

### **3 Geotechnical investigation and evaluation - Geotechnical parameters of formations** **(Relevant paragraph of the paper: GEOTECHNICAL EVALUATION AND GEOTECHNICAL DESIGN PARAMETERS)**

The soil stratigraphy along the examined embankment, presented in the geotechnical longitudinal section (filename: 3-8\_GEOTECHNICAL LONGITUDINAL SECTION.pdf), was determined based on the geological data and the laboratory test results of the evaluated boreholes. More particularly, the geotechnical evaluation of the investigation data was based on the following: a) the results of in situ and laboratory tests of sampling boreholes and b) the Geological Study and the technical-geological evaluation of boreholes.

The laboratory tests results regarding soil physical properties, soil mechanical properties and rock mechanics are summarized in the attached Tables 3-1, 3-2 and 3-3, respectively (Table 3-1\_LabTests\_SoilPhysicalProperties\_All Samples.pdf, Table 3-2\_LabTests\_Soil Mechanical Properties\_All Samples.pdf, Table 3-3\_LabTests\_RockMechanics\_All Samples.pdf).

The laboratory test results conducted on soil and rock specimens are statistically evaluated by determining the minimum (min), maximum (max) and average value as well as the standard deviation for each layer and laboratory test separately. The results of the statistical elaboration for each layer and laboratory test (physical properties, mechanical properties, rock mechanical properties) of the examined embankment are summarized in Tables 3-5, 3-6 and 3-7 (Table 3-5\_StatistAnalysis\_SoilPhysicalProperties\_Page1LayerI\_Page2LayerII.pdf, Table 3-6\_StatistAnalysis\_SoilMechanicalProperties\_Page1LayerI\_Page2LayerII.pdf, Table 3-7\_StatistAnalysis\_RockMechanics\_LayerIII.pdf).

Based on the above, the soil stratigraphy along the embankment from CH. 92+610 to CH. 93+870 includes the following layers:

- **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.
- **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.
- **Layer III:** Gneiss (gn). The formation is characterized as GNEISS of greenish-grey and white color, fractured, moderately weathered and locally weathered.

Based on the results of the statistical evaluation the following data have resulted:

**Layer I:** The percentage of gravels and sand ranges from 0% to 65.0% (average value 9.86%) and from 16.0% to 97.0% (average value 70.8%), respectively. The corresponding percentage of silt and clay ranges from 2.0% to 51.0% (average value 19.3%). The values of plasticity index (IP) of the layer ranges from NP to IP=30.0% (average value 2.77%), whereas the values of natural water (w) range from w=1.40% to 28.7% (average value 17.7%).

**Layer II:** The contained percentages of gravels, sand and fine material (silt and clay) for this layer range from 0% to 4.0% (average value 0.75%), from 2.0% to 46.0% (average value 24.17%) and from 54.0% to 98.0% (average 75.1%), respectively. Values of plasticity index

(IP) range from NP to IP=18.0% (average value 9.0%), whereas the values of natural water content (w) range from w=19.60% to 41.0% (average value 33.5%).

**Layer III:** According to the laboratory tests results it is found that the values of unconfined compressive strength of the formation ranges from  $q_{uc}=14.3\text{MPa}$  to  $q_{uc}=44.5\text{MPa}$  (average value  $q_{uc}=28.0\text{MPa}$ ), whereas the average value of the point load index strength is equal to  $I_{s50}=1.73\text{MPa}$  and  $I_{s50}=1.79\text{MPa}$  parallel and vertically to the direction of drilling, respectively. Based on the rock mass classification of the Geological Study, the geological strength index of gneiss varies from  $GSI=27-37$ , whereas  $m_i=9$ .

In table 3-4 the depth at which each formation is encountered in the evaluated boreholes is presented.

**Table 3-4:** Encountered formations in the evaluated boreholes of the embankment

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

The determination of the geotechnical design parameters of the encountered formations at the area of the embankment is given in detail in the following paragraphs. Geotechnical data from boreholes M8, M12, M16, A2 (executed for the current project) and data from boreholes G48, G50, GN-1 to GN-4 and G103 (executed for the adjacent project of Egnatia Highway) are co-evaluated.

