Geophysical and Geotechnical Studies for Electric Power Plants Project at the Sakarya River Zone, Turkey

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ABSTRACT

Geophysical and geotechnical studies were conducted at a proposed hydroelectric power plants facility at a site north of Sakarya river D100(E-5) bridge in Adapazarı town of Sakarya city in Turkey. The study is aimed at evaluating the competence of the near surface formation as foundation materials. Geophysical and geotechnical methods of investigation were adopted. The geophysical investigation involved the Vertical Electrical Sounding (VES) technique using the Schlumberger configuration, seismic refraction and geotechnical investigative methods comprised of boreholes to 40 m depth with Standard Penetrating Testing (SPT) and undisturbed Shelby tube, disturbed Split Spoon soil sampling, triaxial compresssion and consolidation tests. A total of five Vertical Electrical Soundings (VES) and five seismic refraction profiles from different location within the study area were used for the study. Water content, plasticity, and grain size distribution characteristics are obtained from laboratory testing leading to a classification of soils using the Unified Classification System. The geophysical results revealed three distinct geoelectric sequences which comprises of topsoil, weathered and partially weathered layers. The topsoils composed of clay/silt/sand while the weathered layer composed of silty sand. The partially weathered layer is formed of silty clay. The geotechnical results show that the partly weathered layer has relatively high clay content. Based on the consistency limits of the soils within the area, the soil generally indicate firm plasticity, hence, the soils are expected to exhibit medium swelling potential. There is no evident of geological feature such as fracture/fault within the subsoil in the area.

Introduction

Geophysical methods such as the seismic refraction and electrical resistivity (RC) are used for the site investigation. Geophysical surveys are efficient and cost-effective in providing geotechnical information since they combine high-speed and accuracy in providing subsurface information over large areas (Wang 1979; Seed et al. 1983; Luna and Jadi 2000: Soupios et al. 2005; Momoh et al. 2008; Ozcep et al. 2009; Akin Torinwa OJ et al. 2009; Bozkurt and Kurtulus 2009; Kurtulus et al. 2010). Site characterization is carried out for the construction of hydroelectric power plants in the Sakarya region located in the earthquake-prone areas to determine depth of bedrock, structural mapping and evaluation of subsoil competence. The need to provide information in the subsurface sequence and structure deposition necessary for foundation design needed an integrated geophysical and geotechnical investigations of the area. As a part of investigation, measurements of relevant dynamic parameters both in laboratory and in situ have been made for determination of dynamic deformation and dynamic soil structure analysis.

Description of the Environment of the Investigated Site

The study area is located within Adapazari town north of D100 (E-5) Bridge (Fig.1). The topography is low lying. The site is located within the climate of Marmara region. The main annual temperature is 14° while the rainfall varies between 38-114mm (General directorate of Meteorology of Turkey).Alluviums composed of clay; sand and silt are the major outcrop that occur within the study area.

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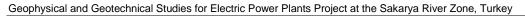[3] Kocaeli University, Engineering Faculty, Department of Geophysics, Kocaeli, Turkey
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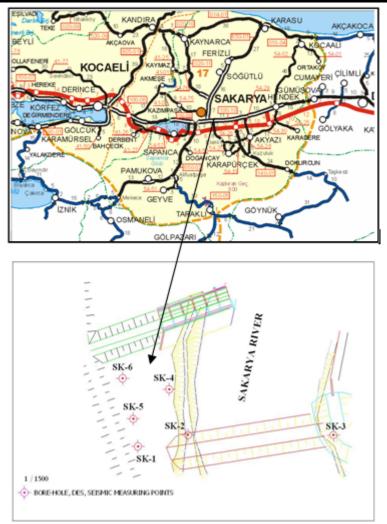*İlgili yazar / Corresponding author: emremazak@gmail.com

Fig.1: Location map of the investigation area

Methodology

Seismic refraction surveys are performed at five stations in the investigation area. The seismic data were recorded by a 12 channel Geometrics Seismic Enhancement (Smart Seis) recorder with a 8 Hz geophone system. A seismic hammer of 8 kg was used as an energy source. The measurements were conducted using offset and geophone intervals at 2m. The geophone coupling was achieved very well and the surface noises were filtered as much as possible to improve the data quality.

