Al Azhar Bulletin of Science Vol. 29, No. 1 (June) 2018, : pp. 15-28

DELINEATION OF GEOTECHNICAL PARAMETERS USING SHALLOW SEISMIC AND ENGINEERING GEOLOGY AT LAND OF COUNCIL OF NATIONAL DEFENSE 6TH OCTOBER CITY – EGYPT.

Ahmed M. Saad[1], M.A.H., Abdel Aziz [2] and Amr Ehab Hassan [1]

1 *Geology Department, Faculty of Science, Al Azhar University, Cairo, Egypt.*

2 Atomic Energy Authority

ABSTRACT

The combination of geophysical data and geotechnical measurements may greatly improve the quality of buildings under construction in civil engineering. The geologic setting and physical and dynamic properties of foundation beds are playing a crucial role in the stability of building, especially in case of the subjection of building to dynamic forces associated with the earthquake occurrence. This research involves geotechnical and shallow seismic refraction for foundation beds at Land of Council of National Defense 6th October City. The shallow seismic refraction technique was also used to evaluate the foundation rock properties in the area by recording the first arrival time of seismic waves and their interpretation in terms of subsurface geoseismic layers and their physical and dynamic properties. The ultimate bearing capacity in the second layer has value ranging between 2876 gm/cm2and 10643gm/cm2, the allowable bearing capacity in the second layer is between 959gm/cm2, to 3842 gm/cm2 and hence the factor of safety is equal 3, this indicates that the bed rock can be classified as cohesive soil. Chemical analysis results reveal content of sulfates and chlorides indicate that most the studied samples are non-aggressive and some samples are moderately aggressive. Mechanically, the frictional angle ranges from 30° to 36° and the ultimate bearing capacity for shallow foundation ranges from 616 kN/m^2 to 1901 kN/m^2 .

Keywords: Geotechnical properties, shallow seismic technique, direct shear box**.**

INTRODUCTION

Sixth of October City is one of the new cities in Egypt. The area has located in the west of Cairo. The area is approximately 117520 m^2 and it lies at about 32 Km from Cairo (Fig. 1). It is rise 160 to 200 m. above sea level. It's bounded from the North by Sheikh Zayed City, from the south by Al Fayuom City, Western Desert from the west and Al Giza from the East.

Geologic Setting of Studied Area

The study area to the West of Cairo has been the subject of intensive geological investigations since the beginning of this century by several authors *(e.g.* ***Beadnell, (1902); Abdel Khalek etal, (1989)***. The exposed stratigraphical succession comprises both sedimentary and volcanic rocks ranging in age from Upper Cretaceous to Quaternary (Fig. 2). The frequent unconformities in the succession indicate structural activity of this area throughout its geological history. The basaltic flows overlie the pre-Miocene rocks. The study area is a typical flat rocky desert plateau. It consists of low relief, terrace like and Northerly dipping scarps (tells) covered by flint cobbles. These scarps border broad shallow valleys controlled by the lithology, slope and prevailing structures. They comprise basalt sheets at the base capped by a series of sands and gravels of the Lower Miocene Moghra Formation. The basalts occur mainly as fissure-erupted flows, associated with the pre-Miocene fault systems which acted as channels for the magma. Both the fault system and the volcanic activity seem to be genetically connected with a regional Mid to Late Tertiary tensional phase that resulted in both the initiation and reactivation of the dominant fault system with East-West and Northwest-Southeast trends, and also in the eruption of the sematic magma that flooded Northern Egypt during the Mid to Late Tertiary.

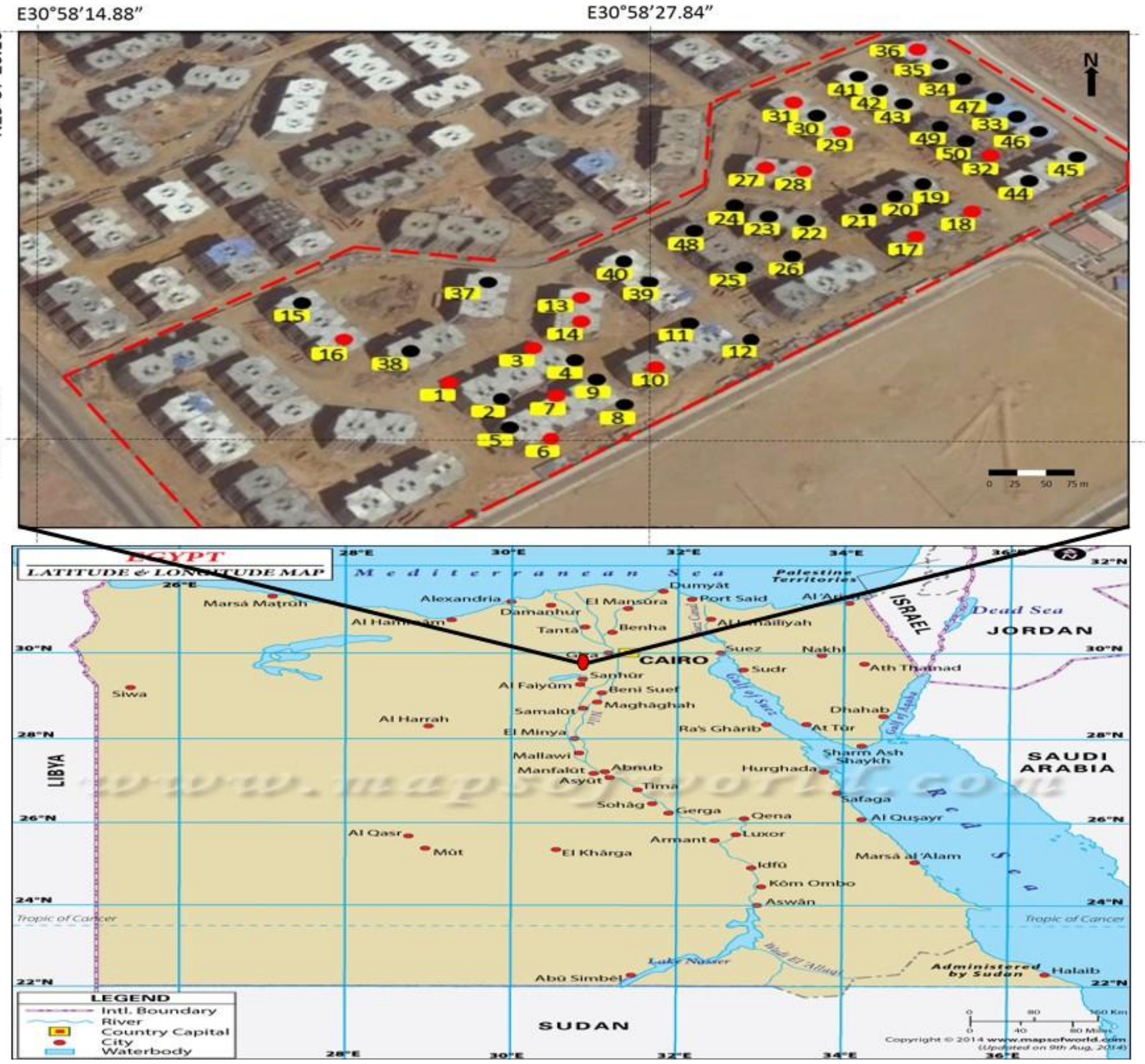

Fig.1: Location map of the study area.

