LOWLAND TECHNOLOGY INTERNATIONAL Vol. 6, No. 1, 9-20, June 2004
International Association of Lowland Technology (IALT), ISSN 1344-9656

SHEAR STRENGTH OF COMPACTED GREEN CLAY PHYSICO-CHEMICAL FACTORS

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ABSTRACT: Shear strength characteristics of Jordan's green clay influenced by physico-chemical factors have been investigated. The investigation was carried out by subjecting soil to treatment by different cations namely, sodium, calcium, and potassium at three the pH-values (pH= 2.0, pH=7.0, pH=12.0). Treated and untreated soils were tested at three different states on the compaction curve namely dry of optimum, optimum (γd=12.5 kN/m3), and wet of optimum. All specimens were tested for shear strength using unconsolidated undrained triaxial loading procedure. Test results indicated that shear strength of the natural soil was the highest in comparison with treated soils. In addition, K-treated soils showed the highest shear strength followed by the Ca-treated soil while, the Na-treated soil was the lowest in shear strength for all the states on the compaction curve except at wet of optimum, where the Ca-treated soil was the highest in shear strength among all the treated soils. At pH=7.0 (Neutral state) the highest shear strength was recorded but as soils changed from acidic to basic shear, strength decreased accordingly.

INTRODUCTION

The engineering industry in Jordan faces a challenge in dealing with the chemical effects ranging from contaminated sites to the effect of corrosion on the metallic components of subsurface elements. Physico-chemical phenomena in soils govern their geotechnical properties and their related behaviors. Existing clay is part of this family, which could be influenced by applying such chemical changes to its structure. As a part of fundamental study, green clay strength is investigated. Such clay has a high potential to cause damages to roads and buildings. Therefore a knowledge base on physicochemical phenomena in the subsurface environment is needed to clarify the geotechnical engineering behavior by altering the exchange complex yielded a single species of cation (Na^+, Ca^{++}, or K^+).

The work conducted by many researchers (Bolt, 1956) and (Olson, 1974) showed that water electrolyte system affects some soil properties such as swelling, compressibility and strength. The physico-chemical aspects related to the system have a profound effect on soil structure, which, in turn, affects the shear strength of a soil but with varying degree depending on the type of the soil. In addition, ground water table, soil characteristics, moisture content, and others largely affect selection of a structure at a given site.

Expansive clayey soils exist at various locations in Jordan. The green clay is located in the capital Amman and other cities like Azraq, Jerash, and Salt. This clay was formed at varying depths ranging from ground surface down to few meters below. This type of soil is believed to have certain effect on the design of structures in Jordan. This paper will investigate the shear strength behavior of this widely existing soil.

This paper will also present and discuss test trial on 160 soil samples to investigate the physico-chemical aspects of Jordan's green clay and mainly pivoted on measuring the concentration of the most predominant cations (i.e. Ca, Na, and K) at different pH values pH=2 (acidic), pH=7 (neutral), and pH=12 (basic) of the bathing solution.

PHYSICO-CHEMICAL ASPECTS OF SHEARING STRENGTH

The term "physical" and "chemical" may be applied here loosely to describe the interaction of soil particles. The physical term describes the interactions, which are controlled by size, shape, packing and physical properties of individual grains and to the frictional forces between them, while the term, chemical is applied to describe the various inter-particle forces such as the diffuse double layers, van der Waals forces and the ionic forces. Shear strength of soil is defined as the resistance of a soil to

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Note: Discussion on this paper is open until December 2004

Table 1 General Properties of Green Clay

Property		Value
Depth of sampling, (m)		2.0
Natural water content, ω (%)		58.0
Specific gravity of solids, G_s		2.76
Liquid Limit, LL (%)		108.0
Plastic Limit, PL (%)		42.0
Shrinkage Limit, SL (%)		21.7
Plasticity Index, PI (%)		59.0
Maximum Dry Unit Weight, γ_{dmax} (kN/m^3)		13.0
Optimum Water Content, OWC (%)		31.5
Cation Exchange Capacity; CEC (meq/100g)		37.4
Particle Size	Sand (%)	7.0
	Silt (%)	29.0
	Clay (%)	64.0
Classification (Unified Soil Classification System)		CH (Fat Clay with high plasticity)

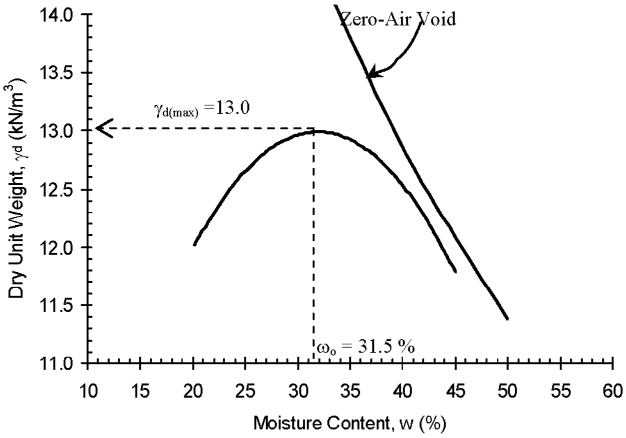


Fig. 1 Moisture content vs. Dry unit weight

withstand shear stress. Numerically, shear strength equal the shearing stress induced under loading in a material under failure. It will be seen that the chemical adsorption of polar fluid influences the coefficient of friction between particles. It is not possible to separate the physical and chemical effects completely.

PROGRAM AND PROCEDURE OF LABORATORY TESTING

Soil Properties

General properties of green clay are shown in Table 1. The relation between the moisture content and dry unit weight using the standard proctor test is shown in Fig. 1.

Samples were prepared for various tests following an extensive process of washing to saturate the soil with sodium, calcium and potassium. Samples of green clay were washed three times in standard solutions (with one normality concentration) containing those cations for three pH-values; acidic (pH=2), neutral (pH=7) and basic (pH=12). In order to facilitate the analysis under same conditions, each washing of the samples was filtered using filter paper and after third wash, samples were oven dried for 24 hours. Consequently, the treated samples were ground, screened and the fractions passing sieve No.10 (2mm) were used in the testing program.