The seismic refraction seismograms were interpreted using GeoSeis computer program. The best fit lines of the first brakes of the signals were determined. The P-and S-wave velocities were computed from the slope of these best fit lines.

A total of five Vertical Electrical Soundings (VES) were conducted at five stations using the Schlumberger configuration. The electrode spacing was opened as much as (AB/2) 60m. The apparent resistivity values were plotted against electrode spacing (AB/2) on a bi-logarithmic graph sheet to generate depth sounding curves. Partial curve matching was carried out on the field curves. The interpretation results (layer resistivity and thickness) were fed into computer for 1-D computer assisted

interpretation involving IPI software. The final interpreted results were used for the preparation of geoelectric sections.
A total of 6 drill holes, were drilled at the depth ranges between 38.0m and 40.0m to figure out the subsurface layers and their geotechnical properties.

Undisturbed (UD) and disturbed (SPT) soil samples were collected at every 1.5m of depth in the boreholes (SK-2, SK-4 and SK-5) as shown in Fig. 1. These samples were preserved in polythene bags and transported to the laboratory within a period of 24 hours after collection. These samples were air dried by spreading them out on trays in a worm room for four days. Geotechnical tests including grain size analysis, Atterberg limits, consolidation and triaxial tests were conducted. Pressiometer tests were applied at the boring holes SK-1, SK-3 and SK-6 to figure out the mechanical properties of the soil layers.
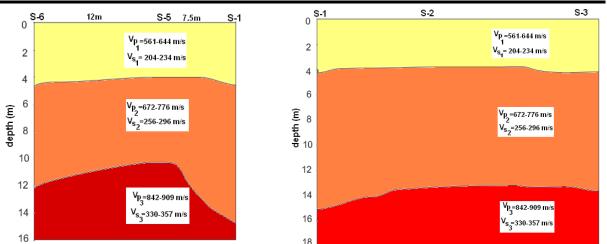
RESULTS AND DISCUSSION

Geophysical Results

Seismic results

Reciprocal seismic refraction studies were conducted. Three layers were identified and their P-and S-wave velocities, and thicknesses were determined from the seismic studies. The topsoil deposits clay/silt/sand with the thickness varies between 3,78m and 4,52m, and P-and S- wave velocities 561-644m/s and 204-234m/s respectively were obtained. Below this layer of P- and S- wave velocities 672-776m/s and 256-296 m/s respectively, the stratum is medium-hard consistency silty sand with the thickness of 11.5 – 14.5m underlain by the third layer formed of hard consistency silty clayey sand with P- and S- wave velocities ranged between 842-909m/s and 330-357m/s respectively (Fig 2). The dynamic properties related to these layers are given in Table 1.

Table 1. Dynamic properties of the soils

Profile No	Layer No	Vp (m/s)	Vs (m/s)	ρ (gr/cm^3)	E (kg/cm^2)	G (kg/cm^2)	σ	h1 (m)	h2 (m)	K (kg/cm^2)	qs (kg/cm^2)	a	To (s)
Sis-1	I	561	204	1,71	2030	713	0,42	4,09	14,45	4442	0,84	3,0	0,62
	II	672	256	1,73	3225	1140	0,41			6303	1,07		
	III	909	357	1,78	6385	2265	0,41			11711	1,52		
Sis-2	I	618	225	1,72	2477	870	0,42	3,78	13,41	5419	0,93	3,0	0,62
	II	753	287	1,75	4093	1447	0,41			8001	1,21		
	III	857	336	1,77	5641	2002	0,41			10346	1,43		
Sis-3	I	644	234	1,73	2703	949	0,42	4,52	13,53	5913	0,97	3,0	0,62
	II	744	284	1,75	3988	1409	0,41			7796	1,19		
	III	872	342	1,77	5849	2075	0,41			10728	1,46		
Sis-4	I	595	216	1,72	2288	804	0,42	4,16	14,41	5006	0,89	3,0	0,62
	II	776	296	1,76	4362	1542	0,41			8528	1,25		
	III	849	333	1,77	5528	1962	0,41			10140	1,41		
Sis-5	I	565	205	1,71	2057	722	0,42	3,95	11,53	4500	0,84	3,0	0,63
	II	738	282	1,75	3927	1388	0,41			7676	1,18		
	III	842	330	1,77	5429	1926	0,41			9958	1,40		

