Geotechnical Design of Embankment: Slope Stability Analyses and Settlement Calculations

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ABSTRACT: *The objective of the paper is the presentation of a case study developed for geotechnical engineering instruction. The examined case study is suitable for undergraduate instruction and refers to the geotechnical design of a railway embankment focusing on slope stability analyses and settlement calculations. The presented geotechnical data have resulted from the geotechnical study implemented by Edafomichaniki S.A. within the framework of the project "Geotechnical Design of the New Railway Alignment between the Existing Line and Kavala's Port". The geotechnical design of a railway embankment is described thoroughly in the current paper and includes the following: a) the evaluation of the available geotechnical data encompassing the determination of soil stratigraphy and geotechnical design parameters of the encountered formations, b) the performance of geotechnical calculations regarding slope stability analyses and settlement calculations with detailed reference to consolidation theory and c) the presentation of the construction methodology of the embankment. The usefulness of the examined case study is highlighted through the achievement of a significant number of learning outcomes. The current paper is accompanied with detailed supplementary material.*

KEYWORDS: geotechnical instruction, embankment, slope stability, settlements, consolidation theory

SITE LOCATION: IJGCH-database.kmz (requires Google Earth)

INTRODUCTION

Case studies constitute a significant component of geotechnical engineering instruction. It should be noted though that an interesting case related to geotechnical practice is not always suitable for instruction (Belokas et al. 2013). For this reason, Orr (2011) has proposed to evaluate the benefits of incorporating case studies in geotechnical courses with regard to the learning outcomes that can be achieved. According to Orr (2011), the learning outcomes depend on fundamental cognitive elements, including knowledge, comprehension and application, as well as on critical thinking qualities such as analysis and evaluation.

The experience provided by case studies is regarded as the basic element connecting the individual aspects of the Soil Mechanics Triangle (Burland 1987) as well as of the recently proposed Geotechnical Design Triangle (Orr 2011). These aspects refer to the ground profile, the soil behavior and the methodology of analyses. More recently, Pantazidou and Orr (2012) have proposed a list of learning outcomes that can be achieved by using case studies in geotechnical instruction.

It should be noted though that the usefulness of case studies used in instruction is limited if these case studies are not well documented. Therefore, detailed supporting material should be prepared in order to accompany the relevant case study and provide the data required for its comprehensive documentation (Belokas et al. 2013, Orr 2011). The above supplementary data should be available in electronic format so that part or all of it can be accesible to the students of the pertinent geotechnical course.

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The literature cases that are accompanied with supplementary material for teaching are rather limited (Pantazidou et al. 2008, Belokas et al. 2013, Orr and Pantazidou 2013, Xenaki et al. 2014, Zekkos et al. 2014), which highlights the significance of the currently presented case study. However, a few basic rules regarding the preparation of case studies for geotechnical instruction have been proposed by several researchers (Herreid 1997, Orr and Pantazidou 2013).

The objective of the current paper is the presentation of a case study suitable for undergraduate instruction in geotechnical engineering courses. The presented case refers to the integrated geotechnical design of a railway embankment, implemented by EDAFOMICHANIKI S.A. within the framework of the project "Geotechnical Design of the New Railway Alignment between the Existing Line and Kavala's Port" located in Greece. The present paper is also accompanied by detailed case material and it is envisioned to assist instructors of undergraduate geotechnical courses who would like to present in class the complete geotechnical design of a real project, with a focus on slope stability analyses and settlement calculations.

SITE LOCATION AND GEOLOGICAL CONDITIONS

The 1.2km-long railway embankment will be constructed at the broader area of Nestos' river in northern Greece, in the close vicinity of Nestos' bridge. It is noted that at the area of the embankment, the railway alignment is adjacent to the alignment of Egnatia Highway. The location of the embankment is shown in Figure 1.

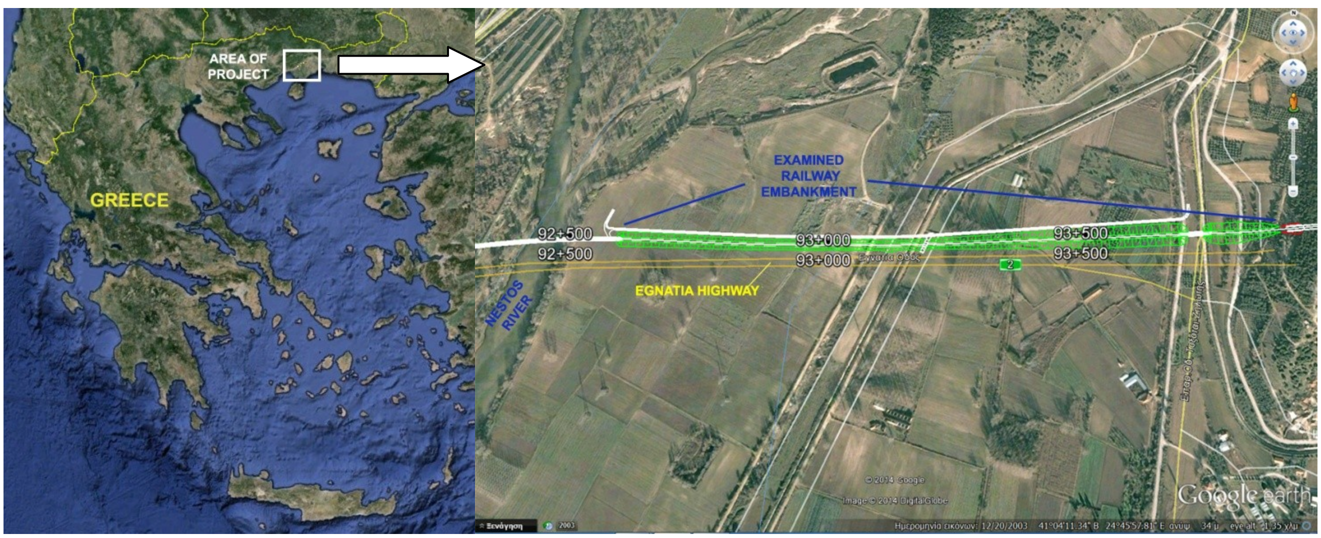

Figure 1. The area of the examined embankment.

In order to investigate the geological and geotechnical conditions in the area of interest, the results of geotechnical investigations from several phases are available. This is one of the reasons for which the examined case study is considered as particularly suitable for undergraduate instruction in the geotechnical engineering field. More specifically, the following geotechnical data are evaluated: a) four (4) sampling boreholes (M8, M12, M16, A2) executed in the framework of the current railway line project and b) seven (7) sampling boreholes (G48, G50, GN-1 to GN-4 and G103) executed in previous investigation phases for the adjacent project of Egnatia Highway.

According to the Geological study, which has been completed for the current project (Megalomastora 2012), the embankment is founded on alluvial deposits (AL) consisting of clayey and sandy-clayey soil materials. In places fine pebbles of marble and gneiss of brown-red colour are also included in the mass of alluvial deposits. The rock formation of gneiss (gn) constitutes the bedrock at the area of interest. The geological plan view and the geological longitudinal section of the examined embankment are presented in Figure 2 and Figure 3, respectively.

