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ORIGINAL PAPER

Determination of the Geotechnical Parameters of Soils Behavior for Safe Future Urban Development, Najran Area, Saudi Arabia: Implications for Settlements Mitigation

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Abstract This paper deals with the geotechnical characterization of Sultana soils (Najran region, Saudi Arabia) by in situ and laboratory tests, for a project proposal for the construction of commercial and government buildings. Geotechnical parameters of the subsoil were analyzed with a view to using the soil as an earth construction material and as a foundation for buildings constructed on the grounds tested. The site was investigated by means of boreholes, trial pits, standard penetration tests, plate load tests, Seismological data, particle size analyses, modified Proctor Tests, Oedometer and direct shear tests. The study contains full description to the subsurface soil conditions and provides recommendations for collapse potential, foundation type, foundation depth, allowable net bearing capacity, estimation for the expected settlement. In this paper a review of the principal results of the in situ and laboratory investigations is described. Results of the study revealed that within the maximum depth of 10.0 m of boring, the soil was found to be composed of brown soil mixture of sand and clayey silt with little gravel, classified as (SM) and (SC) according to USCS classification and as (A-2-4) and A-2-6 according to AASHTO classification. The area was found to consist in general of collapsible soils, classified as collapsible alluvial soils. The results show that the collapse potential of the majority of semi-arid soils tested were classified as severe trouble soils and natural moisture content is an important factor influencing the collapse potential of the soil. In addition, the results of this study provide the parameters to evaluate the soil behavior and the geotechnical model for the foundation bearing capacity and soil settlements evaluation.

Keywords Safe urbanization · Collapse potential · Foundation design · Laboratory tests · In-situ tests · Bearing capacity

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1 Introduction

Urban expansions in major rapidly growing cities such as Najran city with rapid population growth and significant socio-economic problems requires an in-depth knowledge and accurate assessment of the geotechnical hazards and variability of the geological and geotechnical properties of the soils upon which cities are built (Kolat et al. 2012; Donghee et al. 2012; Masoud and Abd El Aal 2017).

Geotechnical investigation varies from site to site depending on the nature of the project, substrata and available funds. Generally, it is carried out in two phases: Field exploration including in situ testing and Laboratory testing of samples retrieved during field investigations. Rapid growth of constructions in Najran region has brought increased focus on geotechnical parameters determination from in situ and laboratory tests for foundation design. Therefore, the geotechnical characterization by in situ and laboratory tests has a great importance because the majority of soils are considered as one of the zones of collapsible soils (Rouaiguia and Dahim 2016).

Research on geotechnical behavior of soils of semi-arid region in Saudi Arabia is relatively scarce in spite of increasing role in the important projects presently planned and under construction. Collapsing soils of the loessial type are reported to exist in several parts of the world, such as the USA, Central and South America, China, Africa, Russia, India and the Middle East (Murthy 2010). For example, in the United States of America, collapsible soils are a significant geologic hazard in Colorado and other Western States in semiarid to arid climates (White et al. 2008).

These soils are subject to rapid changes in volume and settlement in response to wetting, often resulting in severe damage to structures. These soils are strong if dry but collapse if wet, this process sometimes called hydro-collapse. The risk of constructing structures on collapsible soils presents significant challenges to geotechnical engineers due to sudden reduction in volume upon wetting.

Collapsible soils are composed of sand and silt particles in a loose honeycomb structure. It is well known that the bonding is possibly caused by cementation, chemical or physical attraction or negative pore water pressure (suctions). During the process of hydro-collapse the water destroys the cementing agents (clays and salts) which ensure the bonding between the particles of loose structure. This saturation eliminates the clay bonds holding the soil grains together (Mulvey 1992). Collapsible soils result in structural damage such as cracking of the foundation, floors, and walls in response to settlement. In one particular case of soil collapse, 14 houses in a Cedar City, Utah neighborhood had to be jacked off their foundations and relocated due to severe settlement (Rollins et al. 1992). Figure 1 shows the structural damage due to collapsible soils below the foundations in Najran area (Rouaiguia and Dahim 2016).

The identification of collapsible soils and estimation of the collapse potential are major components in appropriate engineering for these moisture-sensitive soil sites. Foundations that are constructed on collapsible soils may undergo large and sudden settlement when the soils become saturated from any unanticipated source of water such as damaged water pipelines, drainage from reservoirs, and rise of groundwater table.

Jennings and Knight (1975) recommended the using of stress level of 200 kPa and calculate the collapse potential according to the following equation:

$$\mathrm{CP} = \frac{\Delta \mathrm{e}}{1 + \mathrm{e}_0} \quad (1)$$

where CP = Collapse potential which is an indication of the degree of bulk volume change soils exhibit due to load and water infiltration. Δe = decrease in void ratio due to wetting and, e_0 = initial void ratio.

Jennings and Knight (1975) have suggested some values for collapse potential which are shown in Table 1.

The change volume of soils by swelling or shrinking, makes clays that are most likely over consolidated (Dhowian et al. 1985) significantly more hazardous for construction purposes, which can causes the subsidence of the land surface, differential settlements, and building collapse and wall cracks (Stavridakis 2006; Hyndman and Hyndman 2014). Also the volumetric changes in clay content are significant in arid and semi-arid areas like as Najran, justifying their close analysis to disclose possible geotechnical hazards. In Najran area, changes in soil water content are often brought about by local site changes, such as leakage from water supply pipes or drains, or can be associated with a pattern of short periods of rainfall followed by long dry periods resulting in seasonal cycles of soil swelling and shrinkage (Cameron 2006; Clayton et al. 2010).

The free swell index (FSI) is another important parameter for foundation design and engineering. It is an index property that was proposed by Holtz and Gibbs in 1956 and turned into an ASTM standard, ASTM D5890. It reflects also the potential for expansion of the soil by comparing the ratio of the volume of soil in water to the volume of soil in

Fig. 1 Structural damage due to collapsible soils below the foundations in Najran area (Rouaiguia and Dahim 2016)

Table 1 Properties of Najran tested soils of trial pit-01and borehole-02

	Moisture content (%)	Specific gravity	Bulk density (g/cm^3)	Dry density (g/cm^3)	Void ratio (e)	Porosity (n)	Degree of saturation S (%)
Trial pit-01 depth (m)							
1.0	4.4	2.64	1.41	1.35	0.86	0.46	13.66
2.0	4.9	2.70	1.60	1.48	0.81	0.45	16.13
2.5	6.1	2.68	1.65	1.49	0.79	0.44	20.43
BH-02 depth (m)							
1.5	6.4	2.65	1.64	1.42	0.82	0.46	20.68
3.0	7.8	2.66	1.61	1.42	0.77	0.46	26.94
4.5	8.5	2.69	1.67	1.50	0.74	0.44	30.89
7.5	9.8	2.70	1.7	1.54	0.72	0.43	36.75
10	10.2	2.75	1.69	1.58	0.68	0.42	41.25

kerosene. The expansion potential for the FSI is shown in Table 2. According to the values shown in Table 3, samples can be considered as low degree of expansion according to Holtz and Gibbs (1956), Chen (1988), and (IS 1498 1970).

