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EFFECT OF SAND ADDITIVES ON THE ENGINEERING PROPERTIES OF FINE GRAINED SOILS

Orabi S. Al Rawi[1], Mohammed N. Assaf[1] and Nidal M. Hussein[2]
[1]Department of Civil Engineering, Isra University, Amman, Jordan
[2]Department of Civil Engineering, University of Petra, Amman, Jordan
E-Mail: orabi.alrawi@iu.edu.jo

ABSTRACT

An experimental research was conducted to investigate the improvement in the engineering characteristics of a cohesive soil after being mixed with gradual increments of sand. To verify the above purpose, several laboratory tests were performed for both the original and mixed soils. These tests were classification, Atterberg limits, permeability, unconfined compression, and direct shear. The results of these tests showed that the values of liquid limit and plasticity index decreased with increasing the percentages of sand additives. Increasing of sand additives also showed an increase in the soils' coefficient of permeability; however, with this parameter, the effect was marginal. Moreover, increasing these additives resulted in an increase in the soils' angle of internal friction and a decrease in its cohesion; in general, for those parameters, the overall trend was increasing the soils' shear strength with increasing sand additives. Out of the results of this research, it was concluded that mixing about 20% of sand material with a cohesive soil had a pronounced influence on the engineering characteristics of the original soil after being mixed, and therefore could enhance its overall engineering behavior.

Keywords: cohesive soils, engineering properties, soils, soil improvement, stabilization.

INTRODUCTION

In soil engineering, it was understood that a fine grained soil is one of the most problematic soils in the word that could show different types of volumetric changes when acted as supported materials below foundations [1]. However, the commonly used term related to fine grained soils is expansive soils. Expansive soils owe their swelling and shrinkage characteristics to the presence of swelling clay minerals. As they get wet, the clay minerals absorb water molecules and expand; conversely, as they dry they shrink, leaving large voids in the soil [2, 3, 4]. In general, fine grained soils have low shear strength, and as a result, may exhibit very low bearing capacity.

A lot of researchers investigated different methods to enhance the physical and mechanical properties of cohesive soils. The majority of them mixed different types of materials with the fine grained soil in order to achieve the above purpose. However, some of these studies are briefly explained below.

Cai *et al.*, (2006) investigated the effect of mixing different ratios of lime and polypropylene fiber on the engineering properties of clayey soils. Measurements of direct shear, unconfined compression, shrinkage and swelling tests had been taken for nine soil mixtures. The results of this research indicated that an increase in lime content showed an initial rise followed by an inadequate decrease in the value of cohesion, friction angle, and unconfined compressive strength of the clayey soil. Moreover, mixing of fiber with clay caused an improvement in shrinkage and strength potential, but produced a decline in swelling potential [5].

Garzon *et al.*, (2016) studied the potential of using different percentages of lime material to improve and stabilize the engineering properties of Spanish phyllite clay. The results showed that the mixture had satisfying compaction attributes and highly to an extremely low coefficient of permeability values [6].

Modarres and Nosoudy (2015) detected the influence of adding coal wastes materials (natural state and after igniting) and hydrated lime powder, on the properties of medium plastic clay. Unconfined compression test, Atterberg limits and California bearing ratio along with swelling tests, were performed for different mix proportions. The analysis of results denoted an improvement in the soil bearing capacity with the addition of coal waste powder and its ash. Moreover, the incorporation of lime with these additives showed significantly higher compressive strength [7].

Pourakbar *et al.*, (2015) fulfilled a study related to utilized Palm oil fuel ash (POFA) in stabilizing soft soils. Several soil tests had been conducted to determine the behavior of clayey soils with adding POFA and cement. Unconfined compressive strength (UCS), Atterberg limit, and Proctor tests had been carried out for the original and mixed soils. This research indicated that adding of POFA and POFA/cement mixture to clayey soil resulted in a significant decrease in the soil plasticity index (PI), a reduction in the optimum moisture content, and an increase in the maximum dry density [8].

Keramatikerman *et al.*, (2016) studied the effect of adding ground granulated blast furnace slag (GGBFS) with lime on the mechanical and strength properties of lime stabilized clay soil. Unconfined compressive strength (UCS), ring shear (RS) and volumetric shrinkage strain (VSS) tests were performed for this study. The results showed that the addition of GGBFS to lime is beneficial in reducing the volumetric shrinkage of lime stabilized clay and that decreasing the volumetric shrinkage behavior. The UCS results explained that the replacement of lime with GGBFS drove to significantly rising compressive strength for all aging periods. The ring shear results also proved that the partial replacement of lime with GGBFS influenced to more significant shear strength [9].

