JIMMA UNIVERSITY

JIMMA UNIVERSITY INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

EVALUATION OF SIUTABILITY OF EXISTING SOILTO USE AS SUBGRADE MATERIAL FOR ROAD CONSTRUCTIONCASE STUDYJIMMA TOWN ,

ECX TO MIKAEL JUCTION

A Project submitted to the School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Engineer in Civil Engineering (Geotechnical Engineering)

BY: BIZUNEH BONSA

May,2016

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Approved by Board of Examiners:

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Chairperson	*Signature*	*Date*

May,2016

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LIST OF ABBREVIATIONS AND ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
CBR	California Bearing Ratio
CH	High Plasticity Clay
CL	Inorganic Silts of Low to Medium Plasticity
DCP	Dynamic Cone Penetrometer
FCBR	Field California Bearing Ratio
FDD	Field Dry Density
GC	Clayey Gravels
GI	Group Index
GM	Silty Gravels
GP	Poorly Graded Gravels
GW	Well Graded Gravel
LI	Liquidity Index
LL	Liquid Limit
MDD	Maximum Dry Density
ML	Inorganic Silts of Low Plasticity
MH	Inorganic Silts of High Plasticity
NCHRP	National Cooperative Highway Research Program of United States of America
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plasticity Limit
S1	ERA Subgrade class with a CBR Range of 0 to 2
S2	ERA Subgrade class with a CBR Range of 3 to 4
S3	ERA Subgrade class with a CBR Range of 5 to 7
S4	ERA Subgrade class with a CBR Range of 8 to 14
SPSS	Statistical Package for Social Science Software
SI	Suitability Index of de-Graft Johnson Equation
USCS	Unified Soil Classification System

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LIST OF SYMBOLS

A	Percent Passing Sieve No. 10 (2mm Sieve Size)
c	Cohesion of Soil
D60	Diameter on the Cumulative Size Distribution Curve where 60 percent of Particles are fines
IP	Plasticity Index
P200, F	Percent Passing Sieve No. 200 (0.075mm Sieve Size)
qu	Bearing Capacity
τ	Shear Strength of Soil
wl	Liquid Limit
w	Moisture Content
α	Standard Significant Error
$\alpha 1$, $\alpha 2$, $\alpha 3$, αn	Coefficients of the Multiple Linear Regression Equation
γ	Bulk density of Soil
β_1, β_2, β_3, βn	Coefficients of the Single Linear Regression Equation
σn	Normal Stress
$\sigma 2$	Statistical Variance
ϕ	Internal Friction Angle
ε	Statistical

CHAPTER ONE

Introduction

1.1 background

Road is built in layers (wearing course, base course, sub-base and sub-grade) from top up to bottom. Of all this, my study could be focused on the sub-grade soil which can be stabilized or non-stabilized soil. The general suitability of sub-grade soil determined from soil classifications. The success or failure of a roadway section is often dependent upon the underlying sub-grade. The strength, stiffness, compressibility and moisture characteristics of the sub-grade can have significant influences on pavement performance and long-term maintenance requirements. The sub grade must be strong enough to resist shear failure and have adequate stiffness to minimize vertical deflection.Stronger and stiffer materials provide a more effective foundation for the riding surface and will be more resistant to stresses from repeated loadings and environmental conditions.

A critical component of the pavement design involves a thorough and reliable characterization of the sub-grade; i.e., the foundation of the pavement riding surface. A number of laboratory methods are available to characterize the strength and stiffness of sub-grade soils including the California Bearing Ratio (CBR), and Atterberg limits and grain size distributions test for estimating soil parameters for use in pavement design methods.

California Bearing Ratio (CBR) is a common and comprehensive test currently practiced in the design of pavement to assess the stiffness modulus and shear strength of subgrade material so as to determine the thickness of overlying pavement layers. In road construction civil engineers always encounter difficulties in obtaining representative CBR value for design of pavement. The type of soil is not the only parameter which affects the CBR value, but it also varies with different soil properties possessed by the soil.

California Bearing Ratio (CBR) is actually an indirect measure which represents comparison of the strength of subgrade, sub-base and base-course material to the strength of standard crushed rock quoted in percentage values. Laboratory CBR test requires relatively large effort to conduct the test and it is time consuming. The alternate method could be to correlate CBR with simpler test results such as soil index properties. These tests are much economical and rapid than CBR test. This project gives an overview to know suitability of existing material as subrade layer that is suited for Jimma town.

Currently, many road construction projects and railway constructions are undergoing in the country. In light of this, the output of the proposed suitability of existing subrade will provide road authorities, railway authorities, consultants and contractors preliminary background information on the value of CBR, for a localized subgrade material, from soil index properties with a benefit of time saving and without incurring any additional cost for carrying out laboratory CBR test.

1.2 Objective of the Study

1.2.1 General Objective

The main objective of this thesis is to determine suitability of existing soil material to use as subgrde layer for construction of new road soil samples recovered from different stations of ECX-Mikeal junction , Jimma Town.

1.2.2 Specific Objectives

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- To determine the physical properties of the existing sub-grade soil.
- To classify the existing sub-grade soil, based on AASHTO
- To identify the Chemical behavior of the existing soil
- To know suitability of existing soil as sub-grde construction and maintenance of any sub-grade around ECX-Mekael junction JimmaTown.

1.3 Scope of the Study

The subject study is desired to conduct a localized research particularly on samples recovered from ECX-Mikeal junction , Jimma town. In order to conduct the proposed find suitability of existing soil material as sub-grade layer for construction of new road ;Eight laboratory test results are used in this research work.

1.4 Methodology

Primarily, in order to address the intended objectives of the study, basic theories and descriptions of CBR test in general and soil index property of sub-grade soil is reviewed. Subsequently, previous works of different researchers with regard to prediction of CBR value from basic soil index properties were assessed.

