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Geotechnical Suitability of Soil for Construction and Pavement Purposes, New Ismailia City, Egypt

Lara Salah[a,*], Ahmed Moatamed Hegazi[a], Salah AbuBakr Abdelhameed[a]

[a] *Geology Department, Faculty of Science, Suez Canal University, 41522, Ismailia, Egypt*

Abstract

The New Ismailia City is located at 30° 34' 31.51"N and 32° 20' 26.27"E, on the eastern side of the Suez Canal, covering an area of about 11km^2. A comprehensive geotechnical field and laboratory testing program, chemical analysis of soil samples, as well as X-ray investigation were carried out. The general subsurface soil sequence encompasses two layers; the upper and lower sand layer, in addition to the backfill. Several lenses of variable composition are encountered at different depths. The SPT values are increasing with depth, which could be attributed to the load resulted from the weight of the overburden soil layers. The New Ismailia City has an acceptable bearing capacity range: 1.0 kg/cm^2 to 1.5 kg/cm^2. The ultimate bearing capacity values of foundation 3.3 kg/cm^2 to 4.99 kg/cm^2, are laying within the allowable range of the bearing capacity, and hence soils are suitable for both construction and pavement purposes. Based on X-ray investigation, the identified clay minerals are montmorillonite, kaolinite and illite. The montmorillonite clay is regarded as very hydrophilic due to their mobile structure, making them highly-expansive and hazardous. So, it's suitable to replace the soil to a depth of 4-7m. The backfilling should be carried out using a mixture of non-cohesive sand and gravel, with percent of fines not exceeds 10%, and soil type A-1-a or A-1-b. Finally, the soil must be compacted to achieve the required maximum dry density and optimum moisture content (maximum dry density: 2.1g/cm^3 & optimum moisture content 6.6%).

Keywords: New Ismailia City, Geotechnical Investigation, Construction and Pavement purposes

1. Introduction

The 2030's Egyptian Sustainable Development Strategy (SDS, 2030) encompasses several economic, agricultural, and industrial national projects. The development of the Suez Canal corridor project is fundamental for this development strategy. The New Ismailia City is located at 30° 34' 31.51"N and 32° 20' 26.27"E, and covers an area of about 11km^2(Figure 1). The study area is dominated by extensive unconsolidated deposits and sedimentary succession of age ranging from Tertiary to Quaternary. The Quaternary sediments are represented by Pleistocene sediments (gravels and sands with clay intercalations), Holocene sediments (Nile silts and clays), and Holocene stabilized dunes (Figure 2**)**. The Tertiary sediments represented by fluvial sand and gravel Hagul Formation (Upper Miocene), which is underlain by Hammath Formation (marine fossiliferous limestone with sandy layer) and Sadat Formation (white limestone with marl) [1] [2] [3].

Generally, the Ismailia area, including the New Ismailia City, is dissected by many structural lineaments of different directions; and are mostly in two directions: NNW-SSE and NW-SE. The NW-SE-oriented set is related to the tectonics of the Gulf of Suez rift. A third minor set of lineaments present is running nearly E-W with small deviations to WNW or ENE directions [2]. However, the lineaments are rare or difficult to be observed in locations showing Quaternary sandy or clayey cover

* Corresponding author.
Email address: larasalah64@gmail.com (Lara Salah)

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such as the New Ismailia City, except for some short lineaments striking N-S, NE-SW, E-W and WNW-ESE.

Concerning geomorphology, the study area is nearly flat with ripple marks, and covered by an extensive sedimentary accumulation, alluvial deposits ranging from Mid-Tertiary (Upper Miocene, Oligocene) to Quaternary age [2]. Two main geomorphic units could be distinguished; these are sand dunes and gravel plains. The sand dunes are N-S-trending linear dunes and barchans of variable sizes and orientations at the area to the east of the Great Bitter Lakes [4]. The gravel plains cover the majority of Ismailia area, and encompass gravels and various sandy, limy and clayey rocks fragments [5]. Furthermore, the ERTS-1 satellite images revealed several other geomorphic units, e.g., wadi alluvium of limy, marly and gypsiferous clays; wadi alluvium of clayey and sandy gravels; sand dunes; marshes and sabkhas; shallow water bodies with seasonal variations: lakes and ponds [2].

The study area is experiencing several geotechnical problems such as lower bearing capacity and reasonable subsidence rate. Hence, a suitable geotechnical approach, both in field and laboratory, should be applied to disclose the appropriateness of the study area for construction and pavement purposes, and suggesting suitable recommendation to solve such geotechnical difficulties.

Generally, the soil in the Suez Canal region exhibits many geotechnical problems, such as the harmful impact on construction and agriculture lands, subsidence caused by sabkha deposits [2]. The presumed tunnel at El-Qersh locality, should designed to sustain a lithostatic stress of 1.2MPa. Moreover, the deep foundation of the El-Ferdan Bridge should be executed using the Bored Piles to a depth great enough to penetrate the high plasticity (CH) clayey lenses [6]. Seven zones have been discovered that are threatened by ground instability, where an average subsidence rate of about –2.7 mm/year was documented [7]. One of these threatened zones is the New Ismailia City. These zones are susceptible to ground instability and are characterized by shallow groundwater level, shallow depth of restriction clay layer, and high swelling potential.

Based on these previously mentioned works, several geotechnical problems are discovered, and hence, more investigations should be carried out at the study area to get more geotechnical information enabling the assessment of the suitability of soil for construction and pavement purposes.

The aim of the current study has been achieved through a comprehensive field and laboratory testing program, X-ray diffraction investigating of clay, analyzing and interpreting the attained result, and then suggesting recommendations for the establishment of construction and pavement purposes.

2. Materials and Methods

The soil samples are both undisturbed and disturbed which collected from thirteen boreholes for the geotechnical and chemical analyses. Four boreholes are present in District no.3; these are: BH-1, BH-2, BH-3 and BH-4 (Figure 3). The subsurface soil profile is shown in Figure 4. Nine boreholes are present in District no.4; these are BH-01, BH-02, BH-3, BH-04, BH-5, BH-6, BH-7, BH-8 and BH-9 (Figure 5). The subsurface soil profile is shown in Figure 6. All samples that were taken from the High-class District are disturbed samples.

The geotechnical field and laboratory testing program was carried according the Egyptian Codes and the ASTM standards. A brief description of these tests is as follows:

2.1. Mechanical Analysis of Soil

Mechanical analysis is the determination of the size range of particles present in a soil, expressed as a percentage of the total dry weight. It includes two techniques; sieve analysis (dry) and hydrometer analysis (wet) [8].

2.2. Moisture Content

The water (or moisture) content is defined as the ratio of the mass of the pore water to the mass of soil solids particles. The test was carried according to [9] ASTM-D2216 (2019).

2.3. Absorption Test

Water absorption is used to determine the amount of water absorbed. The test follows the instructions of [10] ASTM-D570 (1998).

