INVESTIGATION INTO SOME OF THE ENGINEERING PROPERTIES OF RED CLAY SOILS IN BAHIR DAR.

A Thesis

Presented to

School of Graduate Studies

Addis Ababa University

In Partial Fulfillment

of the Requirement for the Degree of

Master of Science in Civil Engineering

BY

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CONTENTS

LIST OF SYMBOLS

a_v	Coefficient of permeability
c_c	Compression Index
C_{cu}	Cohesion value in terms of total stress
C'	Cohesion value in terms of effective stress
C_u	Untrained shear strength
c_v	Coefficient of consolidation
E_s	Modulus of compressibility
G_s	Specific gravity of soil solids
K	Coefficient of permeability
H_{dr}	Drainage depth
I_L	Liquidity index
I_P	plasticity index
q_u	Unconfined compressive strength
U_w	Pore water pressure
W_L	Liquid limit
W_P	Plastic Limit
W_s	Shrinkage limit

ϕ_{cu} Angle of shearing resistance in terms of total stress

ϕ' Angle of shearing resistance in terms effective stress

γ_w Unit weight of water

σ_1 Major principal stress

σ_3 Minor principal stress(cell pressure)

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ABSTRACT

Despite Geotechnical research has not been done in Bahir Dar, from reconnaissance, the majority of the town is covered by red soils comparing to the black soils. The red clay soils are dominantly found in the eastern and western parts of the town. An investigation has been made on the properties of red soils obtained, from locally called 'Abay mado' (Kebele 11), in the eastern part of the town where the expansion is very high for development. Hence the present research is intended to investigate some of the engineering properties of red clay soils in Bahir. After visiting the proposed site, six test pit points were selected. The numbers of test pits were limited because of the constraint in budget and time. Representative disturbed and undisturbed soil samples were collected from open pits by direct excavation manually. The laboratory tests that were carried out includes mineralogical analysis, index properties, consolidation, and shear strength tests and with the aid of these results an analysis is made to determine some characteristics of red clay soils.

The laboratory result shows the red clay soils in Bahir Dar contain the clay mineral kaolinite predominantly. The data resulted from the test is inadequate to provide correlation that relate index properties with shear strength parameters and compressibility characteristics.

1.INTRODUCTION

Bahir Dar, a regional capital, some 565 km North of Addis Ababa was founded in 1915 with the establishment of Ghiorgis Monastery on the southern shore of lake Tana situated at latitude $11^{o}35^{'}$N and Longitude $37^{o}23^{'}$ E. The topography of the town is mainly flat with some small hills on its East and West sides. The existing Master plan of Bahir Dar dates back to the late 1960's. Since then, few revisions in the detail master plan have been made. The master plan in use today covers a land area of about 6000 hectares (Fig 1.1). The town has a potential for expansion in all directions except to the side of Lake Tana. The town has no sewerage system. No systematic soil investigation has been carried out prior to this work.

A thorough and comprehensive geotechnical investigation is an essential requirement to the design and construction of civil engineering projects. The proper design of civil engineering structures like foundation of buildings, retaining walls, high ways, etc. requires adequate knowledge of sub surface conditions at the sites of the structures. Many damages to buildings, roads and other structures founded on soils are mainly due to the lack of proper investigation of substructure condition.

The town of Bahir Dar, having adequate land area for expansion and being an important industrial, commercial, educational and tourist center in the region will have a high potential for future development. A lot of civil engineering structures are under construction, however, nothing has been done on the investigation of soil with respect to the intended urban development plan. Therefore, the objective of the research is to investigate some of the engineering properties of red clay soils in Bahir Dar prevalent in the eastern part of the town, where development has a promising future. After collecting representative disturbed and undisturbed samples from proposed open test pits by direct excavation, following laboratory tests were carried out including mineralogical analysis, index properties, consolidation and shear strength tests.

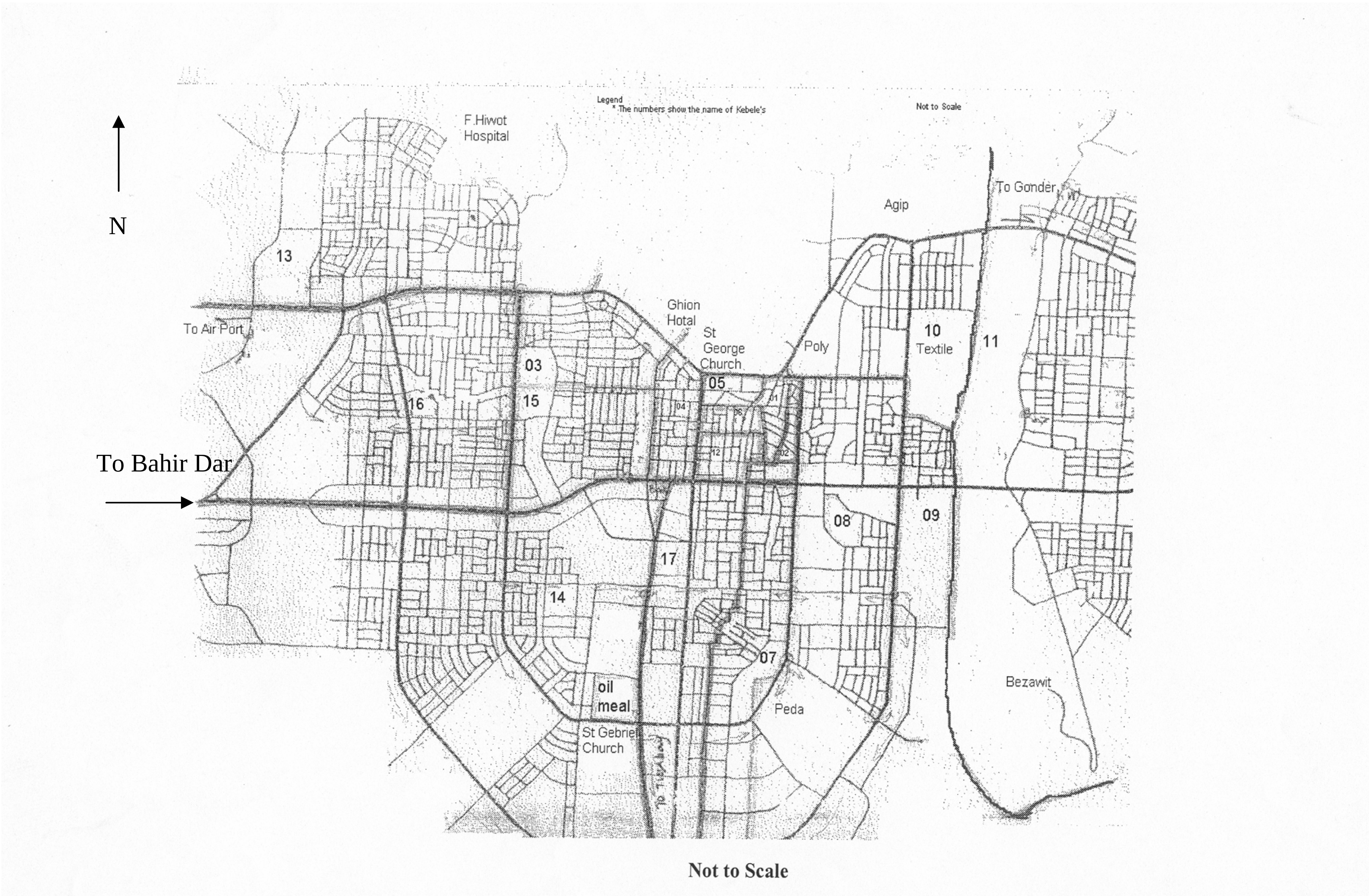


Fig 1.1 Map of Bahir Dar (Courtesy of Municipality of Bahir Dar)

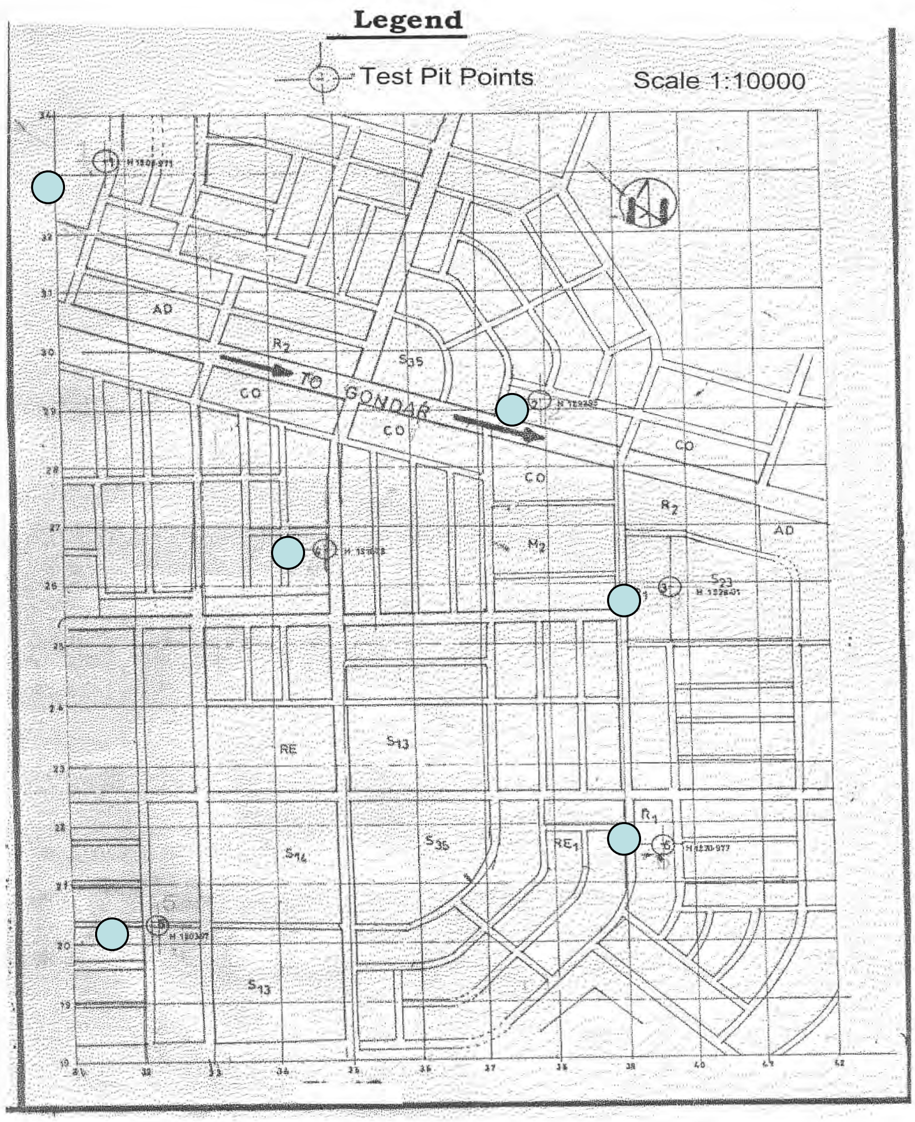


Fig 1.2 Map of Abay Mado (Courtesy of Municipality of Bahir Dar)

2.0 Investigation of Clay Mineralogy

2.1 General

The term clay is applied to the fraction of grains whose equivalent diameter is less than 0.002mm. The individual grains are fragments of a single mineral i.e. a solid compound with a definite chemical composition and unique crystalline structure.

The minerals of clays are formed by the weathering of rocks. Most clay minerals of interest to geotechnical engineers are composed of oxygen and silicon- two of the most abundant elements on earth. Silicates are a group of minerals with a structural unit called the silica tetrahedron. A central silica cation is surrounded by four oxygen anions, one at each corner of the tetrahedron (Fig 2.1a). Silica tetrahedrons combine to form sheets, called silicate sheets (Fig 2.1b). Silicate sheets may contain other structural units such as alumina sheets. Alumina sheets are formed by a combination of alumina minerals, which consist of an aluminum ion surrounded by six oxygen or hydroxyl atoms in an octahedron (Fig 2.1d) [18].

The main groups of clay crystalline materials that make up clays are the minerals kaolinite, illite and montmorillonite.

2.1.1 Kaolinite

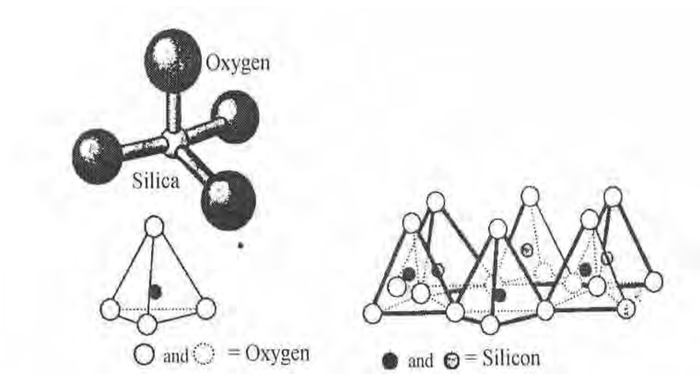
Kaolinite has a structure that consists of one silica sheet and one alumna sheet bonded together in to a layer about 0.72nm (nm = 10^{-9}m) thick and stacked repeatedly (Fig 2.2a). The layers are held together by hydrogen bonds. Kaolinite has a few or no exchangeable cation, and the interlayer bonds are relatively strong preventing any hydration between layers and allowing many layers to build up [20]. Kaolinite is relatively stable and water is unable to penetrate between the layers. Consequently Kaolinite shows little swelling on wetting [22]. Kaolinites are found in soils that have undergone considerable weathering in warm, moist climates. They have low liquid limit and a low activity. Another member of the Kaolinite group appearing in some tropical soils is called halloysite, in which water molecules separate the layers. The halloysites are distinguished by one additional water molecule to the basic kaolinite. In contrast to most other clays, which are flaky, halloysite particles are tabular or rod like.