The unconsolidated undrained (U-U) triaxial compression tests were used to measure shear strength characteristics of both treated and untreated samples. Samples were prepared at prescribed densities and water contents in a cylindrical mold that have 38 mm diameter and 76 mm height.

Saturation of Soil With Exchangeable Cations

Natural soil had been analyzed and shown in Fig.s 2-4. The percentage cations present are 50.9% Na^+, 32.2% Ca^{++}, and 5.5% K^+, in addition to Mg^{++}. Note that Al^{+++} cation, which tightly bonded with the chemical composition of the clay particles, could be replaced under the effect of ratio mass under saturation process with exchangeable cations.

Since the lack of a known standardized procedure to saturate soil with a certain exchangeable cations (Sodium, Calcium and Potassium), The saturation procedure of soil with exchangeable cations (Shibli, 1991) was used in this study. Salts (NaCl, $CaCl_2$, and KCl) as a source for the exchangeable cations Na^+, Ca^{++}, or K^+ and to make them dominant in exchange complex. The following procedure was adopted

Each salt solution was prepared with one normality concentration and buffered for three pH values; acidic (pH =2), neutral (pH = 7), and basic (pH = 12).

The soil is added to the prepared solution with ratio solution to soil ratio of 2, and mixed in the mixer for 90 minutes. Then, the mixture was filtered to determine the percentage of exchangeable cations on the exchanger (soil)

The soil was washed three times for each case as described above.

Amonium acetate (NH_4OAC) was utilized to extract the exchangeable cations. Atomic absorption spectrophotometer was used to measure the concentration of each cation.

The percentage of each cation was measured for each time of washing. Fig.s 2-4 summarized the percentage of exchangeable cations extracted after washing the soil in

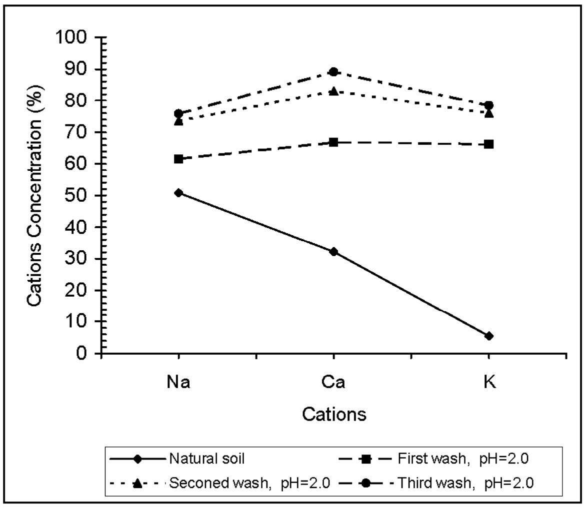


Fig. 2 Percentages of exchangeable cations concentrations after treatment with sodium, calcium and potassium chlorides at pH=2.0

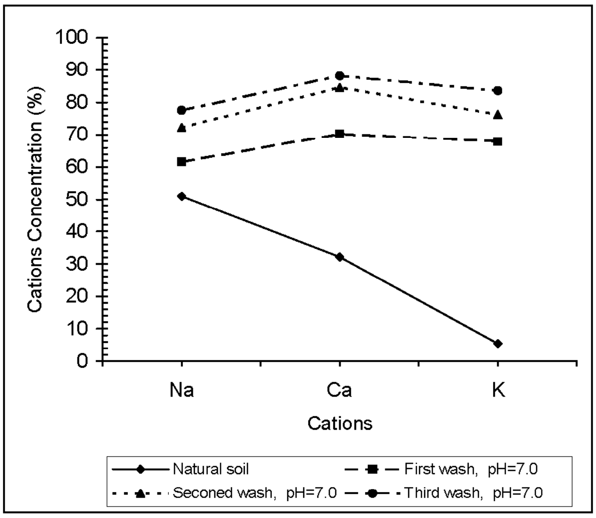


Fig. 3 Percentages of exchangeable cations concentrations after treatment with sodium, calcium and potassium chlorides at pH=7.0

sodium chloride, calcium chloride and potassium chlorideunder three pH values mentioned in step 1.

Fig.s 2 through 4 show percentages of exchangeable cations concentrations after treatment with Sodium, Calcium and Potassium Chlorides at pH=2.0, pH=7.0, and pH=12, respectively.

Samples Preparation

Samples from both treated and untreated soils were prepared to carry out unconsolidated undrained triaxial compression tests. Only soil particles passing sieve No.

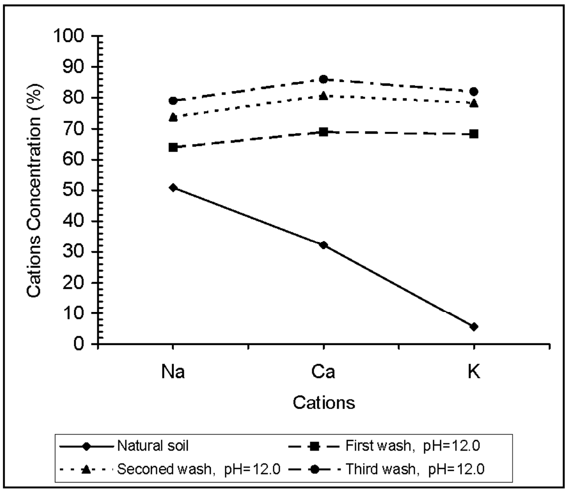


Fig. 4 Percentage of exchangeable cations concentration after treatment with sodium