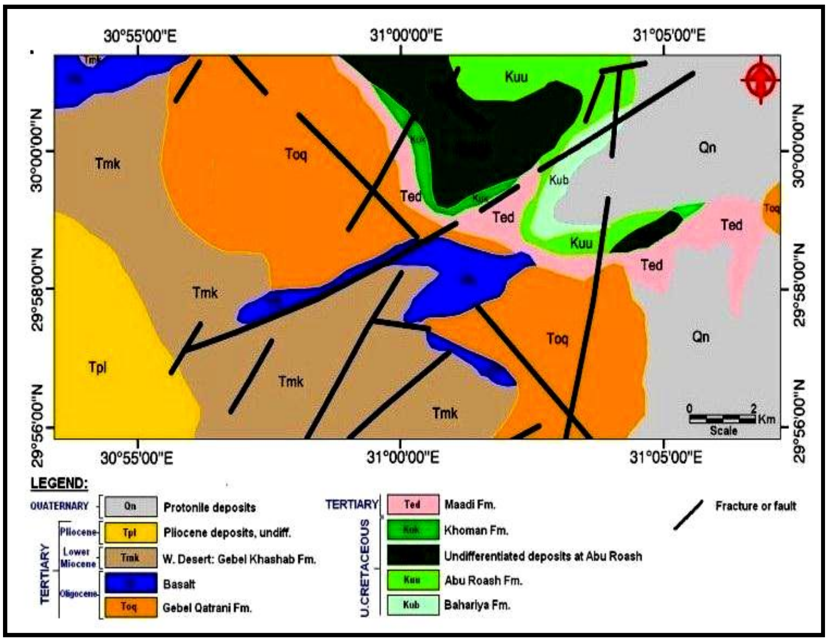

Fig.2): Geological map of October City (modified After Conco 1987).

MATERIAL AND METHODS

Samples were selected thoroughly to cover the different rock types and features present in each area to assist in defining the specifications of the region and propose the best solution or how to treat with its defects program is designed to serve the aim of this research. The laboratory tests were selected to obtain the required physical and mechanical properties of the rocks and soils to evaluate the different foundation beds for Land of Council of National Defense 6th October City. The tests on rocks include: petrography (11 samples), rock quality designation (R.Q.D) (20 samples) and uniaxial compressive strength (U.C.S) (18 samples) were prepared representing the encountered lithologies. The laboratory experiments which were carried out on soils are: shear test (5 samples) and chemical analysis (23 samples).

Interpretation of Shallow Seismic Data:

The seismic refraction method yields quick and relatively inexpensive results for the foundation conditions at these sites, involving the study of the propagation of deformation due to sudden impact in certain media Bell (1980). Four seismic profiles are conducted in the present study with geophone distance of 7.5m. Each profile has two shot points normal and reverse shots of P-waves, while the Vs wave can be estimated by using the theoretical equation $Vp = 1.7\ Vs$. Table (1) shows the variation of the seismic wave velocities reflects the variation in the rock type occupied the shallow section in the area (Fig.3).

Figure 4 shows the velocity waves (Vp) of the second layer range between 1600 to 3100 m/Sec., the thickness distribution maps of the second layers (Fig.5) increase and decrease by the same way and toward the same locations (Northern, Northeastern and Southwestern parts of the study area), because it is of undefined base for the extension of this layer downwardly. Figs. 6, 7and 8) shows the densities distribution map, Ultimate bearing capacity and Allowable bearing capacity of the second layer. Table 2 show the ultimate bearing capacity in the second layer has values ranging between (2876 KPa) and (10643 KPa) while Allowable bearing capacity ranges between (959 KPa) to (3842 KPa). The factor of safety in the studied area equal (3), this indicates that the bed rock can be classified as cohesive soil.

Composite rock is defined as rock mass which constitutes of more than one type of rock in a rock mass (**Mohamed, 2006**) Most of these rock masses are of sedimentary to meta-sedimentary formations and the geological sedimentation process produced an interbedded profile of different rocks, one being weaker than the other. To accomplish this study more than 11 thin sections were prepared representing the encountered lithologies. The uniaxial compressive strength (U.C.S) of the different facies was determined and the samples were prepared according to **(ISRM, 2007).** The petrographic analysis of the studying area indicated the presence of tow sandstone lithofacies and weathered basalt. These are as follows:

Table (1): Distribution of seismic wave velocities along the profiles

Profile No.	Velocity (m/sec) for 1st layer		Velocity (m/sec) for 2nd layer		Expected type of lithology
	P-Wave	S-Wave	P-Wave	S-Wave	
1	900	529.4	2500	1470.6	Sand with gravels and compacted sandstone
2	1000	588.2	3100	1100	Sand with gravels and compacted sandstone
3	800	470.6	2500	1470.6	Sand with gravels and compacted sandstone
4	900	529.4	1600	941.2	Sand with gravels and compacted sandstone