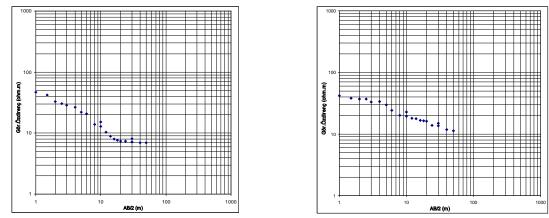Fig. 2. Seismic cross sections: (a) through SK-6 and SK-1, (b) SK-1 and SK-3

Fig. 3. Resistivity curves (a) for RES-3, (b) for RES-5

Geoelectric Parameters

The VES interpretation results were used (Fig. 3) to prepare 2-D geoelectric sections illustrated in Fig. 4. The geoelectric sections describe maximum of three geologic subsurface layers comprising the topsoil, weathered and partly weathered layer. The topsoil is composed of clay/silt/sand with resistivity values range from 18 to 45 Ωm and thickness of between 0.90-1.51m. The weathered layer consists of silty sand with the resistivity value ranging between 19-52 Ωm and partly weathered layer is composed of silty clayey sand with the resistivity values range from 7 to 12 Ωm.

Where Vp and Vs are the P- and S-wave velocities, ρ: density, E: elasticity modulus, G: Shear modulus, σ : Poisson's ration, h_1 and h_2: Thicknesses of the first and second layers, K: Bearing coefficient, qs : Safety bearing capacity, a: Soil magnification, To: Natural soil period.
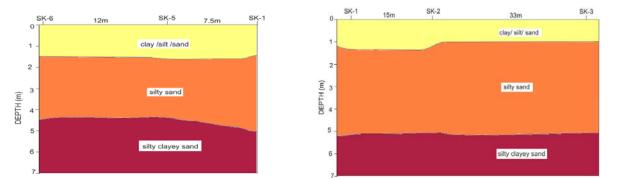
Fig. 4. Geoelectrical cross sections (a) through SK-6 and SK-1, (b) SK-1 and SK-3

Correlation of Seismic refraction and Electrical Resistivity Geoelectrical Sections

Seismic refraction sections and resistivity geoelectrical sections (Figs 2 and 4) were prepared along the same traverses (SK-6- SK-1, and SK-1-SK-3). Comparison of the figures shows that even though the geoelectrical sections look alike, the depth variations for topsoil 2.5-3.0m, for weathered layer 5-6m exist between the seismic refraction and electrical results. This may be due to the fact that two methods respond to different properties of soil.

Geotechnical Results

Index results

Fig. 5 shows the SPT values obtained in bore-hole SK-4 which are very similar to the SPT values of other bore holes. The N_{30} values vary between 23 and 30 indicating that the consistency of the formations ranges from stiff to hard (Peck et al. 1953).
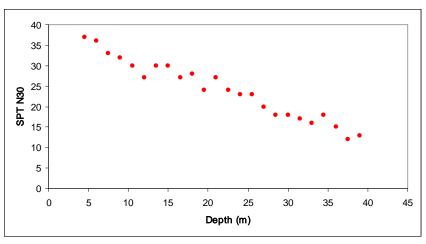

Fig. 5. Variation of SPT N30 values with depth

Table 2 shows the summary of geotechnical results of the SPT and UD samples collected from bore-hole SK-5. The natural water content of tested soil samples varies between 19.65 and 34.0%. This indicates that the natural water content of the soil in this area is of medium at its natural state. From the grading curves (Fig. 6) the soils can be classified as well graded soil. The tested soils have percentage finer (percentage passing 0.075mm) varies between 15.75% and 95.05%. the soils that are largely made up of fine particles are likely to have poor geotechnical properties as foundation materials than soils that are largely made up of coarse particle.