These collapsible soil deposits have a low bearing capacity upon being wetted and may exhibit large settlements when subjected to loading. It is therefore inevitable to treat these soil deposits prior to construction activities in order to prevent differential

Table 2 Atterberg limits of Najran tested soils of trial pit-01and borehole-02

	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Liquidity index (%)	Consistency index (%)
Trial Pit-01 depth (m)					
1.0	17	10.5	6.5	− 0.93	1.93
2.0	17.5	11.3	6.2	− 1.03	2.03
2.5	19.2	12.0	7.2	− 0.81	1.82
BH-02 depth (m)					
1.5	17.7	10.4	7.3	− 0.23	1.23
3.0	18.4	10.5	7.9	− 0.51	1.52
4.5	20.9	11.6	9.3	− 0.33	1.33
7.5	21.5	12.1	9.4	− 0.15	1.15
10	23.7	12.8	10.9	− 0.16	1.16

Table 3 Mechanical analysis of Najran soils (trial pit-01)

	Sand (%)	Silt (%)	Clay (%)	Soil description	USCS	AASHTO
Trial pit-01 depth (m)						
1.0	67.3	27.4	4.9	Brown clayey silty sand	SM	A2-4
2.0	59.5	32.5	7.7	Brown clayey silty sand	SM	A2-4
2.5	57.40	33.9	8.4	Brown clayey silty sand	SM	A2-4
Borehole-02 depth (m)						
1.5	62.4	32.0	5.1	Brown gravely silty sand with some clay	SC	A2-6
3.0	55.2	38.1	6.3	Brown, fine to medium grained silty sand with some clay	SM	A2-6
4.5	51.6	39.5	8.4	Grey to brown, clayey silty sand	SM	A-4
7.5	49.7	42.2	7.6	Grey to brown, clayey silty sand	SM	A-4
10	43.1	46.3	9.8	Grey to brown, clayey silty sand	SM	A2-4

settlements and subsequently potential damages to structures.

Unraveling the accurate spatial geotechnical and geological constraints of these soils and their variability is therefore imperative when investigating land suitability for construction, and for land use management on limited soil resources. Such soil investigations can play an important role in helping to efficiently prioritize areas with cost effective optimization of construction times, efficient setting of hazard mitigation measures, and to design projects for safe extension with appropriate and reliable foundation systems to compensate for geotechnical risks as well as to avoid damage to infrastructural elements, which induces heave cracking and breaking of foundations, slabs, and walls (Dhowian et al. 1988; Parsons and Frost 2002; De Rienzo et al. 2008). However, current geotechnical tests that are widely performed to understand the potential problems of soils can be time-consuming, expensive, and limited.

In practice, when working with collapsible soils geotechnical engineers are faced with several considerable challenges. These include (Houston et al. 2001)

(a) Identification and characterization of collapsible soil sites.
(b) Estimation of the extent and degree of wetting.
(c) Estimation of collapse strains and collapse settlements.
(d) Selection of design and mitigation alternatives.

Foundations for different structures or pavements constructed on collapsible soils may experience sudden and large amounts of settlement.

Foundations for different structures or pavements constructed on collapsible soils may experience sudden and large amounts of settlement. In general, with regard to collapse potential and type of structural loading, the design engineer may consider various types of foundations in order to carry loads safely. Field tests conducted by Rollins et al. (1998) indicate that dynamic compaction treatment provides the most effective means of reducing the settlement of collapsible soils to tolerable limits.

Referring to the Saudi Building Code (SBC-301), the following seismic data could be considered: The proposed site is located at (Region 6), Spectral Response Accelerations (S_s = 0.15 g) and (S_1 = 0.045 g), and the soil profile type is classified as (Class-D). In addition, the shear wave velocity (Vs) is expected within the range (180–370 m/s) and the Liquefaction is not expected during earthquakes.

The scarce data available on geotechnical properties and geotechnical behavior of semiarid climate soils, urge for more research, mainly regarding the huge irrigation projects under planning and the expectation of heavy rainfalls which concentrated in short periods of time while the rest of the climate is characterized by long dry periods.

2 Description and Geology of the Study Area

Najran has several valleys, the most famous of them being Najran's valley. It is encircled by a range of rocky mountains, the highest of which is the 1450-m-high Abu Hamdan Mountain. The climate of Najran is hot during the summer months with an average value of 32 °C and mild during the winter months. Temperatures in winter drop to an average of 6 °C. It is rainy in the mountainous areas. The intensity of rain is about 52.9 mm/year in 2010 and it reached more than 274.2 mm/year during the year of 1996.

The site of the proposed project is located at Sultana (116)/NAJRAN Region as shown in Fig. 2, which is almost at the intersection of the longitude (45° + 00′) and the latitude (18° + 51.50′). During drilling, the natural relative densities of the sub-surface soils were investigated by the regular conduction of the Standard Penetration Test (SPT) each 1.50 m of drilling depths. At each run, the number of blows required to penetrate 15 cm through the soil was registered three times and the sum of blows for the last two penetration trials is the number (N) that could be used for defining the relative density of the cohesionless soils or the consistency of cohesive soils. The results were presented in the boreholes logs.

Field investigation programme was undertaken and from the collected soil samples, laboratory tests were performed. The site was geotechnically investigated via four (04) boreholes, of depth 10.00 m, and one test pits, from depths of 1.5 m, 2.0 m, and 2.5 m as shown in Fig. 3. Figure 4 shows the Poklain during excavation of trial Pit at proposed location in Sultana.

The geology of the ground needs to be carefully assessed and considered in the design process to accurately predict the performance of the foundation system. In the Kingdom of Saudi Arabia, Lower Paleozoic rocks crop out extensively in many areas and are well known from the subsurface (Konert et al. 2001). The study area is entirely located between Wadi Ad Dawasir and Najran which is known as Sultana center. It is located at latitudes 22°36′29″E, and longitudes 31°59′09″N. The distance to Riyadh is about 870 km.

The strata crop out in the area between Wadi Ad Dawasir and Najran in the south. They unconformably on lap the basement of the Arabian Shield to the west. Mapping has revealed that the general dip of the strata invariably is 1°–2° to the east where they disappear beneath the Tuwaiq Escarpment. The study area is found within the arid zone which characterize by a high temperature, high rate of evaporation, low rate of precipitation and low humidity that cause a low recharge of aquifer from outcrop comparing with a high rate of withdrawal from wells. The southern part