For the determination of shear strength parameters (i.e. angle of internal friction and cohesion) laboratory test results from the following types of tests were taken into consideration (Greek Specifications for Soil Mechanics Laboratory Tests E105-86, 1986):

**Triaxial tests (ASTM D2850):**

- Unconsolidated-undrained triaxial tests (UU triaxial tests)
- Consolidated-drained triaxial tests (CD triaxial tests)

- c) Consolidated-undrained triaxial tests with pore pressure measurement (CUPP triaxial tests)

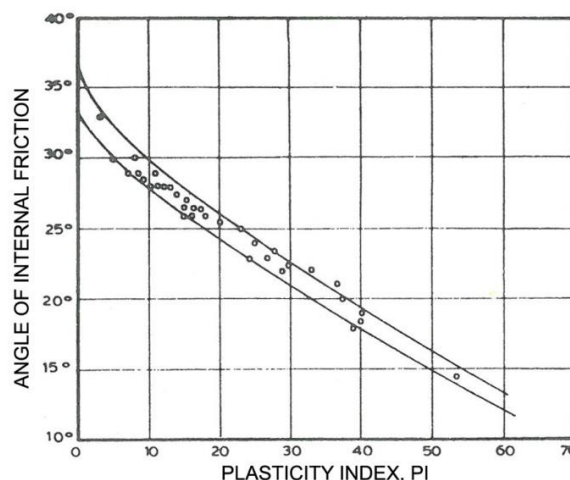
**Direct shear tests (ASTM D3080):**

- a) Unconsolidated-undrained direct shear tests (UU direct shear tests): Shearing is initiated immediately after the vertical load is imposed, therefore the tested sample is not consolidated.
- b) Consolidated-drained direct shear tests (CD direct shear tests): After primary consolidation is completed, shearing is initiated. The selected displacement rate is relatively slow in order to avoid the development of excess pore water pressure at failure (drained conditions).
- c) Consolidated-undrained direct shear tests (CU direct shear tests): After primary consolidation is completed shearing is initiated. The selected displacement rate is relatively quick resulting in the development of excess pore water pressure during shearing, thus inducing undrained conditions.

**Layer I: Sand and gravels-Alluvial deposits of sandy-gravelly composition (ALsg)**

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.

- Grain Size Distribution: According to Unified Soil Classification System (USCS): mainly SM-SP and SM, SC with intercalations of GM-GP and ML, CL
- SPT blow counts:  
 $N_{SPT}=9$  (7 to 11): ALsg of loose deposition - **Layer Ia**  
 $N_{SPT}=20$  (7 to 51): ALsg of medium density – **Layer Ib**  
 $N_{SPT}=56$  (14 to REFUSAL): ALsg (dense) – **Layer Ic**
- Bulk Density  
Wet:  $\gamma_w$  (kN/m<sup>3</sup>)=19.7 (from 17.5 to 22.0)
- Plasticity Index:  $I_p$ (%) =3 (from NP to 30)
- Angle of internal friction  $\phi'$  and cohesion  $c'$   
According to Scheidig:  $\phi'=\arctan(0.58-0.0045I_p)=29^\circ$



According to Sowers:

$\phi'=20+N/4=22^\circ$ : **Layer Ia**  
 $\phi'=20+N/4=25^\circ$ : **Layer Ib**  
 $\phi'=20+N/4=34^\circ$ : **Layer Ic**

From CU direct shear tests:  $\phi=39^\circ$ ,  $c=27.6\text{kPa}$  ( results from 17 tests)

$\phi_{\min}=33^\circ$  (SP-SM sample) to  $\phi_{\max}=45^\circ$  (SM sample)

$c_{\min}=5.0\text{kPa}$  (SP-SM sample) to  $c_{\max}=58.0\text{kPa}$  (SP sample)

From UU direct shear tests:  $\phi=37^\circ$ ,  $c=26.5\text{kPa}$  (results from 13 tests)

$\phi_{\min}=8^\circ$  (SC sample) to  $\phi_{\max}=45^\circ$  (SP-SM sample)

$c_{\min}=2.0\text{kPa}$  (GC sample) to  $c_{\max}=78.0\text{kPa}$  (SM sample)

From CUPP triaxial tests:  $\phi'=40^\circ$ ,  $c'=0\text{kPa}$  (results from 1 test on SM sample)

According to literature data (Soil Mechanics, Table 2, D. Valalas):

for SM soil material:  $\phi'=34\pm3^\circ$ ,  $c'=0\text{ kPa}$

for SP soil material:  $\phi'=36\pm6^\circ$ ,  $c'=0\text{ kPa}$

for SC soil material:  $\phi'=32\pm4^\circ$ ,  $c'=0\text{ kPa}$

for GM soil material:  $\phi'=36\pm4^\circ$ ,  $c'=0\text{ kPa}$

for GP soil material:  $\phi'=33\pm6^\circ$ ,  $c'=0\text{ kPa}$

By taking into consideration the proposed empirical correlations, the laboratory test results and the variation of  $N_{\text{SPT}}$  values with depth, the following values are suggested:

**Proposed design values:**

**Layer Ia** - ALsg of loose deposition:  $\phi'=29^\circ$ ,  $c'=0\text{kPa}$

**Layer Ib** - ALsg of medium density:  $\phi'=32^\circ$ ,  $c'=0\text{kPa}$

**Layer Ic** - ALsg (dense) :  $\phi'=35^\circ$ ,  $c'=0\text{kPa}$

- Compressibility Modulus  $E_s$

From results of oedometer tests for stress value corresponding to the overburden stress of each specimen (results from 5 tests):  $E_s=17620\text{kPa}$

From empirical correlations by literature:

**Layer Ia-ALsg of loose deposition ( $N=9$ )**

According to Tassios & Anagnostopoulos:  $E_s=700 (N+6)=10500\text{kPa}$  (for coarse sand with  $N<15$ )

According to Papadopoulos & Anagnostopoulos:  $E_s=7500+800N=14700\text{kPa}$  (for sand)

According to Papadopoulos & Anagnostopoulos:  $E_s=2600+690N=8810\text{kPa}$  (for silty sand)

According to Menzenbach:  $E_s=7200+490N=11610\text{kPa}$  (for saturated sands)

According to Bowles:  $E_s=3750+250N=6000\text{kPa}$  (for saturated sand)

According to Begemann:  $E_s=1000 (N+6)=15000\text{kPa}$  (for sand with gravels)

According to Menzenbach:  $E_s=3800+1050N=13250\text{kPa}$  (for sand with gravels)

According to Menzenbach:  $E_s=1200+580N=6420\text{kPa}$  (for silts and silty sands with  $PI<15\%$ )

**Layer Ib-ALsg of medium density ( $N=20$ ):**

According to Tassios & Anagnostopoulos:  $E_s=4000+700 (N-6)=13800\text{kPa}$  (for coarse sand with  $N>15$ )

According to Papadopoulos & Anagnostopoulos:  $E_s=7500 +800N=23500\text{kPa}$  (for sand)

According to Papadopoulos & Anagnostopoulos:  $E_s=2600+690N=16400\text{kPa}$  (for silty sand)

According to Menzenbach:  $E_s=7200+490N=17000\text{kPa}$  (for saturated sands)

According to Bowles:  $E_s=3750+250N=8750\text{kPa}$  (for saturated sand)

According to Begemann:  $E_s=4000+1000(N-6)=18000\text{kPa}$  (for sand and gravels)

According to Menzenbach:  $E_s=3800+1050N=24800\text{kPa}$  (for sand and gravels)

According to Menzenbach:  $E_s=1200+580N=12800\text{kPa}$  (for silts and silty sands with  $PI < 15\%$ )

**Layer Ic-ALsg (dense) (N=56):**

According to Tassios & Anagnostopoulos:  $E_s=4000+700(N-6)=39000\text{kPa}$  (for coarse sand with  $N > 15$ )

According to Papadopoulos & Anagnostopoulos:  $E_s=7500+800N=52300\text{kPa}$  (for sand)

According to Papadopoulos & Anagnostopoulos:  $E_s=2600+690N=41240\text{kPa}$  (for silty sand)

According to Menzenbach:  $E_s=7200+490N=34640\text{kPa}$  (for saturated sands)

According to Bowles:  $E_s=3750+250N=17750\text{kPa}$  (for saturated sand)

According to Begemann:  $E_s=4000+1000(N-6)=54000\text{kPa}$  (for sand with gravels)

According to Menzenbach:  $E_s=3800+1050N=62600\text{kPa}$  (for sand with gravels)

According to Menzenbach:  $E_s=1200+580N=33680\text{kPa}$  (for silts and silty sands with  $PI < 15\%$ )

**Proposed design values:**

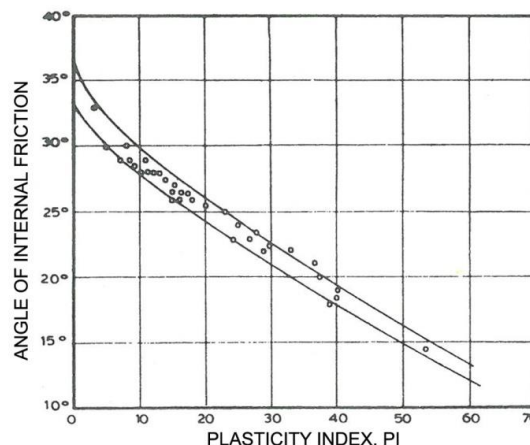
**Layer Ia-ALsg of loose deposition:  $E_s=7.0\text{MPa}$**