The porous layer of alluvial deposits is characterized by varying values of permeability coefficient, depending mainly on the grain size distribution of the formation. Water circulation is locally allowed through the deposits, whereas the groundwater level is expected to be encountered at small depths from the ground surface at the level of Nestos river.

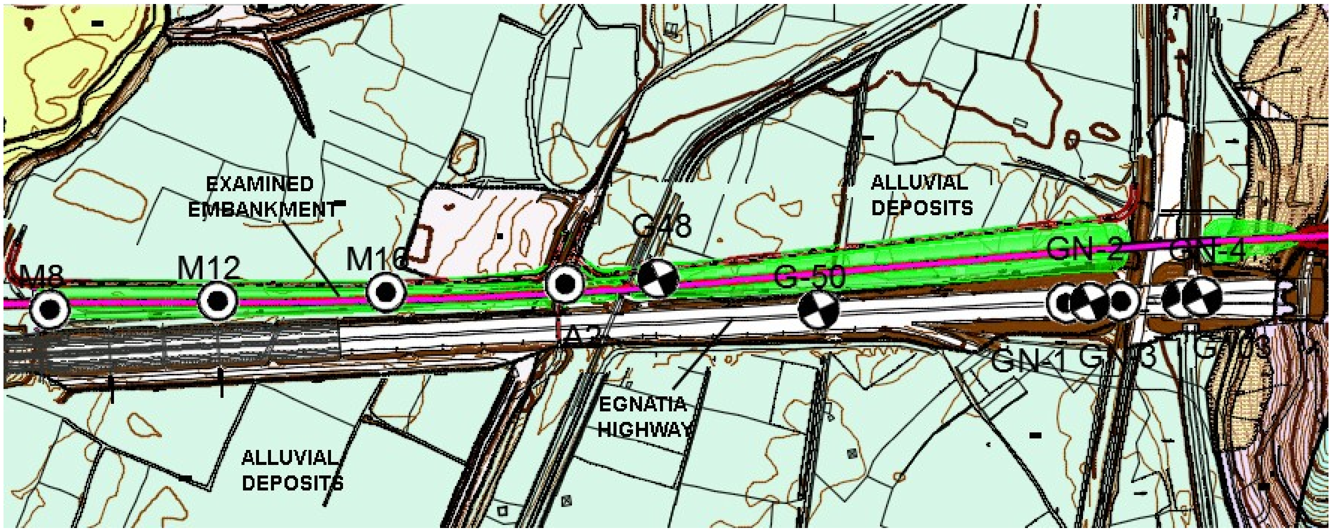

Figure 2. Geological plan view of the embankment with boreholes' locations (Megalomastora 2012).

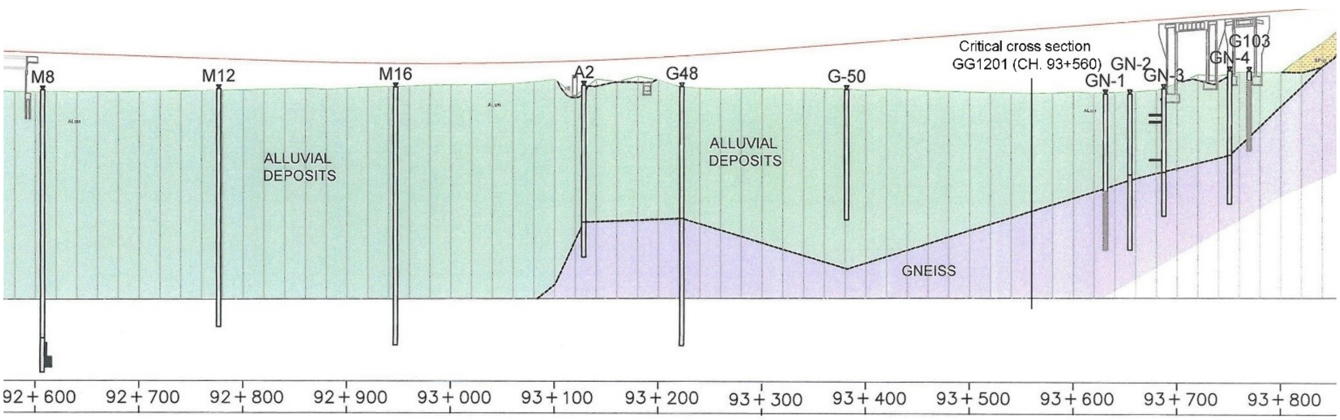

Figure 3. Geological longitudinal section of the embankment (Megalomastora 2012).

The borehole data of the pertinent geotechnical investigations have been taken into account for the preparation of the geological plan view and longitudinal section of the embankment, and resulted in the separation of individual geological layers along the embankment. The engineering geological evaluation of the sampling boreholes located in the examined embankment is given in Table 1.

Measurements from the piezometers installed in the boreholes during the execution of the pertinent geotechnical investigations are evaluated in order to determine the groundwater level (GWL) in the encountered geological formations. The results of the piezometer measurements, obtained after the completion of drilling, indicate that the groundwater level is found at relatively small depth, approximately at 4.0 m below ground level. Detailed data regarding the geology and the groundwater conditions at the area of interest are given in "2-Geological conditions" of the supplementary material.

GEOTECHNICAL EVALUATION AND GEOTECHNICAL DESIGN PARAMETERS

The geotechnical evaluation of the investigation data includes the determination of the detailed soil stratigraphy along the embankment as well as the determination of the geotechnical design parameters of the encountered soil formations. The evaluation is based on the following: a) the results of in situ and laboratory tests and b) the geological study and the technical-geological evaluation of boreholes ("3-Geotechnical evaluation" of the supplementary material).

Table 1. Geological engineering evaluation of boreholes.

No	Borehole	Depth (m) (from-to)	Symbol	Formation
1	M8	0.00-48.00	AL	Alluvial deposits
		48.00-55.00	gn	Gneiss
2	M12	0.00-46.00	AL	Alluvial deposits
3	M16	0.00-50.00	AL	Alluvial deposits
4	A2	0.00-26.30	AL	Alluvial deposits
		26.30-33.00	gn	Gneiss
5	G48	0.00-25.30	AL	Alluvial deposits
		25.30-33.00	mr	Marble
		33.00-44.50	gn	Gneiss
		44.50-45.70	mr	Marble
		45.70-50.00	gn	Gneiss
6	G50	0.00-25.00	AL	Alluvial deposits
7	GN-1	0.00-18.70	AL	Alluvial deposits
		18.70-30.00	gn	Gneiss
8	GN-2	0.00-15.50	AL	Alluvial deposits
		15.50-30.00	gn	Gneiss
9	GN-3	0.00-16.00	AL	Alluvial deposits
		16.00-24.50	gn	Gneiss
10	GN-4	0.00-16.00	AL	Alluvial deposits
		16.00-25.00	gn	Gneiss
11	G103	0.00-1.70	AL	Alluvial deposits
		1.70-15.00	gn	Gneiss

More specifically, the laboratory test results conducted on soil and rock specimens are evaluated by determining the minimum, maximum and average values, as well as the standard deviation, for each layer and laboratory test separately. By combining the results of all laboratory test results regarding soil physical properties, soil and rock mechanical properties of the encountered formations, it became possible to further divide the aforementioned geological layers into geotechnical layers with similar properties. The results of in situ Standard Penetration Tests are also taken into consideration. The statistics for each layer and laboratory test of the examined embankment are given in detail in the provided supplementary material accompanying the current paper ("3-Geotechnical Evaluation").

