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FIELD AND LABORATORY TESTS IN SEMAN DEPOSITS

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ABSTRACT

The construction of the big energetic park near the spilling of the river needed a geotechnical study to be performed by thoroughly investigations of the sand's deposits. We would like to present in these papers all the field investigations, laboratory tests and geotechnical study carried out by "ALTEA&GEOSTUDIO2000" laboratory in order to identify the behavior of soils under static and seismic loads. It is possible to create some geotechnical models based on this study and to determine their behavior under static and seismic conditions.

INTRODUCTION

The great economic development of a country, demands first of all for the energy sources development. This is why, the construction of different energetic parks is planed in Albania: the energetic park of Durres, Vlora, Seman etc near the Adriatic sea. Seman. In the spilling of the Seman's river, spreads the area planed for the construction of a big energetic park. Big gas deposits, roads infrastructure and necessary installations are part of it. "ALTEA&GEOSTUDIO2000" l.t.d laboratory carried out a full geological study, many in – situ soil tests and laboratory tests in order to determine the physical and mechanical properties of the soils in the area where the energetic park was built. As a result, a thorough geotechnical study was prepared which predicted some of the dangerous phenomenon that could occur in that area, characteristic phenomenon and giving the respective recommendations.

THE GEOLOGICAL STUDY

The geological study was carried out by drilling 12 boreholes (BH) of 30 m depth (10 BH) and 80m depth (2BH) Fig.1 (The plan of geological works)

Geomorphology

Geomorphology presents a flat zone with alluvial, maritime and marshy deposits. They have a thickness of more than 100 m in the peripheric parts and 250 m in the center of the area of study. The most characteristic geodynamic phenomenon in this area is the consolidation of the marsh deposits during a long time and the stabilization process of soils under seismic loads (these areas have M=6-6.2).

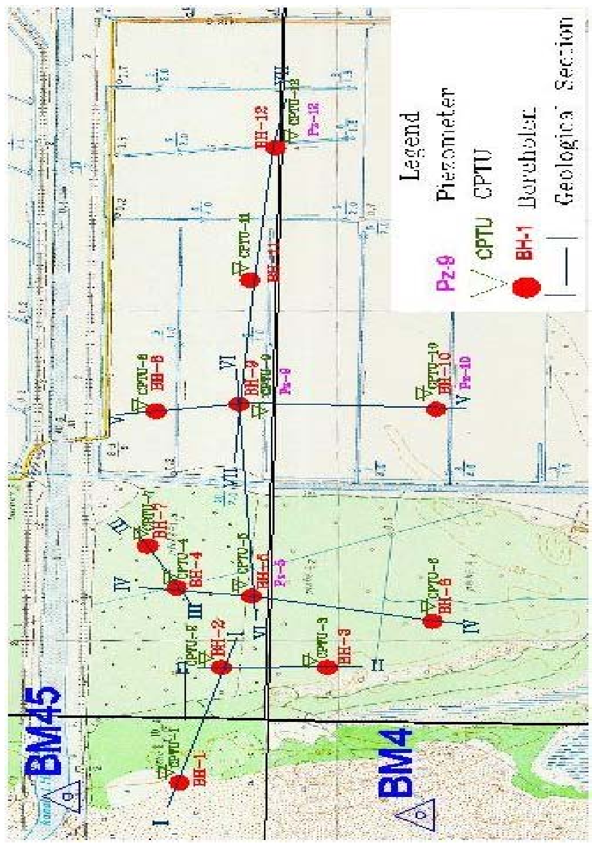

Fig. 1 Locations of the testing bore holes (BH)

Geological and hydro-geological structure

The marsh of Hoxhara, where the energetic park will be constructed, is part of the west depression of Albania where Neogen's and Quaternary's deposits are present. This area represents a deep hole of tectonic origin which during the Quaternary has been filled by swampy deposits. Marshy deposits are combined in here with maritime deposits. The Quaternary deposits are small sized gravels, sands, silty sands, silty clays, clays, peat and organic matters. Neogene's deposits consisting of Mudstone and Sandstone, weathered on top of them are met below the Quaternary deposits. According to the studies made in the Marsh area of Hoxhara, it results that the level of the underground water is almost equal either in winter or in summer: (.5÷1.5) from the ground surface. The chemical analysis show that these are salted waters and aggressive against iron and concrete.

The encountered layers.

