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GEOTECHNICAL EVALUATION OF SOILS FOUND IN KONSO TOWN

Democracy Dilla[1], P. Vasudeva Rao[2]

[1]*Democracy Dilla : lecturer, Dept. of Civil Engineering, Arbaminch University, Ethiopia*
[2]*P. Vasudeva Rao : Assoc. professor, Dept. of Civil Engineering, Arbaminch University, Ethiopia*

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Abstract - *The present study aims to carry out index and engineering properties of soils found in konso town. Konso is one of the fast growing town in Ethiopia. So, geotechnical evaluation of soils found in konso town is very important for pavements, construction of domestic houses, multistory buildings and other structures. Locally available soils are collected in different depths at 8 different locations for experimental work. All the experiments were performed under laboratory controlled condition. It was observed that the soil properties like natural moisture content, free swell index, specific gravity and density values increased, whereas liquid limit and plastic limit values are decreased with increase in depths. Unconfined compression strength values are ranges between 245 to 332 kN/ m2, the cohesion and internal friction values are varies between 160 to 191.34 kN/m2 and 10 to 12º also it was observed from grain size distribution analysis clay fraction varies between 33 to 50.7.*

***Key Words*: compaction, shear strength, free swell index, consolidation, konso**

1. INTRODUCTION

Tropical weathered residual soils are derived from the in situ weathering and decomposition of rock which has not been transported from its original location. Particles of residual soil often consist of aggregates or crystals of weathered mineral matters that breakdown and become progressively finer if the soil is manipulated. As it is outlined in a geological report of the area a peculiar granulite rock is exposed south west of Lake Chamo south of Konso village in Segen valley named as Segen gneiss in addition the granulite rock is found in hammer range of southwestern Ethiopia
Residual soils are formed by the weathering of rocks, through Physical, Chemical and Biological processes. Most commonly, residual soils are formed from igneous or metamorphic parent rocks, but residual soils formed from sedimentary rocks are not uncommon. Chemical processes tend to predominate in the weathering of igneous rocks, whereas physical weathering are so closely interrelated that one process never proceeds without some contribution by the other

Physical weathering includes the effect of such mechanical process as abrasion, expansion, and contraction. Physical weathering produces end products consisting of angular blocks, cobbles, gravel, sand, silt and even clay sized rock flour. The mineral constituents of all these products are exactly like those of the original rock. Chemical weathering, on the other hand, results in the decomposition of rock and the formation of new minerals.

The chemical changes operating in primary minerals of the rocks in temperate or semitropical zones tend to produce end products consisting of clay minerals predominately represented by kaolinite and occasionally by Halloysite and by hydrated or dehydrous Oxides of Iron and Aluminum.

Chemical weathering is favored by warm humid climates, by the process of vegetation and by gentle slope. Thus, tropical and subtropical regions of low relief with abundant rainfall and high temperature are the most susceptible to chemical alterations. Deep, strongly leached red, brown and yellow profiles are manifestations of the effects of sever chemical weathering. Under conditions favorable to tropical weathering, the weathering processes may be so intense and may continue so long that even the clay minerals, which are primarily hydrous aluminum silicates, are destroyed. In the continued weathering the silica is leached and what remains consists merely of Aluminum Oxide such as Gibbsite, or of Hydrous Oxide such as Limonite or Goethite derived from the Iron. This process is known as laterization.

Climate exerts a considerable influence on the rate of weathering. Physical Weathering is more predominant in dry climates while the extent and rate of chemical weathering is largely controlled by the availability of moisture and by temperature. The clay minerals of the soils of the world changed in predictable way with distance from the equator Topography controls the rate of weathering by partly determining the amount of available water for each zone of weathering. Precipitation will tend to run off hills and accumulate soils in valleys and hollows.

Soil profiles developed from basic Igneous Rocks on hillsides the depth of weathering increase down the slope whereas Kalioite / Hallosite are the predominant clay minerals at the top of the slope and Smectite at the bottom of the slope

2. LITERATURE REVIEW

In many countries of Africa and Asia, lateritic soils are the traditional materials for road and airfield construction. Though a good deal of literature is available on lateritic soils and several excellent reviews have been prepared on lateritic soils (Lyon, 1971), The available data on lateritic soils gives the impression that the red color seems to have been accepted by most authors as the most important property by which these soils could be identified. Other obviously significant basic physical properties such as texture, structure, consistency, etc., often were ignored .It is also noted that the lack of uniformity in pretreatment and testing procedures (resulting from association with different standards in different parts of Africa) makes it difficult to compare even textural data on the same soils. It is noted that the major factors influence the engineering properties and field performance of lateritic soils. Those are;

- ✓ Soil forming factors (e.g. parent rock, climate vegetation conditions, topography and drainage conditions).
- ✓ Degree of weathering (degree of laterization) and texture of the soils, genetic soil type, the predominant clay mineral type and depth of sample.