In order to have satisfactory data for utilizing existing soil as sub-grade layer, laboratory tests were conducted by the researcher on samples collected from different localities of ECX-Mikeal junction; Jimma town, so as to get records of test results of CBR values along with the associated soil indices particularly the grain size analysis, Atterberg limits, moisture-density relationships. Then, discussions on sample collection and summary of laboratory test results were presented.

Statistical analyses of test results were carried out and sub-grade were developed and also analyzed to fit the test results. Under the discussions of the obtained results the suitability of the developed existing sub-grade were examined. Finally, a generalized conclusion and recommendation were made.

1.5 Organization of the Study

The thesis is organized and presented under Five Chapters. The first Chapter highlights introduction of the subject study. Chapter two deals with review of published literature. In Chapter three, discussions on sample collection and on test results were made. In Chapter four, correlation and regression analyses were conducted and Under Chapter Five, the conclusion and recommendation were presented. . Organization of the thesis work is presented with a flow chart as follows:

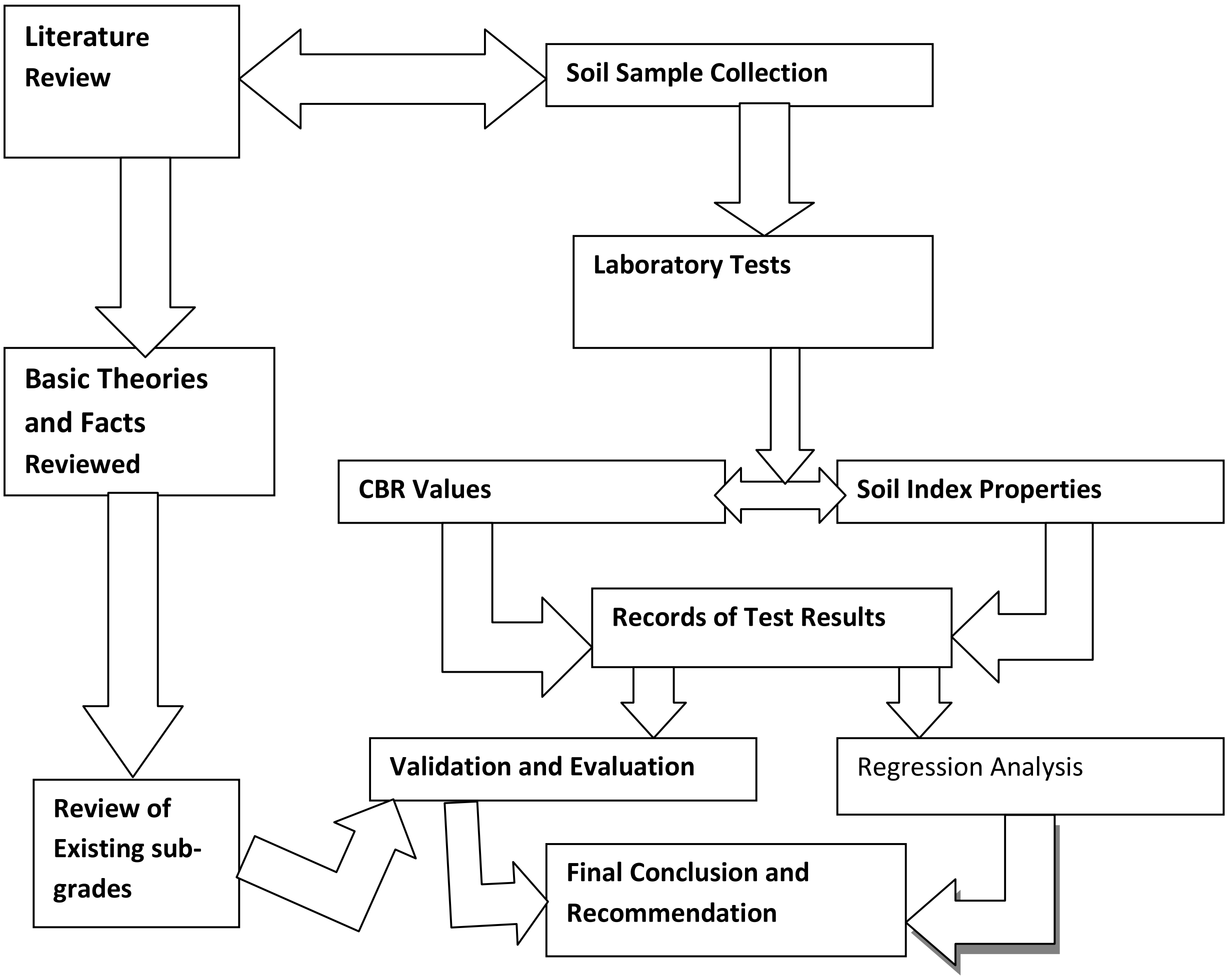

Fig 1.1 flow chart of study

Chapter Two

Literature Review

2.1 California Bearing Ratio (CBR)

2.1.1 General

The California Bearing Ratio (CBR), defined as the ratio of the resistance to penetration of a material to the penetration resistance of a standard crushed stone base material. California Bearing Ratio is the main design input in pavement construction to assess the stiffness modulus and shear strength of subgrade material. The method was developed by the California Division of Highways as part of their study in pavement failure at World War II [1].

With an intention to adopt a more simplified test method to measure the stiffness modulus and shear strength of subgrade soil a simple test that can be used as an index test was devised. This is where CBR test comes into frame in measurement of subgrade strength. The CBR test is a simple strength test that compares the bearing capacity of a material with that of a well graded standard crushed stone base kept in California Division of Highways Laboratory [1]. This means that the standard crushed stone material should have a CBR value of 100%. The resistance of the crushed stone under standardized conditions is well established. Therefore, the purpose of a CBR test is to determine the relative resistance of the subgrade material under the same conditions. The test is an index test, thus it is not a direct measure of stiffness modulus or shear strength.