Considering the results of the above literature; in this research, it was realized that further investigations regarding the enhancement of the engineering properties of fine grained soils (using other mixed materials rather than those suggested in the literature) are still needed to be studied; and hence, the idea of this research had been adopted.

RESEARCH METHODOLOGY

The methodology of this research consisted of two stages. The first stage (the theoretical part) included collecting and reviewing for previous studies concerning the subject of this research; whereas, the second stage (the practical part) was focused on conducting laboratory tests program for natural and mixed soil samples, analyzing for results, and submitting of conclusions.

Through the period of performing the practicalpart, undisturbed fine grained soil samples were obtained from an excavated site at Ma'daba (about 33 km to the south of the Capital Amman); whereas, sand soilwas brought from an adjacent local market. The obtained samples were put inside waterproof plastic bags, then placed in a wooden box and transported to the laboratory.

Mixed soil samples (remoulded samples) comprising the original soil and different sand additives (2%, 5%, 10%, and 20% by weight) had been prepared (with the same initial unit weight and moisture content for the original soil) for laboratory tests program.

Several engineering properties for the original soil and those for the mixed soil samples had been obtainedthroughconducting a set of laboratory tests. These tests were performed according to American Society for Testing and Materials (ASTM) Standards [10], including:

- Water Content.
- Specific Gravity.
- Bulk Density.
- Particle-Size Analysis.
- Atterberg Limits.
- Permeability.
- Unconfined Compression.
- Direct Shear.

As stated before, the major purpose of the above tests was to study the effect of mixing sand additives on the overall characteristics of the fine grainedsoil, and then to recommend the most applicable sand mix ratio that could show significant improvements in the engineering behavior of the cohesive soil.

RESULTS AND DISCUSSIONS

TESTS RESULTS FOR THE ORIGINAL SOIL

The results of testing water content, bulk density, and specific gravity for the original soil are indicated in Table-1; whereas, Table 2 & 3 shows the results of the sieve and hydrometer analysis tests for the same soil, respectively.

Table-1. Water content, bulk density, and specific gravity tests results for the original soil.

Type of test	Results
Water Content,%	14.6
Bulk Density, gm/cm^3	1.83
Specific Gravity	2.69

Table-2. Sieve analysis test results for the original soil.

Sieve number	Mass of retained soil, gm	Retained soil %	Accumulative retained soil, %	Percent finer
4	0	0	0	100
10	1.9	0.3	0.3	99.7
20	3.4	0.5	0.8	99.1
30	3.7	0.6	1.4	98.6
40	4.4	0.7	2.1	97.9
100	4.6	0.7	2.8	97.2
200	6.9	1.1	3.9	96.1

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Table-3. Hydrometer analysis test results for the original soil.

Time sec	Temperature °C	R	R_{cl}	R_{cp}	L cm	D mm	Percent finer
0.25	25	47	48	41.85	8.4	0.073	82.8
1	25	45	46	39.85	8.8	0.037	78.9
2	25	43	44	37.85	9.1	0.027	74.9
4	25	39	40	33.85	9.7	0.019	67.0
8	25	30	31	24.85	11.2	0.014	49.2
15	24	25	26	20.15	12.0	0.011	39.8
30	24	23	24	18.15	12.4	0.008	35.9
60	23	21	22	16.45	12.7	0.005	32.5
1440	24	19	20	14.15	13.2	0.001	28.0

R: hydrometer reading
R_{cl}: corrected reading for effective length
R_{cp}: corrected reading for % finer
L: effective length; andD: diameter of particles

Referring to the results indicated in Tables 2 & 3, the grain size distribution curve for the original soil was drawn in Figure-1. Considering the above results and the obtained relationship, it was realized that the percent of fine grained material was about 96%; whereas, the remaining percent (about 4%) was coarse. In order to classify this soil according to the Unified Classification System, liquid and plastic limit tests were conducted; their results are shown in Table-4. Therefore, based on the Plasticity Chart relationship shown in Figure-2, the soil was classified as (CL).