The extent to which a residual soil has been laterized may be measured by the ratio of silica, SiO2, remaining in the soil (except for discrete pebbles of free quartz that may remain) to the amount of Fe2O3 and Al2O3 that has accumulated. The Silica: Sequioxide ratio gives as

$$\frac{SiO_2}{R_2O_3} = \frac{SiO_2}{Fe_2O_3 + Al_2O_3}$$

The equation given above served as basis for classification of residual soils. Ratios less than 1.33 have sometimes been considered indicative of true laterites, those between 1.33 and 2.00 of lateritic soils and those greater than 2.00 of non-lateritic tropically weathered soils. The soils are broadly differentiated on a genetic basis, determined by soil forming factors. It is a means of identification of lateritic soils. (Lyon, 1971)

The free swell test is performed by slowly pouring 10cm^3 of dry soil which has passed the No. 40 (0.425mm) sieve in to 100 cm^3 graduated cylinder filled with distilled water. After 24 hours, final volume of the suspension being read. Hence, free swell is given as

$$\text{Free swell (FS)} = \frac{V_f - V_o}{V_o} x100$$

Where V_f = Final Volume of the soil
V_o = Initial volume of the soil

The degree of expansivity and possible damage to lightly loaded structures may be qualitatively assessed from Table 2.1 In areas where the soils have high or very high FS values, conventional shallow foundations may not be adequate.

Table 1.1: Degree of expansiveness and Free swell (FS)*(Bujang B.K. Huat, 2013)*

Degree of expansiveness	Free swell (FS)
Low	< 20
Moderate	20-35
High	35-50
Very high	>50

Skempton's colloidal activity is determined as the ratio of the plasticity index of the clay content to fines. He observed that, for a given soil, the plasticity index is directly proportional to the percent of clay-size fraction (i.e., percent by mass finer than 0.002 mm in size).

Activity designated by "A" is defined as A=PI/C

Where C is the percent of clay - size fraction by weight. Activity has been used as an index property to determine the swelling potential of clays (Das, 2002). The low activity of most lateritic soils is due to the mode of weathering which involve the coating of the soil particles with Sesqueoxide, which results in the suppression of the surface activity of clay particles (Lyon, 1971).

The permeability of a soil is a measure of a how easily fluids (usually water) pass through the soil and is related to degree of to which the pores spaces of the soil connected to each other. The permeability of a particular soil is defined by coefficient of permeability, K.

The permeability of the soil is geologically controlled by factors such as the shape minerals grains in the soil, the grain shape and size, the manner in which the shape the grains are held together, soil porosity, density and viscosity of inside soil, degree of soil saturation and type of flow inside the soil .To some extent, The permeability of the soil is controlled by confining the effective stress with in soil mass. Table 2.2 shows permeability of some tropical residual soils

Table 2-.2: Permeability of some Tropical Residual soils

Parent material	Permeability (m/s)	Location	Sources
Basalt	9 x10^{-10} to 9 x 10^{-5}	Brazil	Cost Filho & Vargas Jr (1985)
Granite	1x 10 $^{-9}$ to 1x	Singapor	Poh et al.(1985)

	10^{-8}	e	
Granite	5x 10^{-6} to 2 x 10^{-4}	Hong kong	Lumb (1962)
Granite	5 x 10^{-9} to 5x 10^{-8}	Malaysia	Ting & Ooi (1972)
Gneiss	9x 10^{-7} to 5x 10^{-5}	Brazil	Cost Filho & Vargas Jr (1985)
Gneiss	9 x 10^{-7}	Brazil	Cost Filho & Vargas Jr (1985)

Shear strength is property a material that enables to remain in equilibrium when its surface is not level. Soils in liquid form have virtually no shear strength and even when solid have shear strengths or relatively small magnitudes compared with those exhibited by steel or concrete. The shear strength of a soil is its resistance to shearing stresses. It is a measure of the soil resistance to deformation by continuous displacement of its individual soil particles. Shear strength in soils depends primarily on interactions between particles. Soil derives its shear strength from two sources, Cohesion and frictional resistance between soil particles.