2.1.2 Montmorillonites

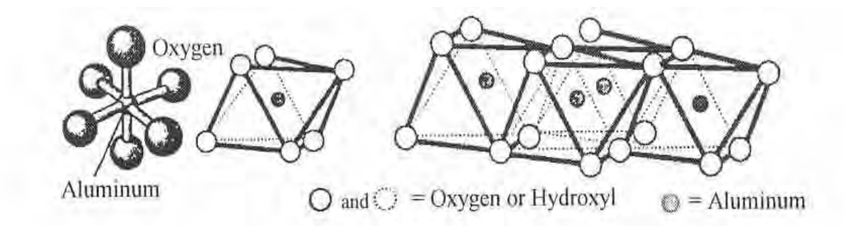
Montmorillonites are made up of sheet like unit comprising an alumna octahedral sheet between two silica tetrahedral sheets, as shown in Fig. 2.2(c).As the electrons rotate around the nucleus of an atom there will be times when there are more electrons on one side of the atom than the other, giving rise to a weak instantaneous dipole. Weak Vander Waals forces hold layers together and the bonding of these sheets is rather weak, resulting in a rather unstable mineral, especially when wet. In fact, montmorillonite display a significant affinity for water, with subsequent swelling and expansion. Its excessive swelling capacity may seriously endanger the stability of overlying structures and road pavements. Bentonite is part of the montmorillonite clay family, usually formed from the weathering of volcanic ash [8].

2.1.3 Illite

The illites are somewhat similar to montmorillonites in the structural units, but are different in their chemical composition. In illite, the layers are separated by potassium ion, where as in montmorillonite the layers are separated by loosely held water and exchangeable metallic ions (Fig 2.2 (b)). Unlike montmorillonite particles, which are extremely small and have a great affinity for water, the illite particles will normally aggregate and there by develop less affinity for water than montmorillonites. Correspondingly, their expansion properties are less. The cation exchange capacity of illite is less than that of montmorillonite. The inner layer bonding by the potassium ions is sufficiently strong. Illites usually occur as a very small, flaky particles mixed with other clay and non-clay materials [8].



(a) *(b)*



(c) *(d)*

Fig. 2.1(a) A silica tetrahedron, (b) Silica sheets, (c) An Aluminum octahedron, and (d) aluminum sheets.

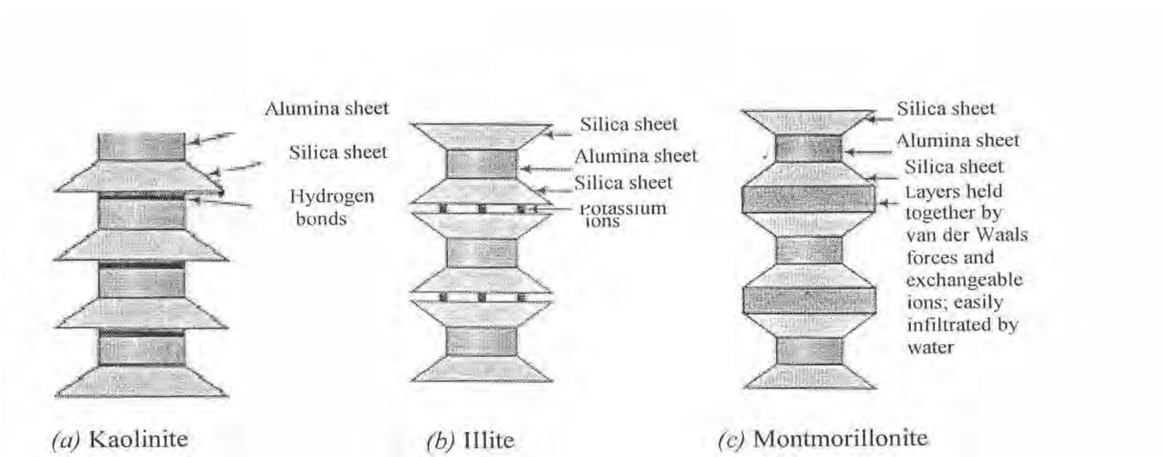


Fig. 2.2 Structure of Kaolinite, illite, and montmorillonite

2.2 Determination of mineralogical composition

Soil mineralogy can be assessed in various ways. Very specialized techniques have been developed for practical purposes, but the most common approaches are x-ray diffraction, thermo-gravimetry, and optical microscopy combined with some form of spectral element identifications. Mineralogical identification using these techniques requires specialized training and procedures. The X- ray diffraction (XRD) technique is by far the most widely used, but is only appropriate for minerals with distinctive crystallography [4]. Since 1923, X -ray analysis became the most widely used method for the investigation of clay minerals [12]. Kaolinite and halloysite are the predominant clay minerals of red soils in Ethiopia [17]. The range in properties of Ethiopian soils prepared by Morin and Parry [17] is attached in appendix A.

To identify the clay minerals of the soil under investigation a few grams of six representative clay samples were taken at the three test pits from the area locally called "Abay Mado" or Kebele11 for x-ray diffraction analysis. The test was conducted by the Ethiopian Geological Survey Mineralogy & Petrography laboratory using X-ray diffraction analysis by powder diffraction method after grinding the bulk soil samples with out separating the clay fraction. The results of the test are shown in Fig 2.4 to 2.9.

Mineralogical analysis shows the red clay soils contain the clay mineral kaolinite dominantly in the range from 10.3 to 24.7%.

FIG 2.4 MINERALOGICAL ANALYSIS OF PIT NO 04, DEPTH 1.5

FIG 2.5MINERALOGICAL ANALYSIS OF PIT NO 04, DEPTH 3.0M

FIG 2.6 MINERALOGICAL ANALYSIS OF PIT NO 05, DEPTH 1.5

FIG 2.7 MINERALOGICAL ANALYSIS OF PIT NO 05, DEPTH 3.0M

FIG 2.8 MINERALOGICAL ANALYSIS OF PIT NO 06, DEPTH 1.5

FIG 2.9 MINERALOGICAL ANALYSIS OF PIT NO 04, DEPTH 3.0 M

3. INDEX PROPERTY TESTS

3.1General

In nature, Soils occur in a large variety. However, soils exhibiting similar behavior can be grouped together to from a particular group. Engineers are continually searching for simplified tests that will increase their knowledge of soils beyond that which can be gained from visual examination without having to resort to the expense, detail, and precision required with engineering properties tests. These simplified tests provide indirect information about the engineering properties of soils and are, therefore, called index tests.

Basic soil properties and parameters can be subdivided into physical, index, and engineering categories. Physical soil properties include particle size and distribution, specific gravity, and water content. Index parameters of cohesive soils include liquid limit, plastic limit, shrinkage limit, and activity. Such parameters are useful to classify cohesive soils and provide correlations with engineering soil properties [15].

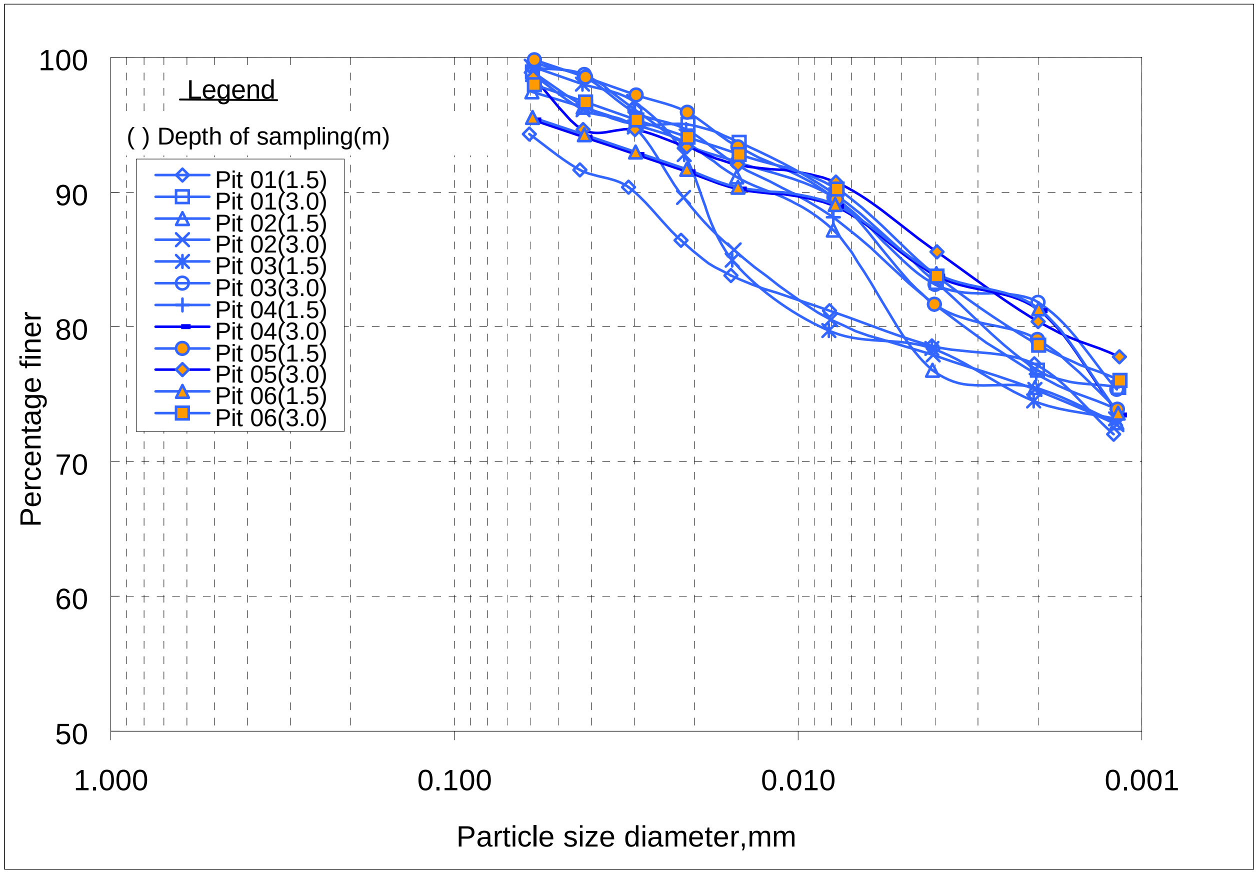
3.2 Determination of index properties

3.2.1 Particle-size distribution

A representative disturbed soil sample was taken from the field and sample was prepared as per BS 1377: part 1:1990. The common laboratory method to determine the size distribution of fine-grained soils is a hydrometer test. Hydrometer test was carried and the details are shown in BS: Part2: 1990.The experimental results were tabulated in Table 3.0 and the particle size distribution curves are shown in Fig.3.1.

Table 3.0. Particle size Analysis test result

Condition of the sample	Location	Test pit No.	Depth m	Sand %	Clay %	Silt %
Partially Sun dried	Abay mado (Kebele 11)	01	1.5	1.23	76.8	21.97
			3.0	1.13	75.0	23.87
		02	1.5	0.96	77.0	22.04
			3.0	0.7	80.4	18.9
		03	1.5	0.73	76.0	23.27
			3.0	0.83	81.8	17.37
		04	1.5	1.53	76.5	21.97
			3.0	0.93	81.2	17.87
		05	1.5	0.93	79.1	19.97
			3.0	1.1	81.3	17.6
		06	1.5	0.7	74.4	24.90
			3.0	0.87	78.6	20.53



coarse	medium	fine	coarse	medium	fine	Clay
Sand			Silt			

Fig.3.1 Particle size distribution curves

3.2.2 Attetrberg limits

In soils containing largely of fine grains, the amount of water present in the voids has a pronounced effect on such engineering properties as shear strength & compressibility. Consistency is used as the basis for their classification. The consistency of a soil is its physical state characteristics at given moisture content.

Atterberg proposed five status of soil consistency [7]. The liquid and plastic limits are used internationally for soil identification and classification and for strength correlations**.** The shrinkage limit is useful in certain geographical areas where soils undergo large volume changes when going through wet and dry cycles. The collision and sticky limits are not generally used in geotechnical engineering work [7].

Liquid limit: To determine the liquid limit of the soils 12 disturbed representative samples were taken from six open test pits by direct excavation and the apparatus used for this test is cone penetrometer. The details of the procedures are given in BS 1377:Part2: 1990. The results of liquid limit tests are shown in Table 3.1.

Plastic limit: Samples for plastic limit test were prepared by the same method followed in liquid limit test. The detail procedures are given in BS 1377: Part 2:1990. The results of plastic limit tests are shown in Table 3.1.