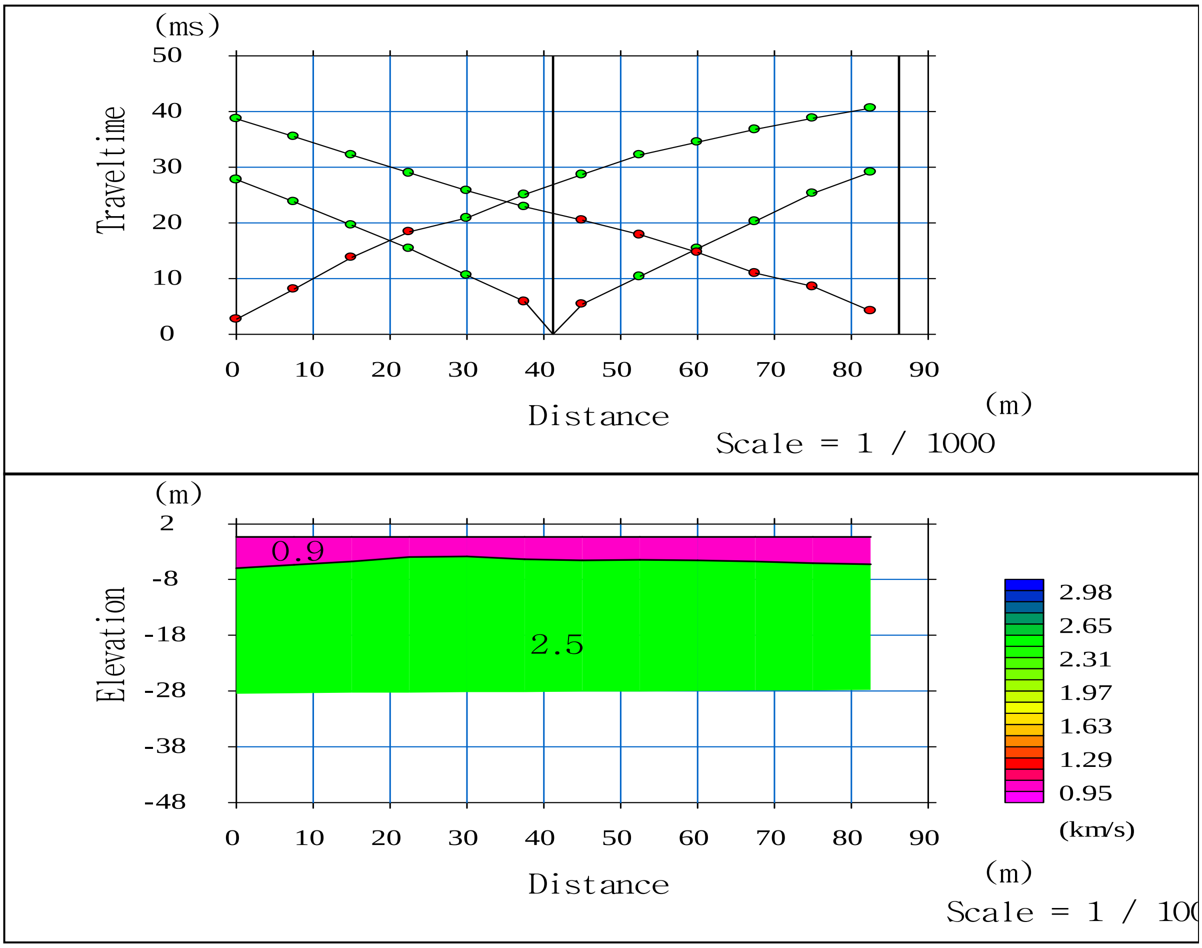

Fig.3): Interpretation of seismic profile No.1)