Shear strength controls the stability of a soil mass under loads. It governs the bearing capacity of soils, the stability of slopes in soils, and the earth pressure against retaining structures and many other problems. All the problems of soil engineering are related in one way or the other with the shear strength of soil

The Unconfined Compression Test procedure similar to the UU test procedure, except that no confining pressure is applied to the specimen (i.e. σ3 is equal to zero).The test is commonly performed in a simple loading frame by applying an axial load to the soil specimen (D.G. Fredlund and H.Rahardjo, 1993) . Unconfined Compression Test was done according to ASTM D 2166. This test method covered the determination of the unconfined compressive strength of cohesive soil in remolded condition, using strain-controlled application of the axial load. The sample was prepared with length to diameter ratio of 2. The load that would produce an axial strain of 1% per minute was applied at the specimen and the load and deformation dial readings were recorded at every 10 to 50 divisions on deformation the dial. The loading was applied until the load (load dial) decreases on the specimen significantly.

General relationship of Consistency and Unconfined Compression Strength (UCS) of clay is shown in Table 2.3.

Table 2.3: Relationship between consistency and UCS

Consistency	Unconfined Compressive Strength q_u (kN/m²)
Very soft	0-25
Soft	25-50
Medium	50-100
Stiff	100-200
Very stiff	200-400
Hard	>400

Consolidation test is performed to determine the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures. From the measured data, the consolidation curve (pressure-void ratio relationship) can be plotted. This data is useful in determining the compression index, the recompression index and the pre-consolidation pressure (or maximum past pressure) of the soil. In addition, the data obtained can also be used to determine the coefficient of consolidation and the coefficient of secondary compression of the soil

The earliest and the most widely used method to determine the pre-consolidation pressure was the one proposed by Casagrande (1936). The method involves locating the point of maximum curvature on the laboratory e-log p curve. From this point, a tangent is drawn to the curve and a horizontal line is also constructed. The angle between these two lines is then bisected. The abscissa of the point of intersection of this bisector with the upward extension of the inclined straight part corresponds to the pre-consolidation pressure. The relative amount of pre-consolidation is usually reported as the over-consolidation ratio.

3. MATERIALS and METHODOLOGY

Accurate values of Index and Engineering properties of the soil samples of the study areas are very much helpful data for construction of any structures. To achieve that information was collected from the city administration which included like town plans to assess the expansion potential of the town and the current distribution of the dwellings and buildings in the town. There were four provinces (namely Doketu, Darra, Duraite and Mechelo). A field visit of the town was conducted to identify suitable locations for pit excavations based on the future expansion plan of the town and the representativeness of the test pit in each province.

Laboratory tests were performed to determine the index and engineering properties of the soil. The tests were conducted according to ASTM standards. Standard procedures for performing laboratory tests were followed. Laboratory tests were conducted in the Soil Mechanics laboratory of Arba Minch University under Oven dried (OD) condition - dried in an oven for 24 hours at 105ºc.

In this study soil specimens were collected in different places from Konso-town. The area of the town is judiciously inspected based on visual variation of soil, after prudent inspection, Eight sampling areas were selected at an intervals of around 1.5 to 1.7 km at different parts of the town, which could represent the whole area of the town. Pits were excavated manually. Disturbed and undisturbed soil samples were collected at 1.5m, 3m and 5m depths from Pits No 1 to 4 and at 2m and 5m depths from Pits No 5 to 8. The soil samples were collected from the following sites: Doketu (TP1 and TP2), Darra (TP3 and TP4), Duraite (TP5 and TP6) and Mechelo (TP7 and TP8).The soil samples taken from test pits were shifted to Laboratory for further investigation and tests were conducted under oven dried condition.

The grain-size distribution of mixed soils was determined by combined sieve and hydrometer analyses. Hydrometer analysis was conducted with Sodium hexa meta phosphate dispersing agent for the soil samples passed on NO 200 sieve size. (0.075mm) and the soils were classified by using USCS and AASHTO classification systems.