Table 2. Index properties of SPT and UD samples collected in bore-hole SK-5

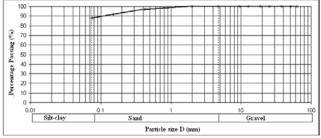| Bore-hole No	Sample	Depth (m.)	Water content (%)	Soil class	Atterberg limits			Compression index (Cc)	Consistency index (Ic)	Consistency	Compressibility	Swelling	Dry strength	Expansion
					LL (%)	PL (%)	PI (%)							
SK-5	SPT-1	4.50-4.95	19.65	SM		NP								
SK-5	SPT-3	7.50-7.95	21.5	SM		NP								
SK-5	SPT-6	12.00-12.45	29.76	CL	47.50	19.62	27.88	0.43	0.64	stiff	high	medium	Medium-high	Not-very slow
SK-5	UD-1	16.00-16.50	27.45	CL	46.24	21.01	25.23	0.42	0.74	stiff	high	medium	Medium-high	Not-very slow
SK-5	UD-2	17.50-18.00	34.005	CH	53.37	19.21	34.16	0.48	0.57	stiff	high	medium	high-Very high	Slow-not
SK-5	SPT-12	21.00-21.45	31.07	CH	52.04	18.72	33.32	0.47	0.63	stiff	high	medium	high-Very high	Slow-not
SK-5	YSK-5	24.00-24.45	27.18	CH	57.56	21.39	36.17	0.52	0.84	stiff	high	high-Very high	high-Very high	Slow-not
SK-5	UD-3	28.00-28.50	28.85	CH	51.34	20.81	30.53	0.46	0.74	stiff	high	medium	high-Very high	Slow-not
SK-5	UD-4	36.50-37.00	26.3	CL	47.50	20.79	26.71	0.43	0.79	stiff	high	medium	Medium-high	Not-very slow

Fig. 6. Typical grain size distribution curve

As shown in Table 2. the liquid limit of the soil samples range from 46.24-57.56%, the plastic limit varies between 19.62-21.39%, and the plasticity index ranges from 26.71-36.17%. The tested soil samples are of hard consistency limits indicating high percentage of clay content in the soil. Generally, soils having high values of liquid and plastic limits are considered poor as foundation materials.

The compressibility index of the soil samples ranges from 0.63-0.84 indicating that the compressibility of the soil is high (Sowers 1979), (Table 2). The plasticity index of the soil samples varies between 25.23-36.17 indicating that the soils are 'plastic-very plastic' and have the dry consistency as 'medium-high' (Leonards 1972). The range of plasticity index shows that the swelling degree of the soil is 'medium-high' (Gibs and Holtz 1956). The dry consistency of the soils 'medium-high' for CL-class, 'high-very high' for CH-class.

Groundwater and Surface Water

The soils in the investigation area have different groundwater potential depending on their lithologic and fine grained ratio. The groundwater readings were performed within 24 hours after the end of drillings. The groundwater levels of the bore-holes range from 2.0-6.70m as shown in Table 3. The water samples collected from the bore-holes were analyzed to investigate harmfulness in term of concrete and steel. The analysis of the results is given in Table 4.

Table 3. Groundwater levels in the bore-holes

Bore-hole No	Ground water level (m)	Bore-hole No	Ground water level (m)
SK-1	6,70	SK-4	6,50
SK-2	3,00	SK-5	6,50
SK-3	2,00	SK-6	6,00

Table 4. The results of analysis of water samples collected from the boreholes

Parameters	SK-1	SK-4	SK-5	SK-6	Up stream	Down stream	TS 266 Limit values
PH	7,45	7,28	7,33	7,21	7,12	7,24	6,5-9,2
Dissolved oxygen mg/lt.	9,6	9,3	9,8	8,6	1,3	1,8	-
Organic matter mgO_2/lt.	3,3	3,1	2,9	2,3	3,4	3,2	3,5
Total hardness $CaCO_3$ (mg/lt)	240	250	230	260	90	80	500
Coliform	38	33	42	36	45	48	50

Permeability Test

The soil samples collected from bore-hole SK-2 (23.50-24m), SK-4 (16.0-16.50m) and SK-5 (17.50-18m) were subjected the fixed-level permeability test. The permeability coefficients were determined for these bore-holes as 5.06×10^{-8} cm/s, 6.00×10^{-8} cm/s and 5.92×10^{-8} cm/s respectively and interpreted as 'not permeable-very low permeable'.

Triaxial test results

Triaxial test was conducted on the UD soil samples collected from different depths of the bore-holes (SK-2, SK-4, SK-5) as shown in Table 5. The cohesion of the soil samples was determined between 38.0 and 53.0 kPa and angle of internal friction was obtained between 2° and 4°.

