Global Scientific JOURNALS

GSJ: Volume 10, Issue 4, April 2022, Online: ISSN 2320-9186

www.globalscientificjournal.com

Investigating settlement behavior of tropical Laterite soils in case of Assosa Town.

Alemu Weyessa Shegro[1], Asmachew Abera Bemanjo[2], Tesfaye Negasa Jaleta[3]

[1]*Water Resources and Irrigation Engineering, Assosa University, Ethiopia*
[2]*Water Resources and Irrigation Engineering, Assosa University Ethiopia*
[3]*Water Resources and Irrigation Engineering, Assosa University Ethiopia*

[1]alemuweyessa@gmail.com
[2]asabera22@gmail.com
[3]yoom2020@gmail.com

ABSTRACT

In general, the settlement due to applied load is achieved through a number of ways, including rearrangement of the soil solid or extrusion of the pore air and/or water. According to this, a decrease of water content of a saturated soil without replacement of the water by air is called a process of consolidation/settlement. However calculating and determining of the settlement behavior of laterite soils is very challenge full due to their sensitivity behavior to sampling techniques. This study was aimed at; investigating consolidation and settlement behavior tropical residual laterite soils of Assosa, investigating how fast is the rate of settlement of the soil of the study area and comparing and discuss the results with other similar studies. The behavior of the soil for logarithmic plot shows that, it is stiff at the beginning of loading and continuously increase with load after the load exceeds the pre-consolidation pressure. But for the linear plots the compression increase with load continuously started from the beginning. The OCR for all tests pits were 9.9, 5.2, 6.6, 6.2, and 3.15 for TP-1, Tp-2, TP-3, Tp-4 and TP-5 respectively. Hence, the soils under investigation are over consolidated as their over consolidation ratios were greater than"1"and the settlement behavior of these soils doesn't show significant variation with in 3m depth. This study recommends that, a further investigation is recommended to investigate the consolidation behavior these soils along the soil profile.

Key words; ***Consolidation, Laterite Soils, Settlement, Tropical Soils.***

1. Introduction

When a soil mass is subjected to a compressive force, like civil engineering structures, such as; building, dam, road embankments…etc. its volume decrease. The property of the soil due to which a decrease in volume occurs under compressive forces is known as the compressibility of soil. The compression of soil can occur due to one or more the following causes; Compression of solid particles and water in the voids, Compression and expulsion of air in the voids and Expulsion of water in the voids. Compression of solid particles is negligibly small and compression of water in the void is also extremely small as water is almost incompressible in the range stress involved in soil engineering therefore compression due to the first cause is not much significant. Air exists only in partially saturated soil and dry soils. The compression of the air is rapid as it is highly compressible. Further, air is expelled quickly as soon as the load is applied. When the soil is fully saturated, compression of soil occurs mainly due to compression and expulsion of water. Therefore, the compression of soil under steady static pressure is known as consolidation. Settlement of a structure is its vertical, downward movement due to volume decrease of the soil on which it is built. In other word, the settlement is the gradual sinking of the structure due to compression of the soil below. A study of consolidation characteristics is extremely useful for forecasting the magnitude and time of the settlement of the structure [12].

2. STUDY AREA AND RESEARCH METHODOLOGY

2.1. Description of the Study Area

Assosa Town is the capital city of Beneshangul Gumuz regional government which located at 675Km from Addis Ababa in south west direction and at 96Km from the Ethio-sudan border. The town has a flat terrain with an elevation of about 1650m above mean sea level. The climatic classification of the Town is warm ("Kola") with a temperature greater than $20^{o}{}_{C}$. It has a moisture index ranging from 50 to 100 (intermediate or moist), i.e.; potential evaporation is mostly greater than precipitation. It has a mean annual rain fall of 1200mm with maximum rain fall from the month of June to September. Mean period of onset of the "Kiremt rain" is 26-30May and mean period of cessation of the "Kiremt rain" is between 23 and 27 October [2].

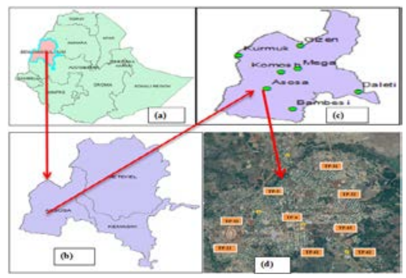

Figure 2 1 The geographical location of Assosa Town

Table 2 1 The mean monthly rainfall and mean monthly maximum and minimum temperature [2].

Month		Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec
Rf		1	25	10	50	100	200	230	200	200	150	25	1
Tem $^{o}{}_{C}$	Max	33	33	33	30	30	30	25	25	25	30	30	30
	Min.	15	15	17	17	20	15	15	15	15	15	15	15

2.2. Geology and Soil Characteristics of Assosa Town

According to geological map of Ethiopia, 1996 the Geological formation of Assosa town and the surrounding area: flood basalt. The flood basalt is a good crushed aggregate material for concrete work, and asphalt work when it exists in sound form. Granite is also observed during the field investigation around Assosa town existing in sound form. All of the urban roads are almost earth roads of red to brown sandy Silty clay soil with a considerable amount of dark to gray soil. The natural ground water table is located at 7m minimum and 35m maximum. All laterite soils requires a minimum rainfall of 750mm for their formation with hot periods and soil geology of mostly basaltic rock as a parent material. It is known that the rate of chemical weathering is controlled by moisture and temperature [2].

2.3. Sampling Techniques and Sample Locations

Firstly, by visual inspection and by field observation, the representative parts of the town were selected to collect samples. Then soil samples were collected by conducting boring/ excavation mechanism by labor force. The test pits were dug by labor force. Some sample pits were very hard to dig and it was very difficult to collect block samples for consolidation test.