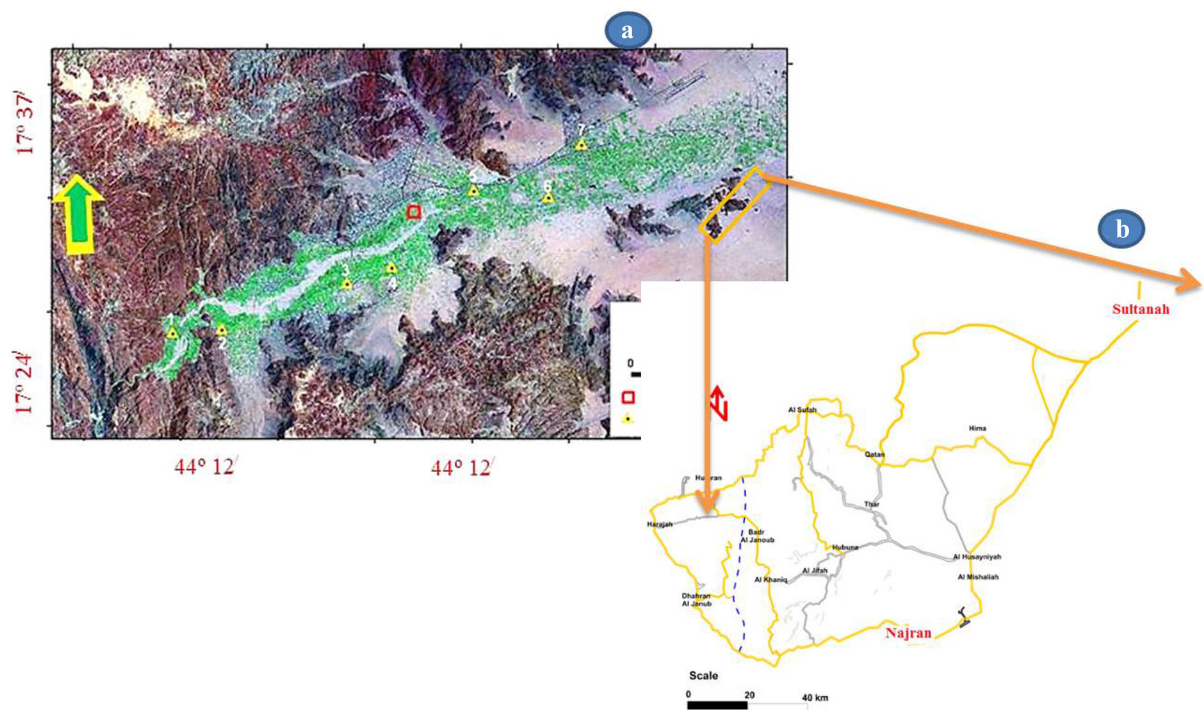

Fig. 2 **a** Landsat-8 operational land imager (OLI) image showing the location map of the study, **b** sketch map of Sultana twain

Fig. 3 Excavated trial pit

Fig. 4 Poklain during excavation of trial pit at proposed location in Sultana

of the formation consists of fluvial sandstone and very minor siltstone and silty clay.

3 Materials and Testing Methods

The geotechnical site was investigated via four (04) boreholes, of depth 10.00 m, and one (01) trial pit, of depth 2.5 m. The boreholes were drilled by CME mobile rig using rotary drilling system. The drilling was started at the existing natural ground surface at the location of each borehole.

Four soil boreholes and one trial pit of the site are illustrated in Fig. 5. The boreholes were focused at locations where buildings were proposed to be constructed. The trial pit was terminated at 2.50 m due to instability of the sidewalls of the pit below this depth. Whereas, the boreholes were terminated at the depth of 10 m. The ground water table was not encountered at the time of the investigation along the entire depth of 10 m.

3.1 Moisture Content and Water Table of Soil Samples

The moisture content of soil samples were calculated according to ASTM D4643–08. The results are shown in Fig. 6. At the site, the moisture content is ranging from 4.4 to 16.7%.

3.2 Atterberg Limits of Soil Samples

Atterberg limits of Sultana (Najran) soils of Borehole 2 are presented in Fig. 7. The plasticity index (PI) of the soil provides an indication of how much clay will shrink or swell. The higher the PI, the greater is the shrink-swell potential. The plasticity index for the soil tested is less than 15% which can be ranged as low expansion potential.

The Properties of Najran tested soils of Trial Pit-01and Borehole-02 of the soil are given in Table 4. Table 5 represents the Atterberg limits of Najran tested soils of Borehole-02.

According to Table 6, all samples were classified as SM, except for Borehole-02–Depth-1.5 m, classified as SC. Grading curves and Atterberg limits, necessary for the classification, were previously presented. Sieve analysis indicated that the average contents of gravel, sand and clayey silt were 3%, 55% and 41% respectively.

3.3 Sieve Analysis and Soil Classifications

The particle size distribution (Fig. 8a, b) for BH-02, BH-03, BH-04, and TP-1. Generally consists of 59.5% sand grains, 32.5% silt grains, and 7.7% clay particles for the sample at depth of 2 m, 49.7% sand grains, 42.2% silt grains, and 7.6% clay particles for 7.5 m

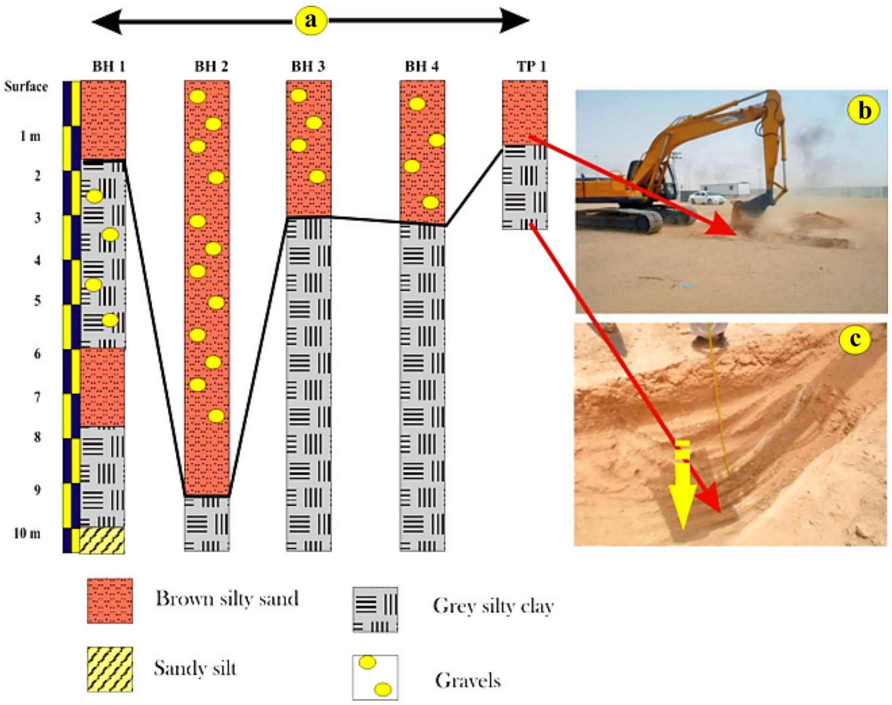

Fig. 5 Borehole BH-01, BH-02, BH-03, BH-04, and trial pit-01 test

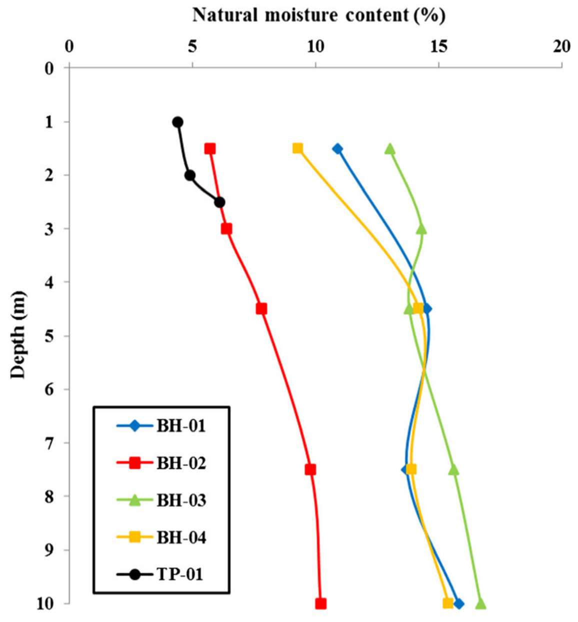

Fig. 6 Natural water content versus depths from different boreholes and trial pit

depth, and 43.1% sand grains, 46.3% silt grains, and 9.8% clay particles for 10 m depth.

3.4 Mineralogy and Chemical Analysis of Soil Samples

X-ray diffraction and energy dispersive spectrometry (EDS) analyses were carried out on the samples. The most predominant soil minerals were: Kaolinite, Smectite, Illite, Montmorillonite, and quartz. Other minerals present in small quantities have been found, including: chloride and feldspars. The clay minerals in the samples were similarity among the majority of analyzed samples.