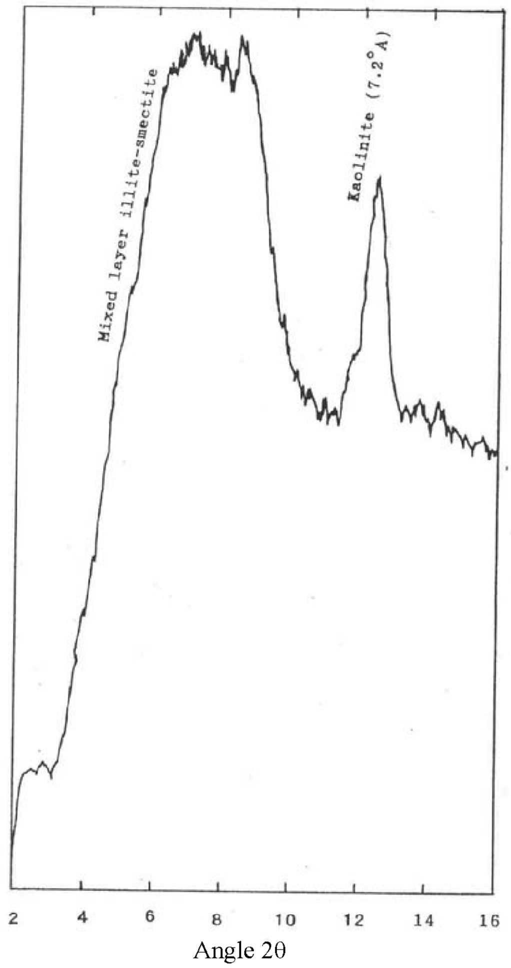


Fig. 5 X-Ray diffraction chart for untreated sample. (wave length, λ = 1.5418° A)

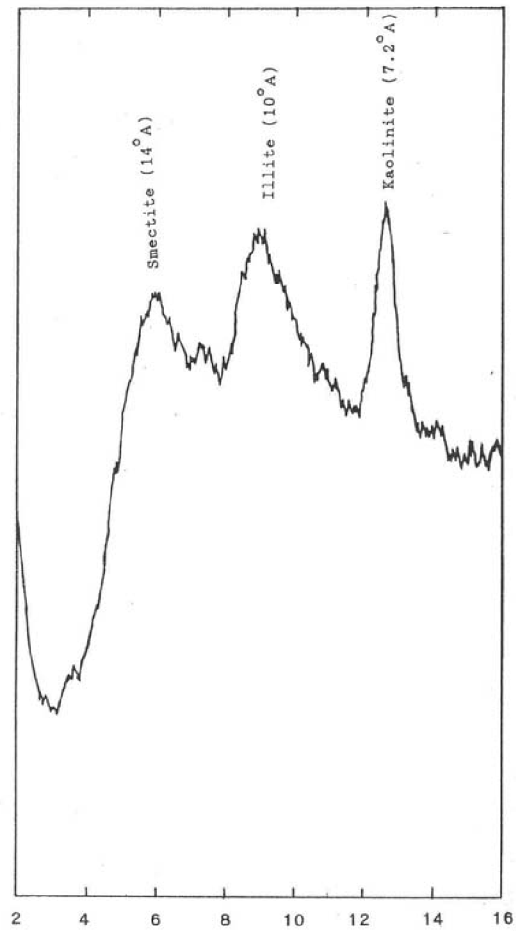


Fig. 6 X-Ray diffraction chart for glycolated sample. (Treated in ethylene glycol for 24 hours)

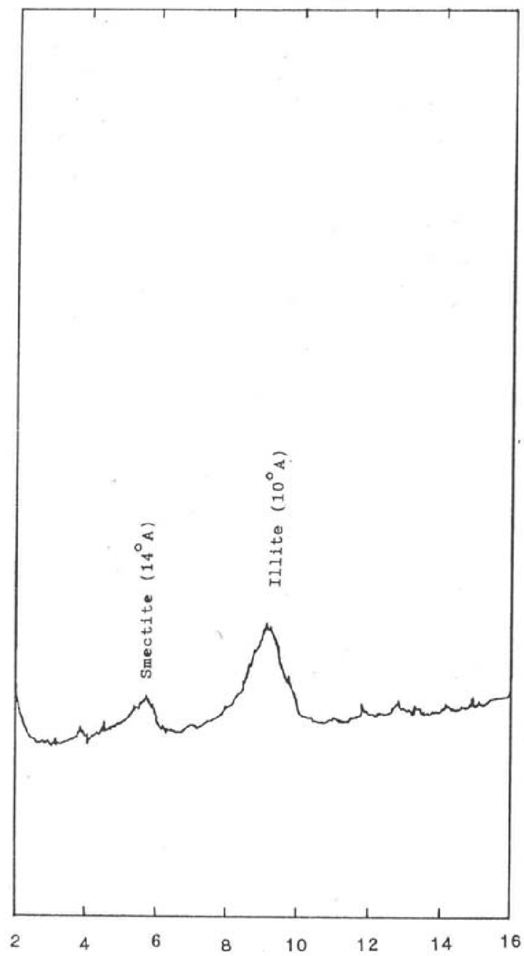


Fig. 7 X-Ray diffraction chart for heated sample. (Heated at 550°C for 2 hours)

(10) were used. After 24 hours of oven drying at 110°c the soil was then kept in the desiccator for an average period of 20 hours before molding to attain the room temperature without adsorbing moisture from the surroundings.

Wet soil at predetermined amounts of distilled water and dry soil were mixed and then compressed in the 38 mm diameter and 76 mm height mold. The tested samples had three different states of densities and water content; dry of optimum, optimum, and wet of optimum as shown in Fig. 1.

Unconsolidated-Undrained Triaxial Tests

The various combinations of the unconsolidated undrained-triaxial compression tests had resulted in about (15500) measured data points (deformation Δl, deviator stress $\Delta\sigma$, degree of saturation Sr, and confining pressure σ_3).

For each case of treated and untreated samples, stress-strain curves were analyzed at different pH value. Results are presented and shown in Tables 2 and 3 for the three states of compaction at four different confining pressures. Also, Tables 2 and 3 summarize the data concerning failure conditions of the tested specimens for the result of UU-triaxial compression tests and also shows the shear strength parameter c and φ obtained for natural soil and at different cations, pH values and degrees of saturation, respectively.

ANALYSIS OF RESULTS

Strength results from UU-triaxial compression test under various cations species, namely (Na, Ca, K), with three pH-values (Acidic, neutral and basic), and different compaction states of water content and dry unit weight, led to the following comments on the test results

Soil Identification And Mineralogical Characteristics

Physical sieve analysis and consistency limit test results, shown in Table 1, revealed that the tested soil according to the Unified Soil Classification System is classified as Fat Clay with high plasticity (CH). Table 4 compares the measured results of liquid limits, plastic limits, specific gravity and the percent of fractions finer than 2 μm of tested green clay with those of kaolinite, illite, and montmorillonite.