Table 5. Triaxial test results

Bore- hole No	Sample type	Depth(m)	Cohesion (kPa)	Internal friction angle (Φ)
SK-2	UD-1	14.00-14.50	40	2
SK-2	UD-2	17.00-17.50	43	2
SK-2	UD-3	23.50-24.00	47	3
SK-4	UD-1	12.50-13.00	38	2
SK-4	UD-2	16.00-16.50	44	2
SK-5	UD-2	17.50-18.00	41	3
SK-5	UD-3	28.00-28.50	53	4

Consolidation properties

Consolidation properties of the soils were conducted on 7 undisturbed samples, collected from 12.50m to a depth of 28.50m, using the odometer device based on ASTM (1985) standards. Graphics of pressure (logarithmic) – void ratio were drawn using the data obtained from tests. Consolidation coefficient (Cv), volumetric compression coefficient (Mv) and compression indices (Cc : compression index and Cr : recompression index) were determined from these graphics. Over consolidation ratio

(OCR) was computed dividing preconsolidation pressure (Pc) to initial effective vertical stress (Po). Preconsolidation pressure and compression indices were graphically determined. The preconsolidation pressure was determined from the laboratory curves by the procedure proposed by Casagrande (Table 6).

Table 6. Consolidation parameters

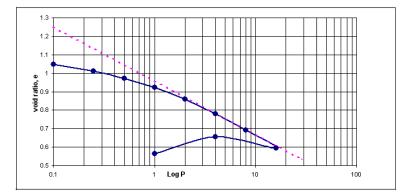| Bore-hole No:	Sample	Depth (m)	P_o	P_c	OCR	e_o	C_r	C_c
SK-4	UD-1	12.5-13.0	0.19	1.8	9.47	1.04	0.082	0.195
SK-4	UD-2	16.0-16-50	0.86	2.8	3.25	1.16	0.073	0.163
SK-5	UD-2	17.50-18.00	0.82	3.1	3.78	1.03	0.07	0.218
SK-2	UD-3	23.5-24.0	0.85	4.0	4.7	0.95	0.082	0.146
SK-5	UD-3	28.0-28.5	0.91	3.8	4.11	0.83	0.072	0.168

Fig. 7. e-logP graphic of consolidation test for bore-hole SK-2

The coefficients of consolidation (Cv) were obtained from the laboratory tests for various depths. For a pressure interval of 1–4 kg/cm^2, an average (Cv) value for consolidation settlement time is taken as 0.0089 cm^2/s. The Cv values are within the range of 0.0075-0.02 cm^2/s for the loads 0.1-16 kg/cm^2 and do not show any relationship with depth (Table 7).

Table 7. Consolidation test results for bore-hole SK-2

Specific gravity Gs = 2.68		Water content W= 20.99		Final void ratio ef=0.5625		First sample height H_0=20mm			Compression index Cc=0.29		
Load (kg/cm^2)	0.1	0.25	0.5	1	2	4	8	16	4	1
H_0 (mm)	20	19.28	18.95	18.58	18.1	17.5	16.77	15.95	15.01	15.59
Last reading	72	105	142	190	250	323	405	499	441	528
dH (mm)	0.72	0.33	0.37	0.48	0.6	0.73	0.82	0.94	-0.58	0.87
Hf (mm)	19.28	18.95	18.58	18.1	17.5	16.77	15.95	15.01	15.59	14.72
de	0.076	0.035	0.039	0.05	0.063	0.077	0.087	0.0998	-0.061	0.09
Void ratio (e)	1.047	1.011	10.97	0.92	0.86	0.78	0.69	0.59	0.65	0.56
Compressibility a_v (cm^2/kg)	0.23	0.16	0.1	0.063	0.087	0.0217	0.012	0.0051		
Volumetric compression mv (cm^2/kgf)	0.11	0.078	0.051	0.033	0.0387	0.012	0.0074	0.0032		
Consolidation time T_{90} (min)	42.15	39.01	30.737 7.41	66.89	80.36	81.42	66.12	73.53		
Cons. Coef. C_v (cm2/min)	0.020	0.02	0.0098	0.01	0.0086	0.0079	0.009	0.007		

Where H_0 : thickness variation, dH: height at the end of the experiment, de: void ratio variation. The e-LogP graphic of the consolidation test is given in Fig 7. As seen from the graphic that the void ratio (e) is found as e=1.25 for point load 0.1 kg/cm^2 , and e=0.534 for point load 29 kg/cm^2.