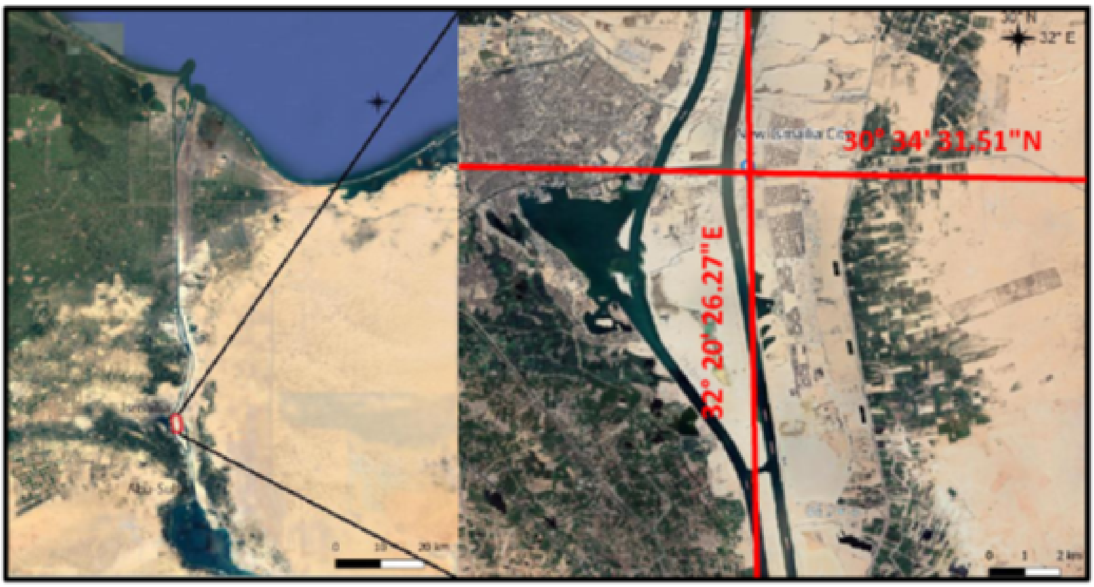

Figure 1: Google Earth image showing the location of theNew Ismailia City

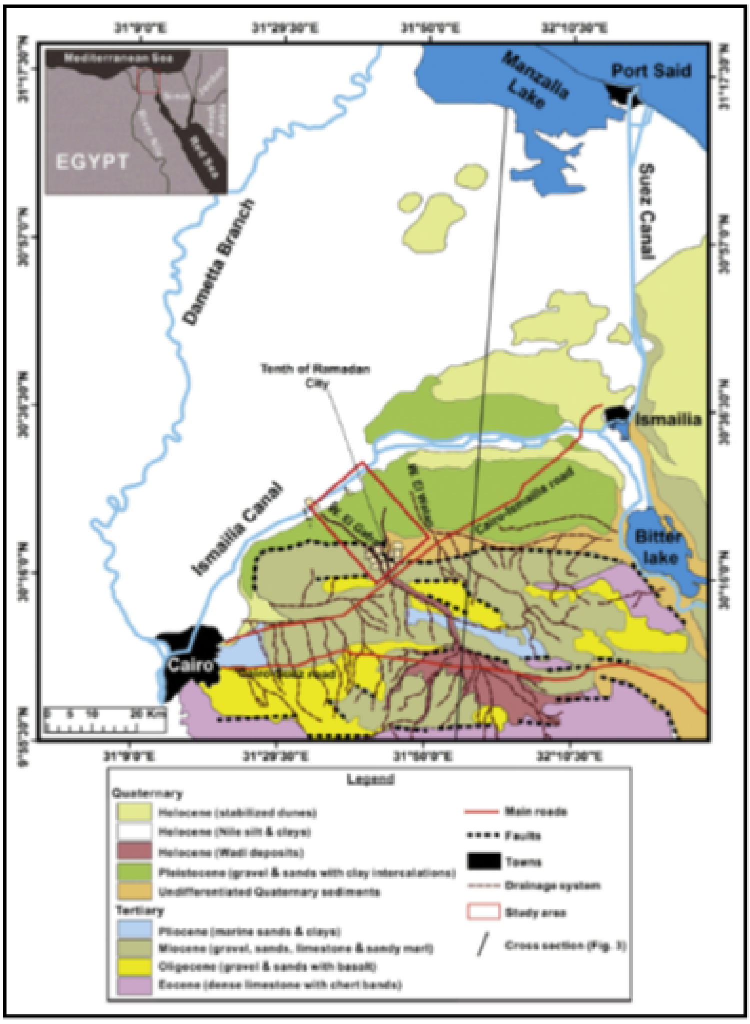

Figure 2: Regional Geological map of the Suez CanalRegion, including the study area [1-3]

2.4. Atterberg Limits

2.4.1. Liquid Limit (LL

The liquid limit is a water content marks the boundary between plastic and fluid-like behavior. This test is carried out according to [11] ASTM-D4318 (2000)

2.4.2. Plastic Limit (PL

The plastic limit is defined as the moist content, in percent, at which the soil crumbles when rolled into threads of 4.2mm diameter. The procedure for the plastic limit test is conducted according to the [11] ASTM-D4318 (2000).

2.5. Modified Proctor Test

The Proctor compaction test is generally used to obtain the maximum dry unit weight of compaction and the optimum moisture content. The procedure for the test is applied according to [12] ASTM D1557 (2021).

2.6. Sand Cone Test

This method used to determine the in-place density and unit weight of soils using a sand cone apparatus, this test is carried out according to [13] ASTM- D1556 (1996).

2.7. California Bearing Ratio (CBR Test

California Bearing Ratio (CBR) test is commonly used to determine the suitability of a soil as a subgrade or subbase for highway and runway design and construction. The procedure for the test is applied according to [14] ASTM D1883 (2021)

2.8. Plate Load Test

The plate loading test is a semi-direct method to evaluate the ultimate bearing pressure of soil to exhibit a given amount of settlement. The plates has dimensions: 40*40*2.5cm (district no.3) and 30*30*2.5cm (district no.4) are used for the test. The load on the plate is applied by a hydraulic jack. The reaction of the jack load is taken by a truck. The settlement of the plate is measured by three dial gauges of sensitivity 0.01 mm placed 90° apart. The dial gauges are fixed to an independent support which remains undisturbed during the test. The test is employed essentially according to [15] Egyptian Code 202 (2001).

2.9. Los Angeles Abrasion Test

The percentage wear of the aggregates due to rubbing with steel balls is determined and is known as Los Angeles Abrasion Value. This test is carried out according to [16] AASHTO-T96 (2002).

2.10. X-Ray Diffraction (XRD Investigation

The mineralogy of studied samples provides the basis for understanding their geotechnical behavior. It also helps to identify types of clay minerals and dominated minerals in the soil so we separated clay from the samples. This analysis was carried out in Geology Department, Faculty of Science, Suez Canal University using a Philips PW1370 X-ray generator fitted with a PW 1390 channel control, a PW1050 vertical goniometer and a digitizer

2.11. Chemical Analysis of Soil Samples

The chemical analyses include pH Measurement, determination of Chloride content, determination of sulfate content, and total dissolved solids (TDS) according to the [17] British Standard (BS) 1377-3 (1990).

3. Results and discussion

3.1. District no 3

Generally, the subsurface soil profile at District no.3 encompasses two layers; the topmost backfill layer and the lower sand layer (Figure 4). The topmost backfill (3m thick) is only encountered in BH-2, and is missed in the other boreholes. It is composed essentially of sand, clay, traces of lime materials, and clayey silt.

The lower sand layer is composed of coarse to medium-grained sand (8-15m thick), in addition to some clay and traces of lime materials. Sometimes, it contains lenses of variable thickness and composition. The groundwater level is encountered at depth ranging from 6m to 7m, which seems to be shallow, and must be taken into consideration during the construction. As an example of the results attained in District no.3, the results of boreholes BH-1 & BH-2 are shown in Tables 1&2.