Table 7 shows the results of the chemical analysis (According to BS 812 and1377) of testing soil samples. It is well known that the soil pH is a measure of the acidity and alkalinity in soils. In this case the PH of Sultana soil varies from 7.85 to 8.08, which they can be considered as alkaline. In addition, the lower percentage of both sulfur trioxide and sodium chlorite did not have a significant effect on Sultana soils.

Figure 9 shows example of Najran soil structure by using Scanning electron microscope (SEM) tests prior collapse for undisturbed samples at depth of 2.5 m and 4.5 m respectively. The results show clay matrices around the silt and sand particles, pore spaces are also shown which are probably responsible for the weaker zones.

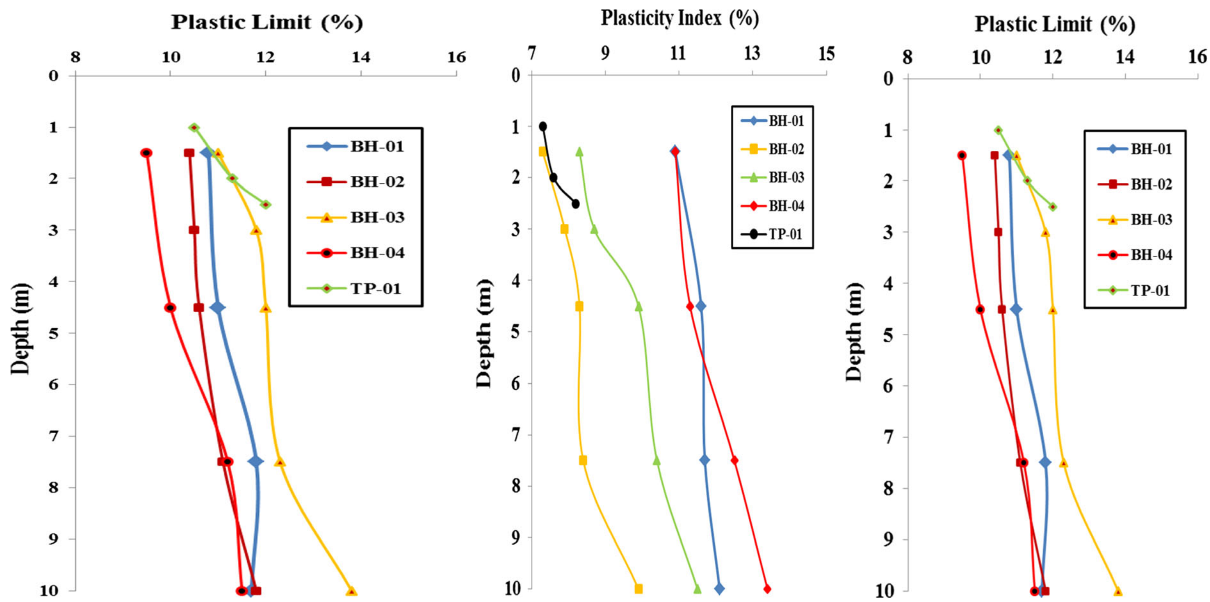

Fig. 7 Atterberg limits values versus depths from different boreholes and trial pit

Table 4 Chemical analysis of testing soil samples

BH	Depth (m)	pH	Chlorides as Cl″ (%)	Sulphtes as SO_3'' (%)	$CaCO_3$ (%)
BH-01	1.5	7.85	0.095	0.053	19.08
	9.0	7.90	0.071	0.036	18.26
BH-02	3.0	7.98	0.063	0.058	6.87
BH-03	4.5	7.94	0.036	0.044	15.19
BH-04	6.0	8.00	0.029	0.022	6.20
TP-01	1.0	8.08	0.083	0.055	16.55

Table 5 Collapse potential (Jennings and Knight 1975), as an indication of potential severity at normal stress of 200 kPa

Collapse potential (%)	Severity of problem
0–1	No problem
1–5	Moderate trouble
5–10	Trouble
10–20	Severe trouble
> 20	Very severe trouble

3.5 The Hydro-Collapse of Najran Soil

The extent of the collapse potential is not well recognized prior to construction in this region which poses a challenge for the geotechnical engineers.

Double Oedometer tests were performed according to ASTM D 5333. Two identical samples with diameter and height of 50 mm and 20 mm respectively are placed in two Oedometers; one tested at in situ natural moisture content and the other is fully saturated before the test begins and then subjected to identical loading. Collapse potential at target stress of 200 kPa is known as the Collapse Index and it is used as an indicator of collapse severity. Murthy (2010) indicates that the amount of collapse for collapsible soils is a function of the relative proportions of each component, including degree of saturation, initial void ratio, stress history of the materials, thickness of the collapsible strata and the amount of added load foundations on collapsible soils.

Results from double Oedometer test for trial pit-01 (1 m depth) are shown in Fig. 10. At a flooding stress level of 200 kPa the degree of hydro collapse was

Table 6 Collapse potential values from double Oedometer tests for samples of trial pit-01 and bore hole-02

Normal stress (kPa)	Collapse potential (%)					
	Trial pit-01 1 m depth	Trial pit-01 2 m depth	Trial pit-01 2.5 m depth	Bore hole-02 4.5 m	Bore hole-02 7.5 m	Bore hole-02 10 m
200	12.9	12.08	11.34	10.05	8.95	7.91
400	13.54	12.47	11.95	10.79	9.7	8.63
800	14.51	13.18	12.29	11.58	10.05	9.28
1600	15.10	14.12	12.84	12.42	10.81	9.76

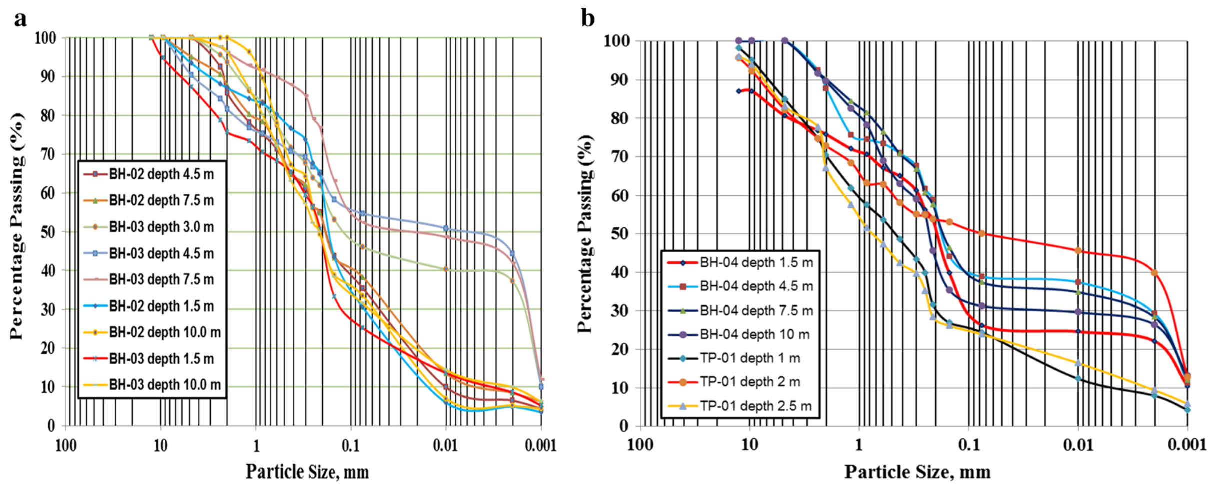

Fig. 8 Particle size distribution for boreholes: **a** BH-02, BH-03 and **b** BH-04, and TP-1 with different depths

Table 7 Expansion potential per the Free Swell Index (Mohan 1977)

FSI	Degree of expansion
> 200	Very high
100–200	High
50–100	Medium
< 50	Low

found to be approximately 12.9%, which can be considered to pose severe collapse problems according to Jennings and Knight (1975). A further increase in applied pressure resulted in additional reductions in the void ratio following the post-wetted curve. The maximum collapse occurred at about 1600 kPa and reached about 15.10%.