By following the above mentioned procedure it is concluded that the soil stratigraphy along the examined embankment includes the following geotechnical layers: a) Layer I: Sand with gravels (Alluvial deposits of sandy-gravelly composition-ALsg). The formation is characterized as Sand of brown to brown-grey color with a small amount of silt and fine gravels of quartz and gneiss origin, loose to very dense, locally with gravels. This layer is further divided into sub-layers depending on the density, by taking into consideration the blow counts N_{SPT} of the standard penetration test. (Layer Ia: ALsg of loose deposition with N_{SPTav}=9, Layer Ib: ALsg of medium density with N_{SPTav}=20, and Layer Ic: ALsg of high density with N_{SPTav}>50) b) Layer II: Clay and silt (CL-ML) (Alluvial deposits of silty-clayey origin-ALcm). The formation is characterized as firm Clay and Silt of grey to brown color with sand and gravels and c) Layer III: Gneiss (gn). The formation is characterized as Gneiss of greenish-grey and white color, fractured, moderately weathered and locally weathered. Table 2 presents the depth at which each geotechnical formation is encountered in the evaluated boreholes. The soil stratigraphy along the examined embankment is presented in the geotechnical longitudinal section of Figure 4.

The geotechnical design parameters for each encountered formation were derived based on the statistical processing of laboratory test resuts in combination with published correlations. The estimation of the critical geotechnical parameters and the proposed design values are described in detail in the supplementary material of the paper ("3-Geotechnical evaluation").

The proposed design values (characteristic values) for each geotechnical layer are summarized in Table 3, which also includes the factored geotechnical parameters which are used in the stability analyses according to Eurocode 7.1. According to the Greek National Annex of Eurocode 7.1, for the overall stability of slopes, Design Approach 3 (DA-3) is adopted for the geotechnical ultimate limit state design under static conditions. According to Eurocode 8 – Part 5 (EN 1998-5) of the Greek National Annex for the stability analyses under seismic conditions, Design Approach 2 is implemented (Variation DA-2*). These methodologies are described in detail in the supplementary material of the current paper ("4-Assumptions of Geotechnical Calculations"). By taking into consideration all the above, the soil parameters used in stability analyses under static conditions (presented in the right part of Table 3) result from the characteristic values (shown in the left part of Table 3) by applying the soil parameters partial factors, γ_M, (set M2) included in Table A4 of "4-4-Assumptions of Geotechnical Calculations" of the supplementary material. In stability analyses under seismic conditions, the characteristic values of soil parameters are used (set M1 with γ_M=1.0, see Table A4 mentioned above) without applying any partial factors on the soil properties.

Table 2. Encountered geotechnical formations in the evaluated boreholes.

Borehole	Depth of encountered formations (from-to) (m)		
	Layer I (ALsg)	Layer II (ALcm)	Layer III (gn)
M8	0.00-48.00	-	48.00-55.00
M12	0.00-20.00 & 22.50-46.00	20.00-22.50	-
M16	0.00-18.45 & 29.20-50.00	18.45-29.20	-
A2	0.00-16.45 & 25.00-26.30	16.45-25.00	26.30-33.00
G-48	0.00-25.30	-	25.30-50.00 (marble and gneiss)
G-50	0.00-11.00 & 20.20-25.00	11.00-20.20	-
GN-1	0.00-18.70	-	18.70-30.00
GN-2	4.20-15.50	0.00-4.20	15.50-30.0
GN-3	4.80-16.00	0.00-4.80	16.00-24.50
GN-4	0.00-16.00	-	16.00-25.50
G103	0.00-1.70	-	1.70-15.00

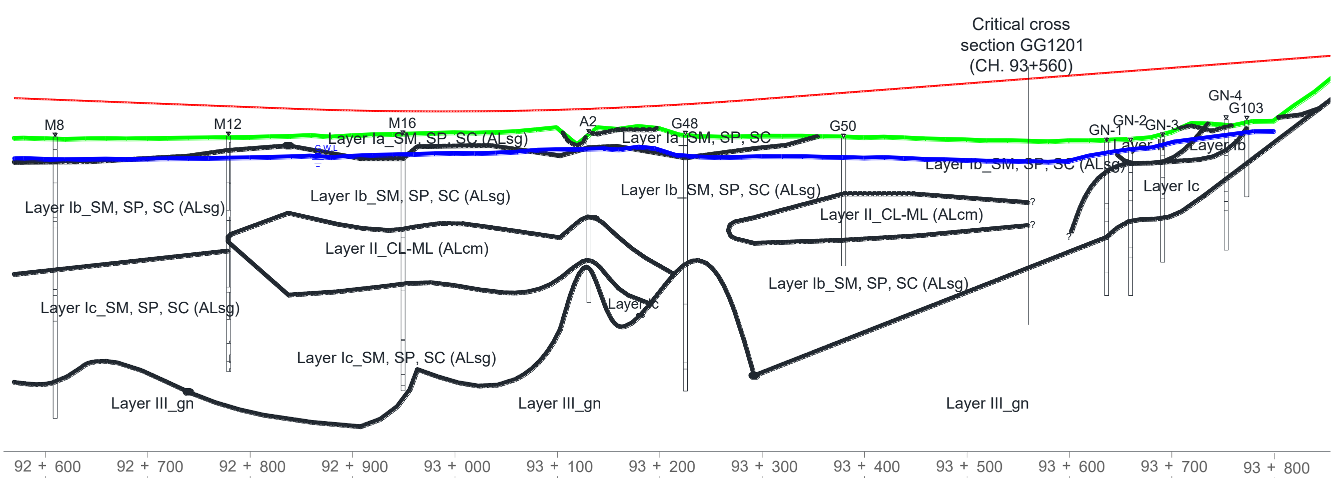

Figure 4. Geotechnical longitudinal section of the embankment.

Table 3. Geotechnical design parameters of encountered formations.

Layer	Geotechnical design parameters (characteristic values)					Geotechnical design parameters (stability analyses according to Eurocode 7.1)			
	γ (kN/m^3)	φ' (o)	c' (kPa)	Es (MPa)	c_u (kPa)	γ (kN/m^3)	φ' (o)	c' (kPa)	c_u (kPa)
Ia / Sand & gravels-loose	19.7	29	0	7	-	19.7	23.9	0	-
Ib / Sand & gravels-medium	19.7	32	0	14	-	19.7	26.6	0	-
Ic / Sand & gravels-dense	19.7	35	0	35	-	19.7	29.3	0	-
II / CL-ML	19.6	28	5	8	35	19.6	23	4	25
III / gn	25.4	26	200	450	-	25.4	21.3	160	-

GEOTECHNICAL CALCULATION ASSUMPTIONS

The embankment has a total length of 1200 m and maximum height equal to 13.0 m, approximately. The critical cross-section used in the geotechnical calculations presented in the current paper 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 specifically, cross-section GG1201 (CH. 93+560) (having height equal to 12.20m) is considered as the most unfavorable cross section in terms of geometric characteristics, which is representative for the area where the embankment is founded on compressible alluvial deposits of significant thickness. The critical cross-section of the embankment used in the geotechnical calculations is presented in Figure 5. The location of the critical section is shown in Figure 4. The thickness of layer II (fine grained layer with consolidation settlements) is conservatively taken equal to the maximum thickness encountered (at the area of borehole G50) and not equal to that corresponding to cross section GG1201. The groundwater level is taken at 4.0 m depth below the ground surface. In the geotechnical calculations, an improvement layer of 1.0 m thickness is taken into consideration.