Generally, the lower sand layer is exposed at the ground surface, but sometimes it is covered by the backfill layer. Based on the Unified Soil Classification System (USCS), the coefficient values (C_u: 2.3-5.5, and C_c values range from 0.18 to 2.3) indicate a soil of poorly-graded sand (SP) ($C_u \leq 6$ and 1 > Cc > 3).

According to AAHSTO system, the lower sand layer is of type A-1-b, which is excellent to good subgrade for pavement purposes. Based on SPT values (27 - >50 blows /cm), the lower sand layer is considered medium to very high dense (Table 3). The equivalent shear strength angle (Φ°) values are ranging from 36 to >40°, which is reasonable value. The SPT values are increasing with depth, for example in BH-1, the recorded SPT value at depth 4m is 27 blows /cm (medium dense), whereas at depth 8m, it reaches a value of 37 blows /cm (high dense).

This increase could be attributed to the load resulted from the weight of the overburden soil layers. The hydraulic conductivity (k) = 0.003-0.008 cm/sec, which is equivalent to that of fine sand (Table 4), and reflecting an intermediate draining condition. This condition must be improved by replacing the soil of high draining condition.

With respect to the lenses, a clayey sand lens (0.2m thick) is encountered in BH-1 at a depth of about 0.5m, a clayey silt lens (0.2m thick) is encountered at a depth of about 0.75m, and limestone fragments lens (0.4m thick) is at a depth 3m. Also, a lens of cemented gravel (0.8m thick) is exposed at the ground surface in BH-3.

Another sandy lime silt lens (ML: low plasticity) is encountered in BH-4 at a depth 1m and has a thickness of about 1.5m.This lens has a low swelling potential, where the liquid limit (LL) and the plasticity index (PI) are 32.1% and 4.3%, respectively (Table 5), and is comparable to that of the kaolinite clay mineral (Table 6). It seems that the majority of these lenses are encountered at shallow depths, so it is recommended to replace them using non-cohesive soils with percent of fines not exceeding 10%, and soil type A-1-a or A-1-b according to AAHSTO system.

3.2. District no 4

Generally, the subsurface soil profile encompasses the upper and sand layers, in addition to the backfill layer (Figure 4). The backfill layer (1-4m thick) is exposed at the ground surface at some localities and it consists of sand with clayey silt and traces of lime which represents the products of drilling and dredging process in the Suez Canal pass-way. As an example, the attained results of BH-1 are shown in Table 7.

The upper sand layer is poorly-graded sand (SP) with calcareous material, which has a thickness ranging from 4m to 11m. The layer is exposed at the surface in BH-5, but it is covered by the backfill layer in BH-8 where it is encountered at a depth of about 4m, whereas it is missed in other boreholes. Based on the classification of Unified Soil Classification System (USCS), the values of coefficients (C_u: 2.61-4.53 and C_c: 1.06-2.75) are related to the poorly-graded sand soil (SP) ($C_u \leq 6$ and 1 > Cc > 3). According to AAHSTO system, the layer is A-3 type, and according to the [15], it is considered medium to very dense, where the SPT values (30 - >50 blows /30 cm), and the equivalent relative density equal to 65- >85%, which is very high (Table 3). The friction angle (Φ°) values are ranging from 36->40°, that equivalent to that of dense to very dense sand (Table 3). The SPT values are displaying increase with depth, for example in BH-8, the SPT value is 49 blows /30 cm (dense) at depth 6m, whereas it reaches >50 blows /30cm (very dense) at depth 8m.

This increase could be attributed to the load resulted from the weight of the overburden layers. The hydraulic conductivity (k) = 0.022-0.048 cm/sec that refer the soil type is equivalent to coarse sand (Table 4), indicating a good draining condition of soil.

On the other hand, the lower sand layer (11-15m thick) is poorly-graded sand (SP) and contains several lenses. The layer is exposed at the surface in BH-3, BH-4, BH-6 & BH-7, but sometimes it is covered by the backfill layer (BH-1, BH-2 &BH-9) where it is encountered at a depth ranging from 1m to 3m. Based on the Unified Soil Classification System (USCS), the coefficients (C_u: 2.61-5.33, and C_c: 1.06-2.75) indicating poorly-graded sand soil (SP) ($C_u \leq 6$ and 1 > C_c > 3).

According to AASHTO classification system, the soil type is A-1-b to A-3 (percent passing No. 10

Table 1: The results of field and laboratory geotechnical tests of Borehole (BH-1), District (3)

Layer	**Sample No**	**Sample Depth (m)**	**Layer Depth (m)**	**Layer thick-ness (m)**	**SPT (N/ 30cm)**	**Bulk density γ (t/m^3)**		**Shear strength angle ($\Phi°$)**
						$D_r = \frac{e_{max}-e}{e_{max}-e_{min}}$	**Description**	
Lower sand layer (Layer no.2)**:** poorly-graded sand, coarse to medium-grained contains several lenses of silt, clay and clayey sand and traces of lime fragments.	1	1 2 3	1	15	35	0.65 – 0.85	Dense	36-40°
	2	4 5			42	0.65 – 0.85	Dense	36-40°
	3	6 7			>50	>0.85	Very dense	>40°

Table 2: The results of field and laboratory geotechnical tests of Borehole (BH-2), District (3).

Layer	**Sample No**	**Sample Depth (m)**	**Layer Depth (m)**	**Layer thick-ness (m)**	**SPT (N/30 cm)**	**Bulk density γ (t/m**		**Shear strength an-gleΦ °**
						Dr $= \frac{e_{max}-e}{e_{max}-e_{min}}$	**Descri-ption**	
Backfill Layer: is composed of sand and clay, with traces of lime materials and fine gravels.		- - -	0	3	-	-	-	-
Lower sand Layer (Layer no.2): is composed of poorly-graded sand coarse to medium, with some clay and traces of lime materials.	1	4 5	3	12	27	0.35 - 0.65	medium	32-36°
	2	6 7			34	0.65 – 0.85	dense	36-40°
	3	8 9			37	0.65 – 0.85	dense	36-40°

Table 3: SPT values, relative density, and equivalent shear strength angle of sandy soil [15]

SPT N/30cm	**Relative density**		**Shear strengthangleΦ °**
	Description	**$D_r = \frac{e_{max}-e}{e_{max}-e_{min}}$**	
0-4	Very loose	0 - 0.15	27-30°
4-10	loose	0.15 – 0.35	30-32°
10-30	medium	0.35 – 0.65	32-36°
30-50	dense	0.65 – 0.85	36-40°
>50	Very dense	>0.85	>40°