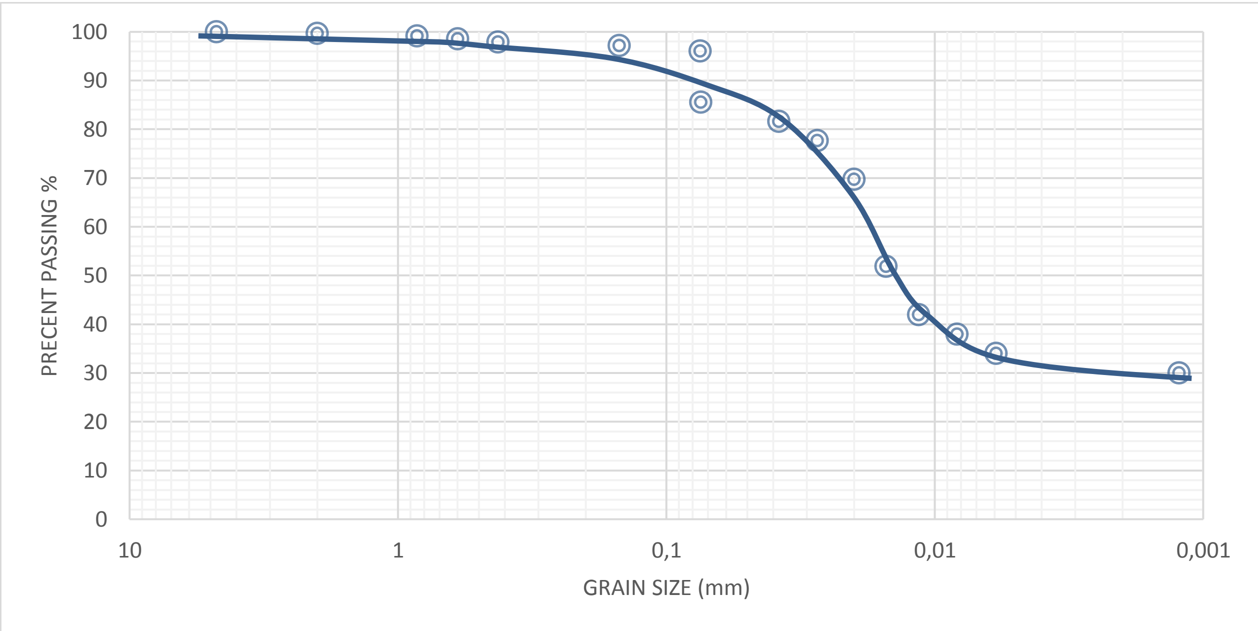

Figure-1. Grain size analysis for the original soil.

Table-4. Liquid and plastic limits tests results for the original soil.

	Liquid limit test results					**Plastic limit test results**		
Test No.	1	2	3	4	5	1	2	3
Number of blows	35	33	23	19	16			
Mass of wet soil sample + container (gm)	48.4	49.0	54.0	48.9	47.3	30.0	31.0	31.1
Mass of dry soil sample + container (gm)	45.3	43.0	46.0	41.0	41.2	30.4	30.4	30.0
Mass of container (gm)	37.9	30.0	29.0	25.0	29.0	26.9	27.1	24.6
Water content (%)	42.6	46.2	46.8	49.6	50.0	20.6	19.1	20.1
Summary of Results	Liquid Limit = 47					Plastic Limit = 20		