Figure 2 2 Photos showing sample collection process**.**

Table 2 2 Description of sample location and sample depth

Designation	Sample location	Sample depth
Tp-1	Kebele (04)	@1m
Tp-2	Medhanialem sefer	@1.5m
Tp-3	Near police commission	@2m
Tp-4	Dipo(Near Red-cross office)	@2.5m
Tp-5	Areb sefer (Near stadium)	@ 3m

2.4. Data Collection Process

The soil specimens for this study were collected in Assosa Town at 6-different sub cities of the town based on the size of the Town and the variation of the soil type found in the town as observed during site visit. Then, by pushing the PVC into the ground to capture the soil and finally the soil sample was covered by plastic bag in order reduce loss of natural water content.

2.5. The Materials Used

The sampling technique for undisturbed sample was done by PVC pipe and some plastic covers were used to control the loss natural moisture content before the test conducted. During the laboratory data analysis time Excel sheets were used to plot the graphs in between different parameters and GIS software was used when the area was delineated.

2.6. Testing Procedures

2.6.1. Test procedures for Consolidation test.

For laterite soils, there is no specialized test procedure/or method to determine their consolidation and settlement behavior. Therefore, the usual test method which is consolidometer (Odometer) device is used to carry out consolidation test to determine the magnitude and rate of settlement in this thesis. All possible precautions were taken to minimize disturbance of the soil or change in moisture and density during specimen preparation by avoiding vibration, distortion and compression and also, great care is taken during sample taking in the test pit, sample transportation and during testing process.

The test is done according to (ASTM-D2435) standard manual and all the test procedure were from the manual. Trim the specimen and insert into a circular consolidation ring of having a diameter of about 50mm and a thickness of 20mm. The porous stone allow water to drain from top and bottom of the sample when vertical pressure is applied. The sample was placed in between the porous stones in a holding cell and setup in a loading frame as shown in figure 3.3 below. Then seating pressure of 5Kpa was applied to keep the specimen from swelling. Immediately after the application seating pressure, the deformation indicator and record was adjusted to zero reading. Then a series of known vertical pressures were applied to the sample using a weight and lever system that forms part of the loading frame of the Odometer.

As each pressure increment was applied, readings of vertical compressions were taken at a regular time interval until movement ceases usually, a load kept for 24hrs then the next load was applied. Once the applied pressure reaches the vertical effective stress that acted on the sample in the field, water is added to the Odometer cell to ensure no water is lost by evaporation. The standard load increment shall consists of load increment ratio of one which is obtained by doubling the pressure on the soil to obtain values of approximately 12,25,50,100,200,400 Kpa etc…and change in height of the specimen on a dial gauge is recorded at 0.1,0.25,0.5,1,2,4,8,15,30,60,120….1440 minutes immediately after the application of the each load increment.

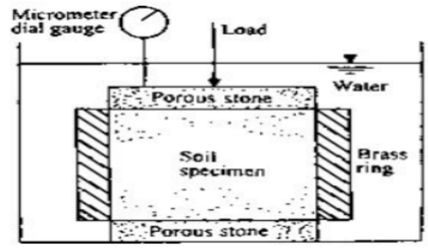

Figure 2 3 Consolidometer from (Das, 2008).

2.7. Data Analysis Methods

Some mathematical expressions and graphical analysis methods were used to determine consolidation parameters.

2.7.1. Pre-Consolidation Pressure (σ_c)

A soil in the field at some depth has been subjected to a certain maximum effective past pressure in its geologic history. This maximum effective past pressure may be equal to or less than the existing effective overburden pressure (σ_0) at the time of sampling. The reduction of effective pressure in the field may be caused by natural geologic process or human processes. During soil sampling, the existing effective overburden pressure is also released. When this specimen is subjected to a consolidation test, small amount of compression will occur when the effective pressure applied is less than the maximum effective overburden pressure in the field to which the soil has been subjected in the past [10]. The two basic definitions of clay based on stress history are:

✓*Normally consolidated*; whose present overburden pressure is the maximum pressure that the soil was subjected to it in the past.

✓*Over-consolidated*; whose present effective overburden pressure is less than that which the soil experienced in the past.

The maximum effective past pressure is called the *pre-consolidation* pressure. Several methods have been proposed for determining the value of the maximum consolidation pressure. These are field method and graphical procedure based on consolidation test results. The pre-consolidation pressure from an *e* versus logσ' plot is generally determined by a graphical procedure suggested by Casagrande, 1936 as in figure 3.4 and the steps are as follows;

1. Visually determine the point p (on the upper curved portion of the e versus logσ plot) that has the maximum curvature.
2. Draw a horizontal line PQ
3. Draw a tangent PR at P.
4. Draw the line PS bisecting the angle QPR
5. Produce the straight line portion of the e-Vs-logσ_c' plot backward to intersect PS at T
6. The effective pressure corresponding to point T is the pre-consolidation pressure σ'_c

The relative amount of pre-consolidation is usually reported as the over-consolidation ratio (OCR) defined as: $OCR = {\sigma'c}/{\sigma o}$ ……………………..2.1

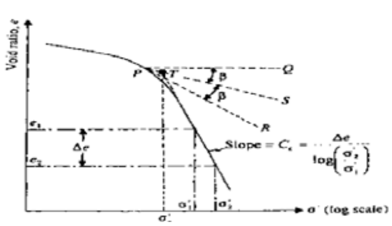

Figure 2 4 Typical e versus logσ plot showing procedure for determination of σ_c and C_c**.**

2.7.2. Coefficient of Consolidation (c_v)

For a given load increment, the coefficient of consolidation C_v can be determined from the laboratory observations of time versus dial reading. There are two laboratory methods that are presently in common use for determination of the coefficient of consolidation. One of them is the *logarithm-of-time method* proposed by Casagrande and Fadum 1940 and the *square-root-of-time method* suggested by Taylor [11]. For this thesis the coefficients of consolidation C_v were determine by using both methods.

i. Logarithm-of-Time Fitting Method

The logarithm-of-time method was originally proposed by Casagrande and Fadum 1940 and can be explained by referring to Figure 3.5 below.