The liquid limit of green clay lies between liquid limits of both illite and montmorillonite but it is much more closer to illite than montmorillonite while for the plastic limits, it lies within the range suggested for illite mineral. Also, the specific gravity of green clay lies within that of illite and montmorillonite. Finally, the grain size, finer than 2 μm, lies between kaolinite and illite. Table 4 also shows the measured cation exchange capacity (CEC) of the tested soil with some published data (Mitchell, 1976) of kaolinite, illite, and montmorillonite. As an overall assessment, Table 4 suggests that CEC of green clay lies within the range suggested for illite.

For identifying natural clay minerals (Shibli, 1991), X-Ray diffraction analysis was performed on clay fraction (less than 2 micron) under three conditions

- Glycolated sample to distinguish between illite and montmorillonite).
- Heated sample to distinguish between kaolinite and chlorite
- Untreated sample

The output of the above X-Ray analysis are shown in Fig.s 5-7, reveals that the clay fraction consist mainly of mixed layer of illite-smectite component and kaolinite as a minor component.

Unconsolidated Undrained Shear Strength Characteristics

Shear strength results of the various tests are presented as a relationship between the p_f and q_f. Fig.s 8 through 11 represent the p-q curves for untreated, Na-treated, Ca-treated, and K-treated soils respectively. It can generally notice that as the degree of saturation increases, the undrained shear strength decreases for the same confining pressure. This is partly due to the flow caused by the decrease in internal friction between the soil particles and to the increase in pore water pressure, which results in decrease in the effective stress.

Effect of pH-Values

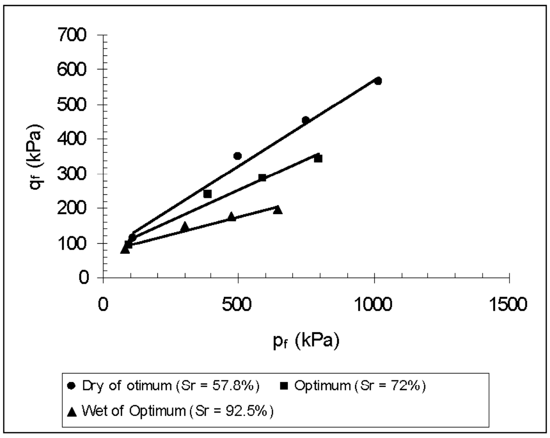


Fig. 8 p_f vs. q_f relationships for untreated soil

The effect of pH-value on the unconsolidated undrained shear strength is also shown in Fig.s 8 through 11. In all cases it was observed that the unconsolidated undrained shear strength in case of untreated soil was higher than soil treated with various pH-values. Also, soils treated with pH=7.0 demonstrated higher shear strength than both the acidic and basic soil in almost all cases.

Effect of Acidic or Basic Soil

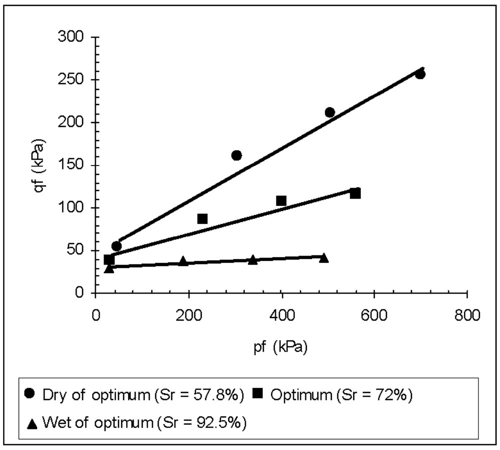
The investigation of whether acidic or basic soils have higher or lower shear strength values is presented in Fig.s 12 through 14. Except for pH =7.0, as the pH-value increases, the deviatoric stress at failure decreases for most of the treated soils. This indicates that the acidic soil posses higher unconsolidated undrained shear strength than basic soil.

Effect of Moisture Content

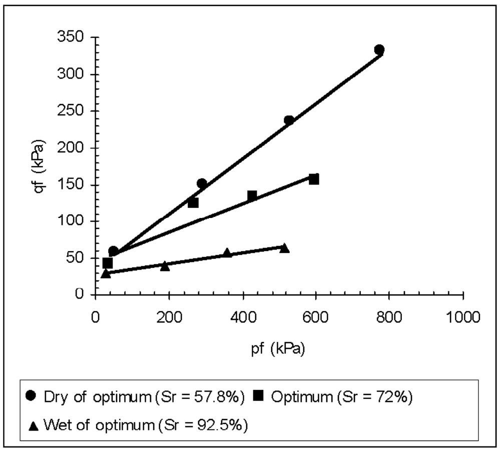
The effects of moisture content on the unconsolidated undrained shear strength for a given cation species and for a given pH-value are shown in Fig. 15 and Table 3. It was observed that, as the moisture content increases the deviatoric stress at failure decreases for all types and conditions of treatment.

This increase in moisture content had resulted in lowering the values of the angle of internal friction, ϕ_u for all the cation species as shown in Fig. 16. As for the cohesion c_u, an increase in moisture content doesn't show major changes in cohesion up to optimum value and thereafter, with further increase in moisture content, cohesion, started to decrease as shown by Fig. 15.

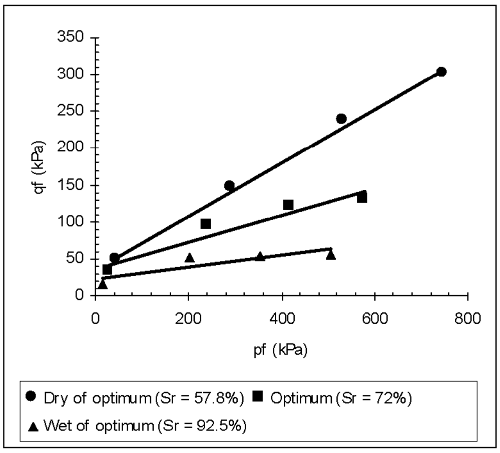
Effect of Confining Pressure on Unconsolidated Undrained Shear Strength