It is emphasized that for the integrated geotechnical design of the embankment, the execution of geotechnical calculations on several representative cross-sections is required. The geotechnical calculations of the above mentioned cross-section was selected to be presented in detail in the current paper as a typical example, the fundamental principles of which were also applied to all examined cross sections.

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, Barnes 2005, Athanasopoulos 1986). At the area of the examined embankment the groundwater level is located at 4.0 m depth below the ground surface. The relatively high groundwater level in combination with the existence of loose sandy and silty-sandy soil layers at the area of the embankment, 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.

For the construction of the main part of the embankment, it is suggested that coarse-grained crushed materials are used. The slope stability analyses and the settlement calculations of the embankment are carried out assuming the following geotechnical parameters for the material of the embankment: Unit weight: γ=20 kN/m^3, Angle of internal friction: φ'=35^o, Cohesion c'=5 kPa and Elasticity modulus E_s = 50 MPa.

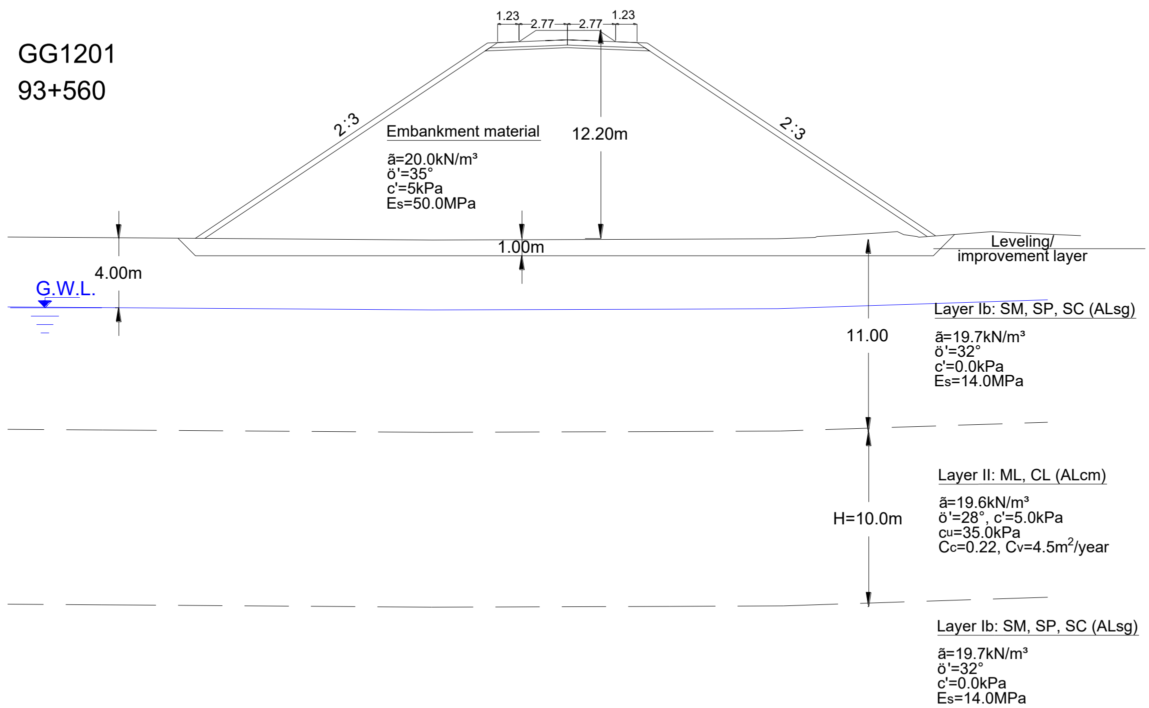

Figure 5. Critical cross-section used in the geotechnical calculations of the embankment.

Slope Stability Analyses of The Embankment

Slope stability analyses are performed according to Eurocode 7.1 and Eurocode 8 for static and seismic loading conditions, respectively. According to the Greek National Annex of Eurocode 7.1, Design Approach 3 (DA-3) is adopted for the geotechnical (GEO) ultimate limit state (ULS) design for the overall stability of slopes. Design Approach 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:

$$E_d = E\,(F_d, X_d\,) = E\,(\gamma_F\, F_k, X_k\,/\,\gamma_M\,) \tag{1}$$
$$R_d = R\,(F_d, X_d\,) = R\,(\gamma_F\, F_k, X_k\,/\,\gamma_M\,) \tag{2}$$

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

$$E_d \le R_d \;\; \text{therefore} \;\;\; E\,(\gamma_F\, F_k, X_k\,/\,\gamma_M\,) \le R\,(\gamma_F\, F_k, X_k\,/\,\gamma_M\,) \tag{3}$$

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_κ: 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) where:

(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 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)

The above mentioned tables from EN 1997-1 are presented in detail in the supplementary material of the current paper ("4-Assumptions of Geotechnical Calculations").

The corresponding slope stability analyses under seismic loading conditions are performed according to Eurocode 8 – Part 5 (EN 1998-5) of the Greek National Annex. More specifically, 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, Table A4 from EN 1997-1) 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.

According to Eurocode 8, the stability verification under seismic conditions may be carried out by means of simplified pseudo-static methods, in which the design seismic inertia forces F_H and F_V acting on the ground mass, for the horizontal and vertical directions respectively, shall be taken as:

$$F_H=0.5\ \alpha\ S\ W \tag{4}$$
$$F_v=\pm 0.5\ F_H\ (\text{for } \alpha_{vg}/\alpha_g>0.6) \tag{5}$$

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
- $\alpha_{vg}/\ \alpha_g$: 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)
- W: the weight of the sliding mass

According to Eurocode 8 the examined embankment is located in a seismic Zone Z1. The design ground acceleration on type A ground for seismic Zone Z1 is equal to α_{gR}/g=0.16. This value is further increased for the other ground types, by multiplying it with soil parameter S>1.0 (Eurocode 8). Due to the proximity of the project to an active seismotectonic fault, even if it is not characterized as seismically active, the design seismic action is conservatively increased by 25%. Therefore the design ground acceleration on type ground A is taken equal to α_{gR} /g=0.20. According to Table 3.1 of Eurocode 8 (Part 1) (EN 1998-1:2004), the soil formations encountered along the examined embankment are classified in the ground types listed in Table 4.

Table 4. Classification of encountered formations according to Eurocode 8.

Formation	Description	Ground type (Eurocode 8)	Ground type description
Ia	Sand and gravels (loose) (Alluvial deposits - ALsg)	D	Deposits of loose-to-medium cohesion-less soil (N_{SPT}<15)
Ib	Sand and gravels (medium density) (Alluvial deposits - ALsg)	C	Deposits of dense or medium dense sand and gravel (N_{SPT}=15-50)
Ic	Sand and gravels (dense) (Alluvial deposits - ALsg)	B	Deposits of very dense or medium dense sand or gravel, characterized by a gradual increase of mechanical properties with depth (N_{SPT}>50)
II	Silt and clay (Alluvial deposits-ALcm)	D	Deposits of predominantly soft-to-firm cohesive soil (N_{SPT}<15, c_u<70kPa)
III	Gneiss (gn)	A	Rock or other rock-like geological formation

The design values of the seismic inertia forces used in slope stability analyses of the critical cross section (GG1201) of the embankment according to Eurocode 8 are: F_H=0.135 W and F_v=0.068 W.