The calculated amount of hydrocollapse at 200 kPa was 12.08% for sample of trial pit-01(2 m) and 11.34 for trial pit-01 (2.5 m) as shown in Figs. 11 and 12 respectively.

Hydrocollapse test results for undisturbed specimens from BH-02 (4.5 m) flooded at different pressures are presented in Fig. 13. It can be seen that the compression of the specimens is insignificant in the dry state as compared to the collapse settlements due to flooding. The post-wetted e-log (P) curve at the end became approximately linear. The collapse potential at 200 kPa was found to be approximately 10.05%.

Figures 14 and 15 show the variation of the voids ratio (e) with the logarithm of the applied pressure (log p) for undisturbed soil samples of BH-02 at depths of 7.5 m and 10 m respectively. The measured hydrocollapse values for stress level at 200 kPa are 8.95% and 7.91% respectively. This reduction in collapse potential values can be explained either by the

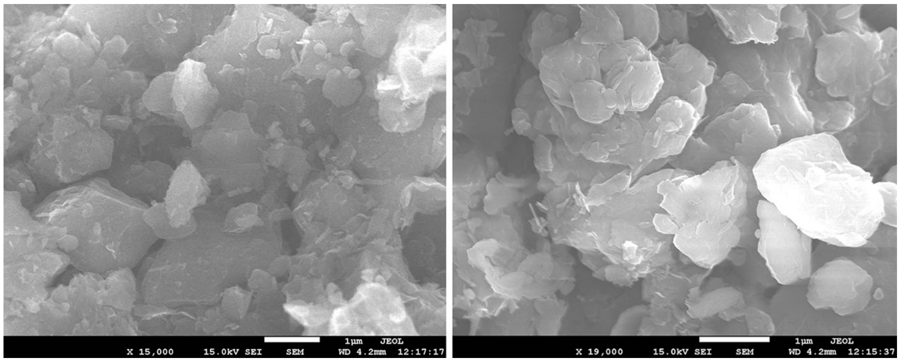

Fig. 9 Soil fabric as shown by scanning electron microscope for Najran Soil (2.5 m and 4.5 m depth) at natural moisture content)

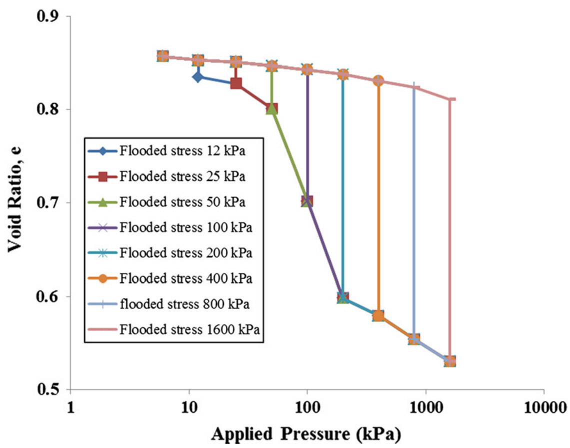

Fig. 10 Results of double Oedometer test on undisturbed Soil sample of trial pit-0 (1 m depth)

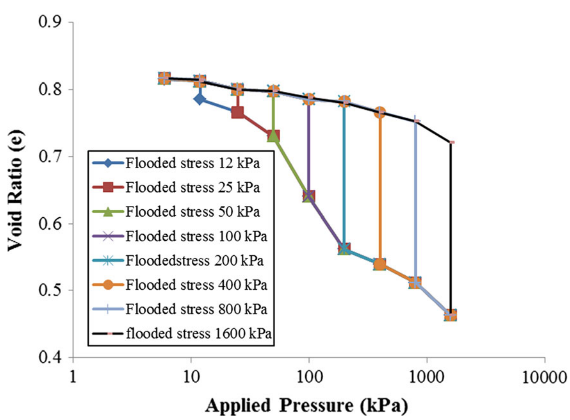

Fig. 11 Results of double Oedometer test on undisturbed soil sample of trial pit-01 (2 m depth)

increase of natural water content or by the increase of overburden pressure, or to other factors. However, the most important parameter is the moisture which causes chemical or physical bonds between the soil particles to weaken, allowing the structure of the soil to collapse.

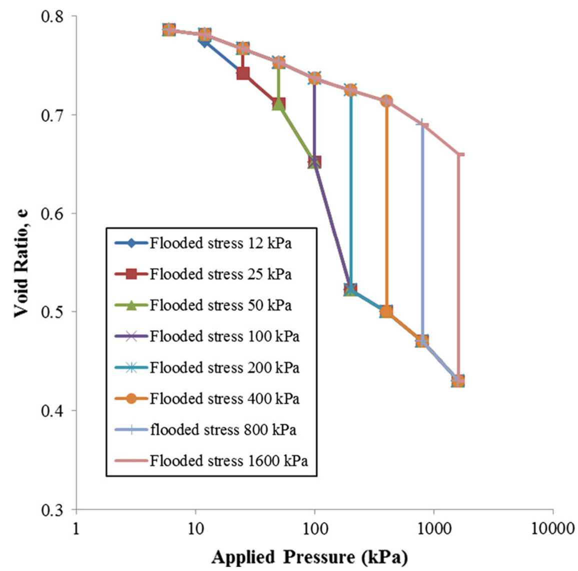

Fig. 12 Results of double Oedometer test on undisturbed soil sample of trial pit-01 (2.5 m) depth

The results of collapse potential values for trial pit-01 and bore hole-02 samples by using double Oedometer tests are summarized in Table 8.

The variations of collapse potential for undisturbed specimens with different natural water content at the same flooding stress of 200 kPa are shown in Fig. 16.

Figure 17 shows the variation of collapse potential for undisturbed specimens with different flooded normal stress for sample from trial pit-01 (2.5 m) depth which indicates that the increase in flooding stress leads to an increase in collapse potential.

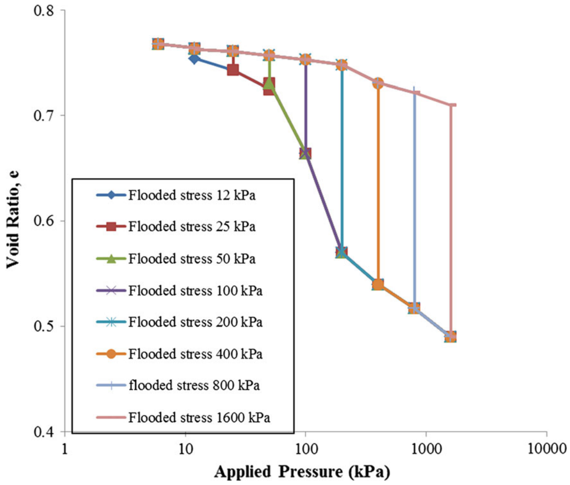

Fig. 13 Results of double Oedometer test on undisturbed soil sample of BH-02 (4.5 m depth)