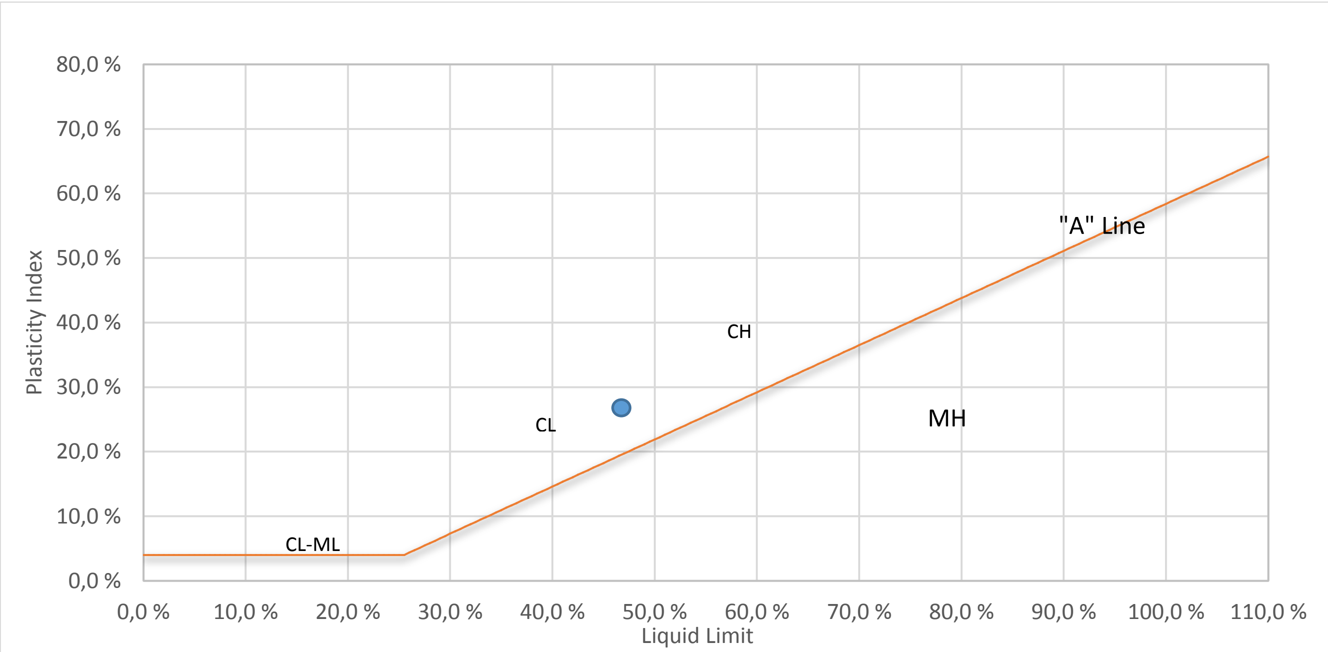

Figure-2. Plasticity chart for classifying the original soil.

The coefficient of permeability of the original soil was obtained using falling head permeability test. According to Das B. and Sobhan K., 2016, the value of this coefficient was calculated using the following equation:

$$K = 2.3 \times \frac{a \times L}{A \times t} \times \log \frac{h}{h_1} \quad (1)$$

Where:

a: Cross-sectional area of the stand pipe = 0.2827 cm^2
L: Length of soil sample = 11.64cm
A: Cross-sectional area of the soil sample =78.5 cm^2
t: Elapsed time of test = 19800 sec
h: Initial head of water = 140.6 cm
h_1: Ending head of water = 133 cm

And therefore, the value of this coefficient was calculated to be 1.17×10^{-7} cm/sec.

The stress-strain relationship for the original soil was obtained using the data of the unconfined compression test results shown in Table-5.

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Table-5. Unconfined compression test results for the original soil.

Deformation dial reading	Proving ring dial reading	Strain*	Corrected area cm^2	Vertical load** kg	Vertical stress, kg/cm^2
0	0	0	11.57	0	0
25	8	0.325521	11.61	4.9	0.4
50	19	0.651042	11.65	11.7	1.0
75	32.5	0.976563	11.69	20.1	1.7
100	46	1.302083	11.73	28.5	2.4
125	57	1.627604	11.77	35.3	3.0
150	67	1.953125	11.81	41.5	3.5
175	75	2.278646	11.85	46.5	3.9
200	81	2.604167	11.88	50.2	4.2
225	79	2.929688	11.92	48.9	4.1
250	75	3.255208	11.96	46.5	3.9

*Considering a sample height of 76.8 mm
** Using a dial gauge factor of 0.62

The shear strength parameters (i.e., the cohesion and friction angle) for the original soil were obtained using the direct shear test results shown in Tables 6, 7, and 8.

Table-6. Direct Shear Test Results (Using a Vertical Pressure of 0.5 kg/cm^2).

Horizontal displacement mm	Load dial gauge reading*	Shear force (S)*kg	Shear stress** kg/cm^2
0.0	0.0	0.00	0.00
0.55	70	10.85	0.30
1.55	145	22.47	0.62
2.40	260	40.3	1.11
3.30	366	56.73	1.57
3.70	384	59.52	1.65
4.18	370	57.35	1.59

* Using a dial gauge factor of 0.155
** Area of soil sample =36cm^2

Table-7. Direct shear test results (Using a vertical pressure of 1 kg/cm^2).

Horizontal displacement mm	Load dial gauge reading	Shear force (S)*kg	Shear stress** kg/cm^2
0.0	0.0	0.00	0.00
0.55	70	10.85	0.30
1.55	110	17.05	0.47
2.40	260	40.30	1.11
3.30	366	56.73	1.57
3.70	399	61.84	1.71
4.18	386	59.73	1.66

* Using a dial gauge factor of 0.155
** Area of soil sample = 36 cm^2

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Table-8. Direct shear test results (Using a vertical pressure of 2 kg/cm^2).