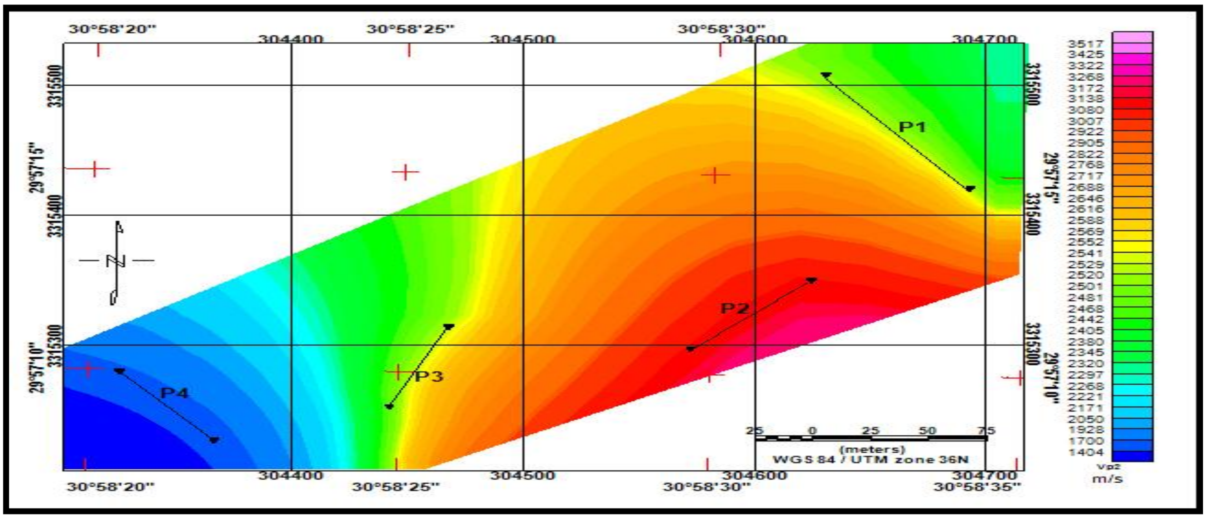

Fig. (4): 2D P-wave velocities of the second layer

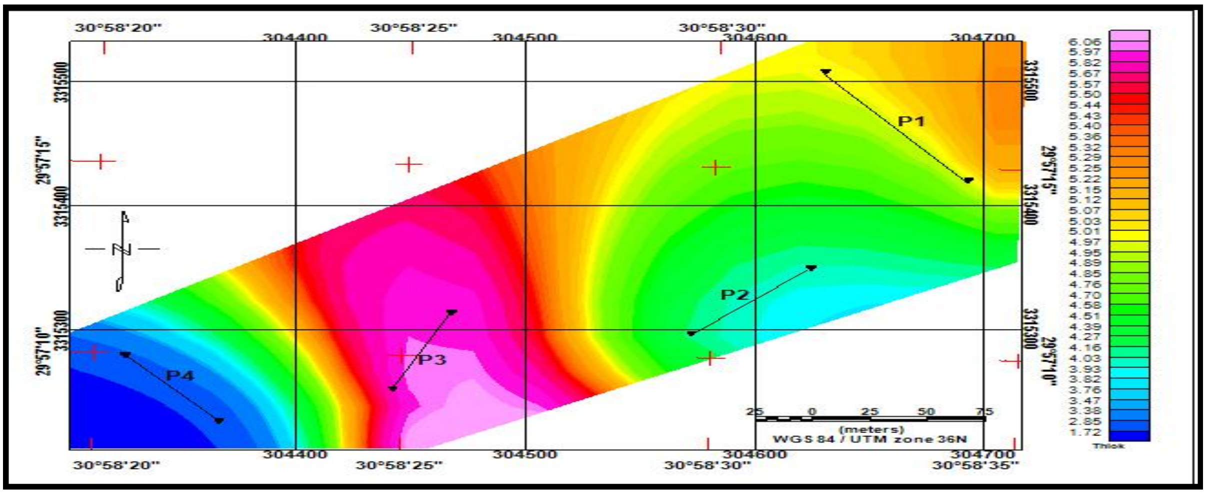

Fig. (5): Thickness distribution maps of the second layer

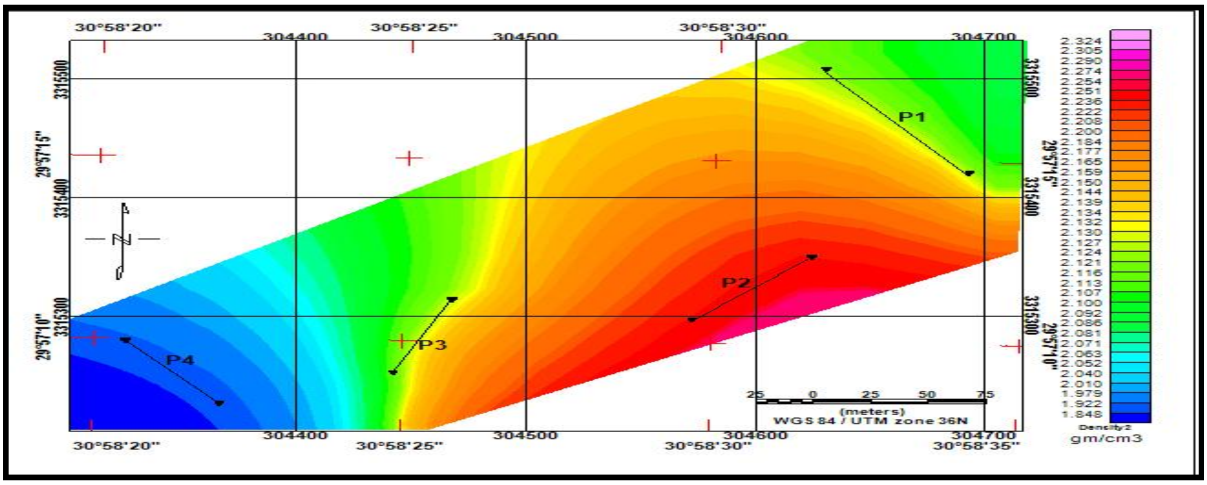

Fig. (6): Density distribution maps of the second layer

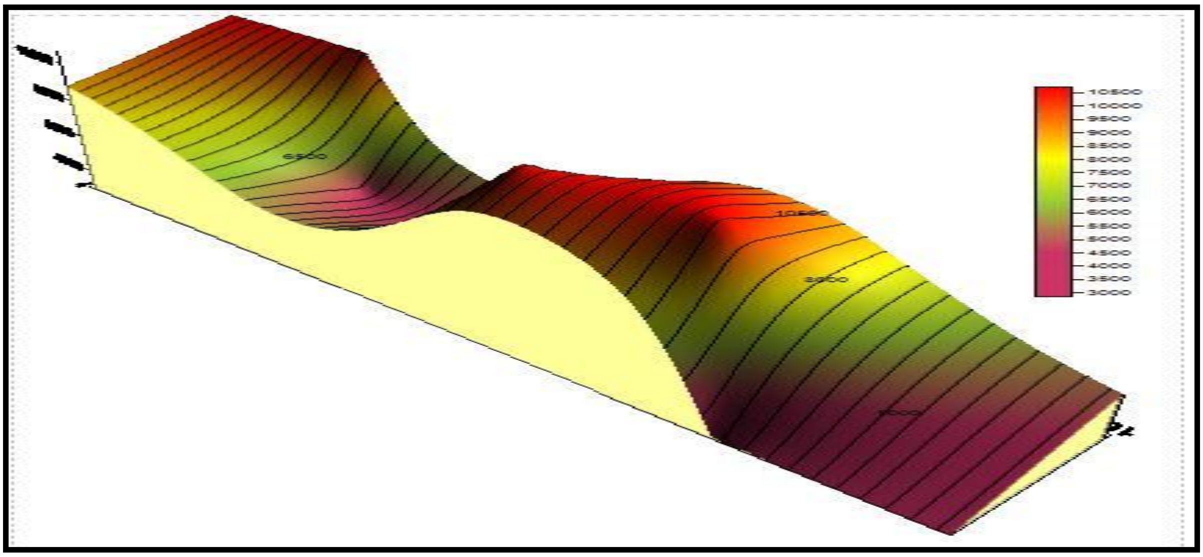

Fig. (7): Ultimate bearing capacity distribution maps of the second layer.

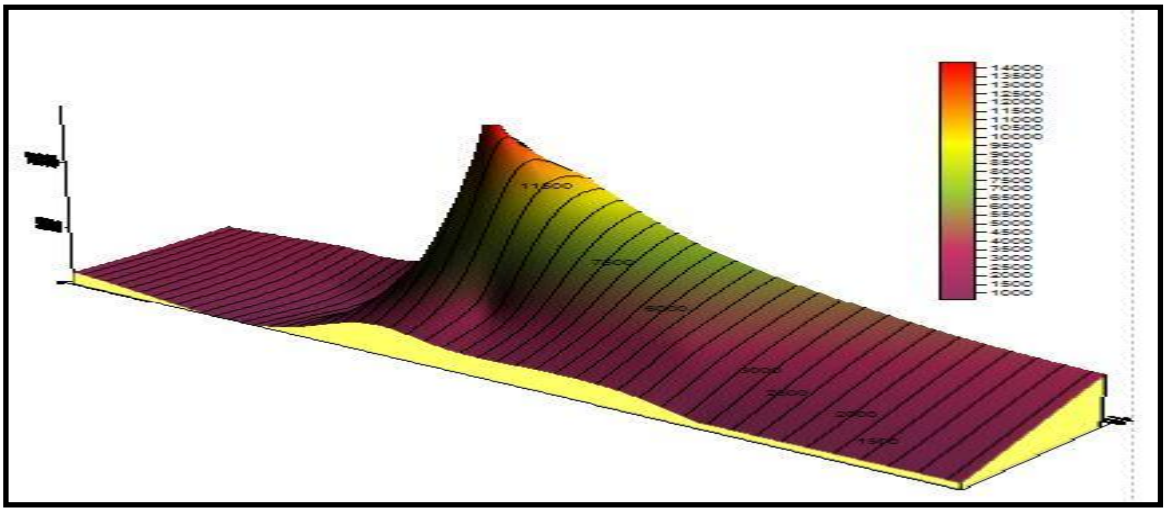

Fig. (8): Allowable bearing capacity distribution maps of the second layer

Table2): Distribution of the values of mechanical rock properties along the profiles.

Profile	V_{p2}	V_{s2}	P_2	α_2	σ_2	μ_2	E_2	K_2	Y_2	C_{i2}	S_{i2}	N_2	Q_{ult2}	Q_{allow2}
1	2500	1470.6	2.1	2.8	0.235	468.0	1155.9	2181.1	0.060	5.2	0.308	3625.27	10643.53	3547.83
2	3100	1100	2.2	7.9	0.428	276.4	789.4	5482.3	-0.712	3.3	0.748	1547.18	4543.22	1514.40
3	2500	1470.6	2.1	2.8	0.235	468.0	1155.9	2181.1	0.060	5.2	0.308	3625.27	10643.53	3547.83
4	1600	941.2	1.8	2.8	0.235	171.4	423.5	799.0	0.060	5.2	0.308	979.43	2877.1	958.70

Graywacky:

This sandstone type is composed of fine to coarse grained, rounded to subrounded, some of grained fractured, quartz grains with normal and wave extinction. The rock fragments are represented by schist, altered basal and altered plagioclase. The components are cemented by micrite and iron oxides (Fig. 9).