The rail traffic load in the slope stability calculations is modelled by applying a distributed load on the crest of the embankment (over 3.0 m width) equal to P=69.27 kPa for both the case of static and seismic loading. This load is further increased in the case of static loading by multiplying it with a partial factor of actions equal to γ_F=1.30. Moreover, in static loading conditions, the shear strength parameters of the encountered formations are reduced through the soil parameters partial factors (γ_M), whereas the resulting soil resistance is divided by a resistance factor equal to γ_R=1.0. Thus, the required "equivalent safety factor" of the analyses is equal to $FS_{equiv.}$=1.0.

For the internal slope stability analyses of the embankment under static and seismic loading conditions, the limit equilibrium method was implemented using Larix-4S software (v. 2.21-Cubus). More particularly Krey's methodology was applied by subdividing the slope into slices of constant width and by assuming circular slip surface. The safety factor is defined through an iterative procedure as the ratio between resisting and overturning moments. The methodology principles as well as the pertinent equations are given in detail in the supplementary material ("4-Assumptions of Geotechnical Calculations"). The above mentioned Krey's methodology is essentially similar to Bishop simplified method (Steiner and Irngartinger 2011) and the resulting safety factors are considered comparable to the values resulting from the application of other methodologies for circular slip surfaces (Fredlund et al. 1981).

Calculations of Soil Settlements Due To Embankment Construction

The geotechnical design of the examined embankment encompasses also the calculation of soil settlements in the critical cross-section (GG1201). The calculated immediate settlements are expected to be completed during the construction of the embankment. Due to the fact that the embankment is founded on alluvial deposits of clayey-silty composition and the groundwater level is encountered at small depth from the ground surface, the development of consolidation settlements is also expected.

For the calculation of consolidation settlements it is assumed that the deformations of the compressible layer will occur only in one dimension. Therefore, the theory of one-dimensional consolidation is implemented. The laboratory oedometer test results for the clayey layer undergoing consolidation indicate low values of preconsolidation stress, p'_c, compared to the effective overburden stress, thus it is considered that the clayey layer is normally consolidated. Apart from the total magnitude of consolidation settlement (ultimate consolidation settlement at time $t=\infty$), the rate of consolidation settlements is also of great significance in the geotechnical design of the embankment.

In order to determine whether the operation of the Railway Line will be affected by the magnitude of the remaining consolidation settlements, the Owner of the Project has specified an available time period equal to 14 months for the completion of the embankment's construction. In case the remaining settlements after the above time period are greater than 3.0 cm (maximum acceptable values decided by the owner of the project), the installation of vertical wick drains is required in order to increase the consolidation rate. As already mentioned, settlement calculations were carried out by taking into consideration a base improvement layer of 1.0 m thickness.

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 sheet used for the pertinent calculations ("5-Results of geotechnical calculations"). In order to obtain more precise results regarding both immediate and consolidation settlements, the examined layer is divided into sub-layers of small thickness. Based on the above mentioned methodology, settlements developed below the embankment axis as well as below the toe of the embankment are calculated. The detailed methodology and the relevant equations regarding the determination of (a) immediate settlements, (b) magnitude of consolidation settlement and (c) rate of consolidation settlement without wick drains and after the installation of wick drains, are presented in the supplementary material of the current paper. The results obtained from the Microsoft excel calculation sheet are verified by applying the provided equations in a solved example ("5-Results of geotechnical calculations").

By thoroughly presenting all the necessary data in the accompanying material, students have the opportunity to perform these calculations several times by changing each time the value of a particular geotechnical parameter. In this way, it becomes possible to investigate the effect of the experimental value variability of each geotechnical parameter on settlement calculation. The coefficient of consolidation is considered as a typical geotechnical parameter for this type of investigation, as it significantly affects the rate of consolidation settlements and thus the possible requirement for installation of vertical wick drains.

RESULTS OF GEOTECHNICAL CALCULATIONS OF THE EMBANKMENT

The results of the geotechnical calculations performed indicatively in the selected critical cross section of the embankment are presented in the following paragraphs. More particularly, results of slope stability analyses and settlement calculations are shown.

Results of Slope Stability Analyses

For the stability analyses according to limit equilibrium theory, Krey's methodology was applied by subdividing the slope into slices of constant width and by assuming circular slip surface (n_x, n_y: number of centers of slip circles in x and y direction, n: number of points on the constraint line, Larix-4S software).

Figures 6 and 7 present the results of slope stability analyses under static and seismic loading conditions using the limit equilibrium method. In these figures the geotechnical parameters of the encountered formations are also included. It should be mentioned that in the above analyses the undrained shear strength is used for the silty-clayey layer (Layer II), whereas only for calculation purposes a small value for the angle of internal friction φ=0.10 is also given as an input parameter. It is emphasized that the geotechnical parameters used for each examined case are different. As already mentioned, in slope stability analyses under static loading conditions the shear strength parameters of the encountered formations are reduced through the soil parameters partial factors (γ_M), whereas in the case of seismic loading conditions the charactestic values of soils parameters are used. Moreover the applied load of P=69.27 kPa, corresponding to the rail traffic load is further increased in case of static loading by multiplying it with a partial factor of actions equal to γ_F=1.30, thus resulting in P=90.0 kPa.