Shrinkage Limit: It is useful for the determination of the swelling and shrinkage capacity of the soil [18]. For the sample taken from the field shrinkage limit was determined from a compacted cylindrical specimen about 38mmin diameter with length between 1diamter and 2 diameters. A typical graphical plot is shown in Fig.3.2. The detail procedures are given in BS 1377: Part 2:1990. The results of Shrinkage limit tests are shown in Table 3.1.

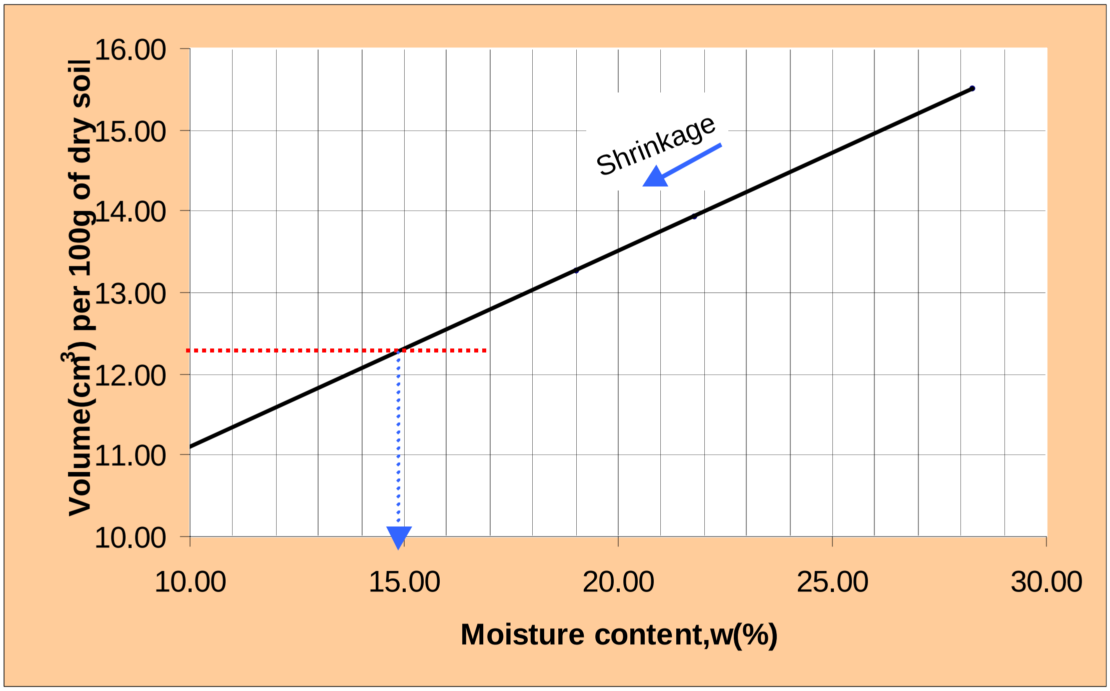


Fig.3.2 Typical Shrinkage curve

3.2.3. Linear Shrinkage

Linear Shrinkage can be used for the evaluation of the swelling potential of clayey soil. For soils with very small clay content the liquid and plastic limit may not produce reliable results to determine the plasticity index. An approximation of the plasticity index may be obtained in such cases by measuring the linear shrinkage and using the following expression [25]:

$$I_P = 2.13 * LS \quad (3.2)$$

The soil specimen was prepared as for the liquid limit test, where 150g of he specimen was taken for the linear shrinkage test. The detail procedures are given in BS: 1377: Part 2:1990. The results obtained for the Atterberg limits and linear shrinkage tests are shown in Table 3.1.

Table 3.1. Test Results of Atterberg limits and linear shrinkage test.

Location	Test pit No.	Depth m	Liquid limit, %	Plastic limit, %	Plasticity Index	Shrinkage Limit %	Linear shrinkage %	Plasticity Index (I_P) =2.13*LS
	01	1.5	62.2	35.50	26.70	15.2	10.67	22.73
		3.0	62.8	36.26	26.54	17.7	9.27	19.75

	02	1.5	64.65	36.96	27.69	16.0	10.35	22.05
		3.0	68.25	38.27	29.98	17.8	11.03	23.49
	03	1.5	60.5	35.22	25.28	14.8	10.10	21.51
Abay Mado		3.0	62.5	34.22	28.28	14.5	11.00	23.43
(Kebele 11)	04	1.5	63.6	33.9	29.7	15.0	12.02	25.60
		3.0	64.4	34.33	30.07	15.2	11.55	24.60
	05	1.5	64	36.15	27.85	14.2	10.03	21.36
		3.0	67.6	36.22	31.38	17.0	12.20	25.99
	06	1.5	60.8	37.22	23.58	17.0	8.67	18.47
		3.0	62.8	36.63	26.17	16.8	9.94	21.17

3.2.4.Free swell

Simple soil property tests, Atterberg limit tests, linear shrinkage tests, and free swell tests can be used for the evaluation of the swelling potential of expansive soils. However, the free swell test is very crude and was used in the early days when refined testing methods were not available [9].

The test is performed by placing 10 ml dry soil specimen, passing through 425µ sieve, into 100ml graduated cylinder and filed with water. The swelled volume was noted after the soil left to settle for about 24hrs.

$$\text{Free swell (\%)} = \frac{\text{Final volume-Initial volume of the soil}}{\text{Intial volume of soil}} \quad X100$$

Table 3.2. Free swell test results

Location	Test Pit No.	Depth	Free swell

		m	%
	01	1.5	9
		3.0	8
	02	1.50	13
		3.0	10
Abay mado	03	1.5	10
(Kebele 11)		3.0	10
	04	1.5	10
		3.0	11
	05	1.5	10
		3.0	9
	06	1.5	9
		3.0	12

As can be seen from the, table the free swell values are low and vary in a narrow range. Those soils having a free swell less than 50% are considered as non-expansive.

3.2.5. Determination of Specific gravity

The soil samples obtained from the field were prepared as specified in BS 1377: Part 1: 1990.The specific gravity test was conducted by the small pyknometer method as described in BS 1377: part 2: 1990. The results are shown on table 3.4.

Table 3.4 Specific gravity of soil

Location	Test pit No	Depth m	Specific gravity
Abay Mado (Kebele 11)	01	1.5	2.75
		3.0	2.78
	02	1.5	2.79
		3.0	2.79
	03	1.5	2.76
		3.0	2.79
	04	1.5	2.80
		3.0	2.83
	05	1.5	2.80
		3.0	2.80
	06	1.5	2.82
		3.0	2.83

3.2.6. In-situ density and Natural moisture content –

In-situ densities were determined using core cutter method since this method is suitable for fine soils free of stones [25]. Natural moisture content was determined by oven-drying method. The natural moisture content is used in calculating the liquidity index (I_L), which indicates the nearness of natural moisture content of the soil to the liquid limit [25].

$$I_L = \frac{W - W_p}{I_P} \quad (3.3)$$

$I_L < 0$: soil is in semi-plastic solid or solid state. $0 < I_L < 1$: soil is in plastic state.

$I_L > 1$: soil is in liquid state .The test results of In-situ density and natural moisture content is given Table 3.5.

Table 3.5.In-situ density, Natural moisture content and liquidity index.

Location	Test pit No	Depth, m	In-situ density, g/cm^3	Natural moisture content (%)	Liquidity index
Abay Mado (Kebele 11)	01	1.5	1.45	34.21	0.06
		3.0	1.71	27.33	-0.34
	02	1.5	1.64	29.52	-0.27
		3.0	1.70	29.40	-0.30
	03	1.5	1.57	31.94	-0.13
		3.0	1.68	32.83	-0.05
	04	1.5	1.52	36.22	0.08
		3.0	1.62	28.58	-0.19
	05	1.5	1.58	37.50	0.05
		3.0	1.69	29.33	-0.22

	06	1.5	1.65	35.87	-0.06
		3.0	1.77	34.45	-0.08

As can be seen from the table the soil can be said in semi plastic solid state, which was also observed visually in the field.

4.0. CONSOLIDATION TESTS

4.1. General

The ultimate change in volume of soil occurring under a change in applied stress depends on the compressibility of the soil skeleton of the soil particles. However, the water in the voids of a saturated soil is relatively incompressible and, if no drainage takes place, change in applied stress results in a corresponding change in pore pressure, and the volume change is negligible. As the drainage takes place, water flows from zone of high excess pore pressure to zone of less or zero excess pore pressure, and thus the excess pore pressure dissipate. The applied stress is then transferred to the soil skeleton and volume change takes place. It is this time dependent volume change of cohesive soils resulting from dissipation of excess pore pressure, which is known as consolidation.

The consolidation of a soil may be considered to consist of two stages: the primary consolidation stage during which the applied pressure increment is transferred from the pore water to the skeleton, and the secondary consolidation stage that follows the end of primary phase. A study of consolidation requires knowledge of the compressibility of the soil skeleton and the rate at which excess pores pressure dissipates which is related to the permeability [3].

The compressibility characteristics of a soil relating both the magnitude and the rate of settlement are usually determined from the consolidation test, using an apparatus called oedometer [7].

4.2. Test procedure and results.

Undisturbed samples in the form of blocks were obtained from the field. Detail procedures were followed as described in BS 1377: Part 5:1990.

4.2.1 Determination of the coefficients of consolidation

The important parameters of a soil are the compression index C_c and the coefficient of consolidation $C_{v.}$ The compression index relates to how much consolidation or settlement will take place .The coefficient of consolidation relates how long will it take for a given degree of consolidation to take place. There are two popular methods that can be used to estimate $c_{v.}$ Taylor (1942) proposed one method called the square root of time method. Casagrande and Fadum (1940) proposed the other method called the log time method [18]. The two methods specified generally show reasonable agreement [5]. The log time method makes use of the early (primary consolidation) and later time response (secondary compression) while the square root time method only utilizes the early time response, which is expected to be straight line. In theory, the square root time method should give good results except when nonlinearities arising from secondary compression cause substantial deviations from the expected straight lines [18]. These deviations are most pronounced in fine grained soils with organic materials. The square root of time method has been used in this work.

4.2.1.1 Square Root of Time Method.

As proposed by Taylor, the dial readings on the ordinate axis to a natural scale and then the corresponding values on the abscissa as the square root of time are plotted. A typical graphical plot is shown in Fig.4.1. The laboratory test results are given in Table 4.1.

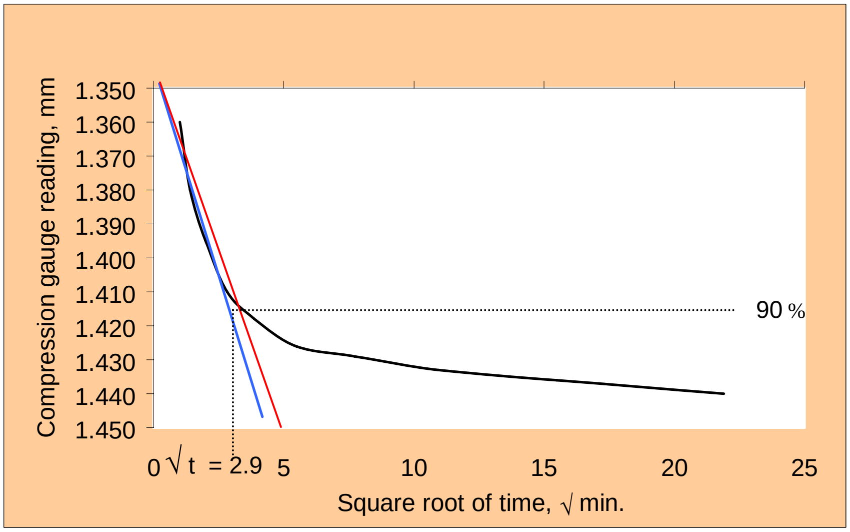


Fig.4.1 Typical curve (Square root of time method) of Pit no.04, Depth 1.5m, loading 800 Kpa.

4.2.1.2 Compression index (C_c)

The slope of the straight line portion of the e – log p plot gives the compression index (C_c). The test data were plotted as Void ratio against log Pressure and C_c was determined from Fig 4.3. The results are shown in Table 4.1.

4.2.1.3 Determination of the precompression stress

An estimate of the precompression pressure can be obtained from the oedometer test results.

Precompression (P_c) was determined from the curves of void ratio versus log pressure curves (Fig 4.3) using an empirical graphical method proposed by Casagrande [25]. The results are shown in Table 4.1. From the results obtained, the soils are found to be highly over-consolidated in their natural state.