Glaucony- Pell-Oobiosparite (Grainstone)

This microfacies is consist of small-medium, rounded to subrounded glaconite grains (10%), faecal pellets (20%), rounded, small- medium Ooides (30%), shell fragments (35%), iron oxides and sparite patches and veinlets are present; the componants are cemented by macrocrystalline calcite cement (Fig. 10).

Weathered olivine basalts consist of Ca-plagioclase, olivine and clinopyroxene (titanoaugite). Epidote occurs as alteration product of Ca- plagioclase. Opaque minerals exhibit reddish brown colors.

Olivine occurs as forsterite phenocrystals of one generation and is not observed in the groundmass (Fig.11). It may occur as cumulates in pyroxenes observed in these basaltic rocks. The basaltic rocks are variably altered. In most types of basalts, only olivine is altered to pale yellowish green to reddish brown iddingsite (Fig.12).

Plagioclase is anorthite and the main constituent of this basalt, present both as

porphyritic crystals and as laths in the groundmass (Fig.13). In the intensively altered types the mineral components are altered mainly to clays and carbonates (Fig.14).

Pyroxene is represented by different varieties: Titanoaugite and diopsidic augite are present as porphyritic crystals and in the groundmass (Fig.15).

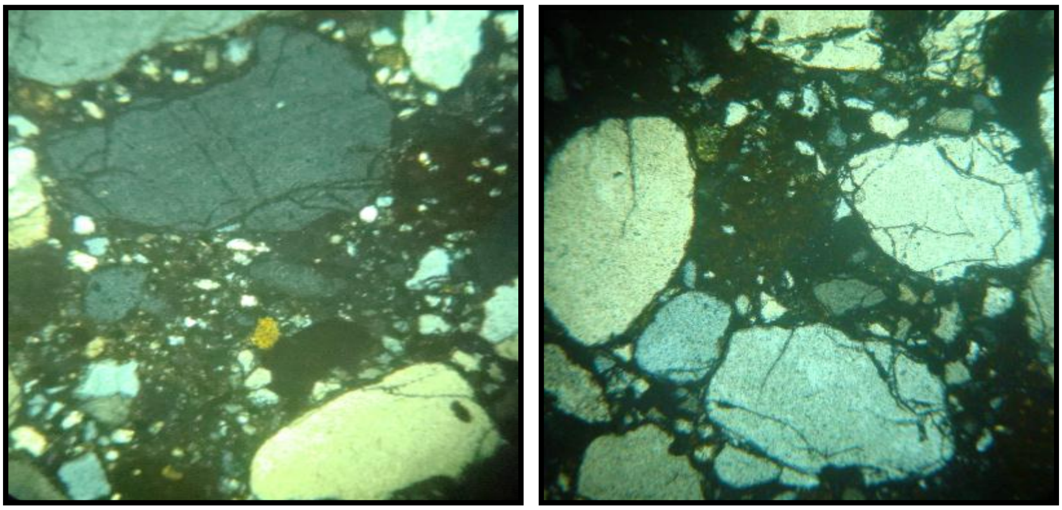

Fig.9): Sandstone graywacky

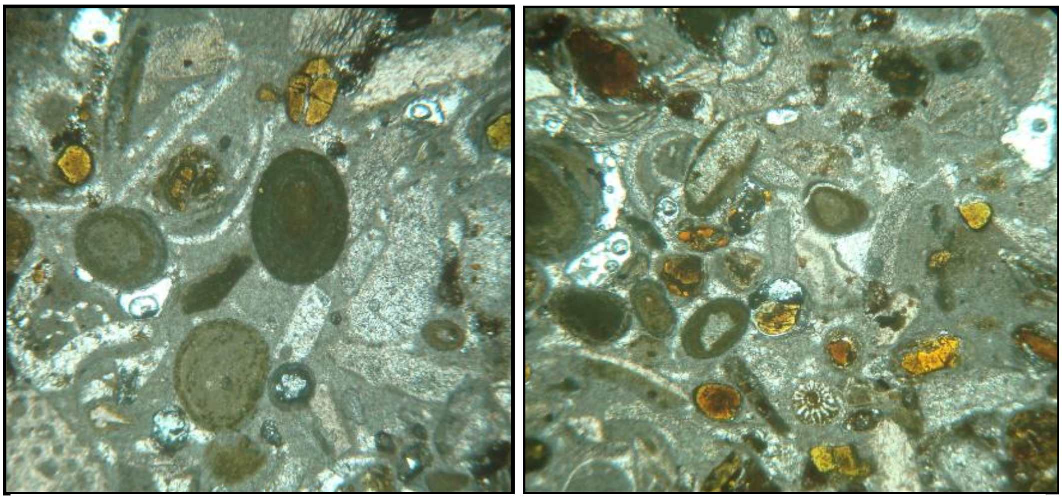

Fig.10): **Glaucony- Pell-Oobiosparite (Grainstone)**

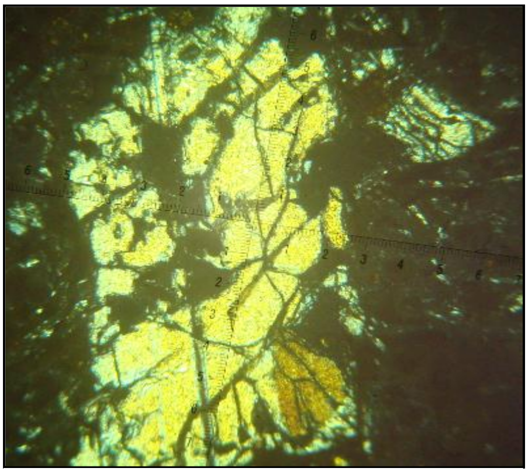

Fig.11): Showing olivine phenocrysts.

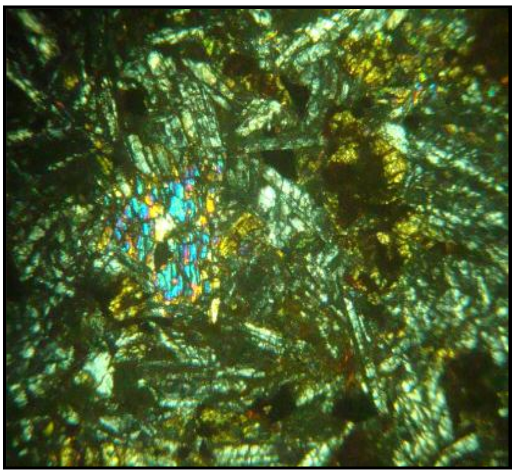

Fig.12): Sowing the alteration of olivine to reddish brown iddingsite.