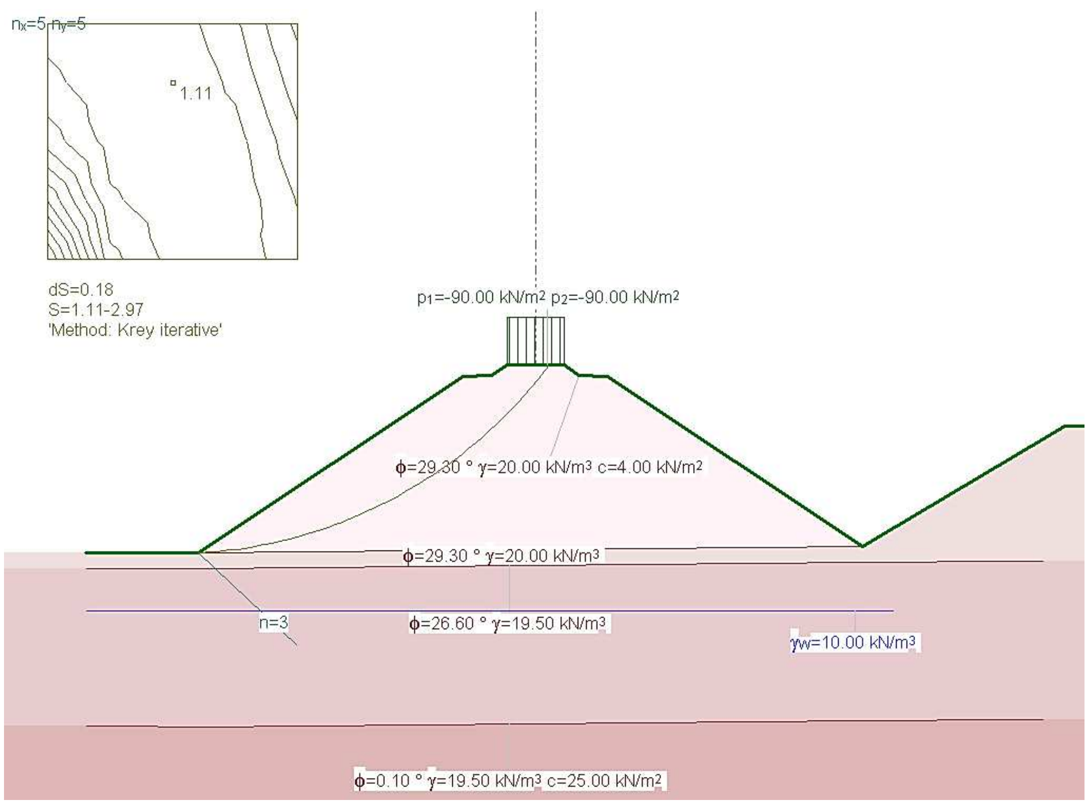

Figure 6. Results of slope stability analyses using limit equilibrium method (static conditions).

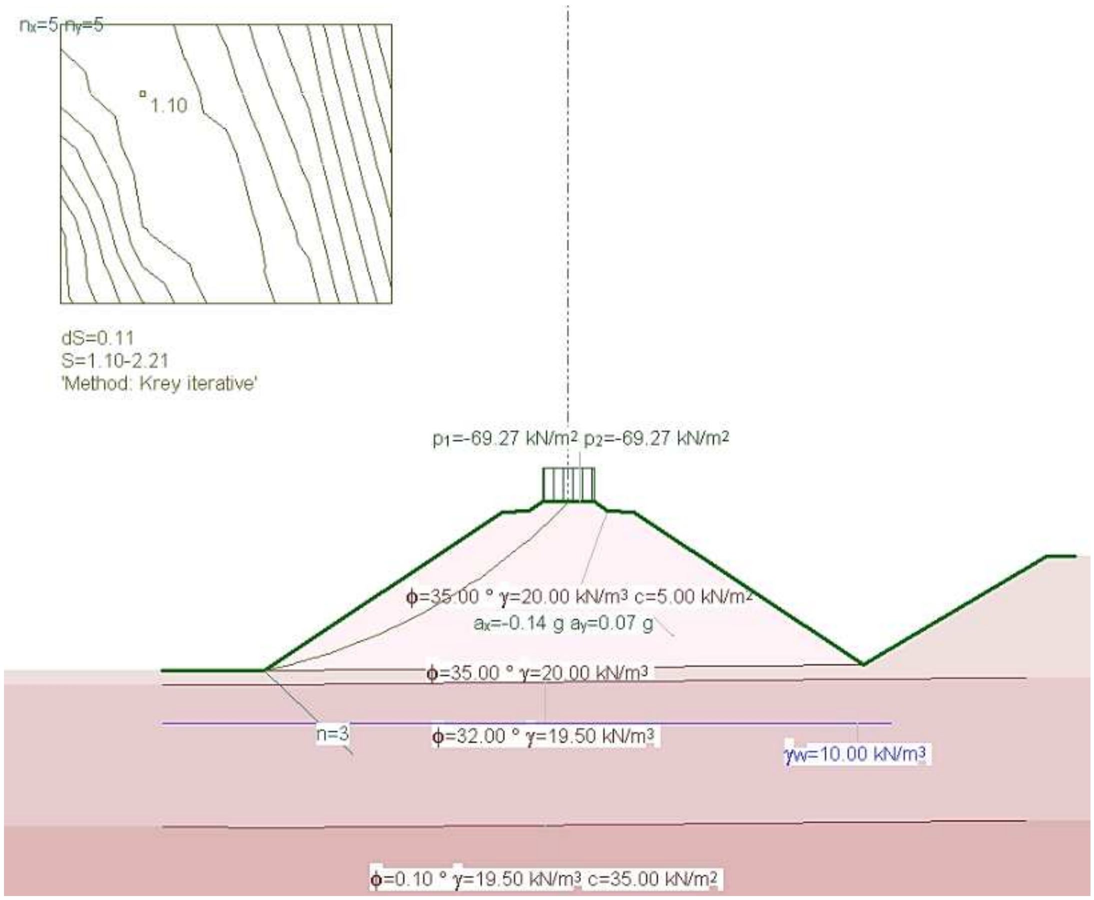

Figure 7. Results of slope stability analyses using limit equilibrium method (seismic conditions).

The results of slope stability analyses with the limit equilibrium method indicate acceptable safety factor values for both static and seismic loading conditions. By observing the results of the analyses depicted in Figures 6 and 7, it is concluded that the resulting safety factors correspond to slip circles passing through the main part of the embankment. Thus, the most unfavorable conditions correspond to internal embankment stability.

Results of Settlement Calculations

The geotechnical design of the embankment also encompasses the calculation of immediate settlements and consolidation settlements. The height of the embankment at the examined critical cross section (GG1201) is equal to 12.20 m, whereas the width at the crest and at the base of the embankment is equal to 7.0 m and 42.30 m, respectively. The ground water level is encountered at 4.0 m depth below ground surface. The geotechnical parameters of the soil formations, which are used in the calculations, are shown in the critical cross section of Figure 5. The results of the calculations for the settlements below the axis of the embankment for each separate layer are summarized in Table 5a, whereas detailed data regarding consolidation settlement of layer II are presented in Table 5b. In the diagram of Figure 8, the variation of settlements at different distances from the axis of the embankment is depicted.

In order to verify that the operation of the Railway Line will not be affected by the development of excessive settlements, the Owner of the Project has specified a maximum acceptable value for remaining settlements which is equal to 3.0 cm. Therefore, following the end of the construction period the long term settlements (due to consolidation) should not exceed 3.0 cm. The time period which is available for the completion of the embankment's construction is determined by the Owner of the project and is considered equal to 14 months. This is also considered as the time available for consolidation settlements to be developed. The settlements' calculations were carried out by taking into consideration a base agregate layer of 1.0 m thickness.

Table 5a. Results of settlement calculations.

Material/Layer (see cross section in Fig. 5)	Settlements (cm)	
	Immediate	Consolidation
Embankment	1.87	-
Leveling/improvement layer	0.49	-
Layer Ib	15.9	-
Layer II	-	31.5
Layer Ib	15.7	-
Total	1.9 + 32.1	31.5

Table 5b. Results of consolidation completion calculations at 14 months.