Atterberg's Limit tests were conducted for oven dried samples (as per the ASTM D4318-00). The OD samples were prepared by putting the samples in an oven at a temperature of 105°C.The wet preparation is also applied for the as received (AR) samples. The portions of the samples passing the No. 40(0.425mm) sieve were used for the preparation of the sample in order to investigate the effect of temperature on the Atterberg Limits.

For UU Triaxial compression test the specimens were prepared as diameter of 38mm and height of 76mm. The dry soil was mixed with natural moisture content and compacted in three layers until the field density is obtained. Three test samples were prepared at each test pits and the test was conducted by applying initial confined pressure and deviatoric load till the soil specimen failed. Similarly the other two samples were tested by increment of the confined pressure followed by diviatoric stress till the soil failed then Mohr circle was developed for shear strength parameter determination.

For consolidation tests, it is used to determine the magnitude and rate of consolidation of soil when it is restrained laterally and drained axially while subjected to incrementally applied controlled-stress loading. Undisturbed soil samples were taken by ring sampler and cut the sides of the sample to be approximately the same as the outside diameter of the ring. Asoil sample diameter of 63mm and 20mm height were used for determination of CC and Cr. The loading pressures were applied as 25kPa, 50kPa, 100kPa, 200kPa, 400kPa, 800kPa and 1600kPa and unloading readings were noted at 1600kPa, 400kPa, 100kPa and 25kPa pressures. The dial gauge readings were recorded for 24 hours

4. RESULTS AND DISCUSSIONS

4.1 Natural Moisture Content

The conventional test for the determination of moisture content is based on the loss of water when a soil is dried to a constant mass at a temperature 105 ºC. To conduct water content test average of three soil samples were taken for better results. The values of the moisture content are summarized in Table 4.1

Table 4.1 Natural Moisture content at oven dried condition 105^{o}C

S.No	Test pit	Depth in (m)	Moisture content (%)
1	TP1	1.5	29.69
		3	31.94
		5	33.05
2	TP2	1.5	32.79
		3	38.96
		5	43.37
3	TP3	1.5	15.5
		3	18.2
		5	20.3
4	TP4	1.5	34
		3	34.8
		5	37.9
5	TP5	2	29.73
		5	30.15
6	TP6	2	27.49
		5	28.22

7	TP7	2	26.2
		5	26.6
8	TP8	2	29.75
		5	32.17

4.2 Free Swell

Free swell test results for oven dried soil samples are summarized in Table 4.2. From the test result it can be seen that the free swell of the soil under investigation ranged from 13% to 43%. Those soils having a free swell less than 50% are considered as low in degree of expansion. Hence all soil samples under investigation are non-expansive soils

Table.4.2 Free swell test results at oven dried Condition

S.No	Test Pit	Depth in (m)	Free Swell (%)
1	TP1	1.5	25
		3	30
		5	38
2	TP2	1.5	38
		3	36
		5	40
3	TP3	1.5	12
		3	13
		5	15
4	TP4	1.5	25
		3	26
		5	30
5	TP5	2	25
		5	28
6	TP6	2	20
		5	22
7	TP7	2	28
		5	28
8	TP8	2	24
		5	26

4.3 Specific Gravity

The specific gravity of laterite soil was found to be increased in specific gravity with depth. This is interpreted as due to a high concentration of iron oxide .The specific gravity was in the range between 2.4 to 2.76. (Table 4.3).Whereas Specific gravity in TP2 and TP3 it is low this can be the type of minerals the soil constitute like Gypsum