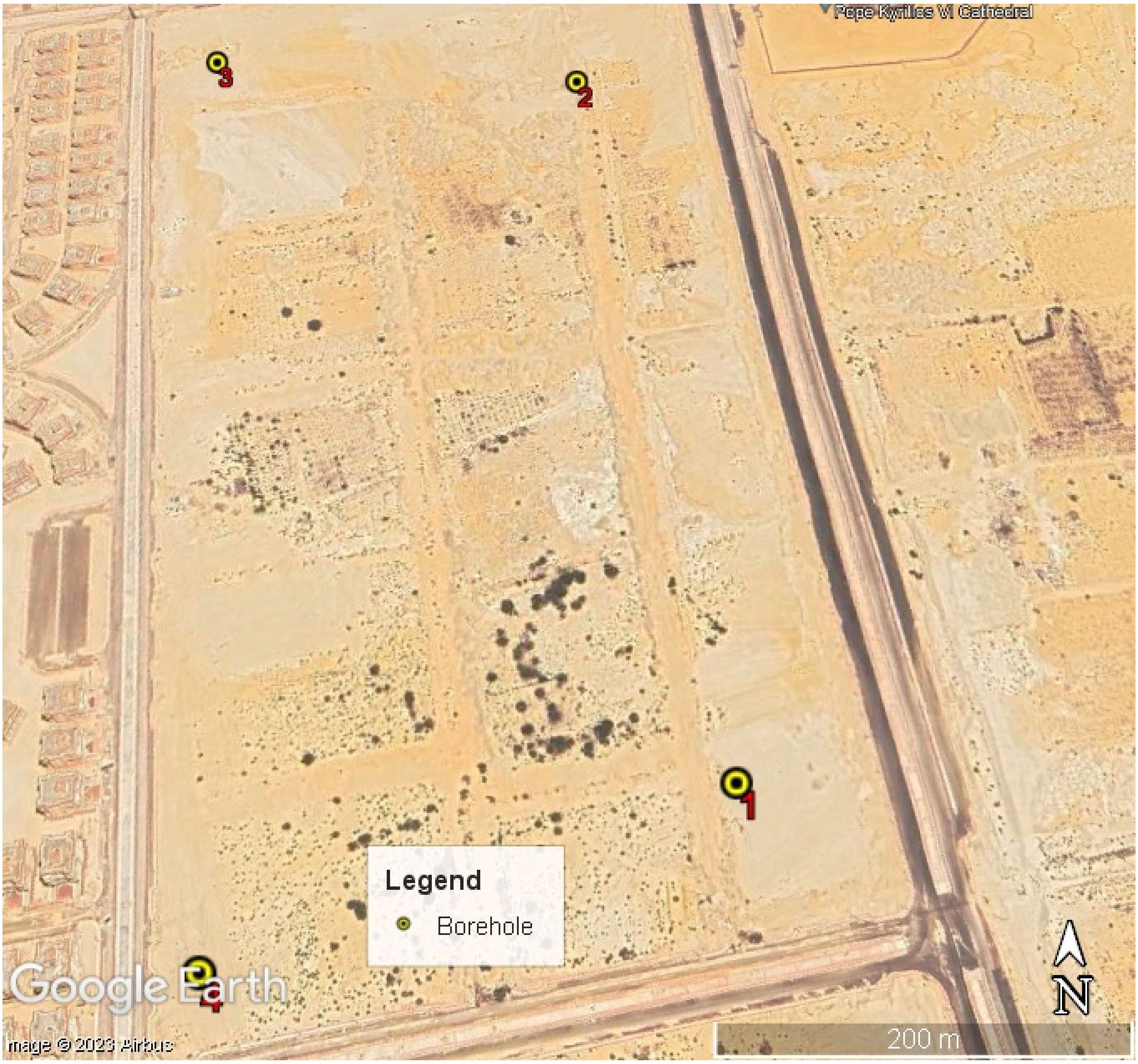

Figure 3: The location of boreholes at District no.3.

Table 4: Typical values of hydraulic conductivity of saturated soils [8].

Soil Type	Hydraulic conductivity (cm/sec)
Clean gravel	100 -1.0
Coarse sand	1 – 0.01
Fine sand	0.01– 0.001
Silty clay	0.001- 0.00001
Clay	< 0.000001

Table 5: Swelling potential of soil based on Atterberg Limits [18].

PI (%)	<2 μ m (%)	<74 μ m (%)	LL%	Swelling Potential
>35	>95	>95	>60	Very high
22-35	60-95	60-95	40-60	High
18-22	30-60	30-60	30-40	Moderate
<18	<30	<30	<30	Low

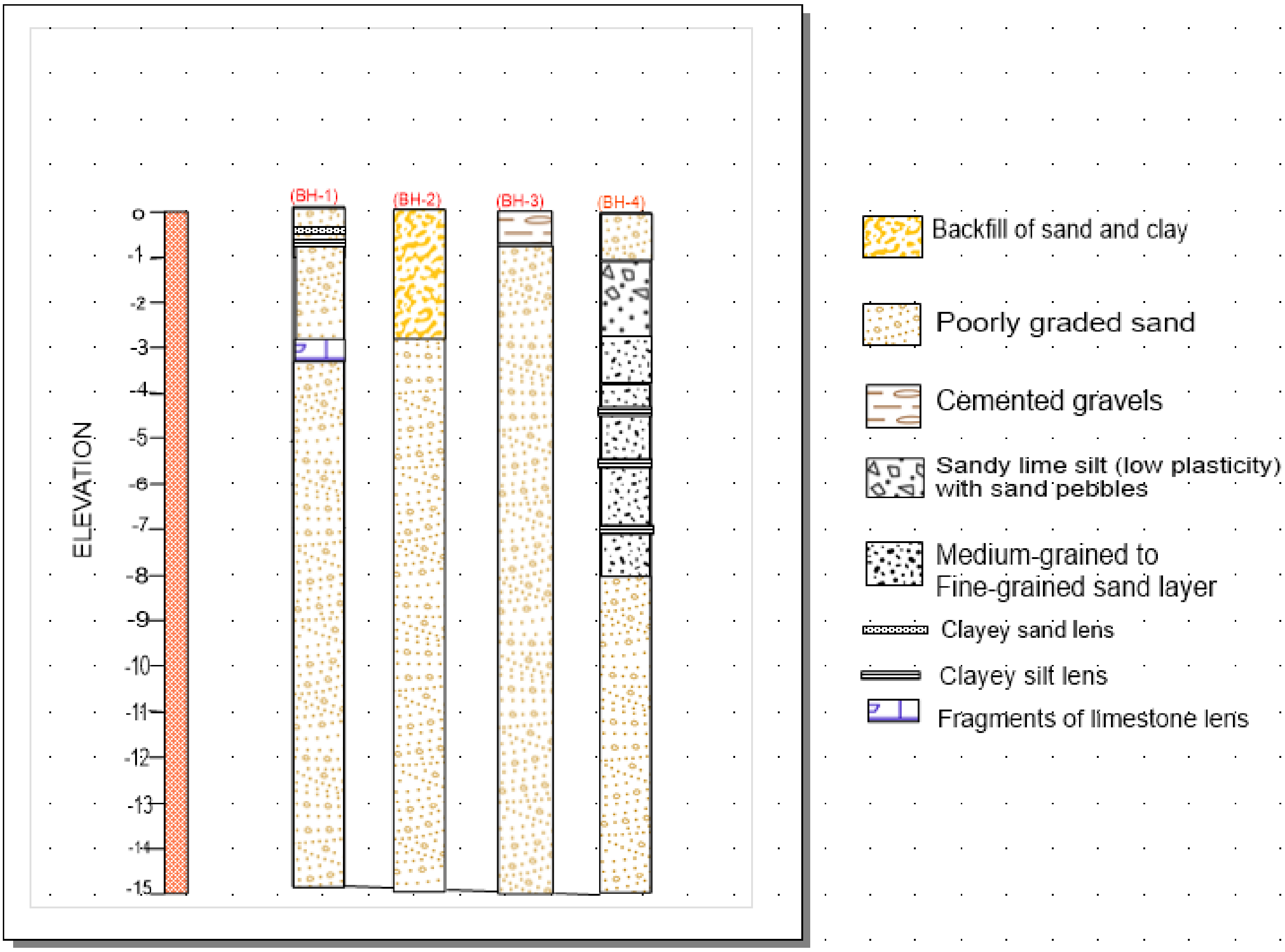

Figure 4: The four boreholes of District no.3, showing the subsurface soil sequence.