We have encountered about 15 layers by drilling the 12 boreholes which by further elaboration of the samples are grouped into 7 characteristic layers. During the field works, SPT and CPTU tests have been performed in the Boreholes. You can see the borehole columns in Fig. 2

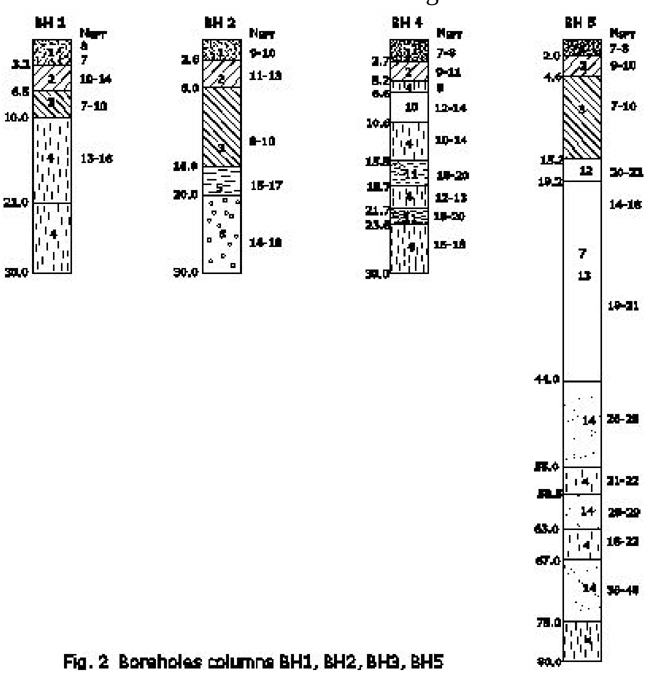

Fig. 2 Boreholes columns BH1, BH2, BH3, BH5

In the same time, we have determined the geological profiles from I-I to VII-VII (see Fig. 1). Based on the geological profiles presented in Fig.3 , we deduce that we have to deal with a very heterogeneous and complicated geology.

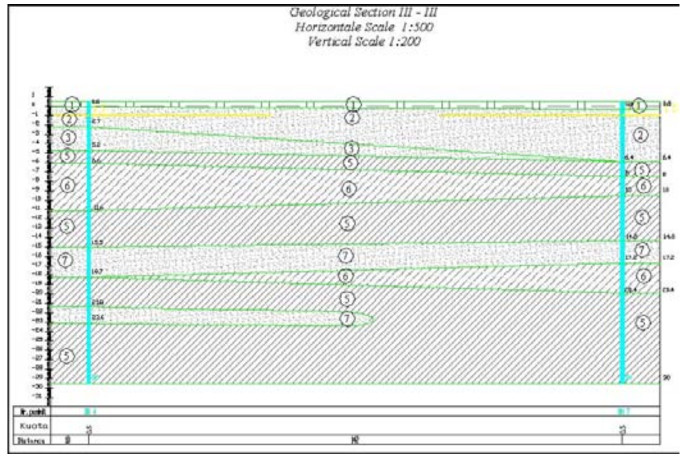

Fig. 3 The Geological Section III-III

The generalized Geotechnical Models

According to the descriptions of layers in the 12 boreholes and based also on the Nspt values, we can create four generalized geotechnical models (Fig.4) and we have evidences of the presence of the 7 following layers:

1- Loose fine beige sand containing organic matters.
2- Loose to medium dense green to gray fine to medium sand.
3- Soft green to gray silty sand + silty clay containing organic matters.
4- Soft to firm green-gray clayey silts or silty clays containing organic matters.
5- Medium dense green to gray sand + stratum of silty clay.
6- Loose to medium dense silty sands to sandy silts.
7- Soft to firm green to gray silty clay +sands.
8-

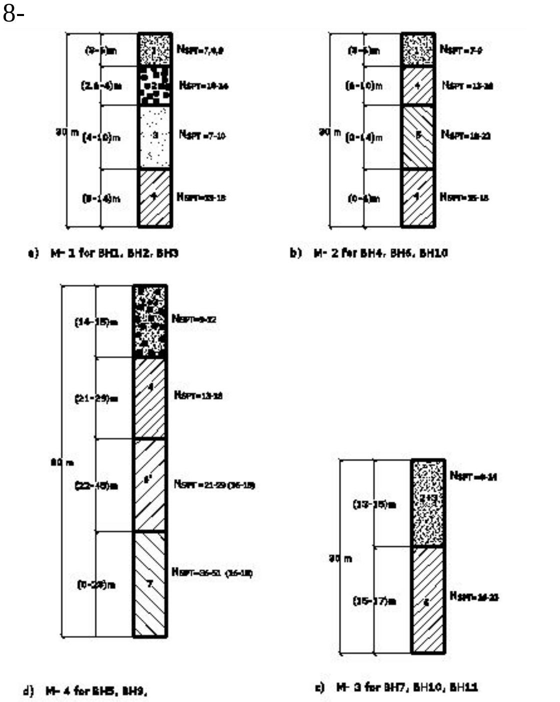

Fig. 4 Generalized Geotechnical Models

IN SITU TESTS

We have carried out three different types of tests:

