* C_v)

S=distance between drains

The degree of consolidation (with vertical drains) is determined using the following equation:

$$U_{tr}=1-e^{\frac{-8*T_r}{F_n}} \qquad (4\text{-}16)$$

where:

$$F_n=\frac{n^2}{n^2-1}\ln(n)-\frac{3*n^2-1}{4*n^2} \qquad (4\text{-}17)$$

for triangular grid of drains: $n=0.525*S/r_w$ (r_w=radius of drains) (4-18)

For the determination of the average degree of consolidation in case vertical drains are installed, the vertical and radial drainage is taken into consideration as follows:

$$U=1-(1-U_{tv})*(1-U_{tr}) \qquad (4\text{-}19)$$

Relevant references

Athanasopoulos, G. A., (1986), “Concise Theory and Problems of Soil Mechanics”, University of Patras Editions (in Greek), 235p.

Barnes, G.E., (2005), “Soil Mechanics-Principles and Practice”, Palgrave Macmillan Edition, 540p.

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