Table 6: Typical values of liquid limit, plastic limit and activity of some clay minerals [8].

Clay Mineral Type	Liquid Limit (LL)	Plastic Limit (PL)	Activity (A)
Kaolinite	35-100	20-40	0.3-0.5
Illite	60-120	35-60	0.5-1.2
Montmorillonite	100-900	50-100	1.5-7.0
Halloysite (hydrated)	50-70	40-60	0.1-0.2
Halloysite (dehydrated)	40-55	30-45	0.4-0.6
Attapulgite	150-250	100-125	0.4-1.3
Allophane	200-250	120-150	0.4-1.3

sieve is 95%, No. 40 sieve is 60%, and No. 200 sieve is 5%). Based on the [15], the layer is considered to be medium to very dense sand soil, where the SPT values is ranging from 30 to >50 blows /30cm, and the equivalent relative density equal to 65- >85%, which is very high (Table 3). Also, the friction angle (Φ°) values are ranging from 36->40°, that equivalent to that of dense to very dense sand (Table 3). The SPT values are displaying an increase with depth, for example in BH-1, the SPT value is 30 blows /30 cm (medium dense) at depth 4m, whereas it reaches 43 blows /30cm (high dense) at depth 10m. The increase in SPT values could be attributed to the load resulted from the weight of the overburden layers. The hydraulic conductivity (k) = 0.017-0.026 cm/sec that refer the soil type is equivalent to that of coarse sand soil (Table 4), indicating

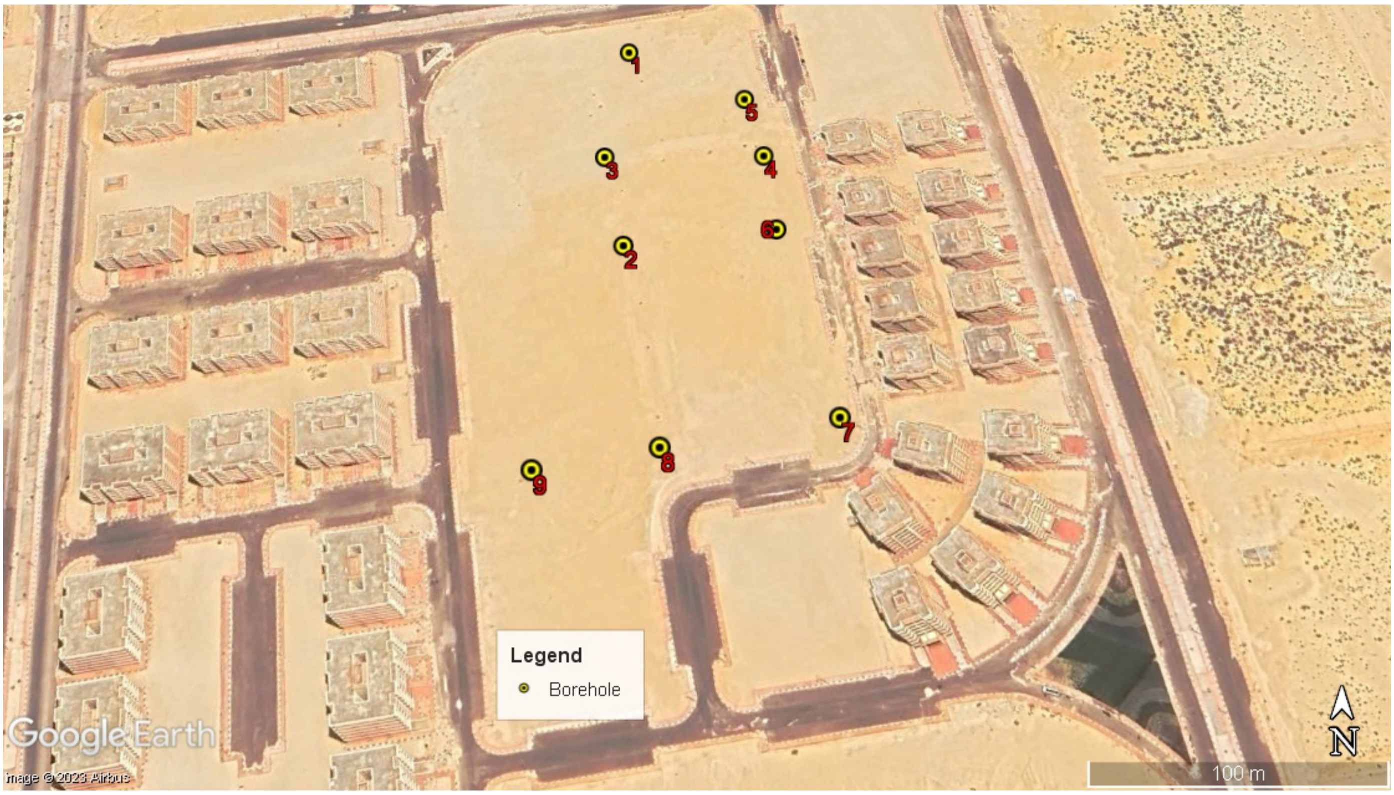

Figure 5: The location of boreholes at District no.4.

good draining condition of soil.

With reference to the lenses, a clayey silt lens (with traces of iron oxides and gypsum crystals) is encountered in BH-1, BH-2 & BH-3 at a depth ranging for 4m to 6m, and has a thickness ranging from 1m to 2m. The lens has a moderate swelling potential (plasticity index (PI): 19.3%), (Table 5), which is comparable to that of the kaolinite clay mineral (Table 6).

A second lens of clayey lime is encountered at a depth 4m, (PI: 19.4%), indicating a soil of moderate swelling potential (Table 5), which is comparable to that of the kaolinite clay mineral (Table 6). A fourth lens of medium-grained sand to silt with traces of lime materials (2m thick) is encountered in BH-4 at a depth 4m. The lens is considered to be dense sand (SPT: 48 blows/cm) and has high relative density is 65-85% (Table 3).

A fifth lens of poorly-graded sand and lime materials (2m thick) with traces of iron oxides is encountered in BH-5 at a depth 4m. The Shear strength angle (Φ°) values from 36->40° that equivalent to that of dense to very dense sand (Table 3). The lens has a low swelling potential, where the plasticity index (PI) is 12.6%, (Table 5).

The plate load test was carried out using successive loading stresses (0.5, 1.0, 1.5, 2.0, 2.5 and 3.0 Kg/cm^2), and for plate load diameter 30cm. As an example, the calculations and results at points 1&2 for District no.4 are listed in Tables (8 &9) and the relationship between the stresses and the corresponding subsidence, point (1, 2) District.4 is shown in Figures (7, 8, 9, & 10). The results show that the ultimate bearing capacity is 1.0 kg/cm^2 (98.66kpa) with corresponding settlement 0.46mm for plate load diameter 30cm for point (1) and the ultimate bearing capacity of soil is 1.5 kg/cm^2 (147.1kpa) with corresponding settlement 1.80 mm for point (2). To calculate the allowable (maximum) bearing capacity of foundation the following formula is applied:

$$q_{ult\,f} = q_{ult\,p} * (B_f/B_p) \qquad (1)$$

Point no.1 = 1* (1/0.3) = 3.33 kg/cm^2 (326.5614kpa)

Point no.2 = 1.5* (1/0.3) = 5 kg/cm^2 (490.3324kpa)

Table 7: The results of field and laboratory geotechnical tests of Borehole (BH-1), District (4).