**Layer Ib-ALsg of medium density:  $E_s=14.0\text{MPa}$**

**Layer Ic-ALsg (dense):  $E_s=35.0\text{MPa}$**

**Layer II: Silt and clay- Alluvial deposits of silty-clayey composition (ALcm)**

- Grain Size distribution: According to USCS: mainly ML and CL
- SPT blow counts:  $N_{\text{SPT}}=12$  (6 to 19)
- Bulk Density  
Wet:  $\gamma_w (\text{kN/m}^3)=19.6$  (from 18.7 to 20.3)
- Plasticity index:  $I_p(\%)=9$  (from NP to 18)
- Angle of internal friction  $\phi'$  and cohesion  $c'$   
According to Scheidig:  $\phi'=\arctan(0.58-0.0045I_p)=28^\circ$



According to Sowers:  $\phi' = 20 + N/4 = 23^\circ$

From CU direct shear tests:  $\phi = 42^\circ$ ,  $c = 15.0 \text{ kPa}$  (results from 2 tests)

$\phi_{\min} = 39^\circ$  (ML sample) to  $\phi_{\max} = 45^\circ$  (CL sample)

$c_{\min} = c_{\max} = 15.0 \text{ kPa}$  (CL & ML samples)

From UU direct shear tests:  $\phi = 50^\circ$ ,  $c = 0 \text{ kPa}$  (results from 1 test)

From CUPP triaxial tests:  $\phi' = 31^\circ$ ,  $c' = 3.0 \text{ kPa}$  (results from 2 tests)

$\phi_{\min} = 29^\circ$  (CL sample) to  $\phi_{\max} = 33^\circ$  (ML sample)

$c_{\min} = 2.0 \text{ kPa}$  (CL sample) to  $c_{\max} = 4.0 \text{ kPa}$  (ML sample)

According to literature data (Soil Mechanics, Table 2, D. Valalas):

for ML soil material:  $\phi' = 33 \pm 4^\circ$ ,  $c' = 0 \text{ kPa}$

for CL soil material:  $\phi' = 27 \pm 4^\circ$ ,  $c' = 20 \pm 10 \text{ kPa}$

Based on the above the following values are proposed:

**Proposed design values:  $\phi' = 28^\circ$ ,  $c' = 5.0 \text{ kPa}$**

These design values have resulted by mainly evaluating the results of laboratory tests and by also taking into consideration the empirical correlations. The results of CUPP triaxial tests are considered as most reliable for the determination of effective shear strength parameters, whereas the number of performed tests and the classification of each tested sample are also evaluated.

- Undrained shear strength

From unconfined compression tests:  $q_{u(\text{average})} = 90.5 \text{ kPa}$ , therefore  $c_{u(\text{average})} = 45.3 \text{ kPa}$

$q_{u\min} = 38 \text{ kPa}$

$q_{u\max} = 143 \text{ kPa}$  (ML sample)

From UU triaxial tests:  $c_u = 28.0 \text{ kPa}$  (results from 3 tests)

$c_{u\min} = 16.0 \text{ kPa}$  (CL sample) to  $c_{u\max} = 47.0 \text{ kPa}$  (ML sample)

**Proposed design values:  $c_u = 35.0 \text{ kPa}$**

- Compressibility Modulus  $E_s$

From results of oedometer tests for stress value corresponding to the overburden stress of each specimen (results from 5 tests):  $E_s = 7500 \text{ kPa}$

From empirical correlations by literature:

According to Menzenbach:  $E_s = 1200 + 580N = 8160 \text{ kPa}$  (for silt and silty sand,  $PI < 15\%$ )

According to Begemann:  $E_s = 1800 + 300N = 5400 \text{ kPa}$  (silt with sand,  $N < 15$ )

According to Papadopoulos & Anagnostopoulos:  $E_s = 3200 + 490N = 9080 \text{ kPa}$  (sandy silt)

**Proposed design values:  $E_s = 8.0 \text{ MPa}$**

- Data from consolidation tests

Compressibility index:  $C_c = 0.22$  (from 0.09 to 0.38)

Initial void ratio:  $e_o = 0.90$  (from 0.64 to 1.10)

Coefficient of consolidation:  $C_v = 4.54 \text{ m}^2/\text{year}$  (from 2.89 to 6.47  $\text{m}^2/\text{year}$ )

Preconsolidation stress,  $p_c$ : low values (it is considered as normally consolidated clay)

The proposed design values for a) compressibility index,  $C_c$ , b) initial void ratio,  $e_o$ , and c) coefficient of consolidation,  $C_v$ , are the average values from laboratory test results (from Page 2 of Table3-6\_StatistAnalysis\_Soil MechanicalProperties\_Page1LayerI\_Page2LayerII).