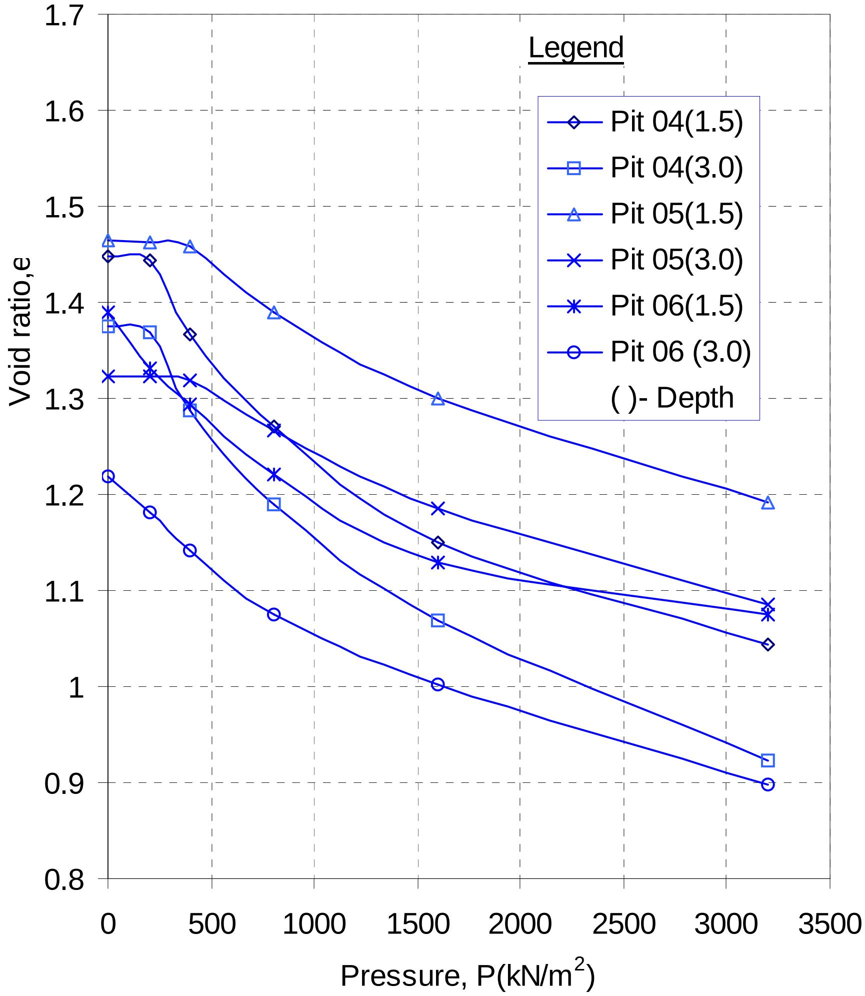


Fig 4.2 Void ratio Vs Pressure

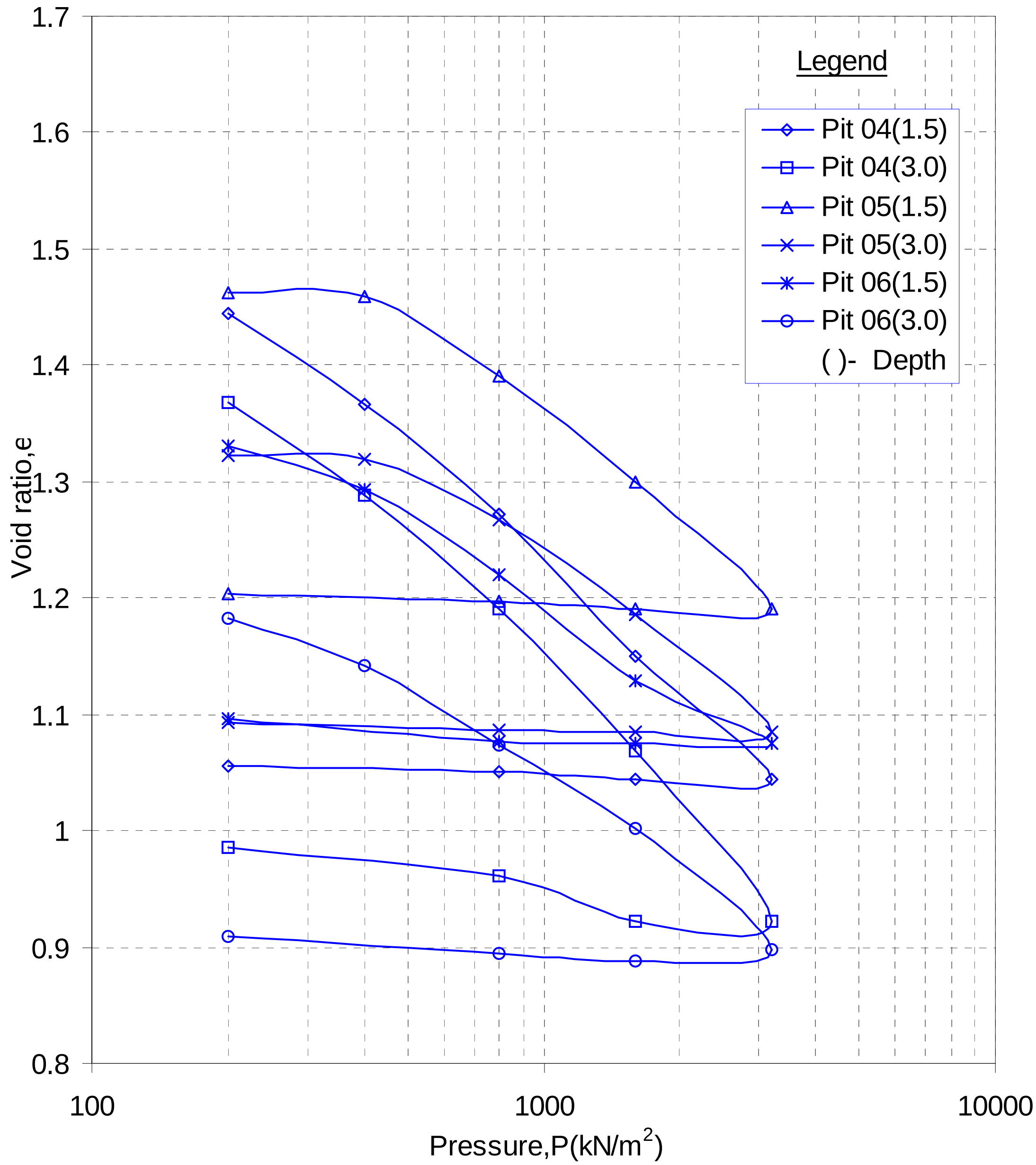


Fig 4.3 Void ratio Vs Log Pressure

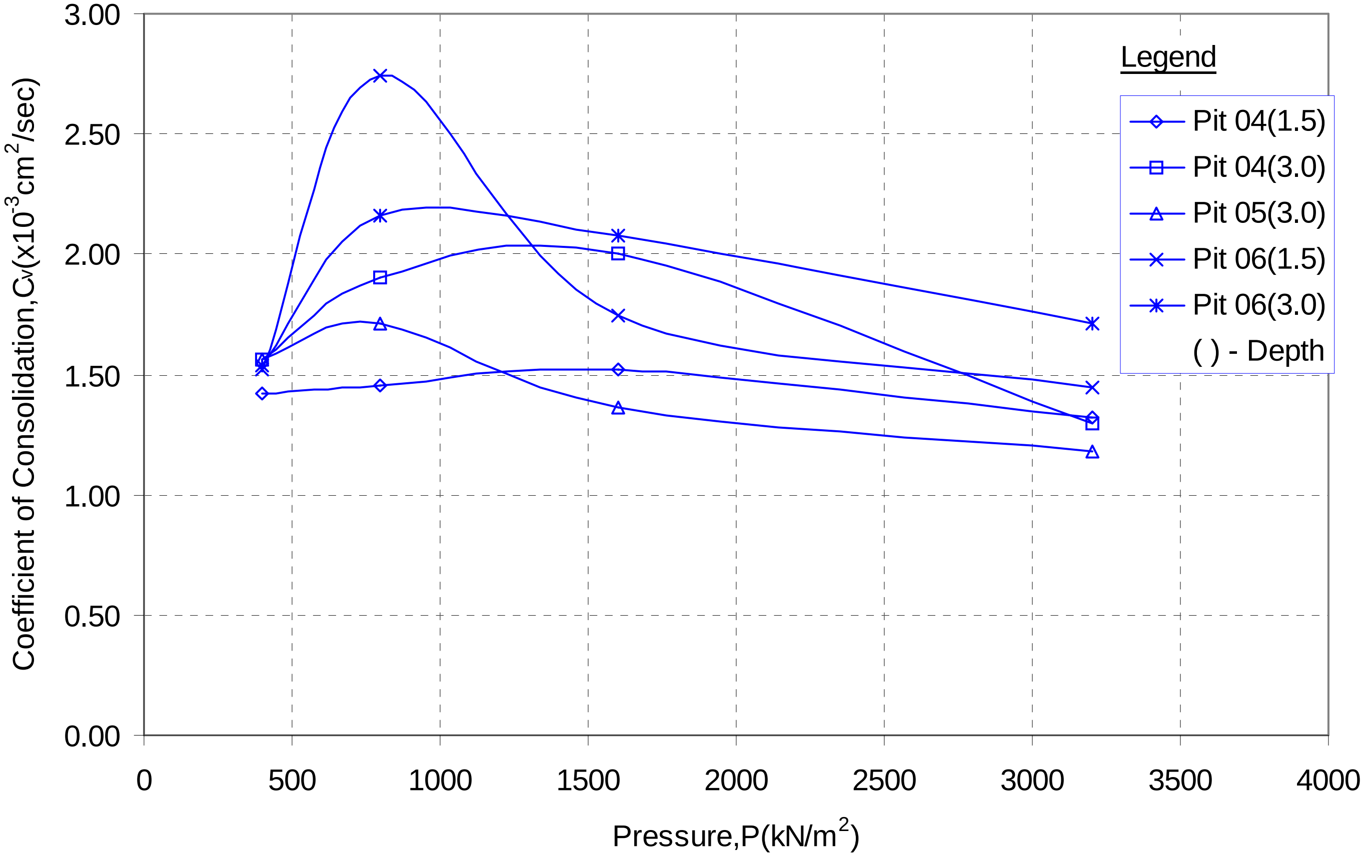


Fig 4.4 Coefficient of Consolidation Vs Pressure

Table 4.1 Consolidation test results

Location	Test pit No	Depth m	Natural moisture content w %	Total unit weight γ kN/m^3	Pressure P kN/m^2	Void ratio e_f	Coefficient of consolidation C_v 10^{-3} cm^2 / sec	Compression Index C_c	Over burden pressure P_o kN/m^2	Pre-consolidation pressure P_c kN/m^2
					0	1.4480				
					200	1.4445	1.42			
	04	1.5	34.07	14.2	400	1.3664	1.45	0.4056	21.3	700
					800	1.2715	1.52			
					1600	1.1494	1.32			
					3200	1.0436				
Abay Mado (Kebele 11)					0	1.3742	1.56			
					200	1.3681	1.90			
	04	3.0	33.12	14.8	400	1.2876	2.00	0.3887	44.3	750

800	1.1898	
1600	1.0690	1.97
		1.30
3200	0.9227	

Table 4.1 (cont'd)

Location	Test pit No.	Depth m	Natural moisture content w %	Total unit weight γ kN/m³	Pressure P kN/m²	Void ratio e_f	Coefficient of consolidation, C_v 10^{-3} cm²/sec	Compression Index C_c	Over burden pressure P_o kN/m²	Pre-consolidation pressure P_c kN/m²
					0	1.4640				
					200	1.4620	1.68			
	05	1.5	31.48	14.9	400	1.4590	2.44	0.3189	22.4	550
					800	1.3906	2.48			
					1600	1.2997	1.15			

1.56

1.71

1.36

					3200	1.1914				
Abay Mado (Kebele 11)	05	3.0	31.39	15.8	0	1.3227	1.97	0.2658	47.4	600
					200	1.3227				
					400	1.3184				
					800	1.2671				
					1600	1.1850				
					3200	1.0850				

Table 4.1 (cont'd)

Location	Test pit No.	Depth m	Natural moisture content w %	Total unit weight γ kN/m^3	Pressure P kN/m^2	Void ratio e_f	Coefficient of consolidation, C_v 10^{-3} cm^2/sec	Compression Index C_c	Over burden pressure P_o kN/m^2	Pre-consolidation pressure P_c kN/m^2

	06	1.5	33.36	15.7	0 200 400 800 1600 3200	1.3886 1.3306 1.2930 1.2199 1.1283 1.0754	1.52 2.74 1.74 1.45	0.3388	23.6	510
Abay Mado (Kebele 11)	06	3.0	32.19	16.9	0 200 400 800 1600 3200	1.2183 1.1819 1.1417 1.0741 1.0011 0.8971	1.54 2.16 2.08 1.71	0.2923	50.6	560

4.2.2 Determination of the Coefficient of Permeability

The coefficient of permeability can be measured using field tests, or tests conducted in the laboratory. Permeability is sometimes also estimated from one dimensional consolidation test. The coefficient of permeability can be obtained from the following relation ship [6].

$$K = \frac{c_v a_v \gamma_w}{1+e} \qquad (4.3)$$

Where c_v = coefficient of consolidation

a_v = coefficient of compressibility

γ_w = unit weight of water

e = void ratio

Using the above equation, the coefficient of permeability as the function of void ratio was calculated from the consolidation test results and shown in Table 4.2. It is noted that a_v, the ratio of change in void ratio to change in pressure, was obtained from Fig 4.2. As shown in Table 4.2, the range of values of coefficient of permeability lies between 10^{-9} and10^{-8} cm/sec, which indicates that the soils are practically impervious. In general, void ratio versus log coefficient of permeability is close to a straight line for nearly all soils [13]. As shown in Fig 4.5, all the soil samples taken from the field conform to this observation.