The CBR test is essentially a measure of the shearing resistance of a soil at a known moisture and density conditions. The method of evaluating CBR is standardized in AASHTO T 193 and ASTM D 1883.

2.1.2 Applications of California Bearing Ratio

The design of pavement thickness in road construction requires the strength of subgrade soil, sub-base and base-course material to be expressed in terms of California Bearing Ratio, so that a stable and economic design achieved. A road section for which a pavement design is undertaken should be sub-divided into subgrade areas where the subgrade CBR can be reasonably expected to be delineated uniform, i.e. without significant variations, in order to utilize it in the design of pavement thickness [2]. It is also used to rate the conditions of an existing pavement layers.

On the other way, the value of CBR is an indicator of the suitability of natural subgrade soil as a construction material. If the CBR value of subgrade is high, it means that the subgrade is strong and as a result, the design of pavement thickness can be reduced in conjunction with the stronger subgrade. Conversely, if the subgrade soil has low CBR value it indicates that the thickness of pavement shall be increased in order to spread the traffic load over a greater area of the weak subgrade or alternatively, the subgrade soil shall be subjected to treatment or stabilization.

2.1.3 Test Methods

The California Bearing Ratio (CBR) test can be carried out both in laboratory and in field. The samples may be prepared in three different ways. Accordingly, (i) the test can be performed on a remolded sample in laboratory, (ii) on undisturbed sample carefully extracted from field and trimmed to closely fit the standard mould in laboratory and finally (iii) an in-situ sample which is entirely tested on field.

2.1.3.1 In-Situ Field Testing

Field in-place tests are used to determine the relative strength of soils, sub-base and some base materials in the condition at which they exist at the time of testing. Field in-place CBR tests are used for the design of flexible pavement components and for other applications in which CBR is the desired strength parameter. If field CBR is to be used directly for evaluation or design without consideration for variation due to change in water content, the test should be conducted under one of the conditions stated in ASTM D 4429-93 [3].

In the field study, once a set of field CBR tools are setup for carrying out the penetration test, the cylindrical plunger is allowed to penetrate the soil at a given rate. The force required to cause the plunger to penetrate the in-situ soil with respect to the penetration depth would be recorded by means of a calibrated proving ring. Later, the results of the in-situ soil shall be compared with the relationship between force and penetration to that of a standard load of a crushed stone base material. A typical in-situ CBR testing apparatus is shown in Figure 2.1 (a).

Fig 2.1 CBR test apparatus .

In order to get a reliable result care measures shall be taken for any construction test activities, such as grading or compacting carried out subsequent to the field in-situ test which will probably invalidate the results of the test. It should be further noted that during in-situ testing the removal of larger-sized particles which may adversely affects the test result is not possible [4]. Therefore, the in-situ test is likely to encounter such problems in coarser types of in-situ material, whereas the laboratory CBR test is limited to particles passing 3/4 inch (19 mm) sieve size [5].

2.1.3.2 Laboratory Testing

Laboratory CBR test is carried out as per the procedure outlined in AASHTO T 193-63 or ASTM D 1883-73. This test method provides the determination of the CBR of a material at optimum water content or a range of water content from a specified compaction test and a specified dry unit weight. The dry unit weight is usually given as a percentage of maximum dry density from the compaction tests of either standard proctor test (ASTM D 698) or modified proctor test (ASTM D 1557). A typical laboratory CBR apparatus is shown in Figure 2.1 (b).

The Laboratory CBR test procedure is based on the principle of a plunger of standard area advancing into a remolded sample at a specified rate of penetration. Prior to the penetration test the soil sample is remolded in laboratory at a desired moisture content and density. The remolded sample may be soaked for 96 hour with a surcharge load not less than 4.52 kg/10 lb that is a representative of the pavement weight in the field. Swell readings are taken during this period at arbitrary selected times. It is worth nothing that the soaking requirement depends on the climate of the study area and on the specifications requirement to be applied in the design [5].

At the end of the soaking period the penetration test is carried out at a rate of 1.27mm/min and the force or load required to cause the penetration will be recorded with respect to the standard penetration depths at each 0.5mm penetration, including the load value at 2.54 mm and 5.08 mm until the total penetration is 12.7mm. The penetration resistance load is then plotted against the penetration depth and correction is made for the load-penetration curve. A typical load-penetration curve correction graph is presented in Figure 2.2.

Using the corrected value taken from the load-penetration curve for 2.54 mm and 5.08 mm penetration, the bearing ratio is calculated by dividing the corrected load by the corresponding standard load, multiplied by 100. Its value ranges from 0 (worst) to 100 (best). If the bearing ratio of 2.54 mm is greater than that of 5.08 mm, the bearing ratio that should be reported for the soil is normally the one at 2.54 mm penetration. When the ratio at 5.08 mm penetration is greater, the test is entirely repeated on a fresh specimen. If the repeated result of 5.08 mm is again greater, the design bearing ratio will be that of 5.08 mm or else, if the bearing ratio of 2.54 mm is greater the design bearing ratio will be that of 2.54 mm penetration [5].

9 In the laboratory test, if the soil sample is remolded using one density and one moisture content, the design CBR value will be the one that satisfies the above bearing ratio criteria. Whereas in the case of a range of densities are used in the test, after getting the bearing ratio for each sample, density versus CBR curve is plotted and the design CBR value of the soil will be the one corresponding to the desired dry density from the Density-CBR plot. The later approach is more practiced in different specifications and also the current research has followed this testing procedure.