### Layer III: Gneiss (gn)

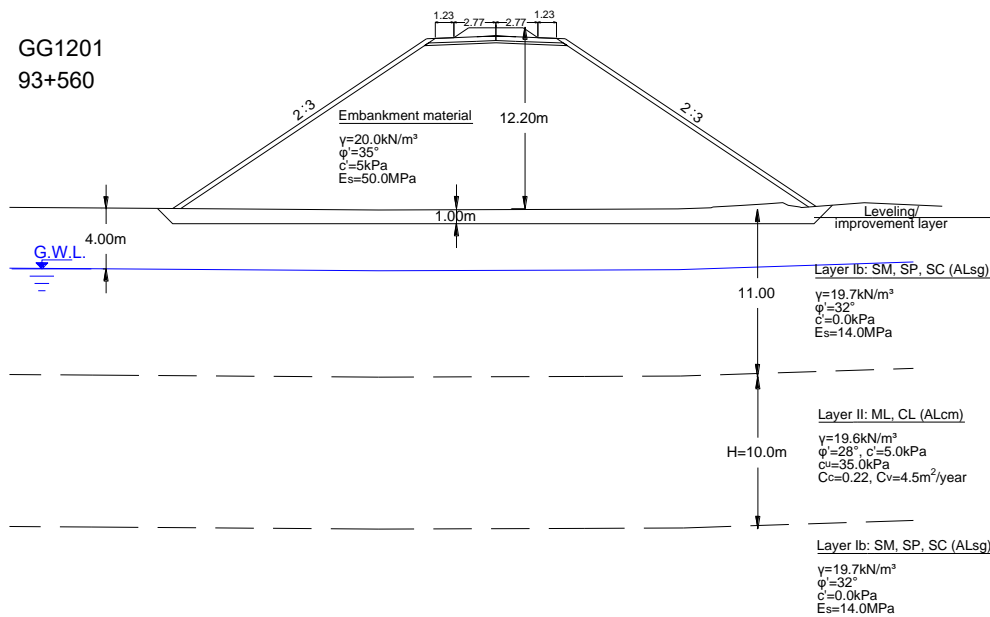
At the area of the examined embankment the bedrock is encountered at great depth from the ground surface and it is assumed that it does not affect the foundation of the embankment. Therefore the determination of the geotechnical parameters for this layer is not required.

The design geotechnical parameters of the encountered soil formations, used in the geotechnical calculations of the embankment, are summarized in the recapitulative Table 3-8. In this table the reduced values of the geotechnical parameters, which are used in slope stability analyses under static loading conditions according to Eurocode 7.1 (Design approach 3, DA-3), are also included.

The critical cross section of the embankment used in the geotechnical calculations is presented in Figure 3-1.

**Table 3-8.** Geotechnical design parameters of encountered formations

Layer	Geotechnical design parameters (characteristic values)					Geotechnical design parameters (stability analyses according to Eurocode 7.1)			
	$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (o)	$c'$ (kPa)	$E_s$ (MPa)	$C_u$ (kPa)	$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (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	-



**Figure 3-1.** Critical cross-section used in the geotechnical calculations

## Seismicity Data

According to Eurocode 8 the examined embankment passes through seismic **Zone Z1**.

The design ground acceleration,  $\alpha_{gR}$ , on type A ground for seismic Zone Z1 is equal to  $\alpha_{gR}/g=0.16$ . This value is further increased for the other types of ground, by multiplying it with soil parameter  $S>1.0$  (Eurocode 8). According to the "Final Geological Study – Seismicity Study" *"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 taken increased by 25%-according to the Greek Code for Seismic Resistant Structures (EAK 2000)."* Therefore the design ground acceleration on type ground A is increased and taken equal to  $\alpha_{gR}/g=0.20$ .