Standard penetration tests SPT in all the boreholes

Based on these tests we have evidenced four generalized geotechnical models. The relation Nspt-Depth for these models is presented in Fig.5.

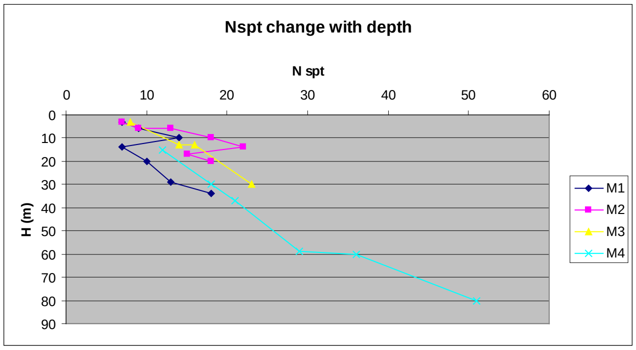

Fig. 5 The change of N_{SPT} in relation to the depth

According to the Nspt tests results, we conclude that:

- Up to 10 m of depth, Nspt=7÷14. The deposit is very loose.
- (10-20)m of depth, for the models M1 and M2, the soil properties worsen : Nspt=7÷10 while for the other models the soil properties are ameliorated : Nspt=18÷20.
- Below 20 m of depth, the soil properties for all the models are ameliorated.
- In the interval (60-75)m, Nspt=25÷43. These Nspt values testify that the soil properties are good.

Measurement of Soil Resistivity on Site for 1m,3m, up to (6÷10)m of depth.

Resistivity is defined as the electrical resistance of a unit volume of a material. Earth resistivity is measured by the Wenner four electrodes method, using a Megger Earth Tester (according to ASTM G 57-58 standard test method).The selected locations for performing the Soil Resistivity Tests are shown in Fig.6

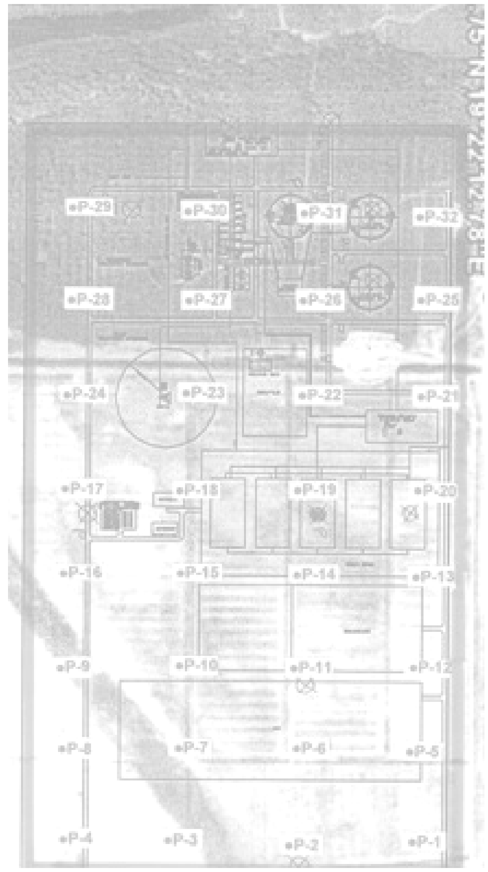

Fig 6 The 32 locations of the soil resistivity measurement

We can see the scheme of these measurements in Fig.7.