Layer	Sample No	Sample Depth (m)	Layer Depth (m)	Layer thickness (m)	SPT (N/30 cm)	Bulk density $\gamma(\mathbf{t/m^3})$ Dr= $\frac{e_{max}-e}{e_{max}-e_{min}}$	Bulk density $\gamma(\mathbf{t/m^3})$ Description	Soil Type AASHTO	Soil Type USCS	Shear strength angle Φ °	LL%	W%	PL%	SL%	PI%
Backfill Layer: is composed of Backfill of sand, some clay and lime traces, clayey silt.		- - -	0	1		-	-				-	-	-	-	-
Lower sand Layer (layer no.2): is composed of poorly graded sand to medium, some clay and traces of lime materials.	1	4	1	4	30 38	0.65 – 0.85 0.65 – 0.85	dense dense	A-3	SP	36-40 36-40	-	-	-	-	-
The clayey silt lens 1: is composed of clayey silt and lime (medium plasticity cohesion), traces of iron oxides and gypsum crystals (brown-gray).	2	5	5	1	-	-	-				38.2	38.5	19.3	14.5	19.2
lens 2: with fragments of clayey silty sand Stone and lime materials.		6	6	1	-	-	-				-	-	-	-	-
Lower sand Layer (layer no.2): is composed of Poorly graded sand to medium, some clay, and traces of lime materials.	3	7 8 9 10 11 12	8	7	43	0.65 – 0.85	dense	A-1-b	SP						

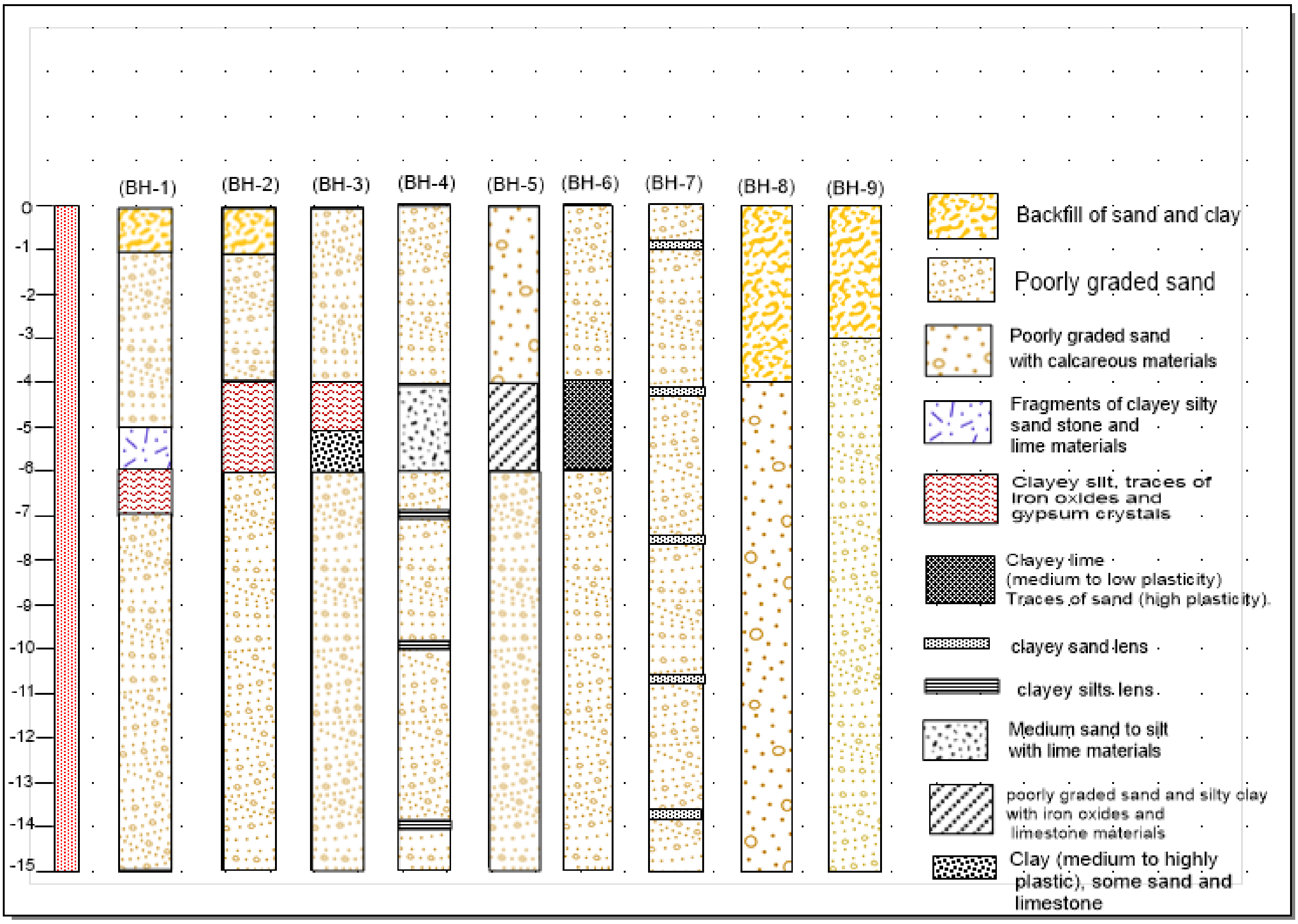

Figure 6: The nine boreholes of District no.4, showing the subsurface soil sequence.

where: $\mathbf{q_{ult}}\ _p$ is ultimate bearing capacity of soil, $\mathbf{q}_{ultf}$ is ultimate bearing capacity of foundation, $\mathbf{B_f}$ is the width of foundation (not less than 1m), and $\mathbf{B_p}$ is diameter of plate used. Based on the [15], (Table 10), and [17], (Table 11), the attained ultimate bearing capacity values of foundation 3.33 kg/cm^2 - 5 kg/cm^2 (326.5614kpa - 490.3324kpa), are laying within the allowable range of the bearing capacity, and hence the soils are suitable for both construction and pavement purposes.

Five modified Proctor testes were carried out in District no.4 for construction purposes with five attempts for each test (Table 12). As an example, the resulted modified Proctor curve for test. No.4 is shown in Figure (11) with Maximum dry density γ_{dmax} = 1.82 g/cm^3, while the optimum moisture content W% (opt) = 10.20%.

Based on the modified Proctor test results, the compaction achieved a reasonable value of maximum dry density (1.68-2.08 g/cm^3) and corresponding optimum moisture contents range is 6.60%-10.3%. These has been taken into consideration when the compaction work is progressing in the field, to know whether the specified unit weight has been achieved or not. According to the results of modified Proctor test, the back filling should be compacted to 100% of maximum dry density; 2.1g/cm^3 and optimum moisture content 6.60% to be suitable for foundation purposes. This has been achieved in the field, where the sand cone tests gave an acceptable compaction percent that ranging from 101.5 to 106.46% (Table 13). According to [19], the achieved dry density must not less than 95% of the dry density determined by modified Proctor test, hence the attained results are acceptable.

The results of the absorption ratio with water after 24 hours = 2.3 %, according to [19], the upper limit of adsorption ratio is 10%, indicating that the attained result is acceptable for pavement purpose.

The soil samples at District no.4, have concentration of total dissolved salts (TDS) (450 - 1122 ppm), sulfates (SO_3) (95- 250 ppm) and chlorides (CL) (70 - 420 ppm) (Table 14). According to the [15], the soil sample are considered non-aggressive soil. Also, the pH values are ranging from 8.95 to 9.5 which are alkaline soil. According to the Egyptian Code of Concrete [20], the pH value of soil of District no.4 has no danger on concrete.