1. Plot the dial reading for specimen deformation for a given load increment against time on semi log graph paper.
2. Plot two points B and C on the upper portion of the consolidation curve which corresponds to time t_1 and t_t respectively. Note that $t_2=4t_1$.
3. The difference of dial reading between B and C is equal to x. locate point D, which is at a distance x above point B.
4. Draw the horizontal line DE. The dial reading corresponding to this line is d_o, which corresponds to 0% consolidation.
5. Project the straight line portion of the primary consolidation and the secondary consolidation to intersect at T. the dial reading corresponding to T is d_{100}, i.e., 100% primarily consolidation.
6. Determine the point V on the consolidation curve that corresponds

to a dial reading of (d_o +d_{100})/2=d_{50}. The time corresponding to point V is t_{50}, i.e., time for 50% consolidation.

7. Determine C_v from the equation T_v= $C_v t/H^2$. The value of T_v for U_{av}=50% is 0.197 [11].

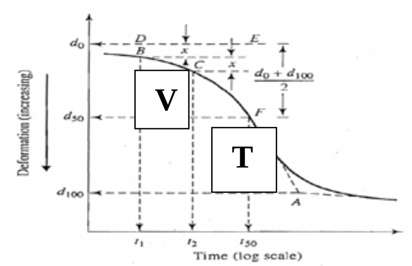

Figure 2 5 Log- of- time method for determining of coefficient of consolidation (C_v) [11].

$C_v=(0.197)*\frac{Hdr^2}{t50}$ … … … … … … … … .2.2

For specimens drained at both top and bottom, H_{dr} equals one-half the average height of the specimen during consolidation.

$H_{dr}=\frac{Ho+Hf}{4}=\frac{Hi-d50}{2}$ … … … … … … … 2.3

ii. Squire Root Time Fitting Method

By plotting of deformation Vs$\sqrt{time}$ as in figure 3.6 have the following steps.

1st Draw a line AB through the early portion of the curve that exhibits a straight line trend. Extrapolate the line back to t=0 and obtain the deformation ordinate representing 0% primary consolidation. 2nd Draw a line AC such that line OC=1.15OB. The abscissa of point D which is the intersection of AC and the consolidation curve, gives the squire root of time for 90% consolidation, 3rd For 90% consolidation, T90=0.848.

Therefore, $T_{90}=\frac{Cv*t90}{H^2dr}$and Hence,

$$Cv = 0.848*\frac{H^2dr}{t90} \dots\dots\dots\dots\dots..2.4$$

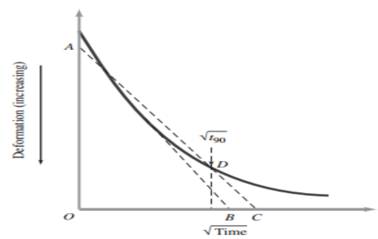

Figure 2 6 A typical graph for the squire –root-of time fitting method [11].

4th The deformation at 100% consolidation is 1/9 more than the difference in deformation between 0 and 90% consolidation. The time for primary consolidation, t100 may be taken at the intersection of the deformation- squire root time curve and this deformation ordinate. 5th the deformation at d_{50}, corresponding to 50% consolidation is equal to the deformation at 5/9 the difference between 0 and 90% consolidation [11].

2.7.3. Compression Index (C_c) and Swelling Index (C_s)

The compression index (C_c) can be determined by graphic construction from laboratory test result for void ratio and pressure curve for logarithmic scale which is equal to the slope of the linear portion of the curve. The slope of the straight portion of the e V_s logσ' plot for normally consolidated soil is referred to as the compression index C_c [12].

$C_c=\frac{e1-e2}{\log\sigma 2'-\log\sigma 1'}=\frac{\Delta e}{(\frac{\log\sigma\ 2}{\log\sigma\ 1})}$ … … … … … … 2.5

Coefficient of Compressibility (a_v)

The coefficient of compressibility is defined as decrease in void ratio per unit increase in effective stress. When the consolidation test results are plotted between void ratio and effective stress arithmetically, the slope of the curve for pertinent stress range that is the coefficient of compressibility a_v can be used for the computation of settlement [12]. As the effective stress increase, the void ratio decrease and the ratio of void ratio to effective stress is negative and for convenience coefficient of compressibility is reported as positive.

$$a_v = \frac{\Delta e}{\Delta \sigma'} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.6$$

Where; a_v=coefficient of compressibility, Δe=change in void ratio, $\Delta\sigma'$=change in stress.