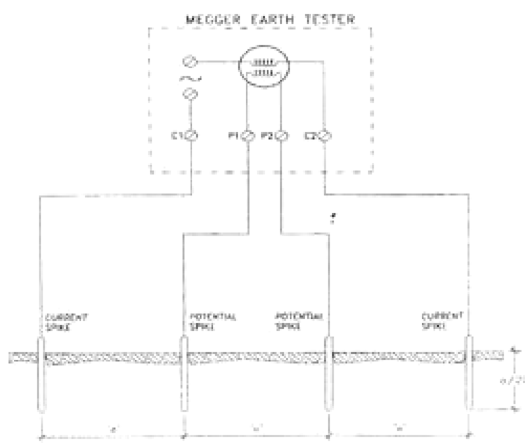

Fig. 7 Soil Sensitivity Measurement Scheme

Based on the measurements , we can calculate the electric resistivity using the formula:

$\rho = 2\pi * a * R$ (ohm*m) where:

a – Space between spikes

R – Resistance reading

ρ - Soil Resistivity

The test results are summarized in the following table:

Table A

Points	Electric Resistivty ρ (ohm*m)
1, 2, 3, 5, 6, 7, 11, 12	2 ÷ 3.5
4, 8, 9, 10	9 ÷ 12
13, 14, 20	1.2 ÷ 1.9
15, 16, 17, 18, 19	4 ÷ 17
21, 22, 23, 24, 25	2.5 ÷ 10
26, 27, 28, 29, 30	20 ÷ 128
31, 32	5 ÷ 33

According to the test results, the study area can be separated in three zones:

- The first zone lies between the points 26,27,28,29,30 where the soil resistivity has the biggest values. This fact is in accordance to the layers met in the boreholes BH1, BH2, BH3, BH4, BH5 and BH6 (Nspt = 7 ÷ 11).
- The second zone lies between the points 21, 22, 29, 25 where the values of soil resistivity are ρ = 2.5 ÷ 10 in accordance to the layers met in the boreholes BH3, BH9, BH10 (Nspt = 8 ÷ 14).
- The third zone lies between the points 1,2,3,4,5,6,7,11,12 where the soil resistivity has the lowest values ρ = 2 ÷ 3.5 in accordance to the layers met in the boreholes BH11, BH12 (Nspt = 13 ÷ 18).

We have also noticed a correspondence between the zones mentioned above and the four geotechnical models as following:

For the first zone are acceptable the geotechnical models M-1 and M-2; for the second zone
is acceptable the geotechnical model M-3 and for the third zone is acceptable the geotechnical model M-4.

Cone Penetration test CPTU

The CPTU tests are performed up to 25 m of depth in all the drilled boreholes.

q_c - Resistance in the cone apex (MPa)
f_s- Friction resistance (MPa)
U - Pore pressure (MPa)