	Degree of consolidation U_{14m}%	Remaining consolidation settlement (cm)	Acceptable YES/NO
without drains	51.7	15.2	NO
with drains	96.8	1.0	YES

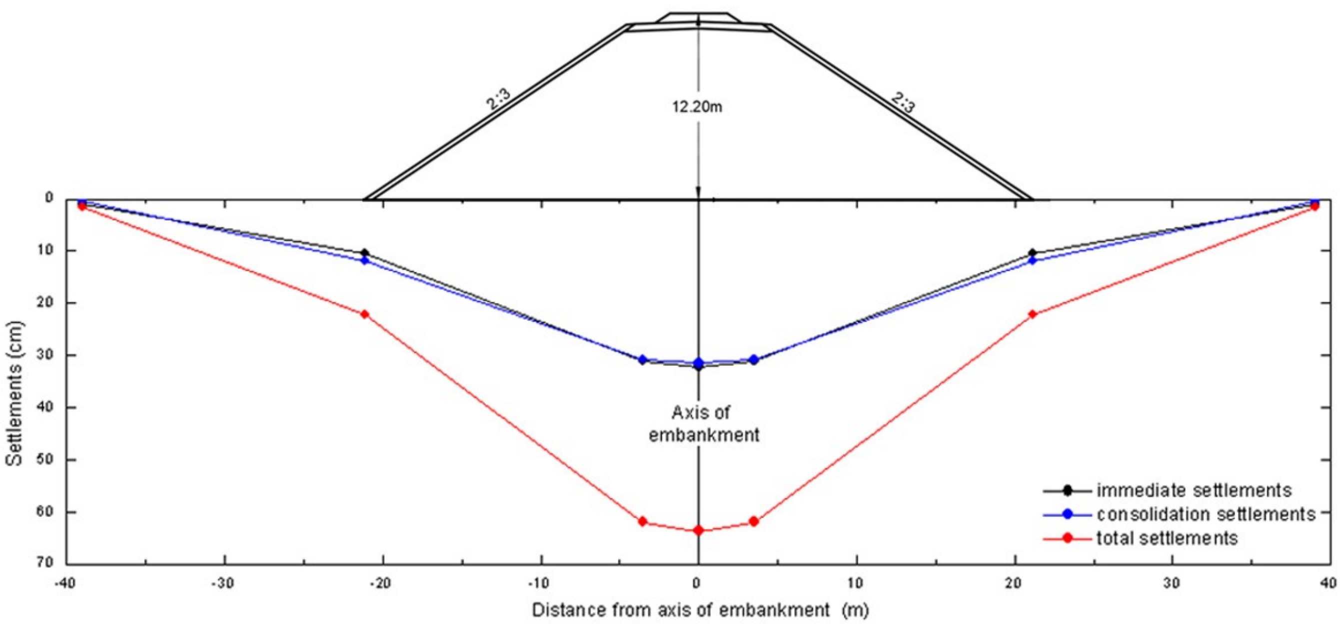

Figure 8. Results of settlement calculations due to the construction of the embankment.

The results of the calculations indicate that the value of remaining settlement after the defined time period of 14 months is equal to 15.2 cm. This is not acceptable because it exceeds the maximum acceptable value of 3.0 cm specified by the Owner of the project, hence, installation of vertical wick drains is required in order to increase the consolidation rate. Therefore, additional calculation settlements were performed by taking into account the installation of wick drains at a triangular grid of 3.0 m (distance between drains S=3.0 m). The radius of the wick drains, r_w, is equal to 0.033 m based on:

$$r_w = \frac{b+t}{\pi} \tag{6}$$

where b=0.10 m is the width of the wick drain and t=0.003 m is its thickness), which corresponds to an equivalent radius of a non-circular wick drain.

The use of the following vertical drains is indicatively suggested: polypropylene drains having width greater than 95 mm and minimum thickness of 3 mm, with discharge capacity q_w (250 kPa, i=0.1) >50x10^{-6} m³/sec and tensile strength >1 kN. The length of the wick drains should at least cover the thickness of the clayey layer subjected to consolidation settlements. The grid spacing was selected in such a way as to increase the consolidation rate and render the remaining settlements acceptable. Moreover, the selected grid spacing lies between the range (S=1.5 to 3.5 m) most commonly adopted for ground improvement. The results of the above calculations indicate a pronounced increase in the degree of consolidation. In particular, when vertical drains are installed, the degree of consolidation is increased from 51.7% to 96.8%. Thus, the remaining settlement after the 14-month period is significantly reduced to 1.0 cm, which is considered acceptable for the operation of the railway line.

By performing similar geotechnical calculations as these presented for the critical cross-section in the current paper in other representative cross-sections along the embankment, the integrated geotechnical design of the embankment is accomplished. In Figure 9 the geotechnical longitudinal section of the embankment is depicted with the proposed measurements including the installation of vertical wick drains at areas where consolidation rate should be increased.

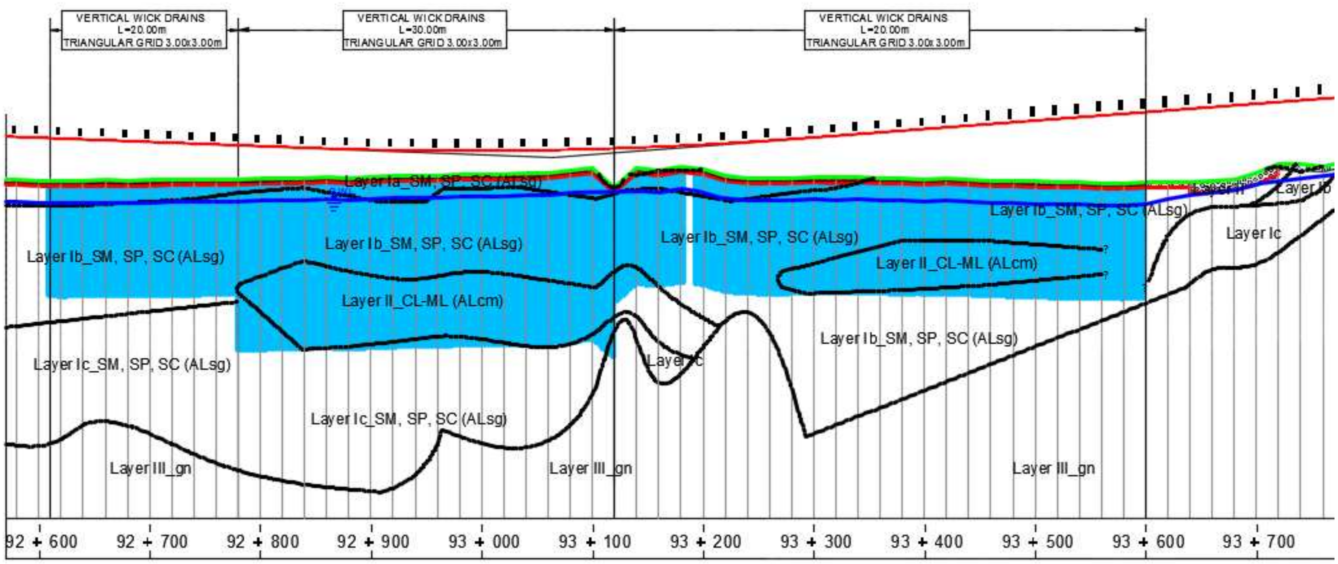

Figure 9. Longitudinal section of the embankment with the proposed installation of vertical drains.

CONSTRUCTION METHODOLOGY OF THE RAILWAY EMBANKMENT

Based on the results of the geotechnical calculations of the examined embankment, it is concluded that at certain areas, where the embankment is founded on alluvial deposits (AL), the installation of prefabricated vertical drains is required in order to increase the consolidation rate. Through the presented case study the students have the opportunity to get familiar with the complex construction methodology and the construction materials of railway embankments with vertical wick drains. Moreover, the students have the opportunity to become acquainted with fundamental data regarding the components of a railway embankment.