(a) pH = 2.0

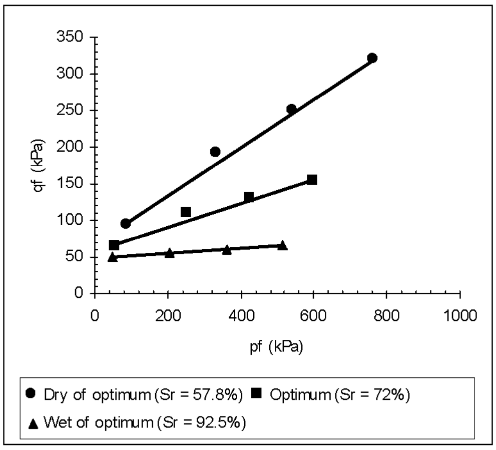


(b) pH = 7.0

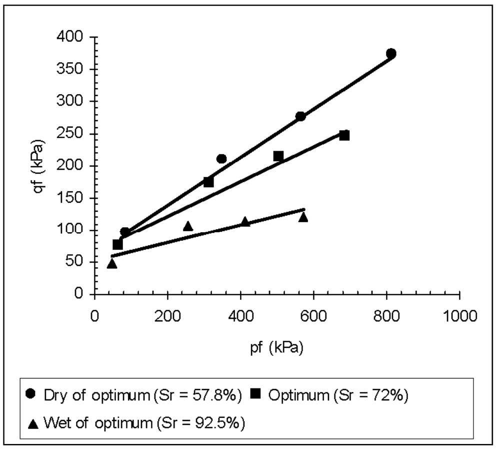


(c) pH =12.0

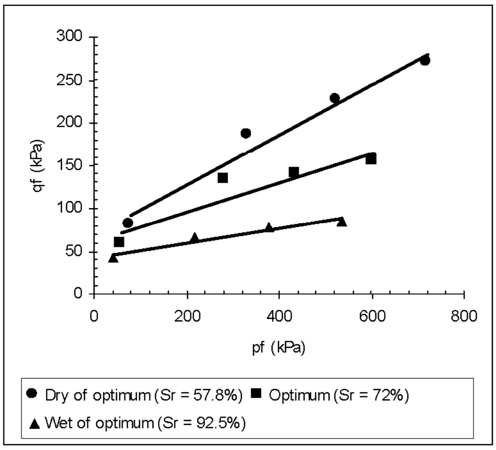
Fig. 9 p_f vs q_f relationships for Na-treated soil



(a) pH = 2.0



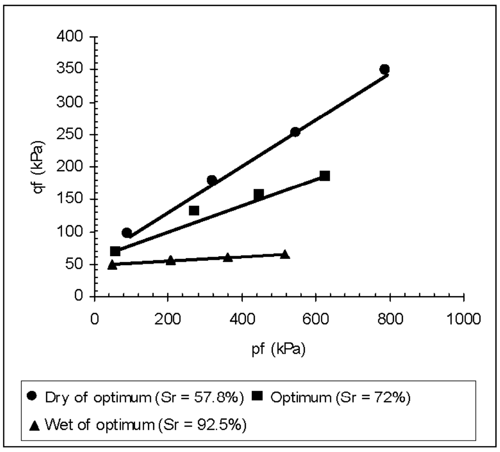
(b) pH =7.0



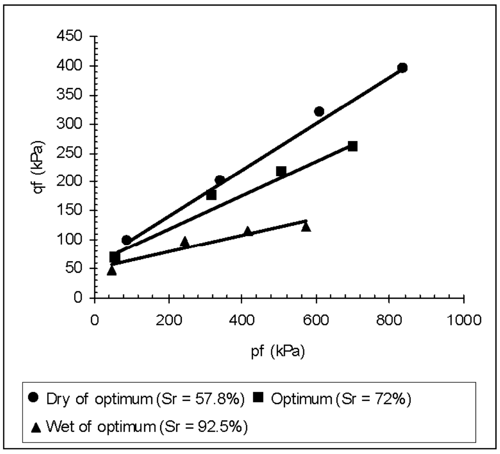
(c) pH =12.0

Fig. 10 pf vs qf relationships for Ca-treated soil

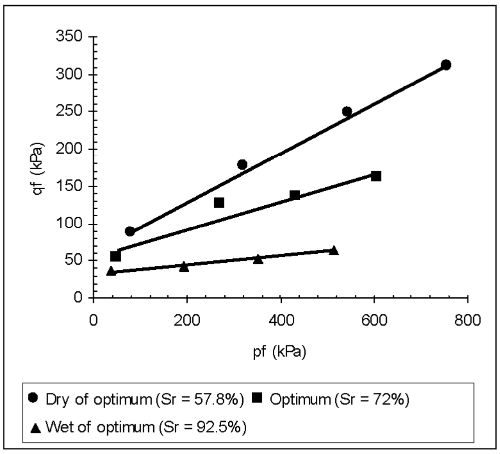
It was observed that as the confining pressure increases, the undrained shear strength also increases. This increase in unconsolidated undrained shear strength with the increase of confining pressure was clearly noticeable in tests condition on soil specimens prepared at a state of dry side of optimum of the compaction curve. This increase with



(a) pH = 2.0



(b) pH =7.0



(c) pH =12.0

Fig. 11 p_f vs q_f relationships for K-treated soil

confining pressure is up to a certain point beyond which an increase in confining pressure has no sound effect on the undrained shear strength. However, on the wet of optimum, the values of undrained shear strength remained

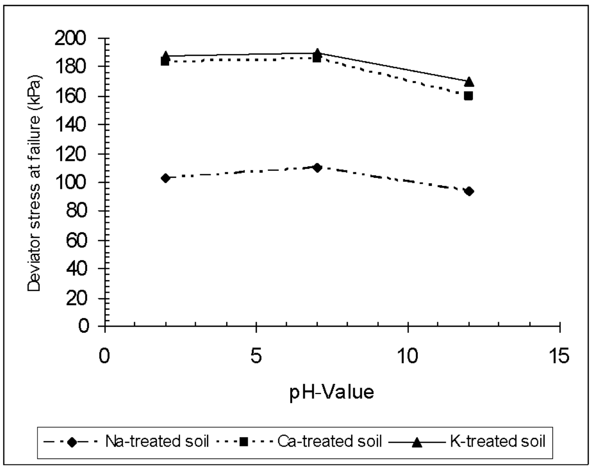


Fig. 12 pH vs. deviator stress at failure ($\Delta\sigma_f$), at zero confining pressure and at dry of optimum. (γ_d = 12.5 kN/m^3, S_r = 57.8 %)