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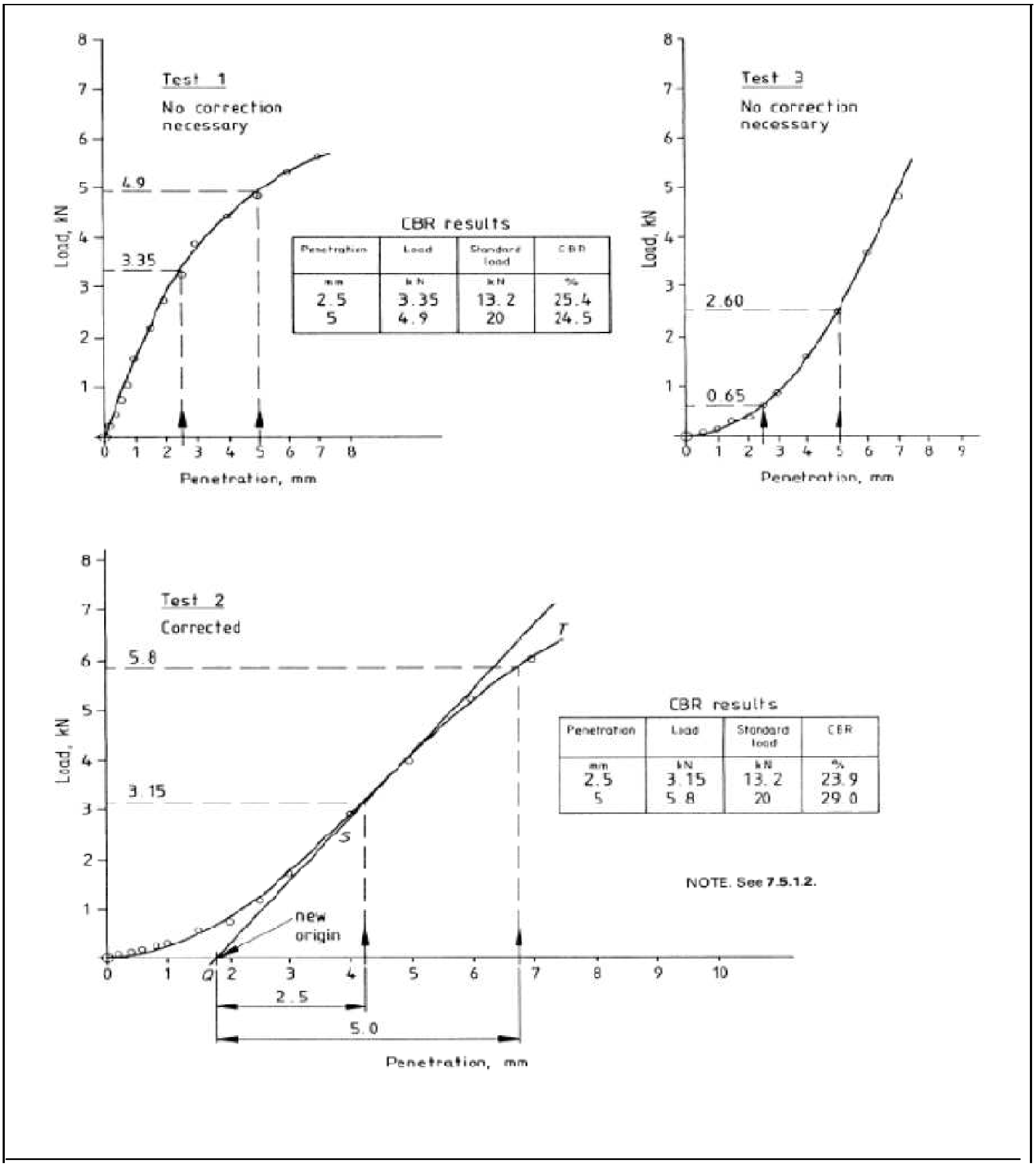


Fig 2.2 : Typical CBR Load-Penetration Correction Graphs.

2.2 Index Property Tests

In nature soil occurs in a large variety. Engineers are continually searching for simplified tests that will increase their knowledge of soils by employing a simple and rapid soil tests. These simplified tests which are indicative of the engineering properties of soils are called index properties [7]. Index properties of cohesive soils are used to characterize the physical and mechanical behavior of soils by making use of parameters such as moisture content, specific gravity, particle size distribution, Atterberg limits and moisture-density relationships. Such parameters are useful to classify cohesive soils and provide correlations with engineering soil properties [8].

2.2.1 Soil Classification

Soils exhibiting similar behavior can be grouped together to form a particular group under different standardized classification systems. A classification scheme provides a method of identifying soils in a particular group that would likely exhibit similar characteristics. There are different classification devises such as USCS and AASHTO classification systems, which are used to specify a certain soil type that is best suitable for a specific application. These classification systems divide the soil into two groups: cohesive or fine-grained soils and cohesion-less or coarse-grained soils.

2.2.1.1 Grain Size Analysis

For coarse grained materials, the grain size distribution is determined by passing soil sample either by wet or dry shaken through a series of sieves placed in order of decreasing standard opening sizes and a pan at the bottom of the stock. Then the percent passing on each sieve is used for further identifying the distribution and gradation of different grain sizes [10]. Particle size analysis tests are carried out in accordance to ASTM D 422-63. Besides, the distribution of different soil particles in a given soil is determined by a sedimentation process using hydrometer test for soil passing 0.075mm sieve size. For a given cohesive soil having the same moisture content, as the percentage of finer material or clay content decreases the shear strength of the soil possibly increases.

2.2.1.2 Moisture Content

Change in moisture content is the most influential parameter that affects the property of soils. Moisture content is defined as the ratio expressed as a percentage of mass of water to mass of soil solids. The moisture content test is carried out in laboratory as per the procedures of AASHTO T 265 or ASTM D 2216 and in field according to AASHTO T 217.

2.2.1.3 Atterberg Limits

Based on their mode of formation and mineralogical composition different soils respond differently for the same moisture content. Albert Atterberg, a Swedish Scientist in 1911 gave an idea of the consistency limit of cohesive soils and proposed a number of tests for defining their properties. The three Atterberg limits which are liquid limit, plastic limit and shrinkage limits are the boundary between each of the two consecutive states of the soil-water phases. Their test is performed only on that portion of a soil which passes the 425mm (No. 40) sieve [9]. A description of phases of soil-water system is shown with schematic diagram in Figure 2.3.