Table 4.2 Relationship between Void ratio and coefficient of permeability

Location	Pit No.	Depth, m	Void ratio	Coefficient of consolidation, 10^{-3} cm^2/sec	Coefficient of compressibility, 10^{-5} m^2/KN	Coefficient of permeability, 10^{-9} cm/sec
Abay Mado (Kebele 11)	04	1.5	1.060	1.32	6.00	3.85
			1.1150	1.44	7.00	4.76
			1.1494	1.52	10.00	7.07
			1.2715	1.45	19.00	12.20
	04	3.0	0.944	1.38	8.00	5.68
			1.026	1.84	10.00	9.08
			1.069	2.00	11.00	10.60
			1.1898	1.90	18.00	15.62
Abay Mado (Kebele 11)	05	1.5	1.208	1.28	6.00	3.48
			1.2680	2.15	7.00	6.64
			1.2997	2.48	8.00	8.63
			1.3906	2.44	14.00	14.26
	05	3.0	1.100	1.20	5.50	3.14
			1.157	1.30	6.50	3.92
			1.185	1.36	8.50	5.29
			1.267	1.71	11.50	8.67
	06	1.5	1.084	1.47	2.00	1.41
			1.110	1.59	3.00	2.26
			1.1283	1.47	6.50	5.50
			1.2199	2.74	13.50	16.70
	06	3.0	0.916	1.74	5.00	4.54
			0.972	1.98	6.00	6.02
			1.0011	2.08	7.00	7.28
			1.0741	2.16	11.00	11.50

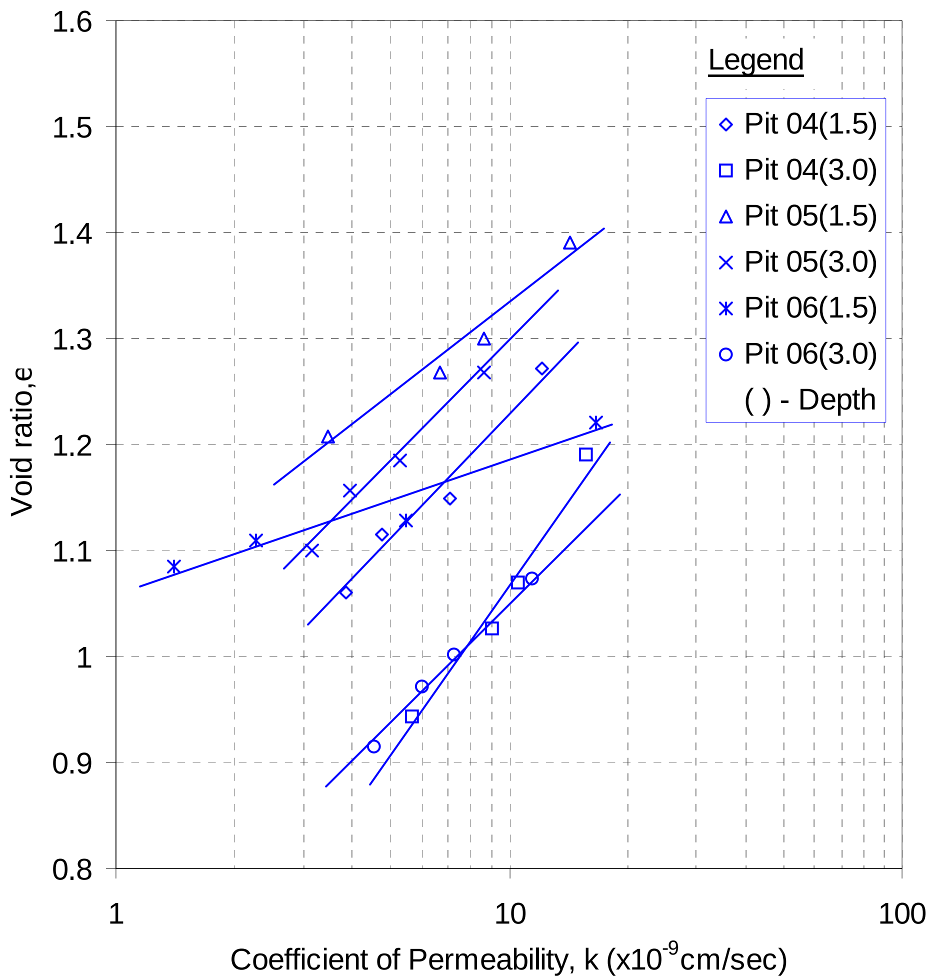


Fig 4.5 Void ratio Vs Log Coefficient of Permeability

4.2.3 Modulus of compressibility

For soils, whose behavior is typically non-linear the ' modulus ' analogous to modulus of elasticity, known also as modulus of compressibility (E_s), is not constant. The compressibility curve is obtained either from the plot of relative settlement versus effective stress or the void ratio against effective stress. The curve may be expressed by the following equation [22].

$$E_s = \frac{d\sigma'}{ds'} = v(\sigma')^w \text{ -------------- (4.4)}$$

$$s' = \frac{\Delta H}{H}$$

v and w are coefficients , v has a unit of kN/m^2

w is dimensionless

σ' is effective normal stress , kN/m^2

v depends on the void ratio, water content and consistency of the sample, it could have values ranging from 50 to 30000 kN/m^2 [22]. w depends on the soil type . It could assume values ranging from 0 to 1 [22]. To make the exponent dimensionless, it is advisable to make σ' also dimensionless dividing by a unit stress σ_e. Equation (4.4) becomes:

$$E_s = \frac{d\sigma'}{ds'} = v(\sigma'_e)^w \text{ -------------- (4.5)}$$

$$\sigma'_e = \frac{\sigma'}{\sigma_e}$$

The tangent of the compressibility curve, which is a function of σ', gives the modulus of compressibility E_s.

From eqation (4.4)

$$ds' = \frac{1}{v}(\sigma')^{-w} d\sigma'$$

$$s' = \int \frac{1}{v}(\sigma')^{-w} d\sigma'$$

$$s' = \frac{1}{v(1-w)}(\sigma')^{1-w} + C \qquad (4.6)$$

Where C is constant of integrations

For $\sigma' = 0$, $s' = 0$ then C= 0

Equation (4.6) becomes;

$$s' = \frac{1}{v(1-w)}(\sigma')^{1-w} \qquad (4.7)$$

$$s' = a(\sigma')^{k} \qquad (4.8)$$

Taking common logarithm, equation (4.8) becomes;

$$Log s' = k Log \sigma' + Log a \qquad (4.9)$$

From a plot of s' versus σ' one obtains a straight-line relationship for some cohesive soils. Other soils give a straight relation ship when results are plotted on double log scale [22].

Using equation (4.9), on the data obtained from one-dimensional consolidation test, relative settlement versus pressure (effective stress) was plotted on Log- Log scale as shown in Fig 4.6.

The values of the coefficients, v and w, were calculated using Fig 4.6 and tabulated on Table 4.4.

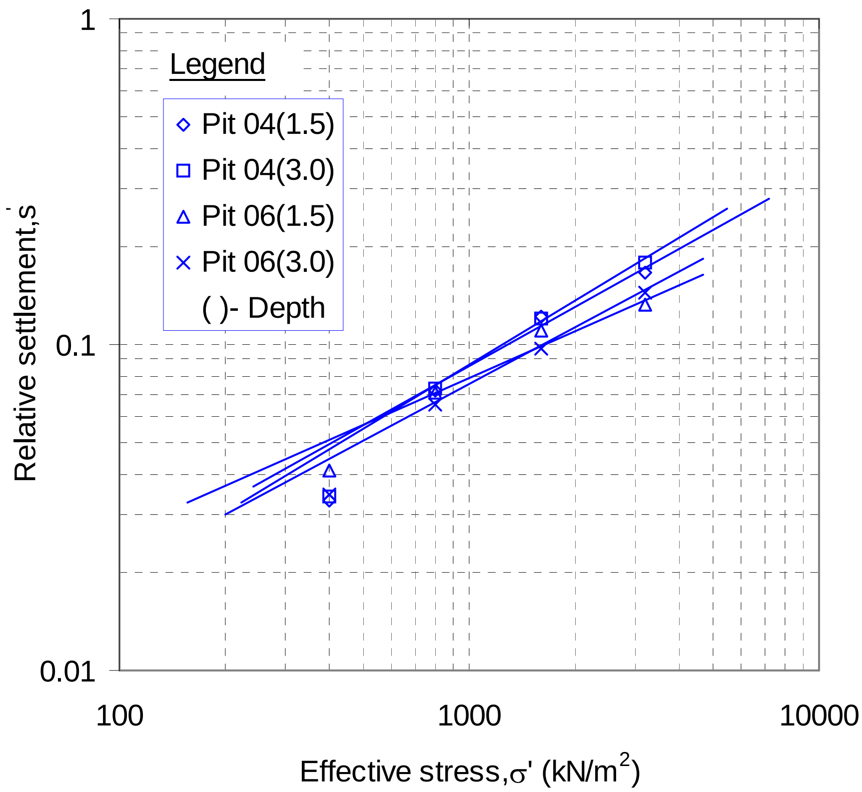


Fig 4.6 Relative settlement Vs Effective Stress (Double Log Scale)

Table 4.3 Total compression and relative settlements

Location	Test pit No.	Depth m	Effective stress, σ' kN/m^2	Total compression, ΔH, cm	Relative settlement $s' = \Delta H/H$
Abay mado (Kebele 11)	04	1.5	200	0.0029	0.00145
			400	0.0667	0.03335
			800	0.1442	0.0721
			1600	0.2440	0.1220
			3200	0.3304	0.1652
	04	3.0	200	0.0048	0.0024
			400	0.0686	0.0343
			800	0.1461	0.07305
			1600	0.2418	0.1209
			3200	0.3578	0.1789
	06	1.5	200	0.0486	0.0243
			400	0.0820	0.0410
			800	0.1432	0.0716
			1600	0.2199	0.10995
			3200	0.2642	0.1321
	06	3.0	200	0.0328	0.0164
			400	0.0690	0.0345
			800	0.1300	0.0650
			1600	0.1958	0.0979
			3200	0.2896	0.1448

Table 4.4 Coefficients, v and w, and equation of the modulus of compressibility.

Location	Test pit No.	Depth m	v kN/m^2	w	Modulus of compressibility, E_s kN/m^2
Abay mado (Kebele 11)	04	1.5	1263	0.4019	$1263\,(\sigma'_e)^{0.4019}$
		3.0	1591	0.3539	$1591\,(\sigma'_e)^{0.3539}$
	06	1.5	631	0.5510	$631\,(\sigma'_e)^{0.5510}$
		3.0	1277	0.4175	$1277\,(\sigma'_e)^{0.4175}$

Substituting the value of the coefficients v and w into Eq.4.5, equation of the modulus of compressibility can be written as shown in Table 4.4. From the above equations the modulus of compressibility are calculated and presented in Table 4.5.

Table 4.5 Modulus of compressibility

Location		Abay mado	(Kebele 11)	
Test pit No.	04		06	
Depth , m	1.5	3.0	1.5	3.0
σ'_e	E_s kN/m^2	E_s kN/m^2	E_s kN/m^2	E_s kN/m^2
0	0	0	0	0
100	8039.024	8118.395	7980.49	8733.55
200	10621.54	10375.4	11692.22	11664.63
300	12501.39	11976.32	14619.19	13816.21
400	14033.67	13259.87	17130.28	15579.41
500	15350.39	14349.46	19371.44	17100.58
600	16517.42	15305.87	21418.58	18453.09
700	17573.09	16164.06	23317.31	19679.74
800	18541.94	16946.25	25097.59	20808.03
900	19440.77	17667.56	26780.40	21856.83
1000	20281.65	18338.77	28381.11	22839.73

It can be observed that the modulus of compressibility increase with depth for a uniform soil formation.

5.0 SHEAR STRENGTH TESTS

5.1. General

The shear strength of soils is an important aspect in many foundation-engineering problems related to stability such as the baring capacity of shallow foundations and piles, the stability of slopes of dams and embankments, and lateral earth pressure on retaining walls.

The purpose of shear strength testing is to establish representative values for the shear strength parameters. The drainage conditions during the test influence the measured values considerably. The most common laboratory methods employed to obtain shear strength parameters are direct shear test, triaxial compression test and unconfined compression test. For this research work, triaxial compression and unconfined compression tests were conducted.

5.2 Triaxial compression test

A widely used and reliable apparatus to determine the shear strength parameters and the stress strain behavior of soil is the triaxial apparatus. The triaxial apparatus is versatile because we can independently control the applied axial and lateral stresses, conduct tests under undrained and drained conditions and it is possible to both control and measure the pore pressure. In the triaxial test, a cylinderical sample of soil usually with a length to diameter ratio of two [2] is subjected to either controlled increases in axial stresses or axial displacements and radial stresses. A membrane laterally confines the sample, and radial stresses are applied by pressuring water in the chamber. The axial stresses are applied by loading a plunger.

The triaxial tests are classified according to the condition of drainage during the test as UU, CU, and CD tests [14].

Unconsolidated Un-drained (UU) tests are carried out by placing a specimen in the chamber and applying lateral pressure without allowing the specimen to consolidate or drain under the confining pressure. Axial load is then applied fairly rapidly without permitting drainage of the specimen.

Consolidated undrained (CU) tests are performed by placing a specimen in the chamber and applying lateral pressure. The specimen is then allowed to consolidate under the all round confining pressure by leaving the drain lines open. The drain lines are then closed, and axial stress is induced without allowing further drainage.