2.7.4. Coefficient of Volume Compressibility (m_v)

The coefficient of volume compressibility (or volume change) is defined as volumetric strain per unit increase in effective stress can be expressed by the equation as;

$$m_v = \frac{-\Delta v/vo}{\Delta \sigma'} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.7$$

Where m_v=coefficient of volume change, V_o=initial volume, Δv=change in volume, and $\Delta\sigma'$=change in effective stress. The volume strain ($\Delta v/vo$) can be expressed in term of either void ratio and the thickness of the specimen.Let e_o be the initial void ratio, let the volume of solid be unity. Therefore the initial volume vo is equal (1+eo), if Δe is the change in void ratio due to the change in volume Δv, then, $\Delta v=\Delta e$ [12].

$$\text{Thus } \frac{\Delta v}{vo} = \frac{\Delta e}{1+eo} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.8$$

Therefore the equation 3.8.Becomes;

$$mv = \frac{\frac{\Delta e}{1+eo}}{\Delta \sigma'} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.9$$

As the area of cross-section of the sample in the consolidometer remains constant, the change in volume is also proportional to change in height. Thus $\Delta V=\Delta H$

$$\text{Therefore;} \frac{\Delta V}{Vo} = \frac{\Delta H}{Ho} \ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.10$$

The relationship between a_v and m_v given as,

$$m_v = \frac{av}{1+eo} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.11$$

2.7.5. Calculation of One-Dimensional Consolidation Settlement (S_c)

The basic principle of one-dimensional consolidation settlement calculation is demonstrated in figure 3.7 below. If a clay layer of total thickness H_t is subjected to an increase of average effective overburden pressure from σ'_o to σ'_1 it will undergo consolidation settlement of ΔH_t and hence the strain can give as;[13]

$$\epsilon = \frac{\Delta Ht}{Ht} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.12$$

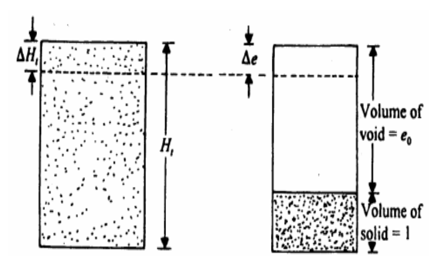

Figure 2 7 Calculation of one dimensional settlement.

Where, ϵ is strain, H_t =thickness of clay layer and ΔH_t=change in height of clay layer. Again if undisturbed laboratory specimen is subjected to the same effective stress increase the void ratio will be decreased by Δe. Thus the strain is equal to;

$\epsilon=\frac{\Delta e}{1+eo}$ … … … … … … … … … … … … … . . 2.13

where, e_o is the initial void ratio at σ'_o. Thus from equation (2.18) and (2.19);

$\Delta H_t=\frac{\Delta e}{1+eo} * Ht$ … … … … … … … … … … . 2.14

From the compression index
$C_c=\frac{e1-e2}{\log\sigma 1'-\log\sigma o'}=\frac{\Delta e}{(\frac{\log\sigma 1'}{\log\sigma 0'})}$ … … … … … 2.15

For over consolidated soils;

$\Delta e=Cc(\log\sigma 1'-\log\sigma c')$ where $\sigma_{1'}=\sigma_{o'}+\Delta\sigma'$ where

Therefore;
$\Delta H_t=\frac{Ht}{1+eo}\Delta e=\frac{Ht}{1+eo}Cc(\log\sigma 1'-\log\sigma c')$where; $\sigma_{1'}=\sigma_{o'}+\Delta\sigma'$

1. By using coefficient of compression index (C_c);

$Sc=(Cc)\frac{Ho}{(1+eo)}\log$ … … … … … … … 2.16

by using coefficient of volume compressibility (m_v)

$Sc=(Ht)(mv)(\sigma f')$ … … … … … … 2.17

Where; m_v= Coefficient of volume compressibility

Sc= One-dimensional consolidation settlement.

H_t=Thickness of soil specimen.

$\Delta\sigma'$ = Induced effective pressure in laboratory

$\sigma_{o'}$ = Existing effective pressure

$\sigma_{c'}$ = pre-consolidation pressure.

C_c=Compression index.

e_o=Initial void ratio.

Δe=Change in void ratio due increase in effective stress.

ΔH_t=Change in height of soil specimen due to increase in effective stress.

3. LABORATORY TEST RESULT AND DISCUSSION

3.1. Test Result and Discussion

The consolidation behavior of the soils were described by plotting the graph between void ratio and effective stress and the values of these parameters for the soils under investigation were shown in the table 3.1 and table 3.2 below.

Table 3 1 Value of void ratios of all test pits for loading of different pressures

Designation	Depth (m)	Loading Pressures in (Kpa)						
		0	50	100	200	400	800	1600
TP-1	1	1.330	1.308	1.235	1.112	0.968	0.831	-
Tp-2	1.5	1.430	1.410	1.350	1.220	1.070	0.910	0.81
Tp-3	2	1.469	1.451	1.416	1.324	1.190	0.994	0.84
Tp-4	2.5	1.646	1.631	1.569	1.482	1.302	1.036	0.820
Tp-5	3	1.74	1.72	1.663	1.524	1.366	1.187	1.088

Table 3 2 Value of void ratios of all test pits for unloading of different pressures.

Designation	Depth (m)	Un loading Pressures in (Kpa)					
		1600	800	400	200	100	50

TP-1	1	-	0.831	0.836	0.841	0.845	0.848
Tp-2	1.5	0.810	0.8102	0.8167	0.8217	0.8251	0.8283
Tp-3	2	0.840	0.8447	0.8521	0.8585	0.8649	0.8693
Tp-4	2.5	0.820	0.8246	0.8311	0.8378	0.8442	0.8489
Tp-5	3	1.088	1.092	1.0996	1.104	1.1078	1.11