The construction sequence and the construction materials of the embankment are described in the following:

- Excavation. Excavation to the required level in order to remove any unsuitable surface material or vegetation.
- Stabilization-Improvement of excavation base. Placement of crushed material from rock excavation products for the stabilization of the excavation base, in order to facilitate machinery operation during compaction of the improvement layer. The stabilization layer is suggested to consist of crushed material of 10-30 cm grain size in combination with separation geotextile.
- Separation geotextile. Placement of non-woven separation geotextile.
- First drainage layer. It is required in order to drain out the groundwater collected through the installed vertical drains and lead it to the longitudinal side drains. It has 0.25 m thickness and consists of crushed materials.

- Installation of prefabricated vertical drains. The length and grid of the drains are determined based on the results of the performed geotechnical analyses.
- Second drainage layer.
- Construction of longitudinal side drains. The groundwater collected from vertical wick drains is led through side drains to adjacent hydraulic systems. The side drains consist of coarse grained materials inside which a perforated plastic drain pipe is installed.
- Placement of non-woven separation geotextile.
- Foundation of embankment. It includes (a) a base layer consisting of crushed material or rockfill free-draining material with maximum particle size equal to 10-15 cm b) drainage layer up to 50 cm above the original ground surface, in order to achieve dissipation of any excess pore water pressure inside the core of the embankment. The drainage layer consists of crushed material with maximum particle size equal to 3" (7.62 cm), fines content (passing sieve No 200) less than 10% and plasticity index (PI)≤4%.
- Core of embankment. Construction of embankment's core with suitable materials. For the construction of the main part of the embankment, materials classified at categories 1.3 to 1.5 or 2.1 to 2.3 or 3.1 and 3.2 of the UIC Code 719R are used.
- Prepared sub-grade layer. This layer consists of crushed materials of QS2 or QS3 categories (UIC Code 719R), whereas the thickness of this layer is determined according to UIC Code 719R.
- Blanket layer and ballast. It consists of well-graded coarse-grained (sands and gravels) materials (materials of QS2 or QS3 categories, UIC Code 719R). The thickness of the blanket layer varies from 37 cm to 42 cm, depending on the type of soil material used for the core of the embankment and the required transversal inclination of this layer. The ballast has minimum thickness of 0.30 – 0.50 m and consists of crushed material with maximum particle diameter equal to 2.0-6.0 cm.
- Soil covering with humus soil. On the slopes of the embankment.

The construction methodology and the construction materials of the embankment are presented in the typical cross section of Figure 10. Further details regarding compaction of the individual layers and the classification of soil materials according to UIC Code 719R are presented in the supplementary material accompanying the current paper ("6-construction methodology").

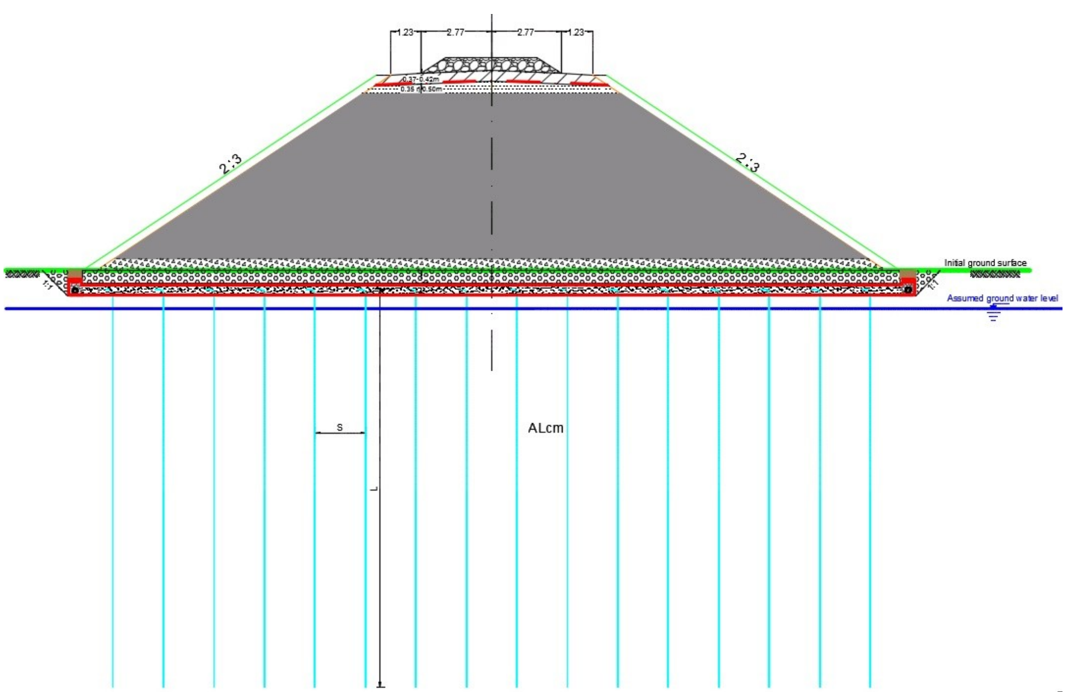

Figure 10. Typical cross section with the construction materials of the railway embankment.

LEARNING OUTCOMES OF THE EXAMINED CASE STUDY

The usefulness of the examined case study is highlighted through the achievement of a significant number of learning outcomes. More specifically the following learning outcomes are highlighted:

1. The identification of potential critical modes of failure in soil due to the construction of a railway embankment. Through the examined case study it is emphasized that the integrated geotechnical design of an embankment involves the execution of slope stability analyses in conjunction with calculations of soil settlements resulting from embankment construction.

2. The application of analytical methods (a) for the analyses of slopes against circular failure and b) for the calculation of immediate settlements as well as of consolidation settlements of load bearing soil layers.

3. The significance of selection of appropriate soil parameters. It is particularly emphasized that the soil strength parameters are crucial when performing slope stability analyses, whereas for implementing settlements' calculations the selection of soil compressibility parameters is significant.

4. The investigation of the variability of the experimental values of geotechnical parameters for the encountered soil formations and its effect on the results of geotechnical analyses as well as on the geotechnical design of the embankment.

5. Determination of the soil profile and the specific soil parameters used for the geotechnical design of the embankment based on the detailed evaluation of the available data from the geotechnical investigation performed in the area of interest.

CONCLUDING REMARKS

The presented case study refers to the geotechnical design of a railway embankment, which was implemented by Edafomichaniki S.A. within the framework of the project "Geotechnical Design of the New Railway Alignment between the Existing Line and Kavala's Port". The paper is compiled in such a way as to constitute a case study that is suitable for undergraduate instruction in the geotechnical engineering field and focuses on slope stability analyses and settlements' calculations of the embankment (both immediate and consolidation settlements).

The current paper is also accompanied by detailed case material which is available in electronic fromat. The supplementary material is necessary in order to assist (a) the instructors of undergraduate geotechnical courses, who would like to present in class the complete geotechnical design of an embankment and (b) to provide students with knowledge, comprehension and application capabilities regarding slope stability analyses and settlement calculations, after the completion of instruction.

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