Consolidated drained (CD) tests are similar to CU tests except that the specimen is allowed to drain as axial load is applied so that high excess pore pressure do not develop. CD tests may take a considerable period of time to run because of the time required for both consolidation under the confining pressure and drainage during application of axial load.

For practical tests both CU and CD yield the same strength provided the tests are performed correctly [18]. Most triaxial testing is CU because the time for testing is less than for CD test. Therefore CU tests were carried out on undisturbed samples obtained, by tube sampling, from the field. The type of triaxial equipment used for this research was strain controlled.

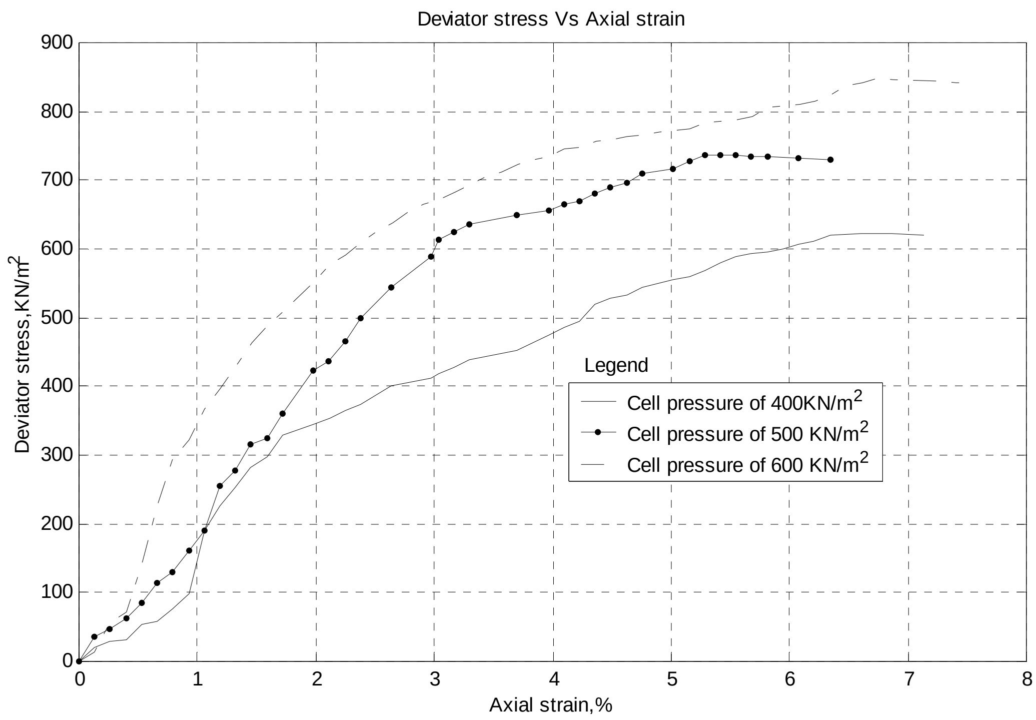
5.2.1 Consolidated Undrained (CU) tests.

The CU tests were carried out on a set of three similar specimens. The specimens were saturated by Applying increments of cell pressure and backpressure alternately. After saturation the cell pressure was raised and isotropically consolidated, keeping the drainage valve open for 24 hrs. Finally the compression stage was followed. The cell pressure was maintained constant while the specimen was sheared under undrained condition at a constant rate of axial deformation until failure occurs. The pore water pressure was measured at each stages of axial load increment. The test procedures are described in BS 1377: Part 8: 1990.

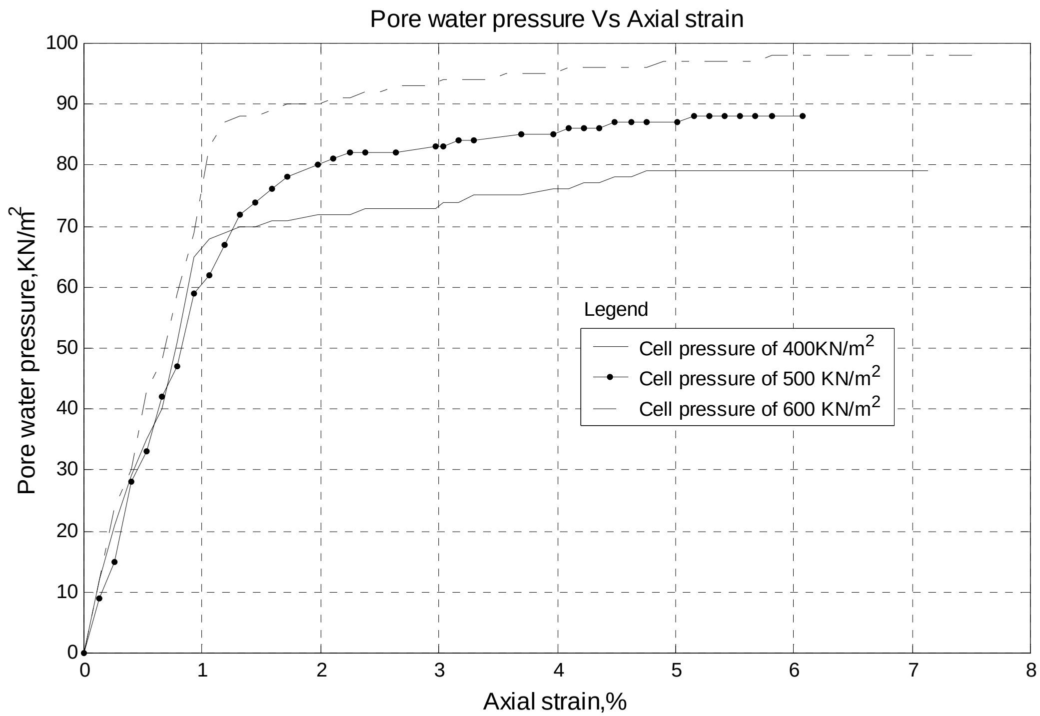
The results of CU tests are shown in Table 5.1. Plots of deviator stress (axial stress)-axial strain, pore water pressure versus axial strain and two sets of Mohr's semi-circles at failure, one representing total stress condition and the other effective stress condition, are shown in Figs.5.2a, b, c, 5.3a,b, c and 5.4a,b, c. The interpretation of the CU tests shows the angle of internal friction and cohesion varies from 18 to19^0, 68 to 90 KN/m^2 and 23 to 24^0, 60 to 84 kN/m^2 for total stress and effective stress conditions respectively.

Table 5.1 Results from Consolidated un-drained tests.

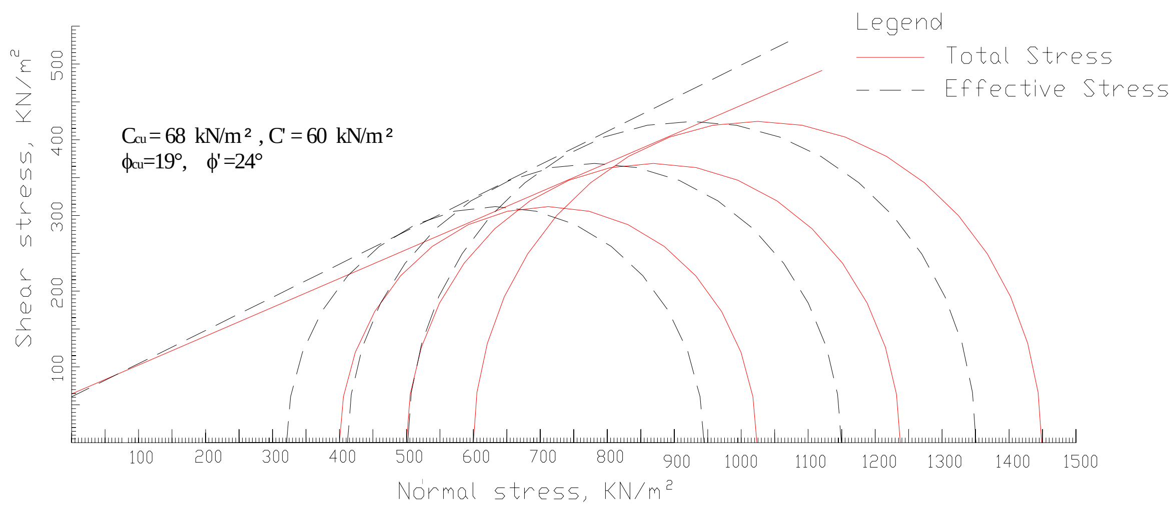
Location	Pit No.	Depth	Cell pressure, σ_3	Deviator stress at failure, $(\sigma_1 - \sigma_3)$	Pore water pressure at failure, U_W	Total stress		Effective stress	
						ϕ_{cu}	C_{cu}	$\phi^{'}$	$C^{'}$
		m	kN/m^2	kN/m^2	KN/m^2	deg.	kN/m^2	deg.	kN/m^2
Abay Mado (Kebele 11)	04	1.5	400	623.43	79	19	68	24	60
			500	737.50	88				
			600	848.58	98				
	05	1.5	400	666.88	84	18	90	23	84
			500	773.51	91				
			600	880.19	105				
	06	3.0	400	627.17	67	18.6	74	24	65
			500	742.33	79				
			600	854.23	85				