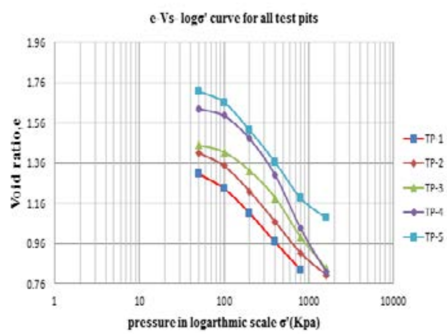

Figure 3 1 e-Vs-log σ' curves for all test pits

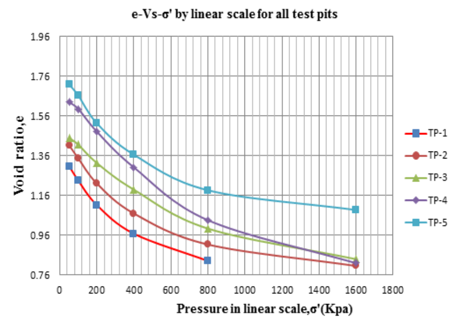

Figure 3 2 e-Vs σ' in linear scale for all test pits

The e-Vs-logσ, curves for all soil samples under investigation for this study are identical in their shapes for both logarithmic and linear scale as shown in figure 3.1 and figure 3.2 above and show continuously compressing behavior after pre-consolidation pressure. So we can understand that, the compressibility of these soils increase with the load applied and finally minimized. As we see on the graphs, the curve due to logarithmic scales shows that the soils are somewhat stiff at the beginning of loading process due to cementation effect and for further increment of the load the compressibility increase with the load and finally decreased. But for the linear scale curves, the compressibility increase continuously with the load.

3.1.1. Pre-Consolidation Pressure (σc')

There are a few graphical methods for determining the pre-consolidation pressure based on laboratory test data. For this thesis pre-consolidation pressures were determined by using Casagrande method and the results are described in fig 3.8-3.12 and table 3.9.

GSJ: Volume 10, Issue 4, April 2022
ISSN 2320-9186

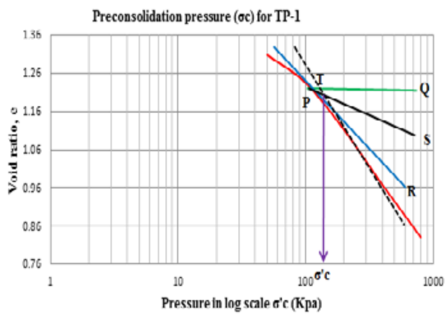

Figure 3 3 Curve to determining pre-consolidation pressure for TP-1

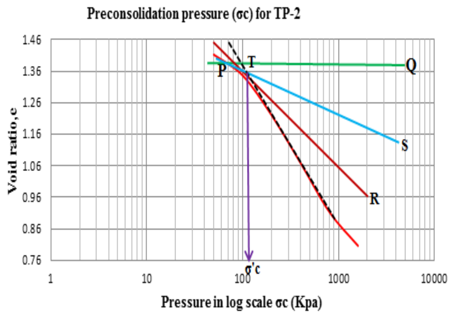

Figure 3 4 Curve to determining pre-consolidation pressure for TP-2

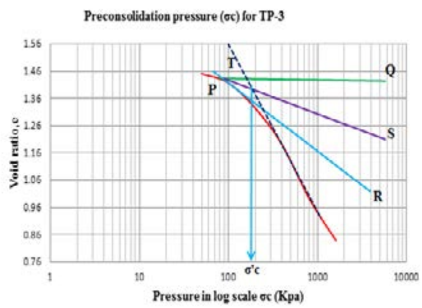

Figure 3 5 Curve to determining pre-consolidation pressure for TP-3**.**

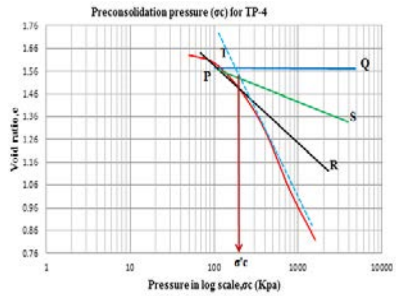

Figure 3 6 Curve to determining pre-consolidation pressure for TP-4

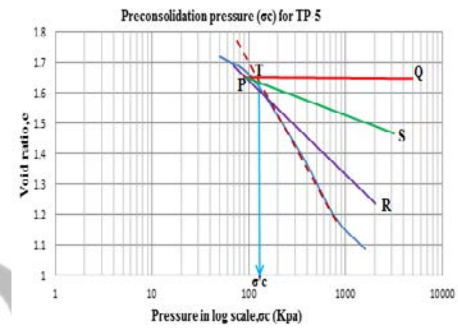

Figure 3 7 e-logσ' curves for determining pre-consolidation pressures for TP-5.

Table 3 3 Pre-consolidation pressures and over-consolidation ratios for different test pits

Pressures in (Kpa)	Designation for each test pits				
	Tp-1	Tp-2	Tp-3	Tp-4	Tp-5
Pre-consolidation pressure (σ_c) (Kpa)	≈140	≈110	≈ 190	≈ 200	≈ 130
Total unit weight γ_b (KN/m^3)	14.13	13.93	14.32	12.85	13.74
Depth of sample (m)	1	1.5	2	2.5	3
Existing overburden (σ_o) (KN/m^2)	14.13	20.89	28.64	32.13	41.22

GSJ: Volume 10, Issue 4, April 2022
ISSN 2320-9186

(OCR)	9.9	5.2	6.6	6.2	3.15

From the above results one can understand that, the soils investigated are too pre-loaded as their over consolidation ratios were greater than 1 for all tests. The apparent pre-consolidation pressure is a consequence of residual cohesive bond between the particles rather than of effective pressure produced by pressure overburden or by desiccation. Generally the results of consolidation test of laterite soils generally indicate that, they are over consolidated. However, the over consolidation phenomenon is also not due to the previous history of pressure but it was due to the cementation in laterite soils which is due to the presence of iron and aluminum sesquioxides and hence, they don't have normal consolidation line [4].