Horizontal displacement mm	Load dial gauge reading	Shear force (S)* kg	Shear stress** kg/cm^2
0.0	0.0	0.00	0.00
0.86	213	33.01	0.91
1.78	289	44.79	1.24
2.63	349	54.09	1.50
3.51	412	63.86	1.77
4.40	455	70.52	1.95
5.29	448	69.44	1.92

* Using a dial gauge factor of 0.155
** Area of soil sample = 36 cm^2

Referring to the above results, shear strength parameters for the original soil were calculated to be 1.5 kg/cm^2 and 13° for the cohesion and friction angle, respectively.

Tests results for the mixed soil samples

Liquid and plastic limits tests were conducted for the mixed soil samples. The determination of "moisture content corresponding to 25 blows" test results (i.e., liquid limit results) for soil samples of various sand additives are presented in Figure-3. This figure shows that the liquid limit values for samples with mixed ratios of 0%, 2%, 5%, 10%, and 20% (by weight) are 47, 44, 40, 39, and 33, respectively. Accordingly, it is clearly indicated that the liquid limit is decreasing with the increase of the sand additive ratio.

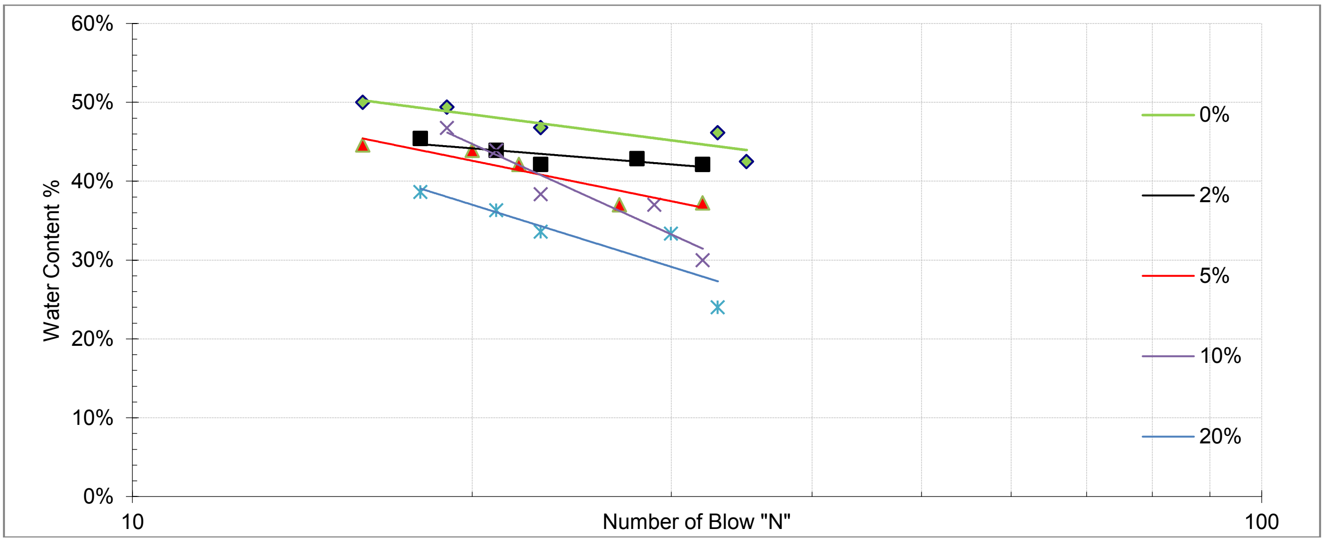

Figure-3. Liquid limit for various sandadditives.

Figure-4 summarizes Atterberg limits tests results for both the original and mixed soil samples. The general trend for this figure shows a decrease of plasticity index with the increase of the soil's sand additive.