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 following ground types (Table 4-9):

**Table 3-9.** Ground types according to Eurocode 8 (Part 1)

EARTHWORK	FORMATION	DESCRIPTION	GROUND TYPE (EUROCODE 8)	GROUND TYPE DESCRIPTION
Embankment from CH. 92+610 to CH. 93+870	Ia	Sand and gravels (loose) (Alluvial deposits - ALsg)	D	Deposits of loose-to-medium cohesionless 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

## Accompanying files

3-1. RECAPITULATIVE TABLE OF RESULTS OF LABORATORY TESTS - SOIL PHYSICAL PROPERTIES

Filename: **3-1\_Table 3-1\_LabTests\_SoilPhysicalProperties\_All\_Samples.pdf**

3-2. RECAPITULATIVE TABLE OF RESULTS OF LABORATORY TESTS - SOIL MECHANICAL PROPERTIES

Filename: **3-2\_Table 3-2\_LabTests\_SoilMechanicalProperties\_All\_Samples.pdf**

3-3. RECAPITULATIVE TABLE OF RESULTS OF LABORATORY TESTS - ROCK MECHANICS

Filename: **3-3\_Table 3-3\_LabTests\_RockMechanics\_All\_Samples.pdf**

3-5. RECAPITULATIVE TABLE OF RESULTS OF STATISTICAL ANALYSIS OF LABORATORY TESTS - SOIL PHYSICAL PROPERTIES

Filename:

**3-5\_Table3-5\_StatistAnalysis\_SoilPhysicalProperties\_Page1LayerI\_Page2LayerII.pdf**



3-6. RECAPITULATIVE TABLE OF RESULTS OF STATISTICAL ANALYSIS OF LABORATORY TESTS - SOIL MECHANICAL PROPERTIES

Filename:

**3-6\_Table3-6\_StatistAnalysis\_SoilMechanicalProperties\_Page1LayerI\_Page2LayerII.pdf**

3-7. RECAPITULATIVE TABLE OF RESULTS OF STATISTICAL ANALYSIS OF LABORATORY TESTS - ROCK MECHANICS

Filename: **3-7\_Table 3-7\_StatistAnalysis\_RockMechanics\_LayerIII.pdf**

3-8. GEOTECHNICAL LONGITUDINAL SECTION.pdf

Filename: **3-8\_GEOTECHNICAL LONGITUDINAL SECTION.pdf**

### **List of symbols**

c: cohesion  
 $C_c$  : compressibility index  
 $c_u$ : undrained shear strength  
 $C_v$ : coefficient of consolidation  
 $E_s$ : compressibility modulus  
 $e_o$ : initial void ratio  
 $\gamma$ : bulk density  
 $\phi$ : angle of internal friction

### **Relevant Reference**

- Begemann, H. (1974). "General report for central and western Europe." Proc. of the European Symp. on Penetration Testing (ESOPT), Stockholm, Sweden, 5-7 June 1974, 29-39.
- Bowles, J.E. (1996). "Foundation analysis and design." McGraw-Hill Ed. 1207p.
- EN 1998-5:2005 "Eurocode 8: Design of Structures for Earthquake Resistance – Part 5: Foundations, Retaining Structures and Geotechnical Aspects" and Greek National Annex to "EN 1998-5:2005 Eurocode 8".
- Menzenbach, E. (1959). "Die Anwendbarkeit von Sonden zur Prüfung der Festigkeitseigenschaften des Baugrundes." Forschungsberichte des Landes Nordrhein – Westfalen. No. 173, Westdeutscher Verlag, Koln.
- Papadopoulos, B.P., and Anagnostopoulos, A.G. (1987). "Groundwater effects on settlements estimation." Proc., 9<sup>th</sup> European Conf. on Soil Mechanics and Foundation Engineering, Dublin, 31 August – 4 September 1987, Vol. 2, 725-729.
- Scheidig, A. (1934). "Der Löss und seine geotechnischen Eigenschaften." Leipzig, as mentioned in: Loizos, A.A. (1977). "Soil Mechanics – Foundation. Part I: The foundation soil". Athens (in Greek).
- Sowers, G.B., Sowers, G.F. (1979). "Introductory soil mechanics and foundations: Geotechnical Engineering", Macmillan Ed.
- Tassios, T.P., and Anagnostopoulos, A.G. (1974). "Penetration testing in Greece." State of the Art Report. Proc. of the European Symp. on Penetration Testing (ESOPT), Stockholm, Sweden, 5-7 June 1974, Vol.1, 65-68.
- Valalas, D. (1996). "Soil mechanics". Kiriakidis Ed., 323p (in Greek).