Bearing Capacity

The minimum bearing capacity of the soil was figured out as qu=3.68 kg/cm^3 considering C=40kPa, $\Phi=2^0$ and Nc=5.63, Nq=1.20, Nγ =0.01 and B=45.0m, Df= 20.0m, K1=1.12 and K2=0.46 in Terzaghi's equation .

$$q = K_{1*}c_*Nc+Cw_*(D_{f*}(\gamma_1\text{-}1)_*Nq)+Cw^*_*(K_{2*}(\gamma_2\text{-}1)_*B_*N\gamma) \tag{1}$$

where
qu : Bearing capacty
c : Cohesion
Nc, Nq, Ny : Coefficients of bearing capacity
D_f : Foundation depth
B : Foundation width
γ_n :Unit weight
K_1- K_2: Coefficient related to foundation shape
The net bearing capacity was calculated from the following equation

$$q_{ult}=qu+(\gamma_n+D_f)=7.27 \text{ kg/cm}^2 \tag{2}$$

and the safety bearing capacity was determined

$$q_{all}=q_{ult}/FS \tag{3}$$

where FS is safety coefficient. Considering FS=4
 q_{all}= 1.82 kg/cm^2
is obtained.The maximum safety bearing capacity of the soils was determined as 2.01 kg/cm^2 considering C=47kPa, $\Phi=3^0$ and Nc=5.90, Nq=1.31, Nγ =0.03 and B=35.0m, Df= 20.0m, K1=1.12 and K2=0.46 in Terzaghi's equation.

Pressiometer test results

Pressiometer test was applied at the depth between 12.0-36.0 m to figure out the mechanical properties of the soils. The elasticity modulus ranges from 89.14 to 36.0 kg/cm^2 and the net limit pressure (PL) varies between 0.5-7.0 kg/cm^2. The bearing capacity was calculated from the following equation

$$q_{ult}=k*(PL^*)_c \tag{4}$$

where,
k=1+0.4*(B/L), B=35.0m, L=D_f =20.0m, and (PL*)$_c$ is the geometric mean of (PL*) values. The Boring log of SK-1 is given in Fig 8., and bearing capacity and the safety bearing capacity determined from the boring log are given in Table 8.

Table 8. Bearing capacity and the safety bearing capacity values determined from pressiometer test conducted in bore-hole SK-1

Depth	Net limit pressure (PL*)	k	(PL*)$_c$	Bearing capacity	Safety bearing capaciy
(m)	(kPa)		kPa	(kPa)	(kPa)
12	175				
14	200				
28	275	1.23	208	256	128
34	200				
36	200				

Compaction results determined by Pressiometer tests

The amounts of the compaction of the soils for the load 60 kPa applied to soils for the foundation width B=35m and depth L=60m were calculated between 3.9-1.4cm.
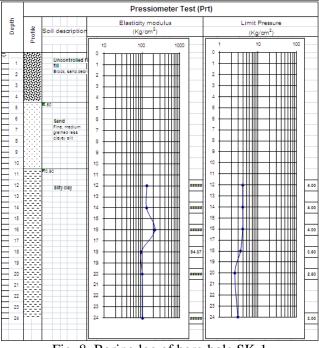
Fig. 8. Boring log of bore-hole SK-1

Bearing Coefficient results

The bearing coefficients in vertical direction were calculated using the SPT N_{30} values determined in bore-holes SK-2, SK-4, and SK-5 with the following equation (Meyerhof 1965)

$Ks = 0.75 \times N_{30}$ (mN/m^3) (5)

The calculated vertical bearing coefficient values for bore-hole SK-2 are given in Table 9 as (t/m^3)

Table 9. Calculated vertical bearing coefficients for bore-hole SK-2.

Depth (m)	SPT N_{30}	Vertical bearing coefficient (t/m^3)
18.00	23	1692
19.50	23	1692
21.00	24	1766
22.50	27	1987
24.00	24	1766
25.50	28	2060
27.00	27	1987
28.50	30	2207
30.00	30	2207

Engineering Analysis and Evaluations
Classification of decomposed soil types

According to 'Regulation about the Structures that will be Built in Disaster Regions, 2007' and Turkish Earthquake Code, the local soil class of the soils were determined as 'Z4', soil group 'D', effective soil acceleration coefficient A0=0.40', building importance factor 'I=1.5' and the spectrum characteristic periods TA and TB as 0.20 and 0.90 (s). Soil dominant vibration period and soil magnification were calculated as 0.62-0.63 (s) and 3.0 respectively.