3.1.2. Coefficient of Consolidation (C_v)

Two laboratory methods used to determine coefficients of consolidation for this thesis.

i. Logarithm-of-Time Fitting Method

For the soil under investigation the coefficients of consolidation by using the logarithm-of-time method were calculated by using the graph shown below in figure 3.8 and the results were shown in table 3.4.

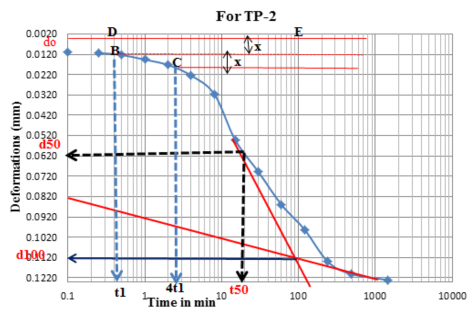

Figure 3 8 Typical Curve for Determining C_v by using log-time Fitting Method for TP-2

Table 3 4 Values of coefficients of consolidations (C_v) by log time fitting method.

Pressures (Kpa)	Tp-1 C_v(cm^2/min) (10^{-3})	Tp-2 C_v(cm^2/min) (10^{-3})	Tp-3 C_v(cm^2/min) (10^{-3})	Tp-4 C_v(cm^2/min) (10^{-3})	Tp-5 C_v(cm^2/min) (10^{-3})
50	05.814	1.52	9.25	9.79	8.89
100	9.10	9.09	6.75	7.15	5.31
200	6.96	8.78	5.87	5.87	4.37
400	7.32	5.01	6.63	6.32	4.74
800	8.58	7.83	6.61	7.07	4.37
1600	-	8.42	6.05	6.88	4.66

ii. Squire Root Time Fitting Method

In a plot of deformation versus the squire root of time drawn for the incremental loading and the coefficients of consolidation were determined as follow.

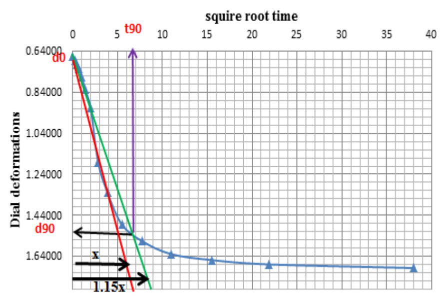

Figure 3 9 Typical curves for determining C_v from squire root time methodfor TP-2

Table 3 5 Values of coefficients of consolidations (C_v) by squire root time fitting method

Pressures (Kpa)	Tp-1 $C_v(cm^2/min)$ (10^{-2})	Tp-2 $C_v(cm^2/min)$ (10^{-2})	Tp-3 $C_v(cm^2/min)$ (10^{-2})	Tp-4 $C_v(cm^2/min)$ (10^{-2})	Tp-5 $C_v(cm^2/min)$ (10^{-2})
50	9.12	1.31	3.44	2.34	2.37
100	2.21	5.12	3.34	5.22	3.287
200	1.26	2.09	4.92	3.42	3.17
400	0.96	1.41	1.91	2.62	2.95
800	1.24	1.61	1.35	2.37	2.24
1600	-	3.21	1.17	2.20	1.46

For the soil under investigation, the coefficients of consolidation calculated from the test result by squire root time fitting method were relatively smaller than coefficients of which calculated by logarithm-time fitting method. Alternatively, the requesting agency may specify a method of its choice.

3.1.3. Compressibility Index (Cc) and Swelling Index (Cs)

The compression index (C_c) and swelling index (Cs) can be determined by graphic construction from laboratory test result for void ratio and pressure curve which is equal to the slope of the linear portion of the curve for loading and unloading respectively [5].

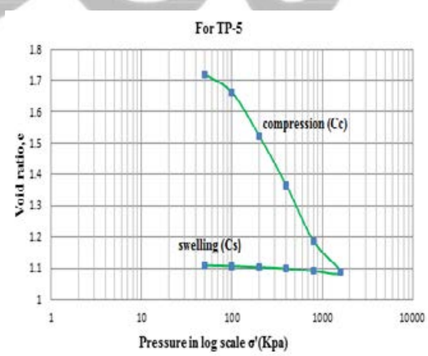

Figure 3 10 a typical curve to determine Cc and Cs for TP-5

Table 3 6 Values compressibility index (C_c) and swelling index (C_s)

Designation	Tp-1	Tp-2	Tp-3	Tp-4	Tp-5
C_c	0.45	0.38	0.51	0.71	0.32
C_s	0.016	0.011	0.024	0.015	0.013

There are several empirical expressions for compression index and swelling index determination is also used for approximate calculation in the absence of laboratory consolidation data. Compression index is extremely useful for determination of settlement caused by consolidation. Residual soils doesn't show normally consolidated behavior rather acting as over consolidated, while considering as a normally consolidated soil the index of compression [4].

3.1.4. Coefficient of Compressibility (a_v)

When the consolidation test results are plotted between void ratio and effective stress arithmetically, the slope of the curve for pertinent stress range that is the coefficient of compressibility a_v can be used for the computation of settlement.