(a) Deviator stress Vs Axial strain

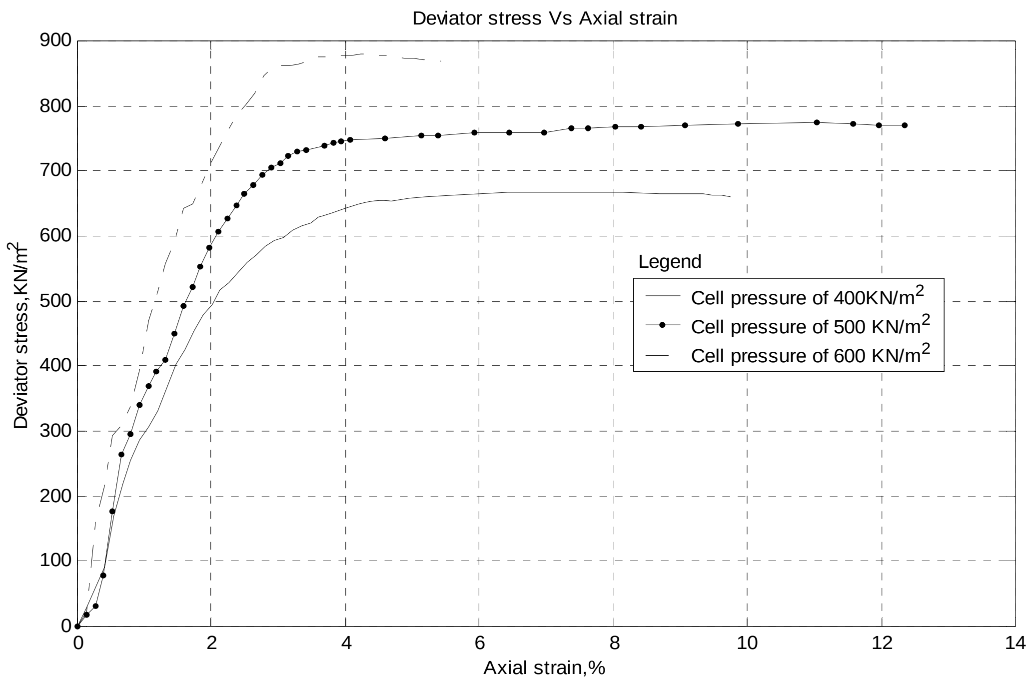


(b) Pore water pressure Vs Axial strain

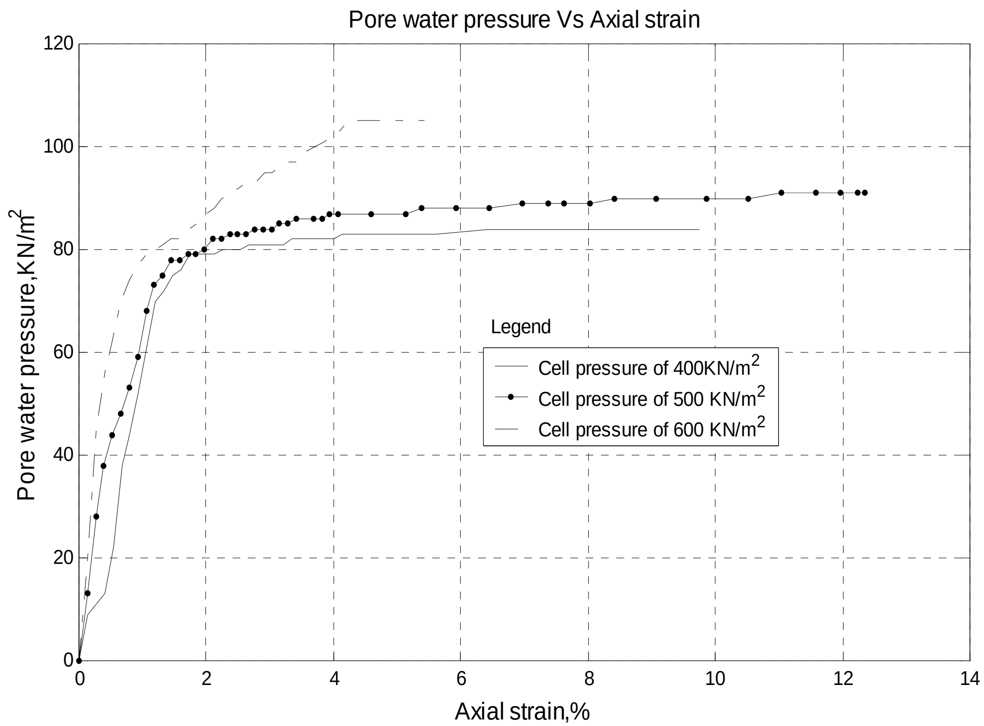


(c) Mohr's semi-circles at failure

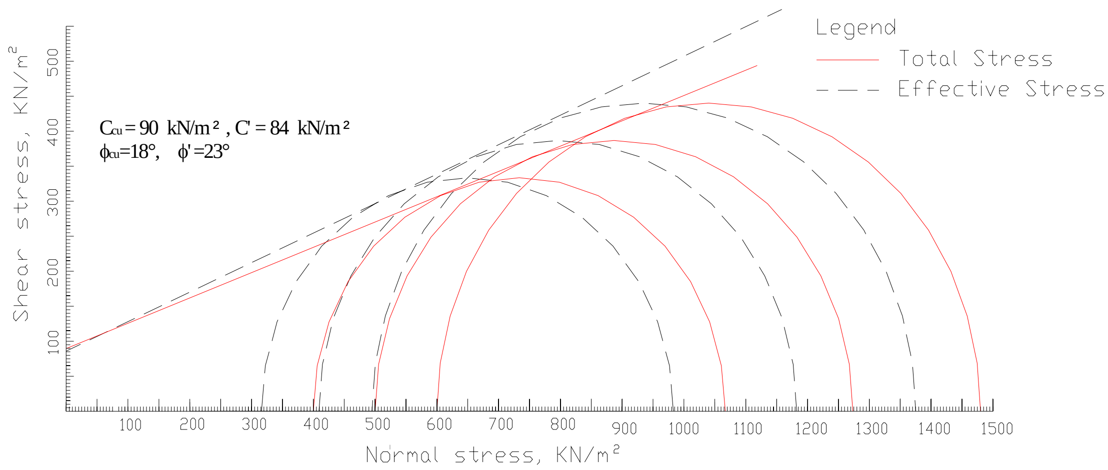
Fig 5.2 Stress- strain and Mohr's semi-circles for CU test at pit no.04.



(a) Deviator stress Vs Axial strain



(b) Pore water pressure Vs Axial strain



(c) Mohr’s semi-circles at failure

Fig 5.3 Stress- strain and Mohr’s semi-circles for CU test at pit no.05.

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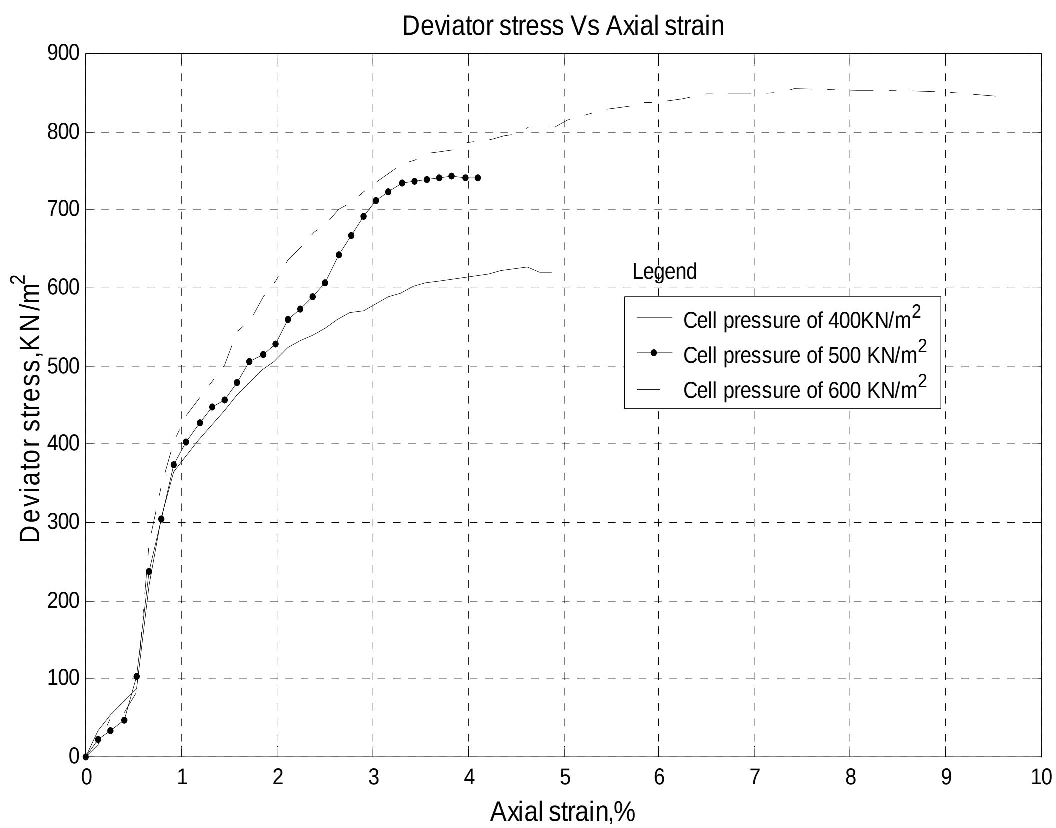
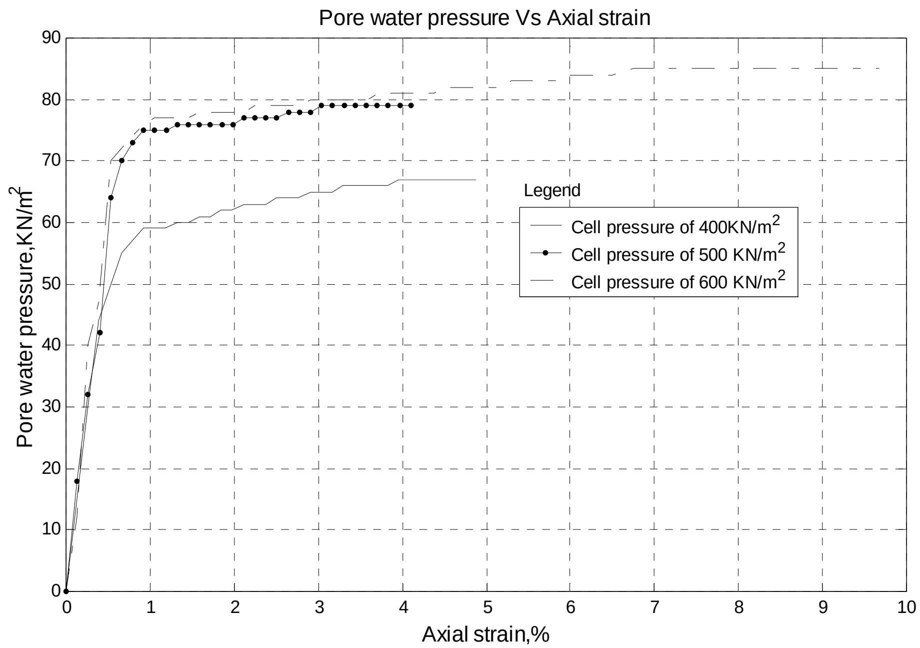
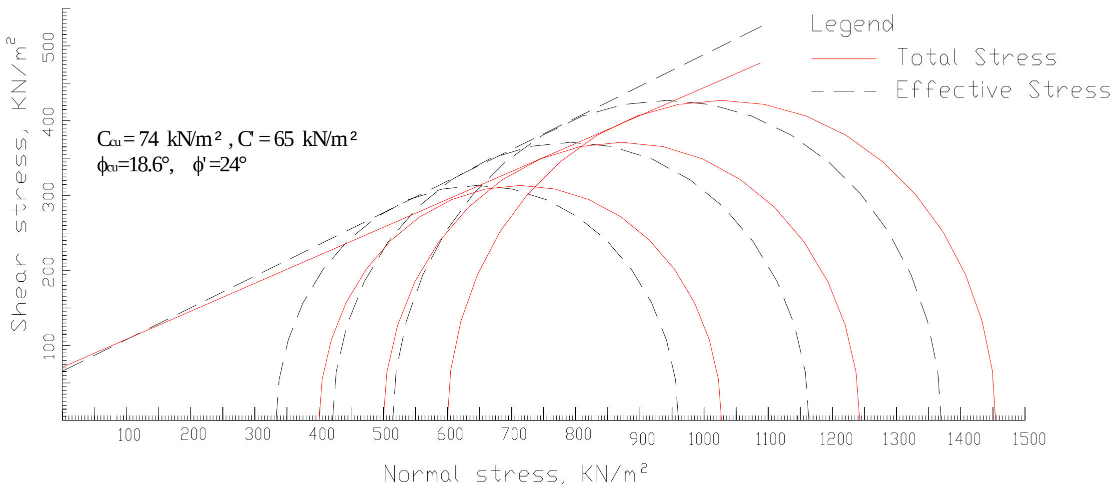


(a) Deviator stress Vs Axial strain



(b) Pore water pressure Vs Axial strain



(c) Mohr's semi-circles at failure

Fig 5.4 Stress- strain and Mohr's semi-circles for CU test at pit no.06.

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5.3 Unconfined compression (UC) test

The unconfined compression test is a special case of the unconsolidated undrained triaxial test. In this case, no confining pressure to the specimen is applied. The UC test is one of the easiest and simplest tests for determining a quick estimate of the shear strength of cohesive soils. The test provides an immediate approximate value of the compressive strength of the soil, either in the undisturbed or the remolded condition. It is also widely used to determine the consistency of saturated clays and other cohesive soils [23]. The UC tests were carried out on 6 undisturbed samples obtained, by tube sampling, from the field. Details of testing procedures are given in BS 1377: Part 7: 1990. The results of UC tests are shown in Table 5.3.

Table 5.3 Results of Unconfined compression test

Location	Test pit	Depth,	Dry density,	Natural moisture	Unconfined

	No	m	g/cm^3	content, %	compressive strength kN/m^2
	04	1.5	1.251	34.14	147.47
		3.0	1.265	33.84	162.20
Abay mado	05	1.5	1.292	34.03	183.69
(Kebele 11)		3.0	1.267	29.88	199.35
	06	1.5	1.311	32.98	198.92
		3.0	1.299	29.95	219.95

6.0 DISCUSSION

6.1 Analysis of the test results

6.1.1 Mineralogical composition

The mineralogy identification result shows the clay mineral that occurs in the red clay soils investigated is predominantly kaolinite, which is expected for the red clay soil as per the study carried out by Morin.

6.1.2 Index properties

The specific gravity lies in the range between 2.75 to 2.83, which is similar to the results obtained by Morin [17] and a previous study carried out in Addis Ababa [21].

To classify these fine-grained soils samples taken from the field, plasticity chart was potted as shown in fig 6.1. According to Unified Soil Classification System these soils are classified as inorganic silts of high plasticity (MH).

Wesley proposed a practical system for classifying all residual soils, based on mineralogical composition and soil micro-and macrostructure; the following three groups are suggested [4]:

Soils without a strong mineralogical influence (Group A), soils without a strong mineralogical influence deriving from clay minerals also commonly found in transported soils (Group B), and soils with a strong mineralogical influence deriving from clay minerals only found in residual soils (Group C). Because of the peculiar behavior of tropical weathered clay mineralogy, the position of the plasticity index versus liquid limit plot falls below the "A- line" and mineralogy identification result shows the clay mineral that occurs in the red clay soils is predominantly kaolinite, therefore, According to Wesley the soil can be classified as Group C.