Liquid Limit: The liquid limit (LL) is the water content, expressed in percent, at which the soil changes from a liquid state to a plastic state and principally it is defined as the water content at which the soil pat cut using standard groove closes for about a distance of 13cm (1/2 in.) at 25 blows of the liquid limit machine (Casagrande Apparatus). The liquid limit of a soil highly depends upon the clay mineral present. The conventional liquid limit test is carried out in accordance of test procedures of AASHTO T 89 or ASTM D 4318. A soil containing high water content is in the liquid state and it offers no shearing resistance.

Plastic Limit: The plastic limit (PL) is the water content, expressed in percentage, below which the soil stops behaving as a plastic material and it begin to crumble when rolled into a thread of soil of 3.0mm diameter. The conventional plastic limit test is carried out as per the procedure of AASHTO T 90 or ASTM D 4318. The soil in the plastic state can be remolded into different shapes. When the water content is reduced the plasticity of the soil decreases changing into semisolid state and it cracks when remolded.

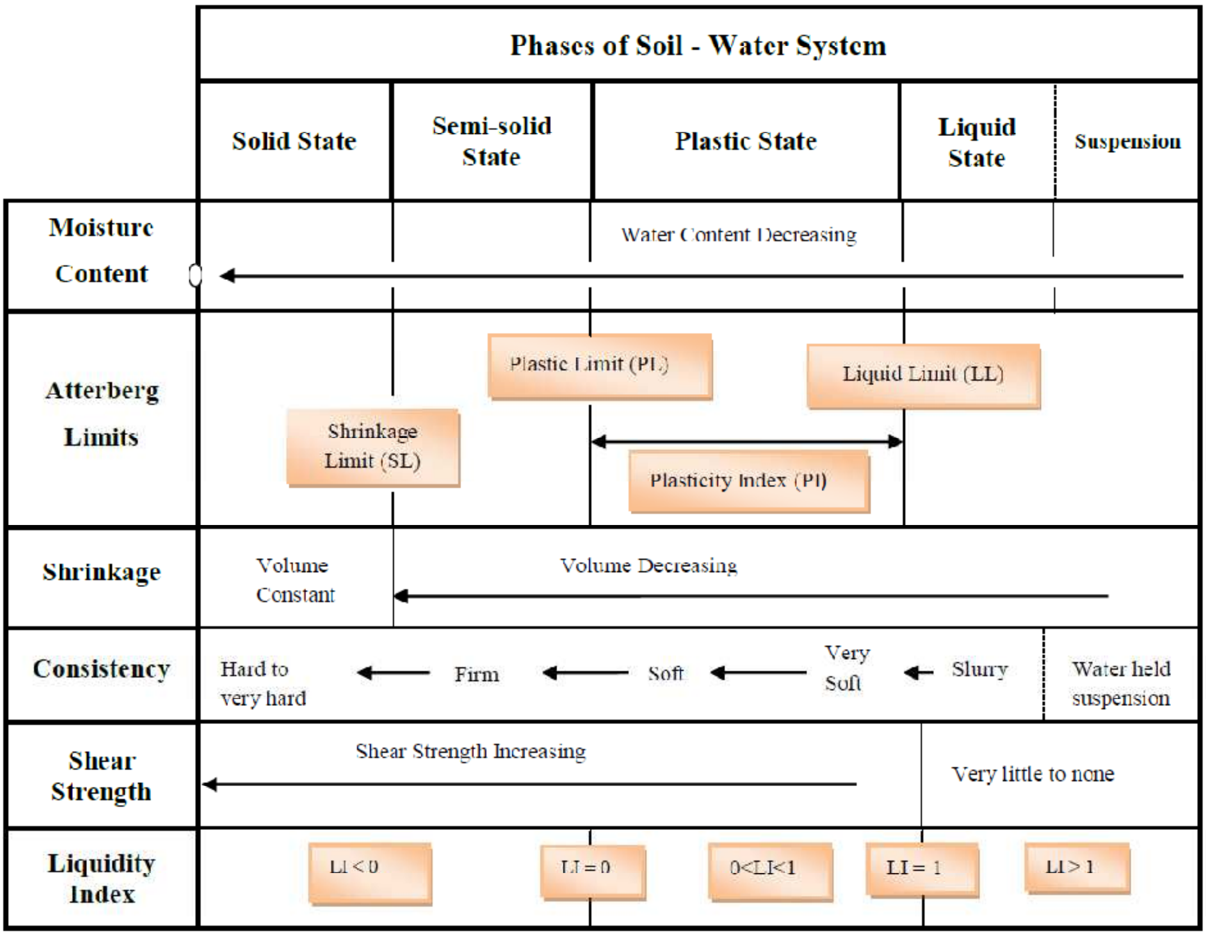


Fig 2.3:Description of Phases of Soil-Water System.

Shrinkage Limit: Is defined as the water content in which the soil changes from semi-solid state to solid state. In other word, it is the moisture content below which no soil volume change occurs with further reduction of water content. The test is carried out as per the procedure of AASHTO T 92 or ASTM D 427.

The amount of water which must be added to change a soil from its plastic limit to liquid limit is an indication of the plasticity of the soil. The degree of plasticity is measured by the plasticity index (PI), which is the numerical difference between liquid limit and plastic limit (PI=LL – PL). The greater the plasticity index means that the soil is more plastic, compressible and the greater volume change characteristic of the soil.

The liquid limit, plastic limit and plasticity index parameters are an integral part of several engineering properties and characterize the fine grained fractions of construction materials. The shear strength of a fine grained soil mainly depends on the consistency of the soil. As shown in Figure 2.3, the shear strength of fine grained soil increases as the moisture content in the soil decreases from the liquid state up to the solid state.