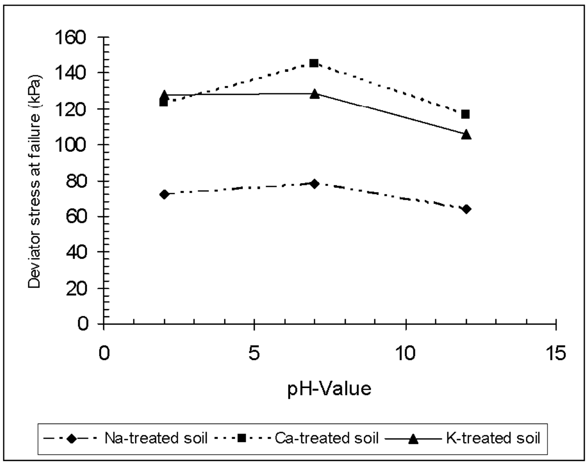


Fig. 13 pH vs. deviator stress at failure ($\Delta\sigma_f$), at zero confining pressure and at optimum. (γ_d = 12.5 kN/m^3, S_r = 72.0 %)

approximately constant regardless of the applied confining pressure.

Stress-Strain Behavior

The behavior of the soils under different conditions of treatment and compaction states are as follows at dry of optimum, it is noted that at low confining pressures (σ_3=0, and σ_3 = 150 kPa), the failure was brittle while at high confining pressures, the soil behaved plastically regardless of the amount of moisture content. As the moisture content increases away from dry of optimum reaching wet of optimum, the sample exhibited a plastic behavior. In addition, under high confining pressures samples at wet of

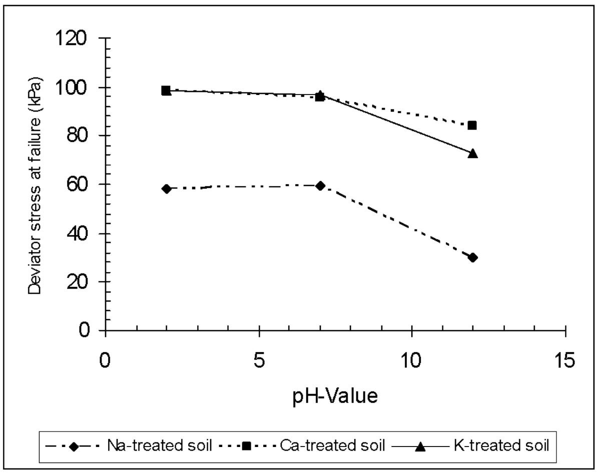


Fig. 14 pH vs. deviator stress at failure ($\Delta\sigma_f$), at zero confining pressure and at wet of optimum. (γ_d = 12.5 kN/m^3, S_r = 92.5 %)

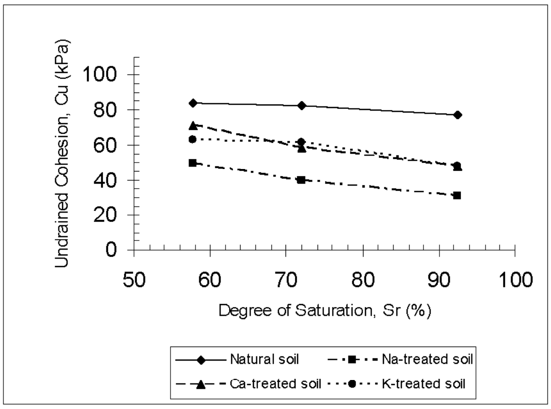
optimum may reach a high degree of saturation state, therefore, less change in volume takes place and the behavior becomes plastic too. For treated soils and under zero confining pressure all soil exhibited brittle failure while at the other confining pressures namely (150, 300, and 450 Kpa) the failure was plastic.

DISCUSSION OF RESULTS

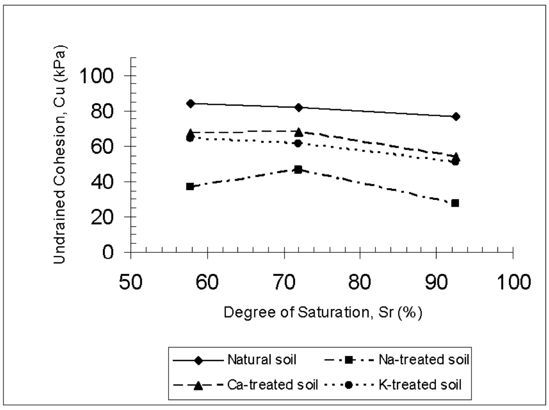
It is generally known that the physico-chemical forces start to play their role once the clay content in a given soil exceeds 10% of the total volume. Thereafter, considerable influence upon shear strength takes place (Shibli, 1991).

The author believes that the exchangeable cations played an important role in altering the structural behavior of this soil. Normally, there are some cations, which are distributed in the structure of the untreated soil. As the added cations enter the structure they will substitute the other cations of higher valances that originally exist in the soil. Such substitution will result in an increase in the double layer thickness, which, in turn, results in more repulsive forces creating dispersive structure. For example, as sodium cation enters the soil structure, it substitutes for original Ca, Mg or Al as a result of mass action. This substitution could create dispersive structure, which has low shear strength as compared to a flocculated structure. Therefore, the Na-treated soil exhibited lower shear strength than Ca-treated soil.