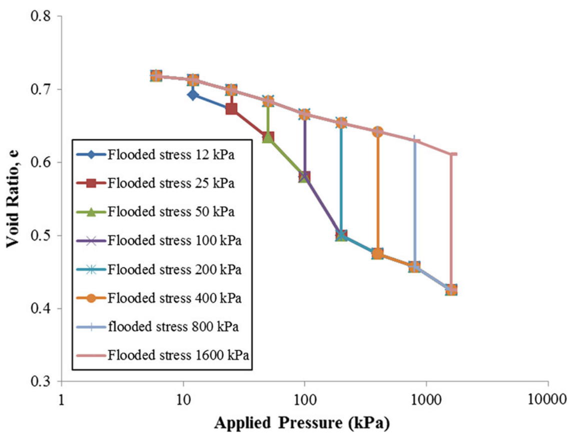

Fig. 14 Results of double Oedometer test on undisturbed soil sample of BH-02 (7.5 m depth)

3.6 Shear Box Tests

Shear strength parameters are crucial for foundations design and stability analyses of slopes. In this study, tests were carried out using samples of 60 mm square and 20 mm in height by using direct shear box, (Whykeham Farrance, UK). Tests were carried out according to ASTM D3080-04 with the aim of examining the shear strength parameters for six undisturbed samples (3 from trial pit-01 and 3 from BH-02) under two conditions: the first condition, samples consolidated and sheared at natural water content, whereas for the second condition the samples consolidated and sheared at wetted conditions. For all

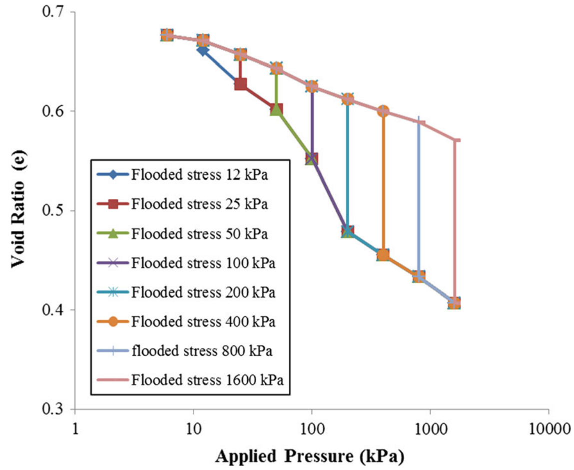

Fig. 15 Results of double Oedometer test on undisturbed soil sample of BH-02 (10 m depth)

tests, the normal stress varies from 25 to 800 kPa. The rate of shear for the shear box was fixed at 0.0158 mm/min, a value that was found by preliminary tests to ensure drained conditions throughout the test. Each test was sheared under normally consolidated conditions.

The results obtained of Shear stress versus displacement of soils from Trial pit-01 at 1 m depth, under natural water content were shown in Fig. 18. From this figure, it can be observed that the maximum shear strength ranges from 12.65 to 327.53 kPa with normal stress ranges from 25 to 800 kPa. However, for the same samples with the same normal stresses under wetted condition, there is a significant reduction in the shear strength values by more than 60% compared to values obtained from shear strengths at natural water content. It was found that the maximum shear strength values for samples at wetted condition ranges from 5.01 to 273.99 kPa, as shown in Fig. 19.

Figure 20 shows the failure envelopes of soils from trial pit-01 at 1 m depth for natural moisture content and wetted conditions. The failure envelopes show nonlinearity at low normal stresses between 0 and 100 kPa for the two tested conditions (natural moisture content and wetted conditions) and the soils show more friction than cohesion. It can be seen from Fig. 20 that the shear strength parameters (C and ϕ) were reduced for wetted conditions compared to natural moisture content conditions. This reduction can be explained by the weakness of the bonding (the fabric structures) of soil due to the wetting of the soil.

Table 8 Soil expansivity predicted by plasticity index

Degree of expansion	Plasticity index, I_p (%)		
	Holtz and Gibbs (1956)	Chen (1988)	IS 1498 (1970)
Low	< 20	0–15	< 12
Medium	12–34	10–35	12–23
High	23–45	20–55	23–32
Very high	> 32	> 35	> 32

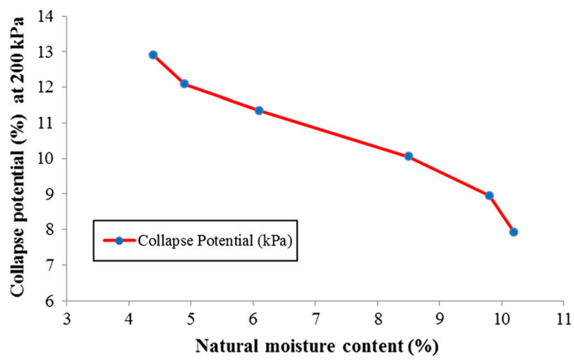

Fig. 16 Relationship between collapse potential and natural moisture content for undisturbed specimens of trial pit-01 and BH-02

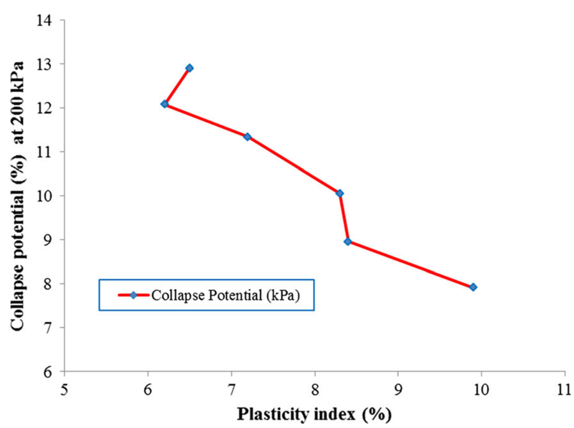

Fig. 17 Relationship between collapse potential and plasticity index for undisturbed specimens of trial pit-01 and BH-02

Table 9 shows the shear strength parameters for all soils tested (trial pit-01 and bore hole-02 at natural moisture content and wetted conditions.

The collapsible soil cohesion could be attributed to the clay particles that cover and bond the soil particles together to form what is apparently stable soil in its dry state. In addition, some soluble materials also can be responsible for creating apparent cohesion between the soil particles, such as gypsum and calcium chloride.

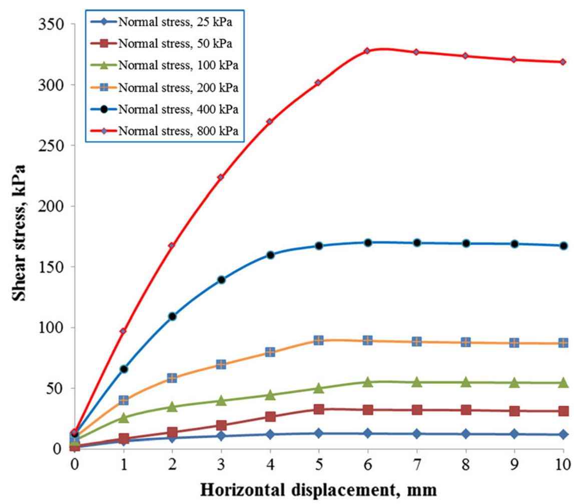

Fig. 18 Shear stress–displacement of soils from Trial pit-01 (1 m) (under natural condition)