2.2.2 Moisture - Density Relationship

Compaction of a soil improves the engineering properties, i.e. it increases the shear strength of the soil and hence, the bearing capacity. It increases the stiffness and thus, reduces future settlement, void ratio and permeability. At lower water content than the optimum the soil is rather stiff and has a lot of void spaces and hence, the dry density is low. On the other hand, at water content more than the optimum the additional water reduces the dry density as it occupies the space that might have been occupied by solid particles [7].

The laboratory standard proctor and modified proctor tests are performed as per (AASHTO T 99 or ASTM D 698) and (AASHTO T 180 or ASTM D 1557) respectively. The tests are performed on disturbed samples of soil particles passing sieve sizes 4.75mm or 19mm mixed with water to form samples at various moisture contents ranging from the dry state to wet state. These samples are compacted in three or five layers at 25 blows per layer in accordance with the specified nominal compaction energy of standard or modified proctor test respectively. Dry density is determined based on the moisture content and the unit weight of compacted soil. The corresponding water content at which the maximum dry density occurs is termed as the optimum moisture content [10].

Grading and Atterberg limits alone are not sufficient to qualify the performance of construction materials since variation of moisture content and density play a considerable role. Different researches show that the moisture content and density conditions have a greater influence, on the value of shear strength of a soil, on coarser materials than fine grained materials.

CHAPTER THREE

Sample Collection and Test Results

3.1 Study area

In order to have sufficient and reliable data for the target analysis, laboratory tests conducted on soil samples obtained from different localities of ECX-Mekael Junction Jimma town. Most of the samples collected from undergoing road construction projects during the excavation stage. A total of Eight disturbed samples were gathered within a reasonable sampling interval. The representative samples selected on the basis of visual identification of a suitable subgrade soil, as such a diversified samples acquired from area Jimma town; ECX-Mikael juction. The location of collected soil samples site is shown with the aid of map in Figure 3.1:

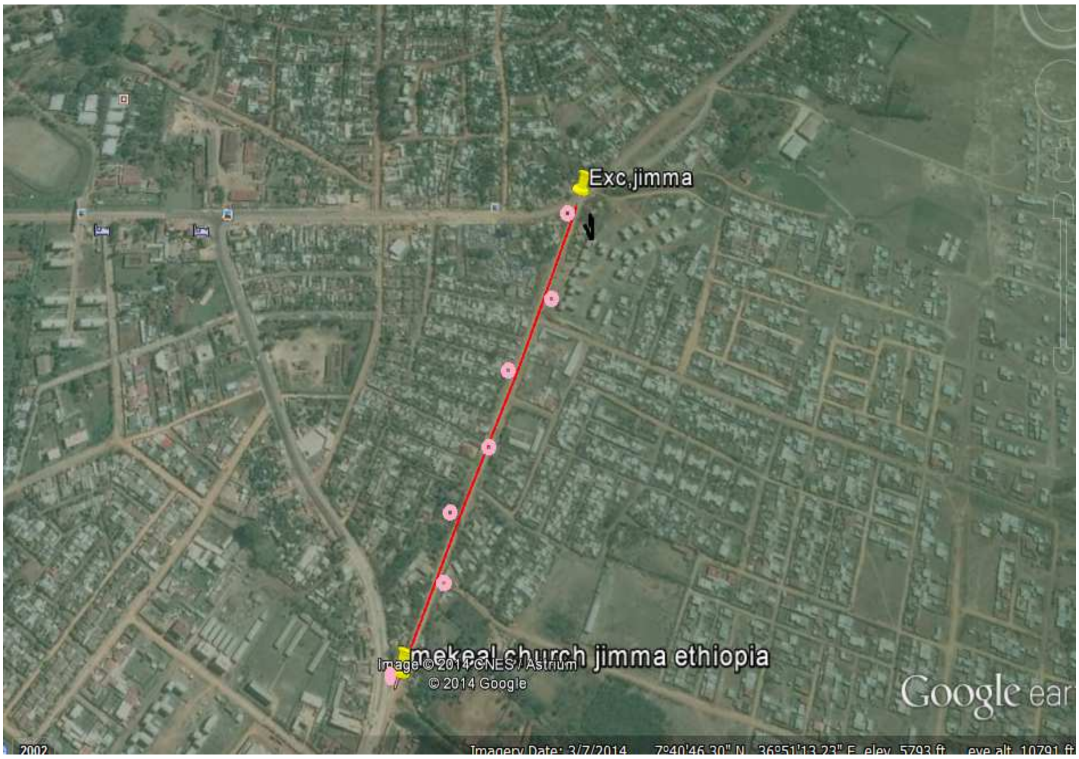


Figure 3.1: Sample Collection from Different Stations of ECX-Mikael junction ;Jimma town

3.2 Laboratory Tests and Results

3.2.1 Discussion on Laboratory Tests

Based on the samples retrieved from the sites, laboratory tests on the eight samples were conducted in the Ethiopian Road Construction corporation(ERCC) district of Jimma laboratory and. In addition, on Jimma-Bonga road construction project laboratory, some tests were carried out in the joint material testing laboratory of LEA,LASA,CORE-JV Supervision Consultants. Accordingly, the following different kinds of tests have been performed:

- Grain size Analysis Test (ASTM D 422-63)
- Liquid Limit Test (ASTM D 4318)
- Plastic Limit Test (ASTM D4318-III)
- Modified Proctor Test (AASHTO T 180)
- Three-point CBR Test (AASHTO T 193)

The above conventional tests were conducted on the eight soil samples and a range of test results achieved. Based on the obtained test results of plasticity and grain size distribution the soil classification was made and the result shows that all the sample are classified as fine grained soil. In accordance to the AASHTO classification system the soil is mainly classified as A-7-6 and also From the conventional Atterberg limit tests, a liquid limit value ranging from 43 up to 97, plasticity limit value of 23 up to 43 and a plasticity index value of 20 up to61 were obtained.