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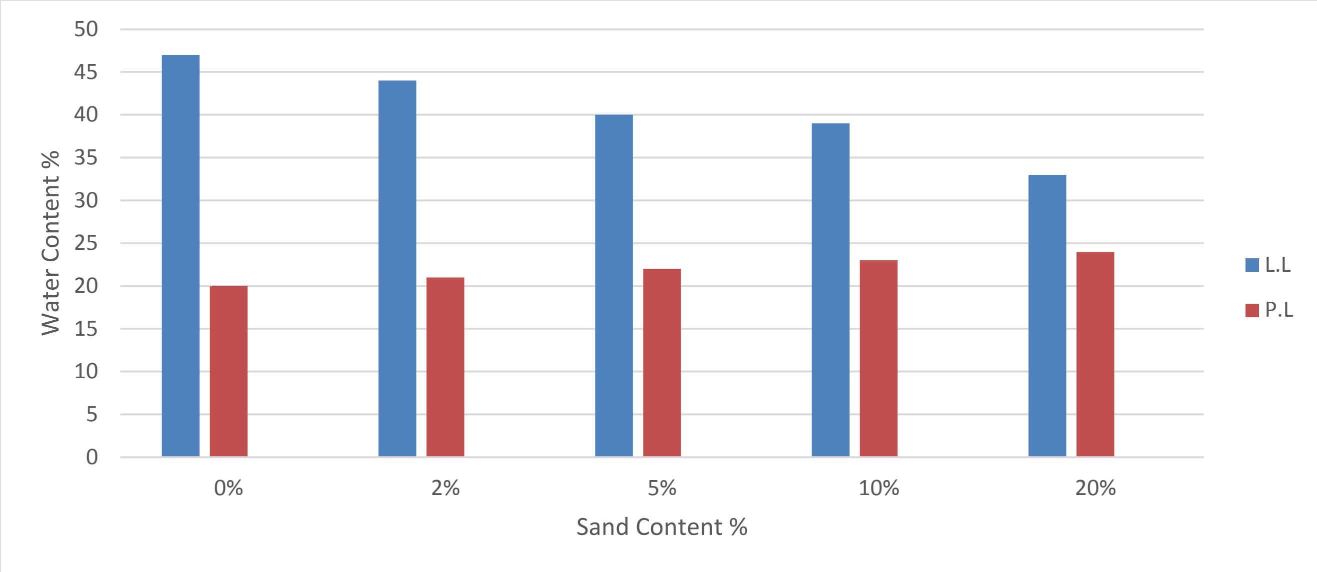

Figure-4. Liquid Limits (L.L) and Plastic Limits (P.L) for various sandadditives.

Falling head permeability tests results for the mixed soil samples are shown in Table 6. Whereas, Figure-5 shows the variation of the coefficient of permeability with the variation of the percentage of sand content. Considering these results, it is clearly indicated that the above coefficient is increasing with the increase of the sand additive ratio.

Table-9. Falling head permeability tests results for various sand ratios.

Sand content, %	2	5	10	20
Cross-sectional area of the stand pipe, cm^2	0.2827	0.2827	0.2827	0.2827
Diameter of soil sample, cm	10	10	10	10
Length of soil sample, cm	11.64	11.64	11.64	11.64
Cross-sectional area of the soil sample, cm^2	78.5	78.5	78.5	78.5
Initial head of water, cm	140.6	140.6	140.6	140.6
Ending head of water, cm	129	115	92	68.8
Elapsed time of test, sec	16200	18000	19800	23400
Coefficient of permeability cm/sec	$2.22*10^{-7}$	$4.68*10^{-7}$	$8.98*10^{-7}$	$1.28*10^{-6}$
Correction Factor RT at t = 21°C	0.976	0.976	0.976	0.976
Coefficient of permeability, cm/sec (corrected to 20°C)	2.17×10^{-7}	4.56×10^{-7}	8.76×10^{-7}	1.24×10^{-6}