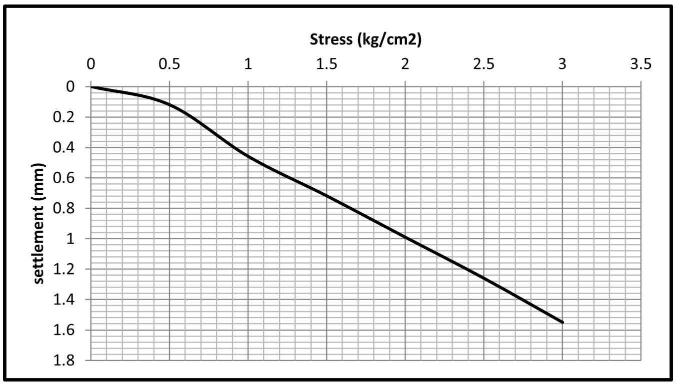

Figure 7: Showing the relationship between the stresses and the corresponding subsidence, point no.1, District.4.

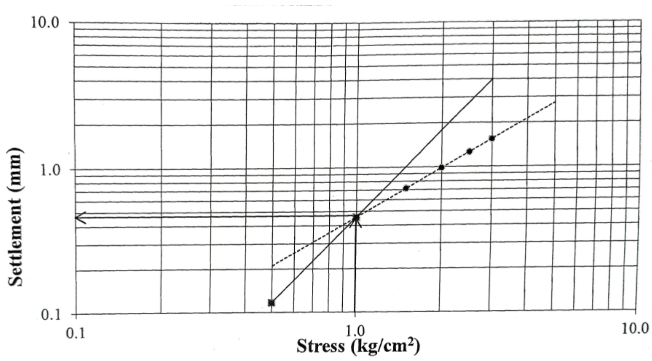

Figure 8: The logarithmic relationship between the vertical stresses and the corresponding settlement, point no.1, District no.4.

Several clayey and silty lenses are encountered in District No.4; hence, it is suitable to replace the soil to a depth of 4-7m. The backfilling should be carried out using a mixture of sand and gravel, non-cohesive with percent of fines not exceeds 10%, and soil type A-1-a or A-1-b. Finally, the soil must

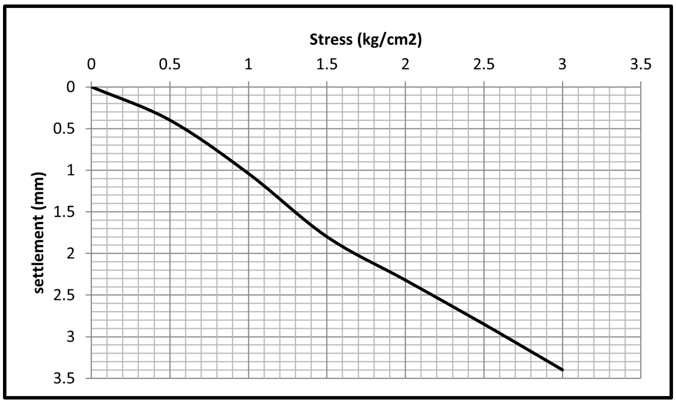

Figure 9: The loading-settlement curve, point no.2, District no.4.

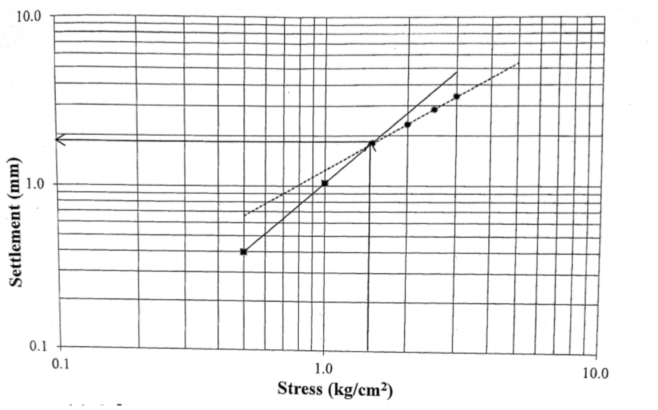

Figure 10: The logarithmic relationship between the vertical stresses and the corresponding subsidence, point no.2, District no.4.

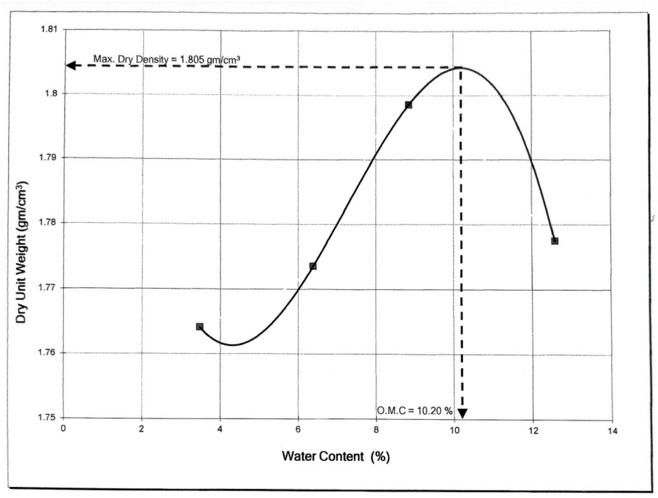

Figure 11: The compaction curve, modified Proctor test no.4, District no.4.

Table 8: Showing load stresses values and corresponding settlement, point (1) District.4

Stress on plate (kg/cm^2)	Plate vertical settlement (mm)			Average settlement (mm)
	Dail No.1	Dail No.2	Dail No.3	
0.00	0.00	0.00	0.00	0.00
0.50	0.195	0.105	0.05	0.12
1.00	0.785	0.425	0.16	0.46
1.50	1.18	0.65	0.32	0.72
2.00	1.555	0.865	0.535	0.99
2.50	1.91	1.09	0.785	1.26
3.00	2.26	1.31	1.075	1.55

Table 9: The results of load stresses and the corresponding settlement, point (2), District no.4.

Stress on plate (kg/cm^2)	Plate vertical settlement (mm)			Average settlement (mm)
	Dail No.1	Dail No.2	Dail No.3	
0.00	0.00	0.00	0.00	0.00
0.50	0.355	0.51	0.33	0.40
1.00	0.95	1.25	0.915	1.04
1.50	1.635	2.145	1.63	1.80
2.00	2.085	2.76	2.11	2.32
2.50	2.555	3.385	2.6	2.85
3.00	3.035	4.015	3.135	3.40

Table 10: Allowable bearing capacity values for soils (in dry condition) according to [15].

Soil Type	Description	Allowable bearing capacity (kPa)	
Gravel or mixture of gravel and sand	High compacted	490.3-686.5	Width of foundation not less than 1 m.
	Medium compacted	392.3- 588.4	
	loose	196.1-392.3	
Coarse to medium sand or mixture of sand with little gravel	Very dense	294.2-490.3	
	Medium to dense	147.1-294.2	
	loose	98.06-196.1	
Fine to medium sand or clayey or silty sand	Very dense	196.1-392.3	
	Medium to dense	147.1-245.2	
	loose	98.06-147.1	

be compacted to achieve the required dry density and optimum moisture content.