A modified proctor test conducted as per AASHTO T 180 D, through which samples compacted at five layers each compacted by 25 uniform blows using 4.54 kg weight of hammer. From the modified proctor test, after plotting moisture-density curve, a range of maximum dry density along with the optimum moisture content were obtained. Similarly, the CBR test was carried out, on samples remolded with OMC using 10, 30 and 65 blows of modified proctor density and soaked for four days. Consequently, after the penetration test were carried out a CBR value ranging from 0 up to 7.8 is obtained at 95% MDD of modified ASHTO proctor density.

For the sake of illustration and easy of reference, the typical test results of a soil sample No. 1 have been demonstrated hereunder from Figures 3.2 up to 3.6

Table 3.1 Existing subgrde Gradation Test

project name Jimma town asphalt road project (ECX-Mikael junction)

Sample station 0+000 @ RHS

Sample A Weight before Washing =2506gm After Washing =2248gm

Sieve(mm)	weight retained	%age retained	%age passing	Specification	
				lower	Upper
50			100	100	100
37.5	0	0.00	100.00	80	100
20	0	0.00	100.00	60	100
4.75	53.48	2.61	97.39	30	100
2	135.07	5.39	92.00	10	100
1.18	325.82	13.00	79.00	17	75
0.425	280.67	11.20	67.80	12	60
0.3	495.36	19.77	59.23	9	50
0.075	674.86	19.95	32.30	5	25
Pan	540.73	10.30	22.00		
Total	2506.00				

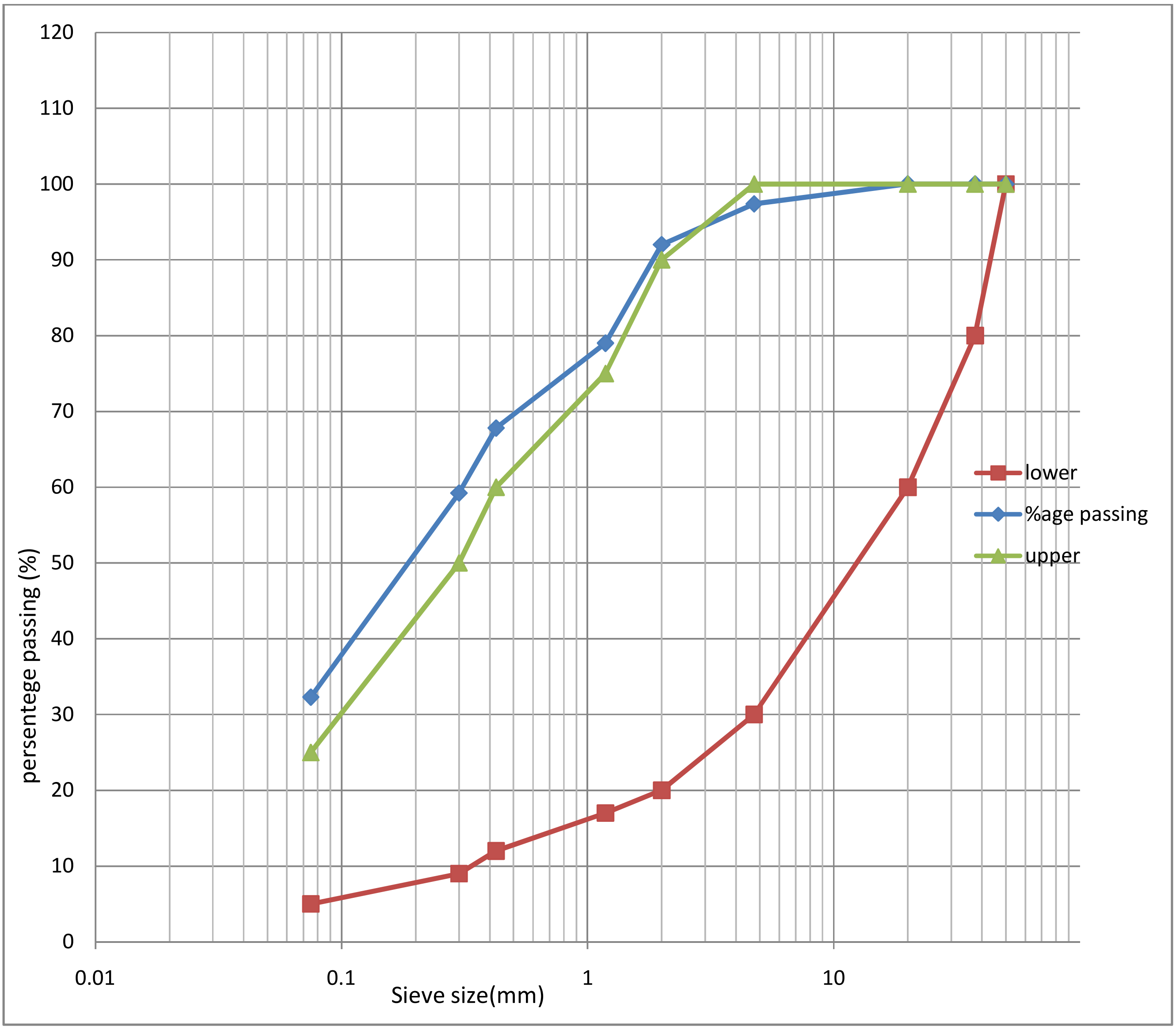


Fig 3.2 Typical Grain Size Analysis Graph.