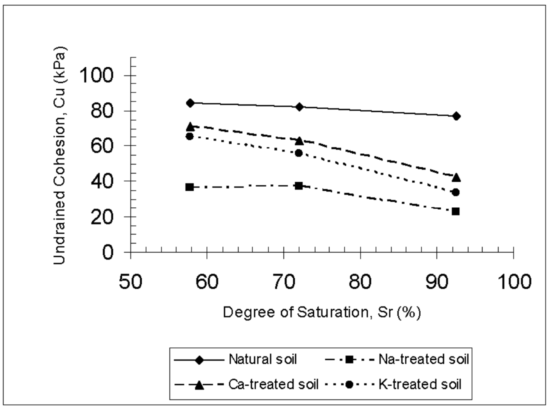
As stated earlier, at low pH-value the structure tends to be more flocculated than dispersive one. This is due to the fact that, at low pH-values edge to face flocculation takes place as a result of positive edge charge to negative surface attraction. On the other hand, as pH-value increases the soil



(a) pH = 2.0



(b) pH =7.0

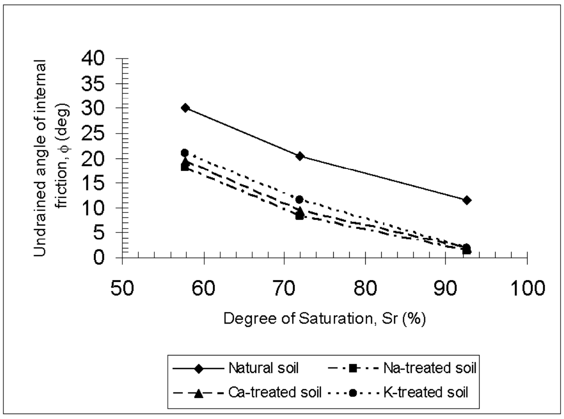


(c) pH =12.0

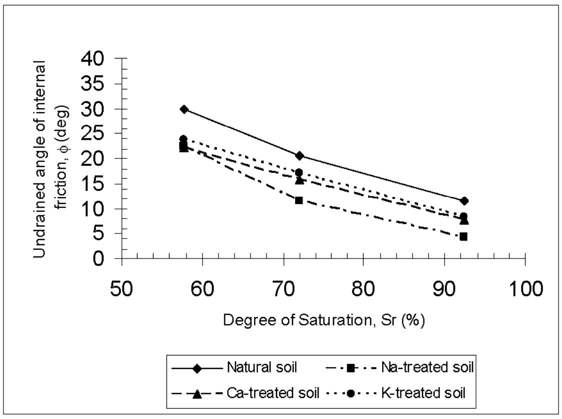
Fig. 15 Undrained cohesion, c_u vs. S_r

tends to be more dispersive due to the negative edge charge to negative surface repulsion. As the soil become more dispersive its shear strength will be reduced as was confirmed by the test result.

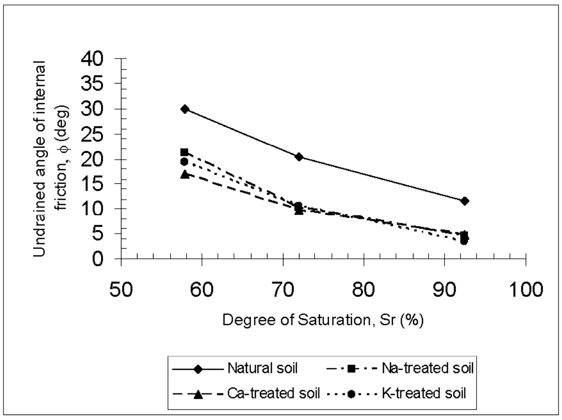
The cohesion of the soil is also a function of its structure. It is determined by the primary ionic bonds



(a) pH = 2.0



(b) pH =7.0



(c) pH =12.0

Fig. 16 Undrained angle of internal friction, φ_u vs. S_r

between the mineral layers and particles, and by the repulsive or attractive forces associated with weaker hydrogen bonds within the diffused double layer (Rosenquist, 1960). As the structure tends to be more dispersive, it is expected that a lower cohesion value will occur. The expansion of the double layer will also result in a reduction of friction between the opposing particles causing lower values of the internal friction angle (ϕ_u). Therefore, a dispersive structure can result in low values of both (c_u and ϕ_u).

In the case of K-treated soils, the highest shear strength was almost obtained, except at wet of optimum, where Ca-treated soils mostly exhibited the highest unconsolidated undrained shear strength due to the higher achieved values of cohesion. Although the potassium ions is monovalent and is expected to yield low shear strength in comparison with Ca-treated soil, the test results, showed the opposite. This can be explained as follows

The hexagonal holes at the bases of the silica tetrahedral sheets allow potassium ions to be near to the center of a charge deficiency (especially with illite mineral since the center of the charge deficiency due to isomorphous substitution is in the silica tetrahedral) (Mitchel 1976 & Shibli, 1991). This fact introduced strong link between clay particles that are attached to each other by such a high attractive force leading to an increase in shear strength.

The molding moisture content plays a significant role in soil structure to such a degree that it affects both the orientation and cohesion of soil particles, which, in turn, affects its shear strength.

As it was observed earlier, the shear strength was the highest at dry of optimum. This is attributed to the fact that at dry of optimum the water deficiency is in its highest value. This deficiency will affect to a high extent the diffused double layer development. As a result a flocculated structure would give rise to higher shear strength. At the wet of optimum, the water deficiency is in its lowest value, resulting in development of a thicker double layer, which, in turn, results in a dispersed structure, thus giving rise to lower shear strength.

It was previously mentioned that cohesion is attributed to the water bond and to the water mineral link. These linkages are a function of the water content of clay. However, if this water content exceeds a certain limit, the action of this linkage is reversed and the cohesion is no more active.

The effect of the confining pressure on the shear strength was noticeable at the dry side of the compaction curve. This is mainly due to the fact that at high confining pressure, and at low water content, a negative pore water pressure develops. The fabric of the sample is more compressed and, therefore, water covers wider area than before resulting in a higher effective stress. At low confining pressure, the negative pore water pressure is effective over lesser area and the average effective stress would be lower.

It should be noted, however, that if the confining pressure were high enough to a degree that it cause a full or near full saturation, any further increase in this confining pressure would not affect the effective stress or the shear

Table 2 Failure conditions and shear strength parameters (c_u and ϕ_u) of tested specimens in UU-triaxial test for natural soil

	Natural soil		
	Dry of optimum (γ_d = 12.5 kN/m^3) Sr = 57.8%	Optimum (γ_d = 12.5 kN/m^3) Sr = 72.0%	Wet of optimum (γ_d = 12.5 kN/m^3) Sr = 92.5%
pH	6.2	6.2	6.2
σ_3 (kPa)	0	0	0
	150	150	150
	300	300	300
	450	450	450
$\Delta\sigma_f$ (kPa)	224.6	191.5	162.5
	701	476.7	300.4
	903.1	574.6	351.2
	1130.6	687.8	393.6
σ_1 (kPa)	224.6	191.5	162.5
	851	626.7	450.4
	1203.1	874.6	651.2
	1580.6	1137.8	843.6
C_u (kPa)	84	82	77
ϕ_u (deg)	30	21	12

strength. This behavior was clearly observed at wet of optimum state where the degree of saturation was at its highest value.