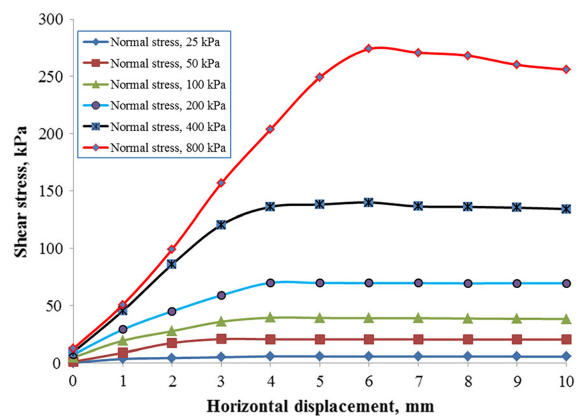

Fig. 19 Shear stress–displacement of soils from trial pit-01 (1 m) (under wetted condition)

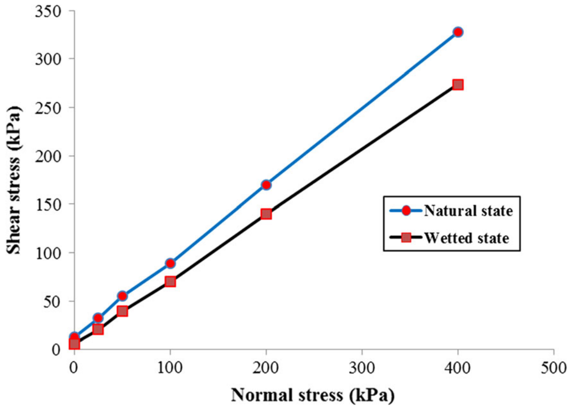

Fig. 20 failure envelopes of soils from trial pit-01 at 1 m depth, under natural moisture content and wetted conditions

3.7 Modified Proctor Test

Five layers of the soil were compacted in a cylindrical metal mold, internal diameter 10.16 cm, a 4.54 kg hammer falling through 457 mm, with 25 blows on each of five layers, for a compactive effort of about 2700 kN/m^3, samples were air-dried passing No. 4 sieve, using an automatic modified Proctor hammer in accordance to AASHTO T180 (ASTM D 1557). Six Proctor tests were carried out on three samples from TP-1 (at depths of 1.0 m, 2.0 m, and 2.5 m) and three samples from BH-02 (at depths of 4.5 m, 7.5 m, and 10 m).

3.8 Procedure of Plate Test

Plate load test is an important field test for determining the allowable pressure and the settlement under foundations for clay and sandy soils (see Fig. 21). Therefore, the plate load test is useful for the selection and design of the foundation. To calculate the safe bearing capacity, an appropriate safety factor is applied. For performing this test, the plate is placed at the desired depth, then the load is applied gradually and the settlement for each increment of load is recorded. The size of the pit must be at least 5 times the size of the plate load (Bp). Regularize the surface with very thin layer of clean and wet sand. The settlement is observed for each increment of load from the dial gauge. The applied load was recorded using a pressure gauge mounted on the hydraulic jack.

3.9 Electrical (Earth) Resistivity Tests

The Wenner Method probably, is considered as one of the more ‘reliable’ methods for testing soils to deeper depths. The Electrical (Earth) Resistivity Tests were carried out on site. The purpose of conducting Earth resistivity tests is to evaluate the subsurface condition and to determine the apparent electrical resistivity values for use in the design of Earthing and grounding system and also for the evaluation of the corrosion potential on buried pipes as well as concrete in general. The test procedure is very simple and can be summarized as follows. Vertical electrical soundings were conducted at the locations.

The electrical resistivity was measured by using Wenner Array (four electrodes) Method in accordance with ANSI/IEEE Std 81-1983. Variations in resistivity at depths were obtained by increasing the spacing of the electrodes. The increased spacing forces the current to flow deeper into the earth in order to

Table 9 Shear strength parameters for samples from trial pit-01 and bore hole-02

Shear strength parameters	Under natural conditions		Under wetted conditions	
	C_{nc} (kN/m^2)	ϕ_{nc} (°)	C_{wc} (kN/m^2)	ϕ_{wc} (°)
Trial pit-01 (1 m)	2.65	32.21	0.31	29.83
Trial pit-01 (2 m)	6.07	31.54	1.65	28.67
Trial pit-01 (2.5 m)	7.02	31.88	1.88	27.77
BH-02 (4.5 m)	9.36	27.42	2.36	24.14
BH-02 (7.5 m)	8.79	28.85	3.01	24.91
BH-02 (10 m)	11.63	25.43	4.68	22.37

C_{nc} cohesion at natural conditions, C_{wc} cohesion at wetted conditions, ϕ_{nc} friction angle at natural conditions, ϕ_{nc} friction angle at wetted conditions

Fig. 21 Plate load test

complete the circuit, thereby increasing the depth of penetration. The depth of penetration was taken a′/20, where (a′) is the spacing of electrodes. The electrodes spacing used were 0.5, 1.0, 2.0, 3.0, 4.0, and 5.0 m. Soil resistivity represents some major corrosion related to soil properties, hence may serve as a good indicator for soil corrosivity.

4 Results and Discussion

The results from laboratory and field tests are necessary to predict the likely settlement that may occur under severe conditions for foundation design in collapsible soils which is a different task for engineers. It should be noted that the maximum moisture content was found in the borehole No. 3 with a value of 16.7%. In addition, the groundwater table was not encountered through the investigated depth, neither during nor after drilling. The specific gravity values vary from 2.64 to 2.75, whereas, the void ratios vary from 0.68 to 0.86. The maximum value of plasticity index for all samples was approximately 10%, they can be considered as low plasticity. In addition, the values of consistency index were greater than one; they can be classified as semi-solid state and is stiff.

According to the test results presented in Fig. 12 and the previous Figs. 10 and 11, it can be seen that there is a unique post-wetted relationship between void ratio and applied pressure for trial pit-01 specimens that were subjected to wetting at different stress levels. The calculated amount of collapse potential (hydrocollapse at 200 kPa) was in the range 7.91–12.9% for both samples from trial pit-01 and bore hole-02. Based on the values given by Jennings and Knight (1975), the samples from trial pit-01 can be considered to pose severe collapse problems. For samples from bore hole-02, the sample at 4.5 m depth can be considered to pose severe collapse problems, however, samples at depths of 7.5 m and 10 m can be considered to pose troubles.

Increasing natural water content leads to an increase of the degree of saturation. Simultaneously, the collapse potential decreases due to the decrease of the volume of voids.

Handy (1973) studied the influence of clay content on Iowa loess and he concluded that soils with clay content less than 16% are subjected to a high probability of collapse; within the range of 16–24%, soils are likely to collapse; within the range of 24–32%, the probability of collapse is 50%; for greater or equal to 32%, soils are usually safe from collapse. For Najran soils, the clay content is less than 16% which is expected to be among the high probability of collapse.

Holtz and Gibbs (1956) demonstrated that for free swell index less than 50%, soils exhibit much volume changes in the field. The average swelling ratio for trial pit-01 and bore hole-02 samples were 37% which are among soils of much volume changes in the field.

The strength parameters were decreased by more than 47% and 11% for cohesion and internal friction angle respectively. The significant changes of the strength parameters were noted for soil from trial pit-01 at 1 m depth, in which the wetting conditions caused a decrease of the angle of internal friction (from 37.21° to 33.83°) as well as for cohesion (from 12.65 to 5.91 kPa).

To calculate the bearing capacity of collapsible soils, it is recommended to take the shear strength parameters determined at wetting conditions into consideration.