Opaque minerals are mainly magnetite and ilmenite, are included in the groundmass. Also, some zoned greenish brown spinel crystals are recorded as xenocrysts with distinct block rim. Amorphous Fe- materials and fine euhedral oxidized magnetite crystals usually surround the olivine xenocrysts (Fig.16). Later weathering and alteration of these rocks resulted in the development of interesting minerals. Minor epidote may be observed in plagioclase crystals as an alteration product (Fig.17). The altered basalts consists essentially of kaolinite, plagioclase, goethite (limonite) with minorchlorite. Occasionally, this basalt is altered into reddish black varieties that are commonly spotted by white color.

Texturally, these rocks may be porphyritic and vesicular textures and also the rocks may be dolertic when in dyke form. Occasionally, these basalts are characterized by its amygdaloidal nature, where the vesicles are filled with chlorite, calcite and amorphous Fe and brown glass. Finally, the alteration of basalt by weathering and its relation to degree of oxidation of iron has been given for hand specimens and thin sections (Fig.18).

Physical and Engineering Properties of Rocks:

The rock mechanics is a branch of mechanics concerned with the response of rock to the force fields of its physical environment **(Juad, 1964).** It is also defined as the theoretical and applied science of studying the mechanical behavior of rocks.

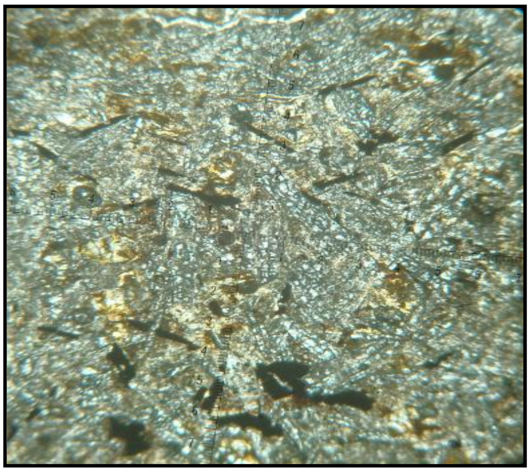

Fig.13): Showing plagioclase laths in the groundmass.

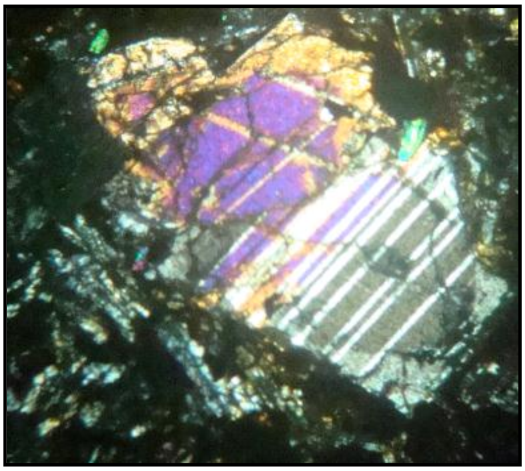

Fig.14): Showing plagioclase intensely altered to carbonate minerals.

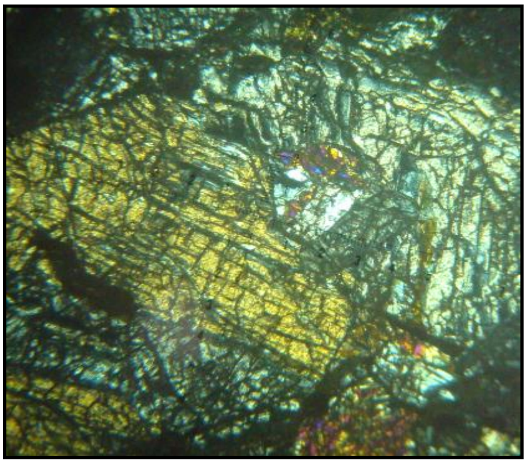

Fig.15): Showing titanoaugite porphyritic crystals.

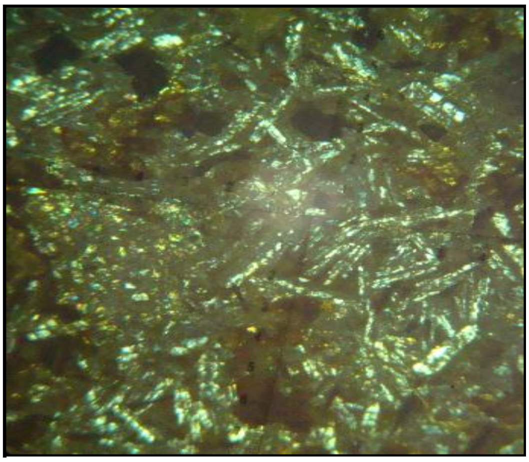

Fig.16): Showing fine euhedral to subhedral oxidized magnetite crystals.

Rock Quality Designation (R.Q.D):

Rock Quality Designation (R.Q.D) is a modification of core recovery, in the only the intact pieces of core that are more than ten centimeters long are added together in calculating length recovered. An (R.Q.D) of 100% indicates 100% core recovery with all pieces equal to or greater than 10 cm in length. Thus it does not imply an unjointed rock mass **(Deere 1967).** The (R.Q.D) described as very poor, poor, fair, good and excellent the results of this test tabulated in table (3).

Unconfined compressive strength:

Unconfined or (uniaxial) compressive strength is normally determined by statically loading a cylinder, cube, and square, of rock to failure, the load being applied across the upper and lower faces of the sample. The results obtained are in part a function of the length breadth ratio of the sample and of the rate of loading. The simplicity of the test is somewhat deceptive **Hawkes and Mellor (1970).** The results of this test are shown in table (4). The uniaxial compressive strength has been used to determine the most appropriate method of tunneling in rock.

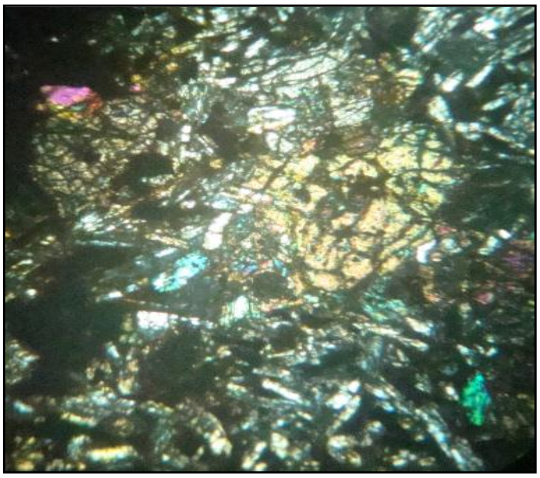

Fig.17): Minor epidote crystals observed in plagioclase crystals as alteration products.