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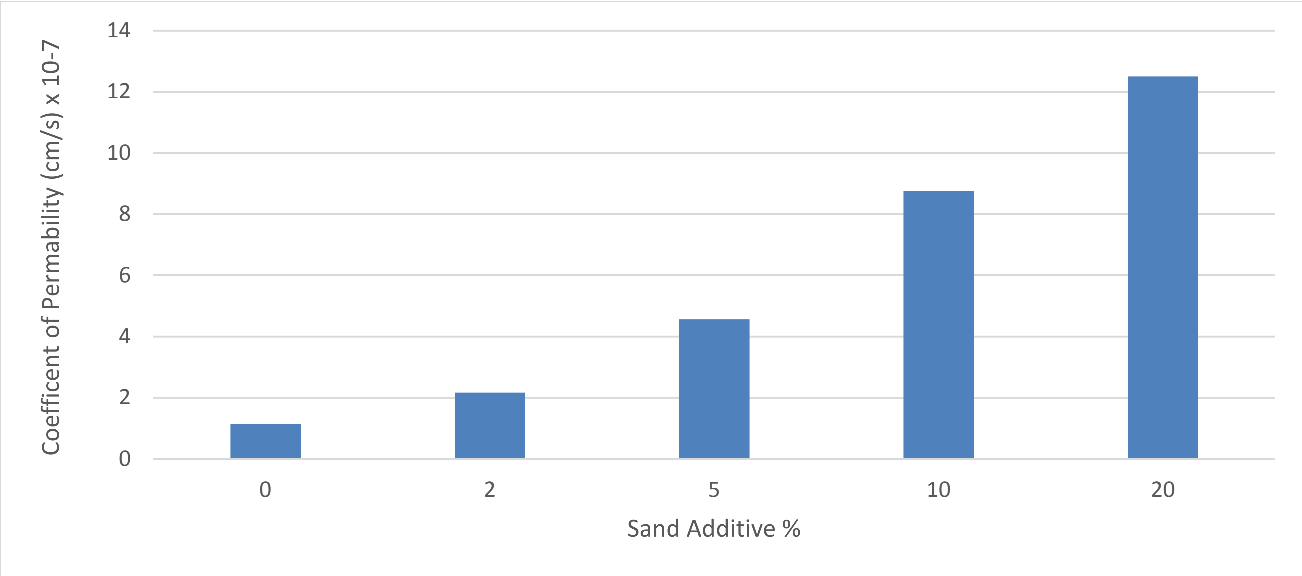

Figure-5. Coefficient of permeability for the original and mixed soil samples.

Figure-6 shows the stress-strain relationship to determine the peak stress for both the original and mixed soil samples. Considering this relationship, it was realized that the peak stress value is relatively increasing with the increase of the sand additive.

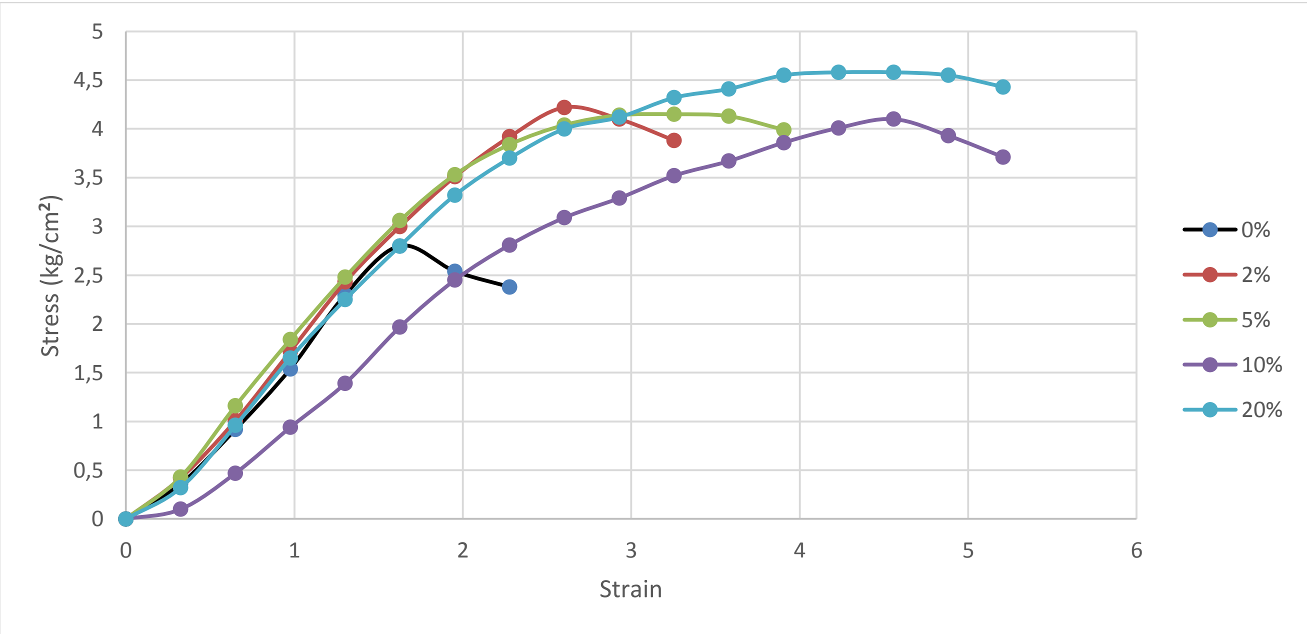

Figure-6. Stress-strain relationship for the original and mixed soil samples.