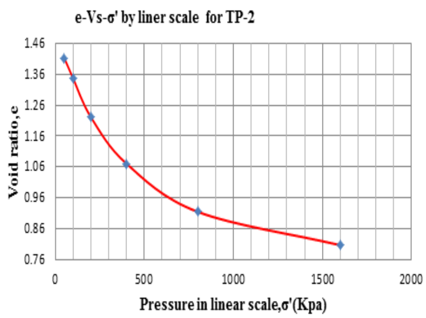

Figure 3 11 Typical curves to determine coefficient of compressibility for TP-2

Table 3 7 Values of coefficient of compressibility (a_v)

Sample designation	Depth (m)	Coefficient of compressibility (a_v) (10^{-3})	Initial void ratio (e_o)
Tp-1	1	0.343	1.33
Tp-2	1.5	0.135	1.427
Tp-3	2	0.193	1.47
Tp-4	2.5	0.270	1.64
Tp-5	3	0.123	1.74

3.1.5. Coefficient of Volume Compressibility (m_v)

Table 3 8 Values of coefficient of volume compressibility

Designation	Depth (m)	Coefficient of compressibility (a_v) (10^{-3})	Initial void ratio (e_o)	Coefficient of volume compressibility (m_v) (10^{-3})
Tp-1	1	0.343	1.33	0.147
Tp-2	1.5	0.135	1.427	0.0556
Tp-3	2	0.193	1.47	0.0781
Tp-4	2.5	0.270	1.64	0.102
Tp-5	3	0.123	1.74	0.0448

3.1.6. Settlement Computation.

The settlement of the soil sample under this thesis was calculated by using coefficients of compressibility index (C_c) and coefficient of volume compressibility (m_v).The settlement behavior of these soils doesn't show significant variation with in 3m depth. This implies that the degree of laterization or cementation effect doesn't vary with in this interval.

1. By using coefficient of compression index (C_c).

Table 3 9 Settlement computation based on coefficient of compression index (C_c)

Desig-nation	depth (m)	γ' (KN/m^3)	σ_o (Kpa)	$\Delta\sigma$ (Kpa)	$\sigma=\sigma_o+\Delta\sigma$ (Kpa)	σc Kpa	C_c	e_o	Height (H_o) (mm)	$H_o/(1+e_o)$	$(\sigma o+\Delta\sigma)/\sigma_c$	S_c(mm)
Tp-1	1	14.13	14.13	800	814.13	140	0.45	1.33	20	8.584	5.81	2.953
Tp-2	1.5	13.93	20.89	1600	1620.895	110	0.38	1.43	20	8.230	14.74	3.654
Tp-3	2	14.23	28.46	1600	1628.46	190	0.42	1.47	20	8.097	8.57	3.173
Tp-4	2.5	12.85	32.13	1600	1632.125	200	0.5	1.64	20	7.576	8.16	3.453
Tp-5	3	13.74	41.22	1600	1641.22	130	0.56	1.74	20	7.299	12.62	4.501

2. By using coefficient of volume compressibility (m_v).

Table 3 10 Settlement computation based on the coefficient of volume compressibility (m_v)

Designation	Depth (m)	Height (H_o) (mm)	Coefficient of volume compressibility (M_v) (10^{-3})	$\sigma_f'=\sigma o'+\Delta\sigma'$ (Kpa)	Settlement (Sc) (mm)
Tp-1	1	20	0.147	814.13	2.394
Tp-2	1.5	20	0.0556	1620.89	1.802
Tp-3	2	20	0.0781	1628.46	2.566
Tp-4	2.5	20	0.102	1632.12	3.330
Tp-5	3	20	0.0448	1642.22	1.471

GSJ: Volume 10, Issue 4, April 2022
ISSN 2320-9186

4. COMPARISON AND DISCUSSION OF TEST RESULT WITH LATERITES SOILS AND SEDIMENTARY SOILS.

4.1. Comparison and Discussion of Consolidation Test Result.

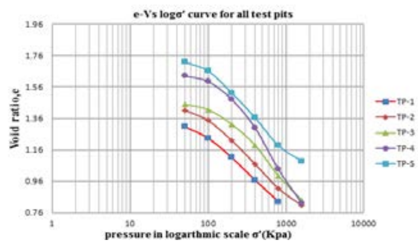

Figure 4 1 e-Vs-log σ' curves for all test pits.

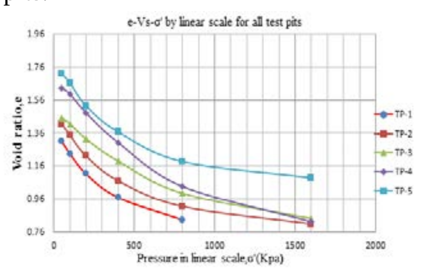

Figure 4 2 e-Vs σ' in linear scale for all test pits.

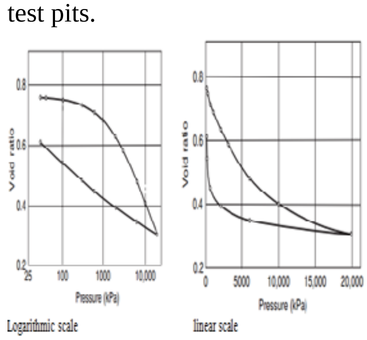

Figure 4 3 Odometer test result from over- consolidated clay of sedimentary soils [6].

The most significant differences in behavior of between residual and sedimentary soils are those associated with their consolidation characteristics. The conventional understanding or interpretation of the consolidation behavior of sedimentary soils is based on their formation mode, namely deposition in sea or lake followed by compression due to self-weight which mean the consolidation behavior of sedimentary soils is highly influenced by stress history. Where as in residual laterite soils stress history has not significant value since they are formed by in-situ weathering of rooks. Thus their formation process is the only determining factor in consolidation behavior of the properties of laterite soils [6]. This research verify that, the consolidation curve of laterite soils as observed in the above figure 4.1 and figure 4.2 with figure 4.3 have similar shape with sedimentary soils. In laterite soils the pre-consolidation pressure was observed in the initial phase of the consolidation curve this is because of the strong cementation due to Iron and Aluminum Sesquioxides, whereas in sedimentary soils the pre-consolidation curve is due to the past stress history [6].