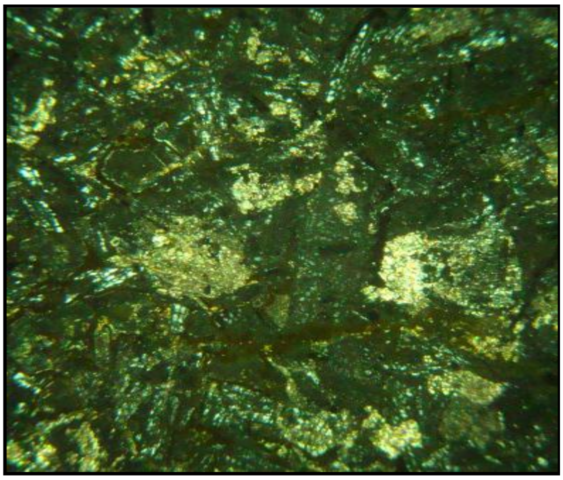

Fig.18): Showing the alteration of basalt by weathering and oxidation of iron minerals.

Table (3): R.Q.D properties of the studied samples.

Sample No.	R.Q.D. (%)	Description	Sample No.	R.Q.D. (%)	Description
1.2	10	Very poor	29.2	20	Very poor
3.2	15	Very poor	30.2	17	Very poor
15.2	15	Very poor	35.1	19	Very poor
22.1	15	Very poor	44.2	19	Very poor
23.2	10	Very poor	45.1	15	Very poor
24.1	15	Very poor	46.1	12	Very poor
25.1	11	Very poor	47.2	10	Very poor
26.1	13	Very poor	48.1	10	Very poor
27.2	10	Very poor	49.2	15	Very poor
28.2	15	Very poor	50.1	31	Poor

Direct Shear Test

A direct shear test is a laboratory test used by geotechnical engineers to measure the shear strength properties of soil. The direct shear test values of the selected samples shown in table 5). For each test, the relationship between the shear stress and horizontal displacement and also the relationship between horizontal displacement and vertical displacement are plotted to determine the shear stress and normal stress at failure (defined as peak stress). Then, the shear stress and normal stress at failure are plotted for each of the three tests to determine the slope (effective friction angle, Φ) and intercept (effective cohesion, c) from the best linear fit of the data. The friction angle is calculated from this formula:

$$\tau = c + \sigma \tan \Phi \qquad (2)$$

where τ =………… shear stress

c =……….. effective cohesion

σ =……….. normal stress

Bearing Capacity

Based on Terzaghi's bearing capacity theory, column load P is resisted by shear stresses at edges of three zones under the footing and the overburden, q=(γ D) above the footing. The first term in the equation is related to cohesion of the soil. The second term is related to the depth of the footing and overburden pressure. The third term is related to the width of the footing and the length of shear stress area. The bearing capacity factors, Nc, Nq, Nγ, are function of internal friction angle, (ϕ).

Terzaghi's bearing capacity equation

Strip footing:

$$Qu = c\ Nc + \gamma\ D\ Nq + 0.5\ \gamma\ B\ N\gamma \qquad (3)$$

Square footing:

$$Qu = 1.3\ c\ Nc + \gamma\ D\ Nq + 0.4\ \gamma\ B\ N\gamma \qquad (4)$$

Circular footing:

$$Qu = 1.3\ c\ Nc + \gamma\ D\ Nq + 0.3\ \gamma\ B\ N\gamma \qquad (5)$$

Where:

C: Cohesion of soil, γ: unit weight of soil, D: depth of footing, B: width of footing (or diameter of circular footing)

Nc, Nq, Nγ: Terzaghi's bearing capacity factors depend on soil friction angle, ϕ.

In the present study, a square foundation is (1.5*1.5), the unit weight of soil is 18kN/m^3, and the depth of the foundation is 1.5m. The results of ultimate soil bearing capacity for different friction angles of the investigated samples are shown in table 6).

The values of bearing capacity are ranges from (269 kN/m2) to (8192 kN/m2). The ultimate bearing capacity increases sharply for a cohesionless soil (c = 0) because for cohesionless soil angle of internal friction (ϕ) is more equal to 30° due to which N_C, Nq and Nγ increase which cause a sharp increase in ultimate bearing capacity. So, this type of soils have good load bearing capacities.

Table (4): Compressive strength result of the studied samples.

Sample No.	Compressive strength Kg/cm2	Description	Sample No.	Compressive strength Kg/cm2	Description
1.2	56.8	Medium weak	29.2	71.8	Medium weak
14.2	35.9	Weak	34.2	61.3	Medium weak
16.2	91.4	Medium weak	36.2	62.5	Medium weak
19.1	75.2	Medium weak	39.1	57.3	Medium weak
22.1	81.9	Medium weak	44.2	72.6	Medium weak
23.2	88.1	Medium weak	45.1	60	Medium weak
24.1	54.7	Medium weak	46.1	54.7	Medium weak
25.1	65.9	Medium weak	48.1	67.2	Medium weak
27.2	95.6	Medium weak	50.1	45.2	weak

Degree of Aggressive for Soil

The chemical analysis, in its simplest sense, is mainly used to determine the degree of aggressive of soils, by determine the organic, sulphate and chloride salts content. The water extraction method can be used for the sulphate, chloride, and PH values. These values are occasionally required to confirm the degree of aggressive for soil. According to the Egyptian code, determines the degree of aggressive for the soil and ground water.

From table (7) the studied samples at in the council of national defense 6th of October City according to SO_3 classified samples (3.1, 10.1, 13.1, 16.1, 17.1, 18.1, 27.1, 32.1 and 36.1) as non-aggressive soil, while some samples as (1.1, 6.1, 7.1, 14.1, 15.1, 17.1.a, 23.1, 28.1.a, 29.1, 31.1, 38.1, 44.1 and 48.1) classified as moderately aggressive soil and sample (28.4) is classified as highly aggressive soil. The PH values indicate all of the samples are non-aggressive soil, while Cl values indicate the most of the samples are non-aggressive.

Table (5): Shear box data of the studied samples.

Sample No.	Normal load (Kg)	Shear load (Kg)	Normal Stress (σ) (Kg/cm2)	Shear Stress (τ) (Kg/cm2)	Friction Angle (Φ)
2.2	5	0.27	0.185	0.01	28
	10	0.68	0.370	0.025	
	15	1.08	0.555	0.04	
5.2	5	0.34	0.185	0.01	21
	10	0.40	0.370	0.015	
	15	0.62	0.555	0.02	
9.2	5	0.36	0.185	0.01	26
	10	0.54	0.370	0.02	
	15	0.81	0.555	0.03	
11.2	5	0.26	0.185	0.009	34
	10	0.37	0.370	0.014	
	15	0.57	0.555	0.02	
12.2	5	0.18	0.185	0.006	45
	10	0.21	0.370	0.008	
	15	0.42	0.555	0.01	

Table 6): The ultimate soil bearing capacity of the studied samples.