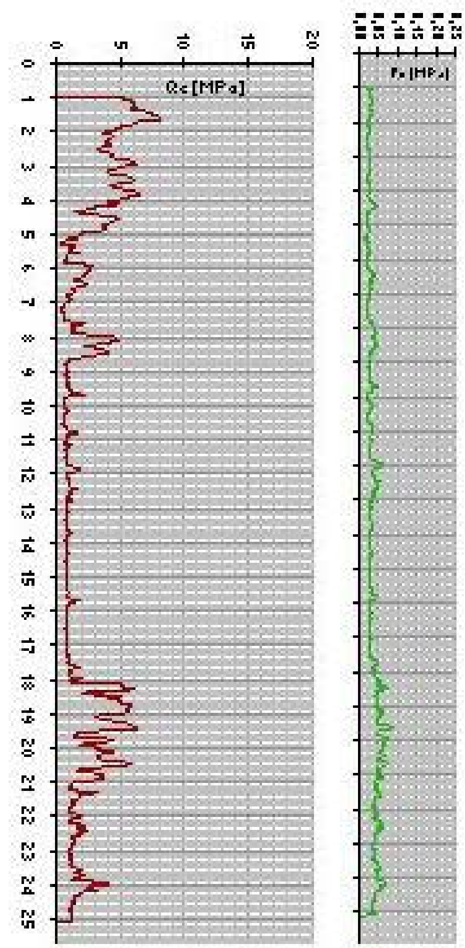

Fig. 8a-I CPTU test Records for the BH-1

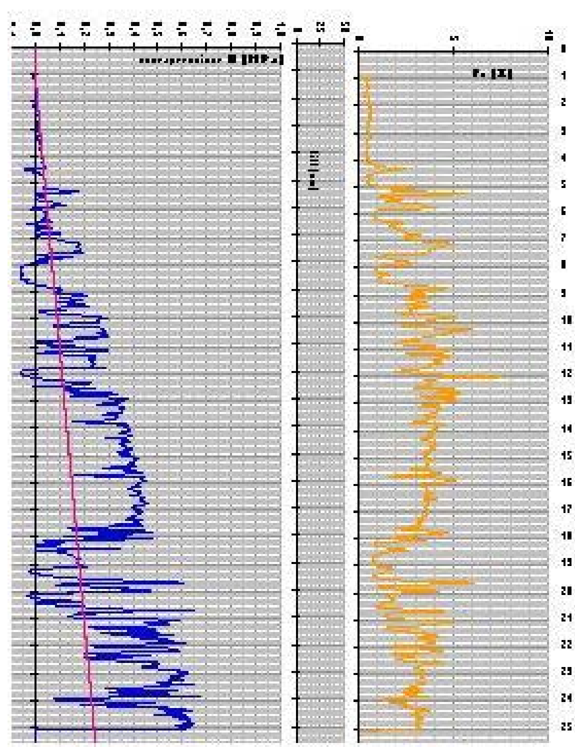

Fig. 8a-II CPTU test Records for the BH-1

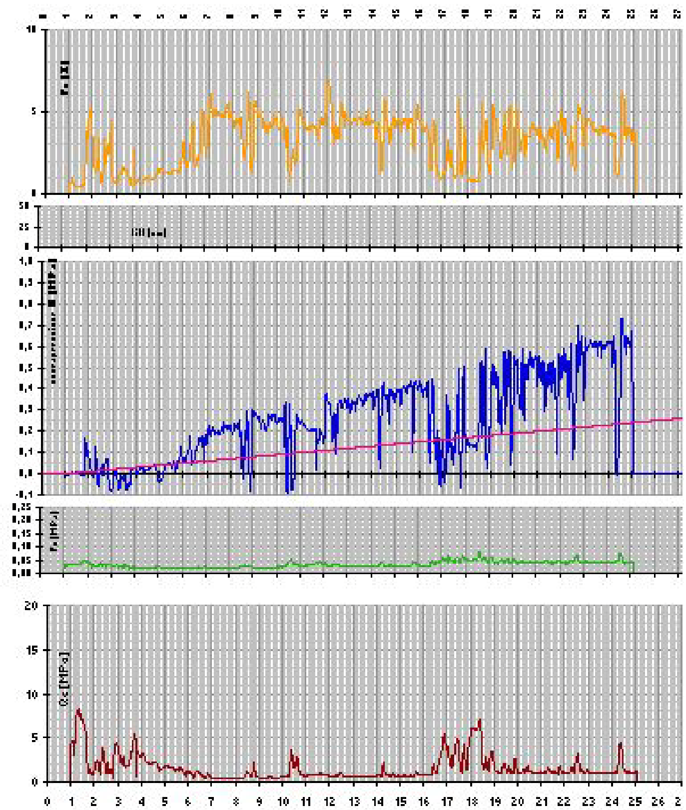

Fig.8b CPTU test records for BH-2

After the measurements we can calculate the Friction Ratio:

$$R_f = \frac{f_s}{q_c} * 100$$

and the Indices of Friction: $I_f = \frac{1}{R_f}$

According to the measurements, we can distinguish some characteristic zones with their respective depths for the geotechnical model M1 (table 1).

Table 1

Depth (m)	q_c (MPa)	f_s (MPa)	U (MPa)
3 ÷ 5	6 ÷ 8	0.02 ÷ 0.03	0.05
5 ÷ 8	1.5 ÷ 5	0.02 ÷ 0.03	0.1 ÷ 0.2
8 ÷ 17	0.08 ÷ 2	0.03 ÷ 0.04	0.3 ÷ 0.4
17 ÷ 25	6 ÷ 9	0.04 ÷ 0.08	0.7 ÷ 0.9

Based on the records taken in all the 12 boreholes we can conclude:

- The friction resistance is nearly constant in almost all the characteristic zones $f_s = (0.02 \div 0.05)$ MPa except for the interval (15 ÷ 17) m depth in borehole BH 5 where its value is $f_s = 0.15$ MPa.
- Based on the test records and test results for the Resistance q_c , Pore Pressure U and Friction Ratio R_f we can divide the study area in four characteristic zones (Table 2):

Table 2

Depth (m)	q_c (MPa)	U (MPa)	R_f (%)
3 ÷ 6	3 ÷ 8	-0.01 ÷ 0.01	0.6 ÷ 1.2
6 ÷ 15	1 ÷ 3	0.01 ÷ 0.04	3 ÷ 4
15 ÷ 17	(7 ÷ 9) or (7 ÷ 10)	-0.08	1 ÷ 3
17 ÷ 25	1 ÷ 3	0.06 ÷ 0.07	4 ÷ 6