Results of liquefaction Analysis

Liquefaction occurs in saturated soil composed of uniform grained silt and fine sand. Liquefaction occurs (based on the Chinese criteria) to have the following characteristics (Seed et al. 1983; Wang 1979);

Percent finer than 0.005 mm (5 microns) ‹15%
Liquid limit ‹ 35 %
Water content › 90% of LL

SM soils are located in the upper levels and CL-CH soils in lower layers. The sand unit with the thickness of 11m will be removed with the foundation excavation that will be 20 m in the investigation area. For that reason liquefaction is not envisaged.

Seismicity of the region

The study area is located in the Marmara region which is an active tectonic zone of Turkey characterized by the transition between the dextral strike-slip regime of the North Anatolian Fault (NAF) and the extension regime of the Aegean Sea. The NAF with a length of 1500 km is the most active component in the tectonic evolution of Anatolia, and is one of the most active and largest strike-slip faults in the world (Ulutaş and Özer 2010). The last destructive earthquakes in NAF and in the vicinity of the site were the 1999 Izmit (Mw 7.4) and the 1999 Duzce (Mw 7.2) earthquakes. The earthquakes caused heavy damage in the region between Adapazarı, Hendek, Akyazı and Arifiye. And also the Kocaeli and Düzce earthquakes highlighted the potential effects of local site conditions on the amplification of ground motions and provided the most extensive earthquake data sets ever recorded in Turkey (Ulutaş et al. 2011).

Ulutaş (2006) assessed the spatial distributions of seismicity and seismic hazard for Marmara region, Northwestern part of Turkey. The earthquake frequency–magnitude relationship is a known way to examine the seismic activity level in any seismogenic area . The frequency–magnitude distribution of earthquakes introduced by Ishomoto and Lida (1939) and Gutenberg and Richter(1944), can be formulated as:

$$\log 10\, N = a - bM \tag{6}$$

where N denotes the exponential distribution of the cumulative number of earthquakes for a single magnitude M, and "a" and "b" are constants describing the activity and slope, respectively. Ulutaş (2006) divided the Marmara region into 9 subregions according to the seismotectonic properties of the areas for the estimation of seismicity parameters. The b value from the Gutenberg–Richter frequency–magnitude distributions was calculated by the classic way and the new alternative method both using the least squares approach. According to the Ulutaş (2006) calculations corresponding to the study area, the a value is found to be equal to 4.26 and b value is found to be equal to 0.57 based on the compiled earthquake catalog by Ulutaş et al. (2003) for the period of 1901-2001. The calculated a and b value show the relation to the tectonics of the area and earthquake activity. The sets of recurrence parameters (a, b-value and Mmax) were estimated for the seismotectonic sources in Marmara region. The recurrence period of 6.0, 6.5 and 7.0 magnitude earthquakes were determined as 47, 66 and 168 years respectively (Ulutaş et al. 2003)

CONCLUSION

The geophysical results revealed three subsurface layers within the investigation area comprising of topsoil, weathered layer and partially weathered layer. The topsoil is thin with the thickness varying 3.78-4.52m and consisted of clay/silt/sand while the weathered layer composed of silty sand with the thickness range from 11.5-14.5 m. The partly weathered layer is formed of silty clayey sand. There is no evident of geological feature such as intensive fault/fracture within the geologic layers. The seismic refraction studies show that the soil layers are of low seismic velocities. The geotechnical results show that the soils are generally of medium natural water content and high clay content as revealed by the percentage passing 0.075mm. SPT N_{30} values varying between 13-32 indicate that the consistency of soils is ranged from 'stiff to hard'. Since the plastic index of the soils within the area are between 26.71-36.1%, the soil can be judged to be 'plastic-very plastic', have dry consistency as 'medium-high', hence, the soils are expected to exhibit 'medium-high' swelling potential.

Deduction from the above is that, the partly weathered layer may be related as relatively good as a foundation material. The foundation of the proposed civil structure can be hosted by this formation. The nature of this layer has to be considered in the design of the foundation.

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