Sample No.	Friction angle (ϕ)	Cohesion (C)	Nc	Nq	Nγ	Qu (kN/m2)
2.2	28	0.00	31.61	17.81	13.70	584
5.2	21	0.00	18.92	8.2	4.31	269
9.2	26	0.00	27.09	14.21	9.84	489
11.2	34	0.00	48.8	36.5	36	1374
12.2	45	0.00	172.28	173.28	325.34	8192

Table (7): Guiding values for some aggressive elements and factors determining aggressive degrees of the soil

Sample No.	Depth (m)	Sulphate	Chloride	PH Value	T.D.S	Degree of aggressive
1.1	3	0.081	0.430	7.40	1.100	M.Aggressive
3.1	6	0.036	0.590	7.35	1.110	Non-Aggressive
6.1	3	0.220	0.340	7.40	0.810	M.Aggressive
7.1	6	0.068	0.550	7.40	1.620	M.Aggressive
10.1	6	0.037	0.420	7.30	0.560	Non-Aggressive
13.1	3	0.045	0.640	7.50	2.640	Non-Aggressive
14.1	6	0.260	0.670	7.45	1.640	M.Aggressive
15.1	3	0.250	0.098	7.35	0.670	M.Aggressive
16.1	6	0.037	0.160	7.30	0.450	Non-Aggressive
17.1	3	0.011	0.320	7.45	1.130	Non-Aggressive
17.1.a	3	0.160	0.072	7.45	0.590	M.Aggressive
18.1	6	0.027	0.460	7.35	0.890	Non-Aggressive
23.1	4	0.170	0.091	7.45	0.620	M.Aggressive
27.1	3	0.043	0.140	7.40	0.420	Non-Aggressive
28.1	6	3.600	0.900	7.35	4.440	Highly Aggressive
28.1.a	2	0.190	0.086	7.40	0.470	M.Aggressive
29.1	6	0.052	0.280	7.50	0.440	M.Aggressive
31.1	3	0.054	0.480	7.30	1.110	M.Aggressive
32.1	3	0.033	0.250	7.45	0.470	Non-Aggressive
36.1	6	0.037	0.089	7.45	0.460	Non-Aggressive
38.1	3	0.160	0.072	7.40	0.520	M.Aggressive
44.1	6	0.170	0.780	7.45	1.500	M.Aggressive
48.1	5	0.220	0.120	7.35	0.810	M.Aggressive

REFERENCES

Abdel Khalek, M. L., El-Sharkawi, M. A., Darwish, M., Hagras, M. and Sehim, A. (1989): Structural history of Abu Roash district, Western Desert, Egypt. Journal African Earth Sciences 9, 435-443P.

ASTM D3080 (2004): Standard test method for direct shear test of soils under consolidated drained conditions. Annual book of ASTM standards. D 3080, Sect. 4, Vol. 04.08. Philadelphia, Pa.

ASTM D-422-63 (1998): Standard test method for particle-size analysis of soils. West Conshohocken, USA: American Society of Testing and Materials (ASTM) International; 1998.

Beadnell, H. J. L. (1902): The Cretaceous region of Abu Roash, near the Pyramids of Giza, Egypt. 48p. Survey Department, Egypt. Cadigan R.A. (1961).

Bell (1980): Study of the propagation of deformation due to sudden impact in certain media**.**

BS, 1981. Code of practice for site investigations. British Standards Institution, London, Vol.5930.

BS, 1990. Methods of test for soils for civil engineering. New purposes. British Standards Institution, London, Vol.1377.

Conoco, (1987): Conoco Coral, and Egyptian General Petroleum Company, 1987 Geological map of Egypt, scale 1:500,000, Cairo, Egypt.

Deere (1967): Design of surface and near—surface construction in rock, Failure and Breakage of Rock—Proceedings of the 8th Symposium on Rock Mechanics, Charles Fairhurst, ed., American Institute of Mining, Metallurgical, and Petroleum Engineers, p. 237–302, New York.

Hawkes and Mellor (1970): Uniaxial testing in rock mechani cs laboratories. Engineering Geology, Vol. 4 No. 3, p. 177- 285

Hazen, A. (1892): Some physical properties of sands and gravels with special reference to their use in filtration. The 24th Annual Report of the State Board of Health of Massachusetts. Public Document No. 34, Boston, Wright and Potter Printing Co., p. 553

Holtz, W.G. and Gibbs, H.J. (1956): Engineering properties of expansive clays. Trans. Am. Soc. Civil. Eng. V. 121, pp.

ISRM, 2007. The Complete ISRM Suggested Methods for Rock Characterization Testing and Monitoring: 1974-2006. In: Ulusay, Hudson (Eds.) Suggested Methods Prepared by the Commission Testing Methods, Int. Soc. Rock Mech. ISRM Turkish National Group, Turkey. p.628.

Mohamed, Z., A.G. Rafeek 2006. Laboratory Design Assessment of Weak Rocks. Al-Jazari International Journal of Civil Engineering, FEC, UiTM, 1(1): 47- 58.

Judd,W . R. (1964): Rock stress, rock mechanics and research, in State of Stress in the Earth's Crust, W. R. Judd, ed., Elsevier, New York, p.5–51.

وصف المعاملات الجيوتقنية باستخدام الموجات الزلزالية الضحلة والجيولوجيا الهندسية علي منطقة ارض الدفاع الوطني بمنطقة السادس من أكتوبر- مصر.

احمد محمود سعد [١]، محمد عبد الحليم عبد العزيز [٢] ، عمرو إيهاب حسن[١]

١- قسم الجيولوجيا، كلية العلوم،جامعة الأزهر، القاهرة، مصر.

٢- هيئة الطاقة الذرية ، القاهرة ، مصر.

يتضمن هذا البحث دراسات جيوتقنية وجيوفيزيقية لمنطقة ارض الدفاع الوطني بمدينة السادس من أكتوبر. ولذلك يهدف هذا البحث إلي دراسة الخواص الجيوتقنية لطبقات الأساس وذلك من خلال دراسة الخواص الفيزيائية والهندسية للتربة باستخدام الجيولوجيا الهندسية والموجات الزلزالية الضحلة وتأثيرها علي الثبات العمراني. وقد أظهرت القياسات الجيوفيزيقية أن قدرة التحمل القصوى للطبقة الثانية من طبقات الأساس تتراوح بين (٢٨٧٦جم/سم^{٢}- ١٠٦٤٣جم/سم^{٢}) وقدرة التحمل المسموح بها تتراوح بين (٩٥٩جم/سم^{٢}-٣٨٤٢جم/سم^{٢}) وهذا دليل علي أن تربة الأساس تكون تربة متماسكة. وقد أظهرت التحاليل الكيميائية التي أجريت علي عينات التربة أنها تربة غير عدوانية الي متوسطة العدوانية. ميكانيكيا، من خلال القص المباشر يتضح أن زاوية الاحتكاك الداخلي تتراوح بين (٢١° - ٤٥°) ، وقدرة التحمل القصوى للتربة تتراوح بين (٢٦٩كيلو نيوتن/متر^{٢} – ٨١٩٢كيلونيوتن/متر^{٢}).