Conclusions from the in-situ tests

According to the Nspt, CPTU and Soil Resistivity tests' results we can conclude that:

- There is a good accordance between Nspt, CPTU tests and Electric Resistivity of the soils.
- The division of the study area in four generalized geotechnical models is relatively exact.
- There are weak, unconsolidated soils (U>0) up to (25-30)m of depth.
- The friction force for the pile foundations will be small.
- There is a good accordance between the "q_c" values and soil classification, Dr, φ, Em, Cu determined by laboratory tests.
- After the elaboration of data of the in-situ tests for the geotechnical models M-1 and M-2 we have determined the mechanical properties of different layers as shown in table 3.

Table 3

Layer	1	2	3	4
Thickness (m)	5	4	7	14
Nspt	7 ÷9	10 ÷ 14	7 ÷10	13÷18
q_c (MPa)	4 ÷7	1.5÷ 5	0.08÷2	3 ÷6
f_s (MPa)	0.02÷0.03	0.01 ÷ 0.02	0.03÷0.04	0.03÷0.05
U (MPa)	0.05	0.1÷0.2	0.3÷0.4	0.2÷0.6
R_f %	0.8÷1	2÷4	3÷5	1÷5
Classification	Fine sand	Fine to Medium sand+silt	Silty sand	Silty clay
Dr	Loose	Loose	Very loose	-
Em (MPa)	10 ÷20	10 ÷20	10 ÷20	-
Cu (KPa)	-	-	-	37 ÷40
Φ (˚)	26	28	27	-

- There is a good accordance between the three zones determined by the electric resistivity tests and the geotechnical models M-1, M-2, M-3 and M-4.

LABORATORY TESTS

There was a great number of soil samples (disturbed and undisturbed) taken from all the layers encountered in 12 drilled boreholes.

The laboratory tests performed on these samples are as following:

- Grain size distribution,
- Plasticity Limits LL, PL and PI, liquidity index $I_L = \dfrac{w - PL}{PI}$
- Moisture content w, unit weight γ, specific gravity Gs.
- coefficient of permeability k, modulus of compression E, coefficient of consolidation Cv and shear strength of soil φ, c.

We have plotted a graph by the grain size distribution tests data (table 4, Fig.9) . Based on this graph we conclude that:

- The dominant fractions for the layers of type 3,4,5,6,7 in the study area are *silt* and *fine sand* (60 ÷72)%.
- Then come *fine to medium sands* (64 ÷66)% for the layers of type 1, 2.
- Clay particles in the end with a percentage of <30 %.

Table 4

Layer	Particles Percentages			
	< 0.005	< 0.002	< 0.075	< 0.63
1	11.40	4.60	30.50	64.90
2	10.60	7.70	24.60	67.70
3	36.50	22.80	71.20	6.00
4	47.00	29.40	63.80	6.80
5	20.90	12.80	67.40	19.80
6	15.3	10.70	59.70	29.60
7	-	24.60	71.60	3.90

This vast presence of unconsolidated and very loose fine sands and silts may be a major cause for the liquefaction phenomenon in cases of earthquakes'.
The maximum magnitude of an earthquake expected in the study area is M = 6 ÷ 6.2.

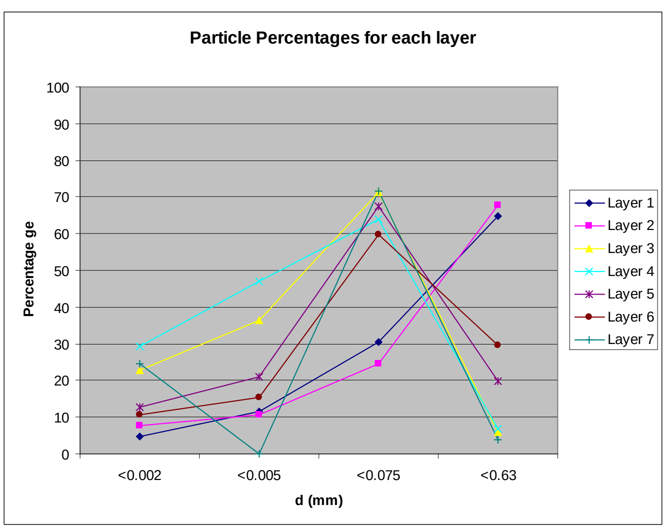

Fig. 9 The percentage of the particles for the seven layers

The above mentioned layers are classified after UCS as shown in table 5:

Table 5

Layer	LL (%)	PI	Passing No. 200 Sieve	Classification
1	-	-	30.50	SM
2	-	-	24.60	SM
3	43.60	22.60	71.20	CL
4	33.46	15.56	63.80	ML
5	28.00	14.45	67.40	ML
6	23.71	8.94	59.70	CL-ML
7	39.71	20.01	71.60	CL

<u>The physical conditions</u>
(porosity "e", density "Dr" and degree of saturation S, Liquidity Index IL)

The physical properties of the layers determined from laboratory tests are summarized in table 6.

Table 6

Type of Layer							
	1	2	3	4	5	6	7
Nspt	7 ÷9	10÷ 14	7 ÷10	14 ÷15	8 ÷12	16 ÷18	16 ÷18
W (%)	20.63	22.11	38.98	35.52	25.18	21.38	32.77
LL (%)	-	-	43.63	33.46	27.77	23.71	39.17
PI	-	-	22.59	15.56	14.45	8.94	20.01
IL	-	-	0.79	1.13	0.82	0.74	0.68
e	0.650	0.695	0.880	0.950	0.910	0.643	1.094
S	0.85	0.82	Plastic liquid	Plastic liquid	Plastic liquid	Plastic soft	Plastic soft
Dr (%)	30	40	-	-	-	-	-

Based on the tests results we conclude that we have to deal with weak soils with low bearing capacity which calculated after the classical method resulted [σ] = (100 ÷180) KPa.
The graphical presentation of the relation between the bearing capacity of soils and depth for the geotechnical model M-1 is shown in Fig. 10

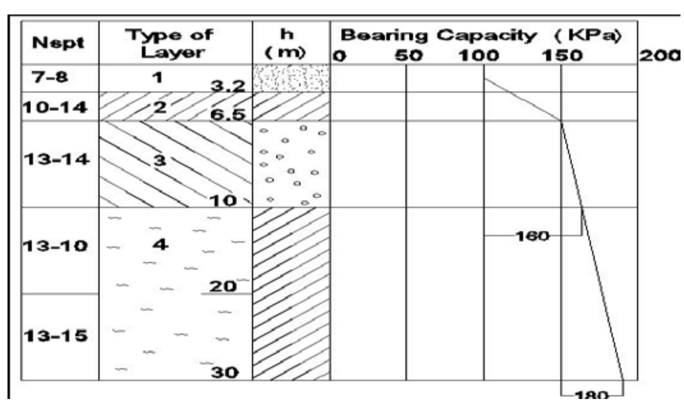

Fig. 10 Change of the soil bearing capacity

The results for the oedometric and permeability tests are given in table 7. Based on that we conclude that the layers 3, 5, 6 are very compressible which will cause important settlements to the objects that will be constructed there. So, if we analyze the geotechnical model M-1 we can calculate the settlements of the foundation of a gas deposit (diameter =30m, depth =2 m, transmitted pressure P= 100 KPa) See Fig. 11.

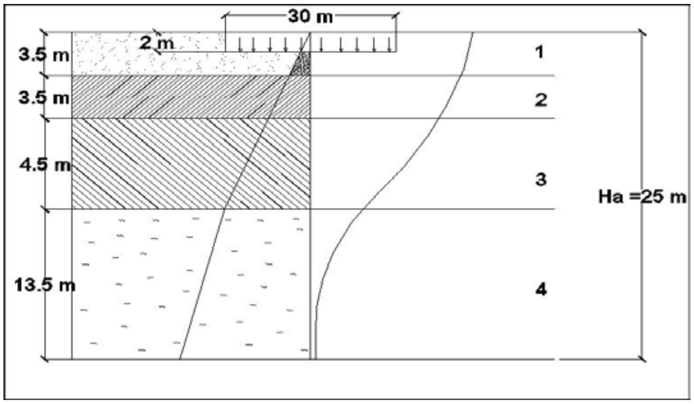

Fig. 11 Active zone under gases deposits foundation located in the model M-1

Results: Active zone Ha= 25 m; The settlements are S= (12-15) cmThe settlement values may exceed the limit values.