3.3. High-class District

Generally, the subsurface soil sequence encompasses mainly three layers; the upper, the middle, and the lower sand layers, in addition to the topmost backfill layer (Figure 12). The backfill layer (2m thick) consists of sand and clayey silt with traces of lime materials and fine gravels. It represents the product of drilling and dredging of Suez Canal. The upper and lower layer has the same composition, which is poorly-graded sand (SP) with lime fragments. The middle layer, which separating the upper and lower layers is composed of silty sand with lime traces.

Table 11: Typical values of soil bearing capacity according to [17].

Soil type	Bearing value (kPa)	Remarks
Dense gravel or dense sand and ravel	> 600	Width of foundation not less than 1m. Water table at least at the depth equal to the width of foundation, below base of foundation.
Dense gravel or medium dense sand and gravel	200-600	-
Loose gravel or loose sand and gravel	< 200	-
Compact sand	> 300	-
Medium dense sand	100 - 300	-
Very stiff boulder clays and hard clays	300 - 600	Susceptible to long term consolidation settlement
Stiff clays	150 - 300	-
Firm clays	75 -150	-
Soft clays and silts	< 75	-

Table 12: The results of Modified Proctor tests, District no.4.

Testes No.	Maximum dry density (g/cm^3)	Optimum moisture content (%)
1	2.08	6.6%
2	2.03	10.3%
3	1.74	9.4%
4	1.82	10.2%
5	1.68	9.2%

Table 13: The results of Sand Cone Test no.1, District no.4.

Number of samples points	Water content (%)	Density in site (gm/cm^3)	Dry density (gm/cm^3)	Correction factor for maximum dry density (according to the ratio of gravel greater than 25.4 mm)	Compaction ratio (%)
1	4.42	2.19	2.10	1.000	115.3
2	4.37	2.18	2.09	1.000	115.3
3	4.21	2.23	2.14	1.000	106.46
4	4.30	2.21	2.12	1.000	115.3
5	4.30	2.14	2.05	1.000	112.6

Table 14: The degree of aggressive for the soil and ground water [15].

Degree of Aggressive	Highly aggressive		Aggressive		Moderately aggressive		Non-aggressive	
	G.W (ppm)	Soil (ppm)	G.W (ppm)	Soil (ppm)	G.W (ppm)	Soil (ppm)	G.W (ppm)	Soil (ppm)
SO_3	> 5000	> 20,000	1000-5000	5000-20,000	300 -1000	1000-5000	< 300	< 1000
Cl	> 2000 ppm		1000-2000 ppm.		300-1000 ppm.		< 300 ppm	
pH value	< 4.5		5-6		6-7		7-8	

Generally, the upper sand layer (2 - 2.5m thick) is exposed at the ground surface, but sometimes it is covered the backfill layer and is encountered at a depth 0.5 - 2m. It is overlain by a thin layer (0.5m thick) which is exposed at the ground surface and is composed of gypsum and limestone materials. The middle layer (5m thick) is composed of silty sand with lime traces and is encountered at a depth of about 3m. The lower layer is composed of poorly-graded sand and lime fragments (depth: 8m) with thickness 4m.

Figure 12: A field photograph showing the soil sequence at the High-class District.

The results of field and laboratory tests are shown in Table 15. Based on the Unified Soil Classification System (USCS), the coefficient ranges (C_u: 1.47-2.78, and C_c: 0.94-1.20) indicating poorly-graded sand (SP) soil ($C_u \leq 4$ and $1 > Cc > 3$). The hydraulic conductivity (k) = 0.0169-0.026 cm/sec, so, the soil type is equivalent to that of coarse sand (Table 4). According to AAHSTO system, the layer is A-3.

High moisture within the pavement system has harmful effects on pavement performance, where it reduces the strength and stiffness of the pavement foundation materials, promotes contamination of coarse granular material due to fines migration, and can cause swelling and subsequent consolidation [21]. Hence the range of moisture content (0.7 - 3.5%) is acceptable and has no dangerous effects on soil performance.

Based on interpretation of the X-ray diffraction two samples of (medium to fine sand (powder samples) with some clay (separate -2μm), the pattern by X'Pert High Score version 2 reveals that the layer consists of Quartz, Moganite, and the Gypsum (earth Rose). Based on the scheme of [22], the identified clay minerals are aluminum-rich montmorillonite, Kaolinite and illite. The montmorillonite clay mineral has the most expansive potential hazard, and it is regarded as very hydrophilic due to their mobile structure, making them highly expansive [23].

For soil swell–shrinking application, [24] defined four classes linking shrink–swell potential and montmorillonite content: low swelling potential (<10%), moderate swelling potential (between 10% and 50%), high swelling potential (between 50% and 70%) and very high swelling potential (> 70%). So, the results refer to moderate swelling potential [25].

Table 15: The results of field and laboratory geotechnicaltests, High-class District

Layer	Sample No	Sample Depth (m)	Layer Depth (m)	Layer thickness (m)	Silt Percent%	Clay percent%	W%	Soil Type AASHTO	Soil Type USCS
Backfill Layer is composed of Backfill of sand, some clayey silt and lime traces, the product of drilling and dredging of Suez Canal.	-	1	0	2	-	-	-	-	-
Sand Layer (Layer no.1): is composed of medium to fine sand, with clay and traces of lime materials	1	2	2	3	16	18.4	1.4	A-3	SP
capped with thin layer of gypsum and limestone materials (0.5m thick).	2	3			4.01	0.09	2.1	A-3	SP
The sand Layer (Medium to Fine sand) (Layer no.2):	3	5					3.5	A-3	SP
is composed of Medium to fine sand silty (cohesive)	4	6	5	2	-	-	0.7	A-3	SP
and traces of lime materials.	5	6					3.3	A-3	SP
Sand Layer (Layer no.3): It is similar to layer no.1 and is composed of poorly-graded sand. Medium to fine with clay and traces of lime materials.	6	7	7	4	-	-	2	A-3	SP

4. Conclusion

Based on the present geotechnical study, the following recommendations are suggested:

• The attained ultimate bearing capacity values of foundation 3.32 kg/cm^2 -5 kg/cm^2 (326.5kpa - 490.3kpa), are laying within the allowable range of the bearing capacity, and hence the soils are suitable for both construction and pavement purposes.

• The back filling, in the study area, should be carried out using a mixture of sand and gravel, non-cohesive with percent of fines not exceeds 10%, and soil type A-1-a or A-1-b, finally the soil must be compacted to achieve the required dry density and optimum moisture content. This is essential for constructing building consisting of five floor plus bed floor.

• The applied stress from foundation must not exceeds the allowable bearing capacity of soil to prevent shear failure in the soil, so the allowable load should not exceed 1.1kg/cm^2 - 1.6 kg/cm^2.

• According to the results of modified Proctor test, the backfilling should be compacted to 100% of maximum dry density; 2.1g/cm^3 and optimum moisture content 6.60% to be suitable for foundation purposes. This has been achieved in the field, where the sand cone tests, according to [19], gave an acceptable compaction percent ranging (101.5% to 106.46%).

• The admitted CBR value for sub-base and base materials are 30% and 80% minimum, respectively [14], hence, the soil of the study area is suitable as sub-base and base (CBR: 86.9%).

• The X-ray diffraction investigation reveals that the identified clay minerals are montmorillonite, kaolinite and illite. So, the soil layers which contain clayey lenses encountered at shallow depth must be removed, whereas the deeper ones must be fixed.

• The soil sample are considered non-aggressive soil. Also, the pH values are ranging from 8.95 to 9.5 which are alkaline soil. According to the Egyptian Code of Concrete [20], the pH value of soil of District no.4 has no danger on concrete.

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