The brittle behavior of the soil at dry of optimum could be attributed to the flocculated structure and negative pore water pressure at low confining pressure. Under high confining pressure plastic behavior dominated, since fabric is compressed more and the specimens have bulging shape.

The plastic behavior was also observed as the moisture content increased due to the reduction in the effective compactive effort. However, as the soil fabric became weaker and plastic, they yield more rapidly before reaching the full shear strength.

For wet samples, the deformation under compaction pressure was higher. Therefore, the residual strength was greater and more strain was needed to mobilize shear resistance in a given direction. Also under higher confining pressure wet samples may reach complete saturation and consequently less volume change may take place and as a result a plastic behavior was observed.

For treated soils, the behavior under zero confining pressure is brittle while for other confining pressures, the behavior was plastic. This is due to the effect of cations which changes the structure of the soil toward orientation as a result of increasing diffused double layer so the fabrics are easily compressed under compaction effort unless at dry of optimum which means the soil aggregate becomes weaker and more plastic.

CONCLUSIONS

From the previous analysis and discussion, the following conclusions were reached

1) The natural (untreated) Azraq green clay experienced the highest value of angle of internal friction (ϕ_u), cohesion (c_u) and shear strength in comparison with same soil that was treated with three different pH-values namely (pH-2.0, pH=7.0 and pH= 12.0).

2) The K-treated soils exhibited almost the highest unconsolidated undrained shear strength in comparison with Ca-and Na-treated soils, except at the wet of optimum side of the compaction curve were the Ca-treated soil demonstrated the highest unconsolidated undrained shear strength. In all cases, the Na-treated soil experienced the lowest unconsolidated undrained shear strength.

4) At low confining pressure, the treated soils experienced plastic deformation regardless of the degree of saturation. At complete or near complete saturation, the confining pressure had no sound effect on volume change and the behavior remained plastic.

5) With the increase in moisture content, the cohesion increased up to an optimum value after which it started to

Table 3 Failure conditions and shear strength parameters (c_u and ϕ_u) of tested specimens in UU-triaxial test for all treated soils

	Dry of optimum (γ_d = 12.5 kN/m^3) Sr = 57.8%			Optimum (γ_d = 12.5 kN/m^3) Sr = 72.0%			Wet of optimum (γ_d = 12.5 kN/m^3) Sr = 92.5%		
				Na-treated soil					
pH	2	7	12	2	7	12	2	7	12
σ_3 (kPa)	0	0	0	0	0	0	0	0	0
	150	150	150	150	150	150	150	150	150
	300	300	300	300	300	300	300	300	300
	450	450	450	450	450	450	450	450	450
$\Delta\sigma_f$ (kPa)	102.7	110.2	93.8	72.6	78.2	64.4	58	59.6	30.3
	317.4	294.8	290.3	170.3	243.8	187.5	76.4	80.9	103.9
	417.9	466.3	471.6	209.1	265.8	238.7	78.2	116.3	107.4
	506.9	658.6	600.9	228.1	307.1	260.1	82.6	129.6	111.8
σ_1 (kPa)	102.7	110.2	93.8	72.6	78.2	64.4	58	59.6	30.3
	467.4	444.8	440.3	320.3	393.8	337.5	226.4	230.9	253.9
	717.9	766.3	771.6	509.1	565.8	538.7	378.2	416.3	407.4
	956.9	1108.6	1050.9	678.1	757.1	710.1	532.6	579.6	561.8
C_u (kPa)	49.4	37	37	40	46.4	37.4	30.4	27.6	23
ϕ_u (deg)	18	22.5	21.2	8.4	11.4	10.3	1.4	4.4	4.53
				Ca-treated soil					
$\Delta\sigma_f$ (kPa)	183	185.6	159.7	122.9	145.6	116.7	98.5	95.8	83.7
	378.2	413.1	368	216.2	342.2	263.8	111.9	212.3	133.7
	494.5	545.1	451.6	255.9	421.3	276.8	121.4	225.6	156.3
	635.4	742.2	540.1	304.9	486.2	308.4	130.9	242.3	170
σ_1 (kPa)	183	185.6	159.7	122.9	145.6	116.7	98.5	95.8	83.7
	528.2	563.1	518	366.2	492.2	413.8	261.9	362.3	283.7
	794.5	845.1	751.6	555.9	721.3	576.8	421.4	525.6	456.3
	1085.4	1192.2	990.1	754.9	936.2	758.4	580.9	692.3	620
C_u (kPa)	71.3	67.5	71.4	58	68	63	48.1	54	42.8
ϕ_u (deg)	19.3	22.1	17	9.4	15.8	9.8	2	7.8	4.9
				K-treated soil					
$\Delta\sigma_f$ (kPa)	187.4	189.8	169.6	128.4	129.1	105.6	98.5	96.6	73.1
	349	396.1	349	257.5	349.4	250.5	111.9	194	87.2
	500	635	493	305.7	425.6	269	121.4	231	105.5
	691	783	617	362.2	511.3	319	130.9	248	129
σ_1 (kPa)	187.4	189.8	169.6	128.4	129.1	105.6	98.5	96.6	73.1
	499	546.1	499	407.5	499.4	400.5	261.9	344	237.2
	800	935	793	605.7	725.6	569	421.4	531	405.5
	1141	1233	1067	812.2	961.3	769	580.9	698	579
C_u (kPa)	62.8	64.3	65	61	61.4	55.8	48.1	50.7	33.4
ϕ_u (deg)	21	23.7	19.4	11.6	17.1	10.6	2	8.2	3.4

decrease. However, this increase in moisture content resulted in a decrease of the unconsolidated undrained angle of internal friction.

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