As illustrated in Figure-7 and Table-10; an increase in the sand additive for the original soil samples caused a distinguished decrease in the cohesion values, and an increase in the friction angle. However, this behavior may be due to the fact that the addition of sand caused an increase in the friction between soil particles, and also a reduction in the interlocking action.

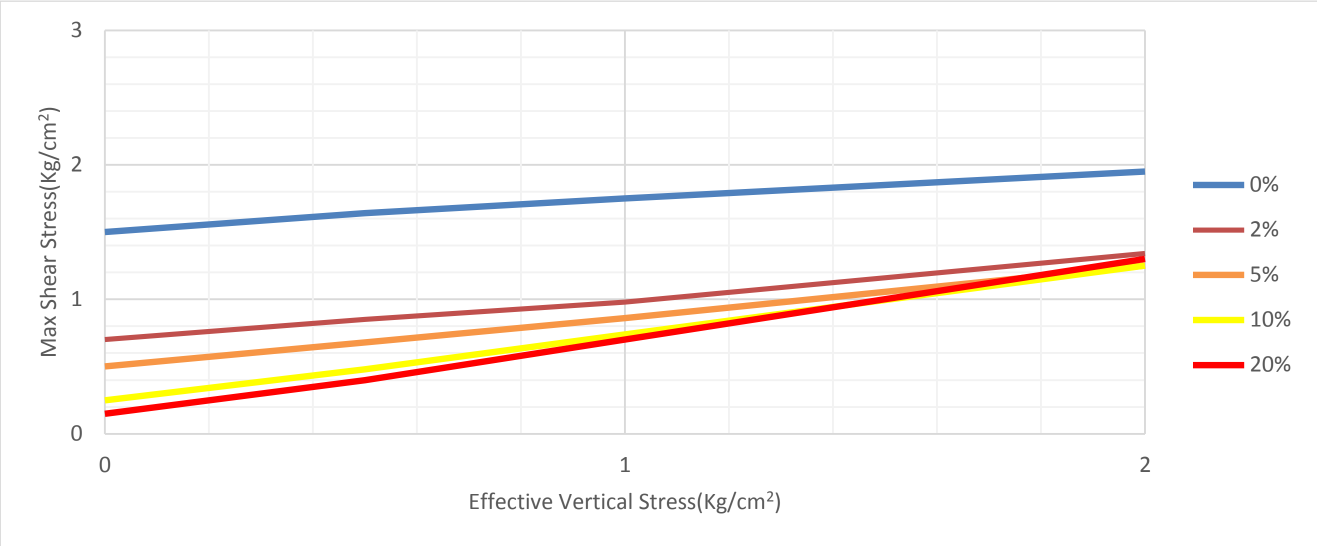

Figure-7. Direct shear test results.

Table-10. Results of cohesion and friction angle for the original and mixed soil samples.

Soil type	Cohesion kg/cm^2	Ø Degree
Original soil sample	1.50	13
Mixed soil sample (with 2% sand additive)	0.70	18
Mixed soil sample (with 5% sand additive)	0.50	21
Mixed soil sample (with 10% sand additive)	0.25	27
Mixed soil sample (with 20% sand additive)	0.15	30

CONCLUSIONS

Considering the results of this research; the following conclusions are drawn:

a) Increasing of sand additives to the cohesive soil resulted a decrease in the liquid limit and plasticity index that could show a lower ability for the soil to expand and shrink, and then lead to considerable (positive) effects on the performance of highways or buildings resting on these types of soil mixes.

b) Increasing of sand additives to the fine grained soil may lead the soil to be more permeable material (i.e., a decrease in the possibility of using the soil mix as an impermeable material in some civil engineering projects).

c) Sand additives showed a general increase in the unconfined compressive strength. However, higher ratios of sand additives showed amarginal increase in these values.

d) Considering the shear strength parameters, the results showed that increasing of sand additives caused a decrease in the soils' cohesion and an increase in its friction angle (i.e., a general increase in the soils' shear strength).

e) For the five types of the conducted tests (i.e., classification, Atterberg limits, permeability, unconfined compression, and direct shear); a sand additive of 20% by weightshowed a significant influence on the results; therefore, mixing of 20% sand with the cohesive soil may create clear effects on the overall engineering behavior of the resulted soil (provided that the recommended mixed soil are carried out by compacted layers based on the available standards).

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