Table.2.3 Values of specific gravity at oven dried condition

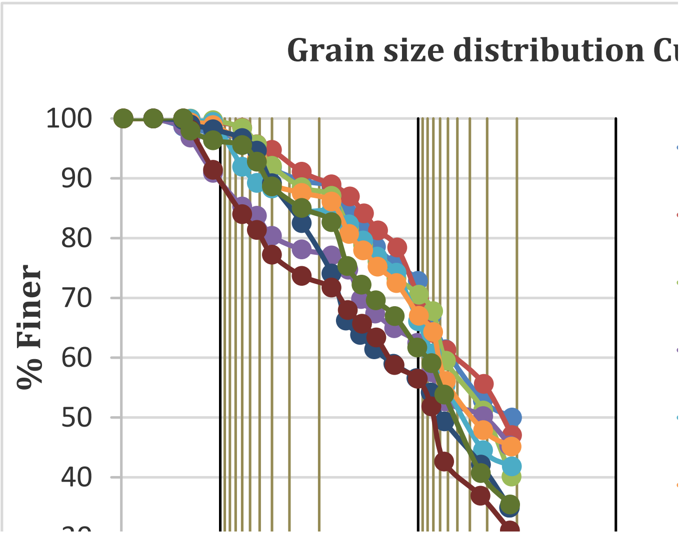
Sl.No.	Test Pit	Depth in (m)	Specific Gravity
1	TP1	1.5	2.61
		3	2.65
		5	2.67
2	TP2	1.5	2.48
		3	2.52
		5	2.51
3	TP3	1.5	2.4
		3	2.41
		5	2.45
4	TP4	1.5	2.7
		3	2.72
		5	2.73
5	TP5	2	2.69
		5	2.71
6	TP6	2	2.71
		5	2.74
7	TP7	2	2.66
		5	2.68
8	TP8	2	2.75
		5	2.76

4.4 Grain size Analysis

The grain size analysis distribution curves for soil samples under investigation at oven dried condition are shown in Fig 4-1.The results obtained from the grain size analyses indicated that the dominant proportion of soil particle in the research area was clay. The percentage of soil passed through sieve No. 200 (0.75mm) was ranging from 71.7%

to 94.8, which indicated that the soil in the study area was fine grained soil.

Fig 4-1: Grain size distribution curves at oven dried condition



4.5 Consistency Limits

Atterberg Limit values for konso city soil is shown on Table 4-4

Table 4-4: Atterberg Limit values at oven dried condition

Test pits	Depth (m)	Test condition	Liquid Limit (%)	Plastic Limit (%)	Plasticity index
TP1	1.5	As received	62.2	39.64	22.56
		Air dried	58.9	35.63	23.27
		Oven dried	57.2	36.44	20.76
		Unsoaked	49.9	28.65	21.25
	3	As received	59.9	37.06	22.74
		Air dried	56.2	34.18	22.02
		Oven dried	53	34.83	18.17
		Unsoaked	51.6	31.5	20.1
	5	As received	53.7	33.58	20.12
		Air dried	51	32	19
		Oven dried	48.9	29.1	19.8
		Unsoaked	47.8	28.7	19.1
	1.5	As received	62.8	37.35	25.45
		Air dried	57	34.86	22.14
		Unsoaked	56.2	35.79	20.41
TP2	3	As received	56	33.71	22.29
		Air dried	54.2	35.01	19.19
		Unsoaked	53.6	34.77	18.83
	5	As received	55	35.77	19.23
		Air dried	53.2	34.67	18.53
		Unsoaked	53.2	32.8	20.4
TP3	1.5	As received	54.6	34.37	20.37
		Air dried	47.7	28.14	19.56
		Unsoaked	45.8	26.55	19.25
	5	As received	48.8	29.15	19.25
		Air dried	46.4	27.27	19.13
		Unsoaked	43.2	24.1	19.3
TP4	1.5	As received	64.2	38.9	25.3
		Air dried	62.5	37.29	25.21
		Oven dried	59.7	38.15	21.55
		Unsoaked	59.4	38.04	21.36
	3	As received	59.4	35.65	23.75
		Air dried	56.2	33.81	22.39
		Oven dried	51.8	33.75	18.05
		Unsoaked	51.5	33.24	18.26
TP5	2	As received	58.4	37.25	21.15
		Oven dried	55.3	36.23	19.07
		Unsoaked	52.5	32	21.5
	5	As received	57.8	36.02	21.78
		Oven dried	52.2	32.5	19.7
		Unsoaked	52.2	40.16	20.7
TP6	2	As received	61.1	40.03	21.07
		Oven dried	54.4	35.08	19.32
		Unsoaked	52.7	33.04	19.66
	5	As received	56.5	36.73	19.77
		Oven dried	50.6	31.5	19.1
		Unsoaked	46	26.5	19.5
TP7	2	As received	58.7	38.21	20.49
		Air dried	54.4	34.9	19.5
		Oven dried	53.7	34.88	18.82
		Unsoaked	49.9	30.78	19.12
	5	As received	56.7	36.84	19.86
		Air dried	56.6	38.37	18.23
		Oven dried	51	31.97	19.03
		Unsoaked	48.9	29.5	19.4
	2	As received	58	36.95	21.05
		Air dried	54.8	36.34	18.46
		Oven dried	54.4	36.21	18.19