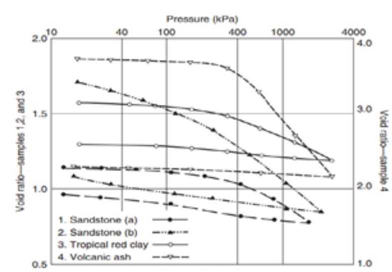

Figure 4 4 Consolidation curve for loading and unloading in log-scale residual soils [6].

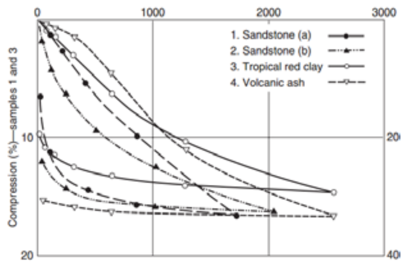

Figure 4 5 Consolidation curves for loading and unloading in linear scale for residual soils [6].

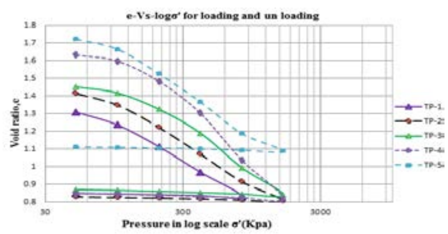

Figure 4 6 Consolidation curves for loading and unloading in logarithmic scale

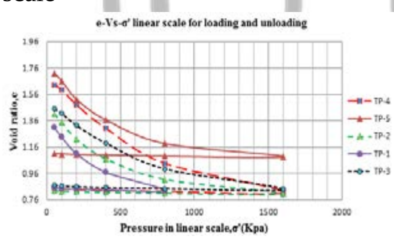

Figure 4 7 Consolidation curve in linear plot for loading and unloading conditions

As described in the literature review, the formation of laterite residual soils doesn't involve the process of sedimentation and consolidation that are essential components of sedimentary soil formation. The plots shown in figure 4.4 using logarithmic scale for residual soils has similar shape with the soils under investigation in this thesis as shown in figure 4.6 and also when they are plotted by using linear scale they do have similar shape as shown in figures 4.5 and figure 4.7.

Residual soils do not have a virgin consolidation line. In figure 4.6 the soils under investigation were plotted in the logarithmic scale and in figure 4.7 they were plotted using a linear scale for pressure. The use this plots is to examine the relative shapes of the compression curves and to compare the compressibility of soils. According to this, TP-4 is considerably more compressible than others, whereas, TP-2 and TP-3 have relatively more compressibility than TP-1 and TP-5 as shown in figure 4.6 which plotted by using logarithmic scale. But here TP-1 was compressed with a maximum effective pressure of 800Kpa during the consolidation test which is lower than the rest samples which were loaded with a maximum effective pressure 1600Kpa.The graphs are all concave from below and their shapes possibly suggest the vertical yield stress at a point of maximum curvature. For the sedimentary soils, the term vertical yield stress can be caused by many factors of which stress history is the most important factor whose influence is limited to sedimentary soils. However, in residual soils the vertical yield stress is due to the laterization process/ cementation process due to the presence of Iron and Aluminum sesquioxides which have cementing effects [6].

5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

- ✓ The consolidation curves for the soils under investigation show that the soils have continuously compressing behavior after pre-consolidation pressure. The plot was made in between void ratio versus effective pressure by using both logarithmic and linear scale. Results from logarithmic scale shows that the soil is less compressible at initial phases and when the load exceeds the pre-consolidation pressure, the soil compress rapidly and finally minimized. But for linear scale compressibility increases continuously with load.

- ✓ The soils under investigation were over-consolidated and hence, they don't have normal consolidation line due to the strong cementation that bonded the soil particles together rather than the previous stress history..

5.2. Recommendation

In this study, consolidation test was conducted on soil samples collected at different place with different depth and hence, a further investigation is recommended to investigate the consolidation behavior along the soil profile.

6. References

[1]. CIRIA, (1995).Laterite in road pavements.

[2]. Wakuma F., (2007). Investigating the Index Properties of Residual Tropical Soils of Western Ethiopia (The Case of Asossa), a thesis presented to School of Graduate Studies, Addis Ababa University.

[3]. Raychaudhuri S.P., 1980. The occurrence, distribution, classification and management of laterite and lateritc soils.JOURNEE GEORGES AUBERT, New Delhi, India.http://www.bondy.ird.fr/pleins_textes/cahiers/PTP/1423.PDF

[4].Adugna.T.,(2015).consolidation and settlement of Residual laterite soil in Western Ethiopia (the case of Tongo-Begi Road Project contract-1,Tongo-Gidami)

[5].Haile B., 2014 "investigation into Some Engineering properties of soils found in woliso Town"

[6]. Laurence D.Wesly,2010.Fundamentals of Soil Mechanics For Sedimentary and Residual Soil, Published by John Wiley and Sons,Inc,Hoboken,New Jersey.

[7]. G.E. Blight & E.C Leong, 2012, 2nd edition , mechanics of residual soils.

[8].Bujang B.K Huat, David G. Toll &Arun Prasad, "Hand book of tropical residual soils".

[9].Das, 2008 3rd edition "Advanced soil mechanics".

[10].V.N.S. Murthy "Geotechnical Engineering"

[11].Braja M. Das 3rd edition "Fundamental of Geotechnical Engineering"

[12].Arora, 2004 "Soil mechanics and foundation Engineering"

[13]. Braja M. Das 2nd edition "Advanced soil Mechanics, 1997).