Gurtug and Sridharan (2004) proposed correlations for optimum moisture content and maximum dry unit weight with plastic limit (PL) of cohesive soils. These correlations can be expressed for modified Proctor test as:

$$\begin{aligned}\omega_{opt}(\%) &= 0.65(\mathrm{PL}) \text{ and } \gamma_{d\,max}(\mathrm{kN/m^3}) \\ &= 22.68\,e^{-0.012(\mathrm{PL})}\end{aligned}$$

The modified Proctor parameters for the samples tested compared with values proposed by Gurtug and Sridharan (2004) are shown in Table 10.

It can be seen from Table 10 that the parameters of the modified Proctor test for trial pit-01 and borehole-02 do not match well with values proposed by Gurtug and Sridharan (2004). This observation indicates that the proposed correlations need some modifications in order to be considered for non-cohesive soils.

Results of electrical resistivity measurements are shown in Fig. 22. Table 11 shows the Soil corrosivity ratings based on soil resistivity (Roberge 2007). The range of apparent resistivity values obtained at location is illustrated in Table 3. The test results of Soil Resistivity shall be used for the design of earthing system. According to the typical resistivity values for different soils and rocks. The Najran soils can be grouped as essentially non-corrosive and also regarding the material it can be arranged as between (shale, dry clay, silts) and (sand, gravel) (Table 12).

Soil resistivity represents some major corrosion related soil properties, hence may serve as a good indicator for soil corrosivity.

Results from chemical analysis indicated that the values of (PH) ranging from 7.85 to 8.08, the content of (Cl) varies from 0.029 to 0.095% which represents a negligible to moderate exposure, and the content of (SO_3) was from 0.022 to 0.058% which also represents a negligible exposure.

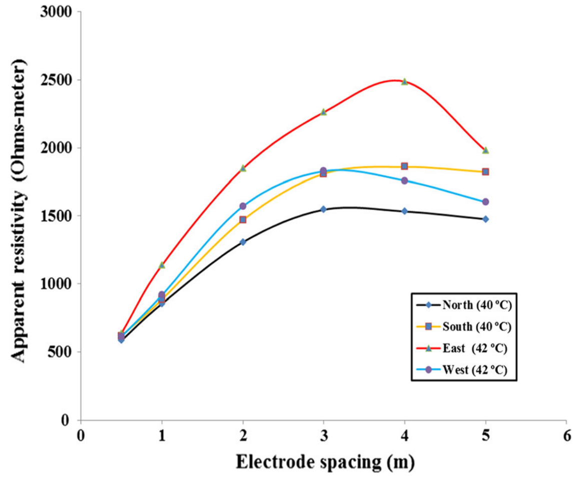

Fig. 22 Relation between electrode spacing and apparent resistivity

Table 11 Soil corrosivity ratings based on soil resistivity (Roberge 2007)

Soil resistivity (Ohm m)	Corrosivity rating
> 200	Essentially non-corrosive
100–200	Mildly corrosive
50–100	Moderately corrosive
30–50	Corrosive
10–30	Highly corrosive
< 10	Extremely corrosive

Table 10 The modified proctor parameters for trial pit-01 and bore hole-02 samples

	Trial pit-01 and bore hole-02		Gurtug and Sridharan (2004)	
	Maximum dry density (kN/m^3)	Optimum moisture content (%)	Maximum dry density (kN/m^3)	Optimum moisture content (%)
TP-01 depth 1.0 m	16.50	6.53	19.99	6.82
TP-01 depth 2.0 m	16.92	7.84	19.80	7.34
TP-01 depth 2.5 m	17.31	7.09	19.63	7.8
Borehole-02 4.5 m	17.65	11.4	19.73	7.54
Borehole-02 7.5 m	18.47	12.3	19.61	7.86
Borehole-02 10 m	19.53	13.70	19.45	8.32

Table 12 Values of resistivity expressed in units of ohm m (after Peck et al. 1974)

Material	Resistivity (ohm-m)
Clay and saturated silt	0–100
Sandy clay and wet silty sand	100–250
Clayey sand and saturated sand	250–500
Sand	500–1500
Gravel	1500–5000
Weathered rock	1000–2000
Sound rock	1500–40,000

In general, with regard to collapse potential and type of structural loading, the design engineer may consider various types of foundations in order to carry loads safely. In many cases, when the subsoil foundation is found to be collapsible, deep foundations such as piles or piers maybe used to transmit foundation loads to deeper bearing strata below the collapsible soil deposit. By using the classic formulas, which have form similar to Meyerhof equation (Das 1998), the allowable net bearing stress for Najran soil is 1.50 kg/cm^2 which could satisfy a factor of safety larger than 3.00 and prevent any unexpected conditions. Bowles (1988) provides three general and practical methods to combat the collapsing potential of soil. These are: compaction of the soil to $\gamma_d \geq 15.5$ kN/m^3, use of different types of admixture such as Portland cement during compaction, and use of piles through the collapsible soils to a more competent underlying stratum. Therefore, compaction techniques with either conventional impact or vibratory rollers can be used for shallow depths up to 1.50 m as far as γ_d greater than 15.5 kN/m^3 obtained by modified Proctor test. All the expected elastic settlements are less than the allowable values, as per Saudi Building Code (SBC-303).

In cases where it is feasible to support the structure on shallow foundations in or above the collapsing soils, the use of continuous strip footings may provide a more economical and safer foundation than isolated footings (Clemence and Finbarr 1981). Differential settlements between columns can be minimized, and a more equitable distribution of stresses may be achieved with the use of strip footing design. The results from laboratory or field tests can be used to predict the likely settlement that may occur under severe conditions. In many cases, deep foundations, such as piles, piers etc., may be used to transmit foundation loads to deeper bearing strata below the collapsible soil deposit.

5 Conclusions

The following conclusions presented below are based on information developed from the field and laboratory investigations.

The calculated amount of collapse potential at vertical stress of 200 kPa was in the range of 7.91–12.9% which reveals that some soils are considered to pose troubles and other to pose severe collapse problems.

- Direct shear test, indicated that the angle of internal friction (ϕ) was ranging between (25.43° and 32.21°) and the cohesion strength (c) from 2.65 kN/m^2 and 11.63 kN/m^2 under natural conditions, Whereas, for wetted conditions, the angle of internal friction (ϕ) was ranging between (22.37.43° and 29.83°) and the cohesion strength (c) from 0.31 and 4.68 kN/m$^{2.}$
- The reduction in shear strength parameters can be due to the presence of water which weakens or destroys bonding material between particles that can severely reduce the bearing capacity of the original soil.

The above results correspond an average classification between (SM) and (SC) regarding the USCS system and between (A-2-4) and (A-2-6) concerning the AASHTO system.

Modified Proctor indicated that the maximum dry density of the soil was 19.53 kN/m^3 and the corresponding optimum moisture content was 13.68%.

- The Free swelling test indicated low ability to swell with the increase of moisture content where the average swelling ratio was 37%.

Chemical Analysis, indicated the followings:

- The values of (pH) = 7.85–8.08.
- The content of (Cl) = 0.029–0.095% … (negligible to moderate exposure).
- The content of (SO_3) = 0.022–0.058%… (negligible exposure).

Based on the studied soil properties and the expected structures, the followings could be recommended:

- Among the methods for treating collapsible soils is to densify their structure by compaction.
- Continuous strip foundations perform better than isolated footings since strip foundations can withstand differential settlement and, hence, minimize damage to the structural framing system.
- Deep foundation system using piles or piers, which derive support from strata below the collapsible soils or the zone of possible wetting, can be used for structures of heavy weights.

Compaction techniques with either conventional impact or vibratory rollers can be used for shallow depths up to 1.50 m. Well compacting and leveling the existing ground surface to satisfy at least 95% of the maximum dry density of soil at the ground surface.

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