TP8		Unsoaked	53.2	34.24	18.96
	5	As received	59.1	38.65	20.45
		Air dried	55.2	35.82	19.38
		Oven dried	54.1	34.37	19.73
		Unsoaked	50.5	32.27	18.27

4.6 Triaxial Unconsolidated Undrained (UU) Test

Based on the results obtained from Triaxial compression test for the soil samples at different locations were conducted Unconsolidated Undrained tests and the results are summarized on Table 4-5, The values of internal friction vary from 10° to 12º and cohesion vary from 160.76 to 191.34 kN/m2

Table 4-5: Unconsolidated Undrained (UU) test Results

Test Pit	Depth (m)	Dry density g/cm3	NMC (%)	qu (kPa)	Su (kPa)
TP1	3	1.71	30	245	122.5
TP1	5	1.77	33	282	141
TP4	5	1.79	38	250	125
TP7	2	1.75	25.4	332	166
TP7	5	1.76	26	326	163
TP8	2	1.65	31	294	147
TP8	5	1.66	32.3	315	157.5

4.7 Unconfined Compression Strength

Based on the results obtained from Stress-Strain curves the values of unconfined compressive strength varies from 245 to 332 kN/m2.The results are shown in Table 4.6.

Table 4-6: UCS test results

Test Pit	Depth (m)	Dry density g/cm3	NMC (%)	qu (kPa)	Su (kPa)
TP1	3	1.71	30	245	122.5
TP1	5	1.77	33	282	141
TP4	5	1.79	38	250	125
TP7	2	1.75	25.4	332	166
TP7	5	1.76	26	326	163
TP8	2	1.65	31	294	147
TP8	5	1.66	32.3	315	157.5

4.8 Consolidation test

From one-dimensional consolidation test results, which is summarized in Table 4.7, the values of the compression indices Cc range from 0.273 to 0.37 which were in the normal range of common silty clay soils.

Table 4-7a: Consolidation test results

Location	Depth (m)	Unit Weights (KN/m3	C_c	C_r	Po
Deketuo (TP1)	5	17.17	0.283	0.033	85.85
Darra (TP4)	5	17.59	0.370	0.044	87.95
Duraite (TP6)	5	18.72	0.340	0.040	93.6
Mechelo (TP7)	5	17.28	0.312	0.037	86.4
Mechelo (TP8)	5	16.25	0.273	0.032	81.25

Table 4-7b: Consolidation test results

Location	Depth (m)	Unit Weights (KN/m3	Cv 10^{-6} mm^2/min	a_v m2/KN	K 10^{-6} cm/sec
Deketuo (TP1)	5	17.17	0.646	0.322	1.734
Darra (TP4)	5	17.59	0.79	0.184	2.394
Duraite (TP6)	5	18.72	0.646	0.355	1.743
Mechelo (TP7)	5	17.28	0.667	0.328	1.79
Mechelo (TP8)	5	16.25	0.678	0.329	1.81

5. Conclusions

Based on the investigations made on the soils of Konso Town, the following conclusions are drawn.

- ✓ Laterite soils of Konso area are characterized by high concentration of Iron Oxide, Aluminum Oxide (Sesqueoxide) and Kaolinite minerals.

- ✓ The soil samples results subjected to fall below A-line under MH (inorganic silt with medium strength).
- ✓ The natural moisture content values ranges from 15.5% to 43.37%
- ✓ The liquid limit and plastic limit values varies from 48.9 to 57.2% and 29.1 to 36.44% respectively.
- ✓ Specific gravity test results for this study varies from 2.40 up to 2.76.
- ✓ Soils variation in percentage of particle sizes in the different test procedures is not significant.
- ✓ The Values of friction angle vary from 10 to 12º and cohesion vary from 160 to 191.34kN/m^2.
- ✓ The values of unconfined compressive strength range from 245 to 332 kN/ m^2, which was in the ranges of very stiff consistency.
- ✓ From one-dimensional consolidation test, the Compression Index Cc ranged from 0.273-0.37 and coefficient of Permeability ranged from1.81x10^{-6} to 2.39x10^{-6} cm/sec. which indicates that the soil in the area is relatively impervious.

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