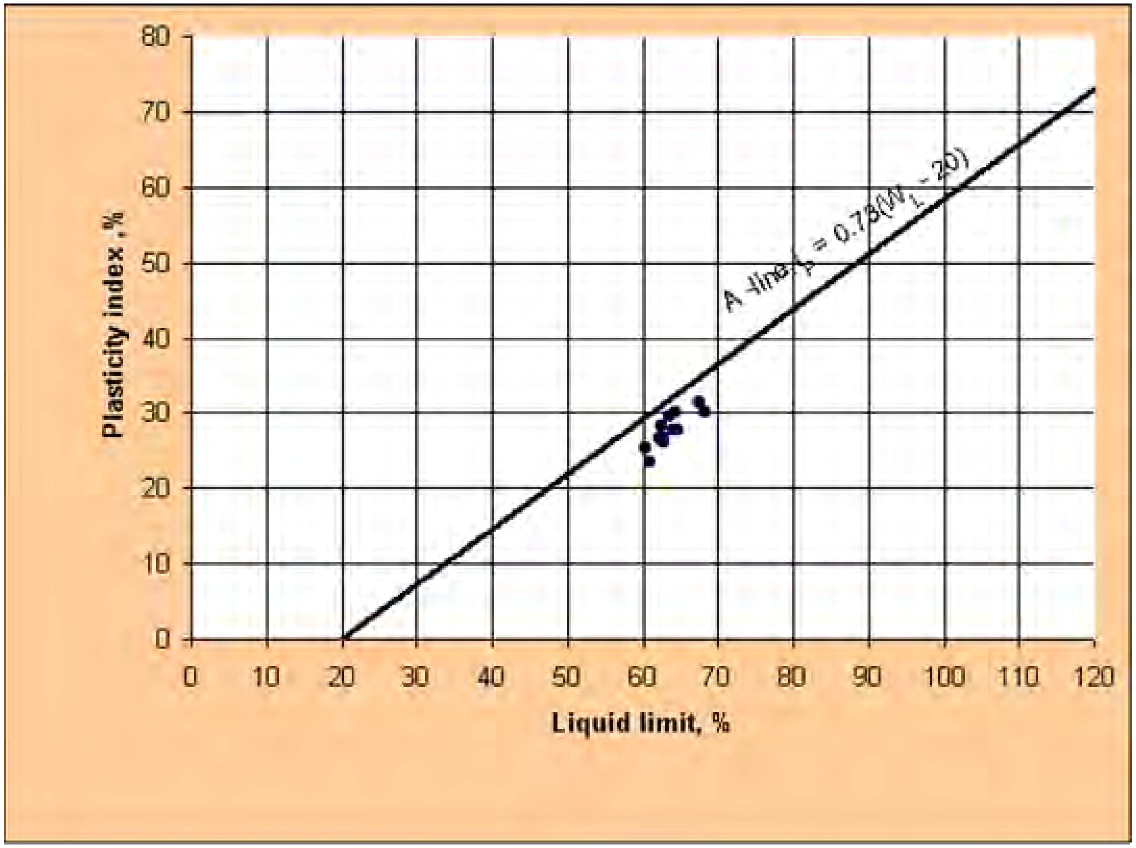

Fig. 6.1 Plasticity Chart (25)

6.1.3 Consolidation test results

The values of c_v lie in the range from $1.30x10^{-3}$ cm^2/sec to $2.74x10^{-3}$ cm^2/sec and the values of the compression indices range from 0.2658 to 0.4056 which are in the normal range of common soils.

From one dimensional consolidation test result, the values of the modulus compressibility are obtained for the given effective stress. However, these values are a bit higher than the normal values. There was an attention to consider an average value of E_s but due to the variation in the coefficients v and w with each other it is better to consider as they are.

6.1.4 Shear strength test results

From the result of CU tests, the values of internal friction and cohesion vary from 18 to19^0, 68to90 kN/m^2 and 23 to 24^0, 65 to 84 kN/m^2 for total stress and effective stress conditions respectively. The friction angles and cohesion are typical for the soil investigated.

The values of unconfined compressive strength lie from 147 to 220 kN/m^2, at corresponding moisture content of 30 to 34 %, which are close to the results obtained by Morin [17]

6.2 Aspects of the test results

The laboratory test results of this investigation can be compared with the other research data as shown in the following table.

Table 6.2 Comparison of Red clay soil data

	Morin and Parry [17]	Previous research [21]	Present research
Soil type	Red clay	Red clay	Red clay
Location	Ethiopia	Addis Ababa	Bahir Dar town.
Clay content %	34 - 76	58 - 70	74 - 82
Activity	-	<0.75	0.56
Clay minerals	Kaolinite, Halloysite, Montmorillonite	-	Kaolinite

W_L %	44- 66	56- 75	61- 68
W_s %	10- 30	14- 22	9- 12
I_P %	14- 30	29- 47	24- 31
G_s	2.61- 2.90	2.66- 2.77	2.75- 2.83
q_u KN/m^2	146.5- 251	-	148- 220

As shown in table 6.2, the ranges of values are close to the results obtained by Morin [17] and.

previous research [21]

6.3 Correlation of test results

6.3.1 Relationship between Atterberg limits and clay content

Atterberg limits for a soil are related to the amount of water that is attracted to the surface of the soil particles. Because of the great increase in surface area per mass with decreasing particle size, it may be expected that the amount of attracted water will be largely influenced by the amount of clay that is present in the soil. The plasticity of clays arises almost exclusively from the interaction between the clay fraction and the water. Provided the mineral composition of the clay size fraction is constant, the plasticity index is directly proportional to the clay content of the soil [12]. On the other hand, plasticity is greatly influenced by the mineral composition of the clay fraction. On the basis of this reasoning, Skempton (1953) defined a quantity called activity [13]. It is the ratio of plasticity index to the percent of clay -size particles. Soil is said to be inactive when activity is smaller than 0.75; normal when it is between 0.75 and 1.25, and active when it is greater than 1.25 [12]. It should be noted that the activity of a given soil would be a function of the type of clay mineral present [10]. Generally, the relative level of activity expected is low for kaolinite, medium for illite, high for montmorillonite [8]. For the samples taken the plasticity index versus clay content was plotted as shown in Fig.6.3 and the plot shows the soils exhibit an activity less than 0.75 indicating that they are inactive.

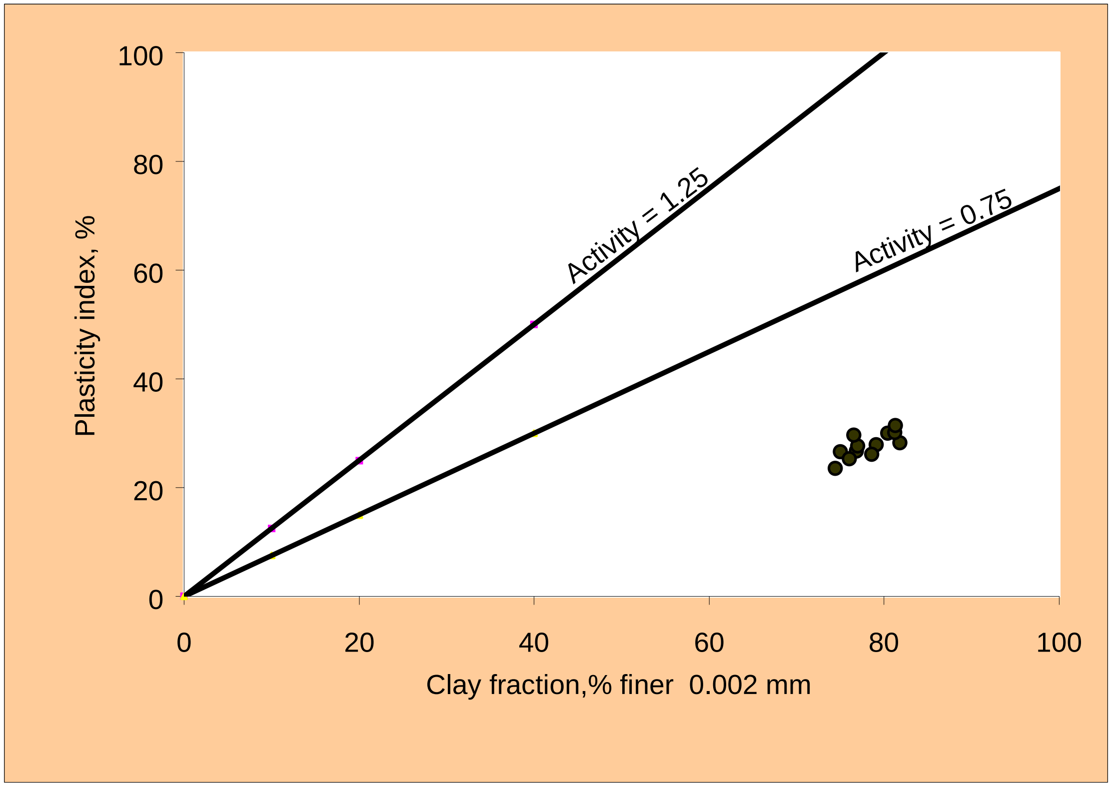

Fig.6.3 Activity Chart (12)

6.3.2 Relationship between plasticity index and shrinkage limit.

The relationship between plasticity index and shrinkage limit is shown in Fig.6.4 and the relation ship obtained is as follows:

$$I_P = 2.0661w_s + 5.9448 \qquad (6.4)$$

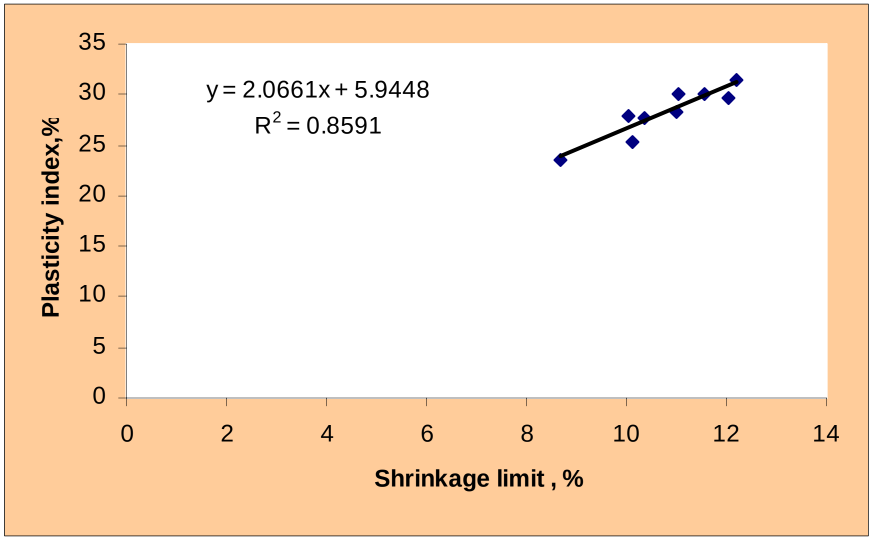

Fig.6.4 Relation ship between Plasticity index and Shrinkage limit

Table 6.3 Some of the laboratory and correlated result of I_P

w_s	I_P	$I_P = 2.0661w_s + 5.9448$
10.67	26.7	27.99
9.27	26.54	25.10
9.94	26.17	26.48

Some of the values of the lab results, w_s and I_P, which were not inserted into the correlation, were taken and the I_P values obtained from the lab and calculated from the correlation equation were compared as shown in Table 6.3.

7.0 CONCLUSIONS AND RECOMMENDATONS

The following conclusions can be drawn from the results of investigation on the red clay soils in Bahir Dar.

1. The red clay soils in Bahir Dar contain the clay mineral kaolinite predominantly

2. The values of, specific gravity range from 2.75 to 2.83, I_P range from 23.58 to 30.07 and clay fraction range from 74.4 % to 81.80 %

3. The index properties show the clay soils are not expansive.

4. The coefficient of permeability estimated from one-dimensional consolidation test lies between 10^{-9} and10^{-8} cm/sec and shows that the soil is relatively impervious.

5. The values of unconfined compressive strength range from 147kN to 220 kN/m^2 at natural moisture content of 30 to 34 %.

6. The values of ϕ' and C' are in the range from 23 to 24^0 and 60 to 84 kN/m^2 respectively.

7. Correlations that relate index proprieties with shear strength parameters were not done due to insufficient data. Further studies can shade light on this aspect of the problem.

8. The values of the precompression pressures are relatively higher than corresponding values of commonly encountered soils therefore further studies are required.

9. It is recommended that the above values should be refined with larger size of data.

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APPENDIX

Appendix A

Table 3.1 Ranges in properties of Ethiopian soils according to Morin and Parry (17)

	Black clays	Red clays	Other clays	Lacustrine soils
Parent Rock	Olivine basalt, basalt, trachyte	Olivine basalt, basalt, trachyte	Olivine basalt, basalt trachyte, ignimbrite, etc.	
Rainfall (in/year)	9 - 53	48 - 92	0.4 - 48	8 - 41
Temperature (^{0}F)	54 - 68	57 - 68	62 - 82	66 - 80
Drainage	Poor to good	Fair to good	Poor to good	Poor
Principal clay minerals	Montmorillonite, Kaolinite, Halloysite	Kaolinite, Halloysite Montmorillonite	Montmorillonite, Kaolinite	Montmorillonite
PH Value	7.2 - 8.4	5.1 - 6.8	6.3 - 0.1	8.0 - 9.1
Principal Cations	Ca, Mg, K	Ca, Mg, K	Ca	Ca
Cations exchange Capacity (me/100g)	42 - 95	30 - 77	22 - 90	61 (one test)
Clay (2μ), %	13 - 75	34 - 76	5 - 51	3 - 41
Liquid limit (%)	37 - 88	44 - 66	NP - 57	NP - 70
Plasticity index (%)	11 - 48	14 - 30	NP - 35	NP - 35
Shrinkage limit (%)	7 - 28	10 - 30	11 - 19	15 - 20
Specific gravity	2.62 - 2.94	2.61 - 2.90	2.61 - 2.90	2.61 - 2.92

Organic content (%)	2 - 7+	1 - 4+	2 - 7+	4+
Compaction test				
Max density (1b/ft^3)	69 - 101	74 - 106	73 - 104	76 - 90
Optimum Moisture (%)	40 - 17	38 - 29	28 - 21	30 - 27
CBR test values	2 - 8	6 - 9	-	-
Unconfined compressive				
Strength (1b/ft^2)	2020 - 5580	3060 - 5240	1290 - 3320	1350 - 4020
Expansion pressure (1b/ft^2)	2040 - 2000	440 - 2000	120 - 3000	400 - 2400

Declaration

I, the undersigned, declare that this thesis is my original work and that all sources of materials used for the thesis have been dully acknowledged.

Fasil Abagena