Table 7

Layer	e_0	Cv* 10^{-2} (m^2/ sec)	Eo (KPa)	K (cm/sec)
1	0.650	-	$1.05*10^4$	9.92
2	0.695	-	$1.15*10^4$	4.01
3	0.880	9.6	$0.5*10^4$	$6.89*10^{-2}$
4	0.950	38.6	$0.6*10^4$	$4.63*10^{-2}$
5	0.910	-	$0.7*10^4$	$3.15*10^{-3}$
6	0.643	-	$0.8*10^4$	$2.95*10^{-3}$
7	1.094	16.9	$0.89*10^4$	$9.8*10^{-3}$

The large presence of sand and silt particles in the layers determines a high value of the coefficient of permeability: Kmed= 0.059cm/sec, and as a result a short time of the primary consolidation(a few days)

The results of direct shear tests

Internal friction angle φ and cohesion c are summarized in table 8. By using this data we can calculate the bearing capacity "R" after the "Limit state" Theory.

Table 8

Type of Layer	φ (°)	c (KPa)	R (KPa)	R_1 (KPa)	R_2 (KPa)
1	26	-	192	128	96
2	28	-	219	146	110
3	23	7	203	135	101
4	20	11	194	129	97
5	24	5	202	135	101
6	22	4	174	116	87
7	19	32	299	200	150

According to the "Limit state "Theory, and by considering safety factor Fs= 1.5 ÷2, the above layers appear to have low bearing capacity. (R_1 and R2 are the allowable bearing capacities respectively for Fs =1.5 and Fs=2; R_1 =R/1.5, R_2 = R/2)

THE BEHAVIOR OF THE BASEMENT UNDER STATIC AND DYNAMIC LOADS

The data collected from geological, geotechnical and seismological studies permit us to predict the future behavior of soils under static and seismic loads.

Static loads

In this case we have to pay attention to:

- Low bearing capacity of shallow foundations
- Low bearing capacity of deep foundations (pile foundations) because of the small friction force of the different layers (the layer of fine sand, silty sand or silty clay in plastic liquid conditions).
- Enormous settlements of the basement because the layers are loose, unconsolidated and very compressible.

These settlements can cause ultimate limit state or service limit state in the construction.

- The time of primary consolidation will be longer than normally (several months) because of the high pore pressure resulted from CPTU tests.

Dynamic loads

Dynamic loads from possible earthquakes can cause dangerous situations such as:

-Possibility of liquefaction because:

- the thicknesses of loose deposits are enormous,
- the deposits are mostly fine sands and soft silts or silty sands.
- the density of deposits is Dr<50%
- the deposits are under the groundwater table.

In these conditions we must determine: the maximum acceleration "a_{max}" =n*g on the ground surface, the possibilities of liquefaction and the potential of a possible liquefaction.

- If the liquefaction phenomenon has big chances to happen, then before the construction begins the area must be improved by using gravel piles which serve as vertical drainage or by using a combined foundation slab will piles.
- In order to construct safe foundations we must evaluate the reduction of the bearing capacity of the soil because of the PGA.

CONCLUSIONS AND RECOMMENDATIONS

The main deposits in the construction area of the energetic park are maritime, marshy and alluvion.Q_{4kt} , Q_{4dt} , Q_4,

represented by silty sands, sands and slimy clays, peaty clays and gravel from slightly consolidated to normally consolidated. Below them , there are Neogene's deposits composed by mudstone, sandstone and conglomerates.
The thickness of the deposits varies from 80m to 100 m. The groundwater table is (0.5÷1.5)m from the ground surface. The water is aggressive to concrete and iron.

The bearing capacity of the layers is low or very low.

The geological sections show that the geological composition is very complicated and heterogeneous, nevertheless we can use for calculations four generalized geotechnical models. We distinguished seven layers in these models.

The in-situ and laboratory tests results confirmed our four geotechnical models.

The constructed buildings in this area will have considerable settlements under static loads and as a consequence the may appear the ultimate limit state ore the service limit state.

Under seismic loads, the deposits of this area can be liquefied, can loose stability and undergo to supplement enormous settlements. This is why is necessary a seismic study on

The behavior of the four geotechnical models and hereby evaluating the soil-structure interaction. The soils of this area belong to E classification category after EC- 8.

In order to assure the safety of the buildings that will be built in this area we have to make three provisions:

- the improvement of the building area by injecting silica gels in the ground, by explosion in order to obtain soils more compacted with Dr>50.
- By using vertical and horizontal drainages in order to eliminate premises for the liquefaction phenomenon.
- By using combined foundation: mat or slab with piles.

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