Table 3.2 SOIL CONSISTENCY TEST RESULT

(TEST METHOD : AASHTO T89 , T90)

Project: Jimma town ECX-Mikael junction

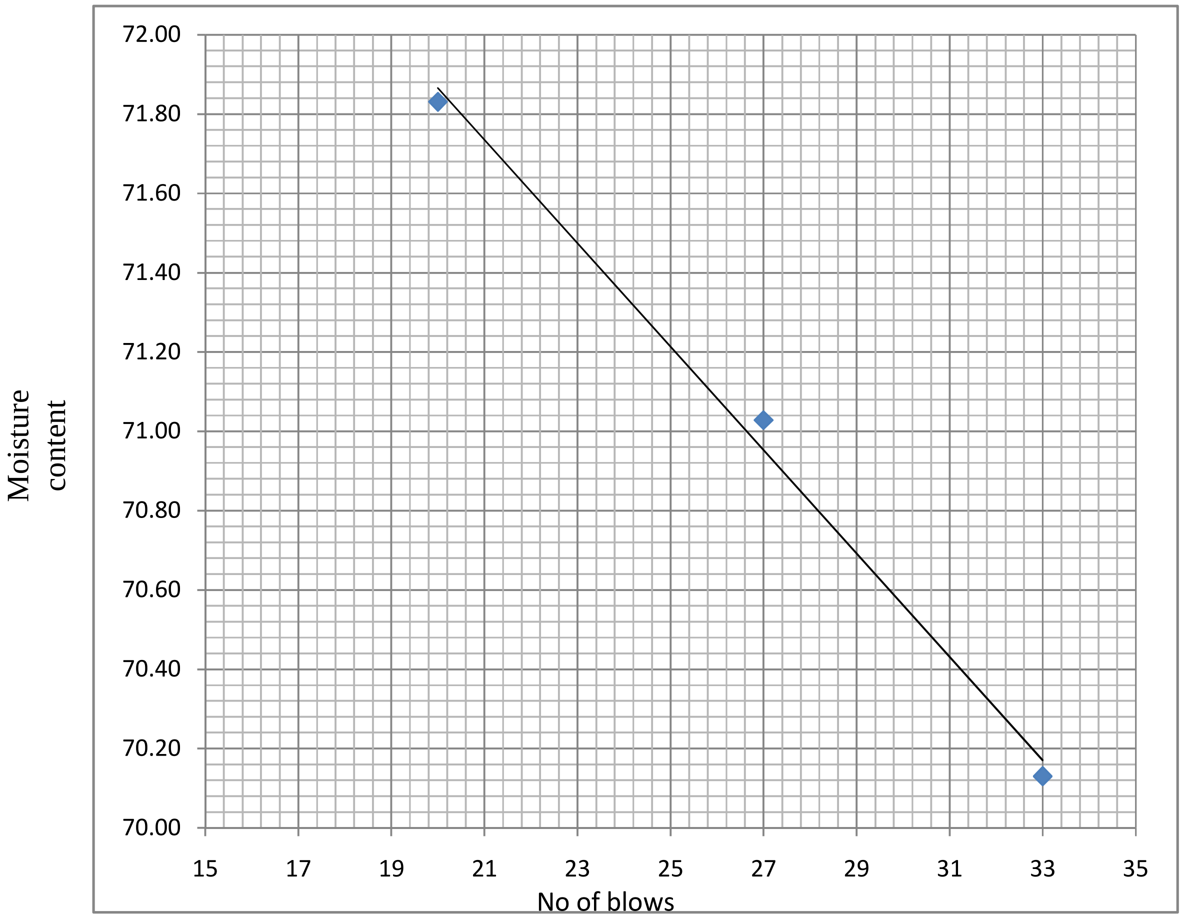
Sampling Station : 00+000 RHS

Material Type: Subgrade

PI Specification Limit for : ≤ 15 for Capping layer and Subgared , ≤ 20 for Upper Layer and ≤ 25 for lower layer of Ordinary Fill

	Liquid Limit			Plastic Limit	
No. of Blows	33	27	20		
Container Number	AV. liquid. Lim.	A1	F	X6	5
Wt. of Container + Wet Soil (g) = (W_1)	57.70	55.60	55.80	28.00	29.12
Wt. of Container + Dry Soil (g) = (W_2)	41.50	40.40	40.50	25.60	26.40
Wt. of Container (g) = (W_3)	18.40	19.00	19.20	18.80	19.00
Weight of Moisture (g) = (W_1 - W_2) = A	16.20	15.20	15.30	2.40	2.72
Weight of Dry Soil (g) = (W_2 - W_3) = B	23.10	21.40	21.30	6.80	7.40
Moisture Content (%) = (A / B)x 100	70.13	71.03	71.83	35.29	36.76
	AV. liquid. Lim.	71.00		AV. Plas. Lim.	36.03

RHS is right hand side and LHS is left hand side



LIQUIDLIMIT	LL	71.0
PLASTIC LIMIT	PL	36.0
PLASTICITY INDEX =	LL - PL	35

Figure 3.3: Typical Liquid Limit Graph (Flow Curve)

Table 3.3:Moisture Density Relationship Of Soil

DENSITY	SAMPLING STATION	0+000		SAMPLE TAKEN
	REPRESENT SECTION	ECX-Mikael junction		DATE SAMPLED
	VISUAL DESCRIPTION	Deep test		DATE TESTED
	PURPOSE :	*Sub grade*		SOURCE STATION
	TRIAL NUMBER	1	2	3
	WEIGHT OF SOIL + MOLD g	8881	9119	8862
	WEIGHT OF MOLD g	5107	5107	5107
	WEIGHT OF SOIL g	3,774	4,012	3,755
	VOLUME OF MOLD cc	2116	2116	2116
	WET DENSITY OF SOIL g/cc	1.78	1.90	1.77
MOISTURE	CONTAINER NUMBER	5	C5	6
	WET SOIL + CONTAINER g	427.7	416.7	399.5
	DRY SOIL + CONTAINER g	356.6	336.9	311.4
	WEIGHT OF WATER g	71.1	79.8	88.1
	WEIGHT OF CONTAINER g	52.2	51.60	52.40
	WEIGHT OF DRY SOIL g	304.4	285.3	259
	MOISTURE CONTENT %	23.36	27.97	34.02
DRY DENSITY OF SOIL g/cc		1.45	1.48	1.32

	MDD :	1.495	g/cc
	OMC :	26.80	%

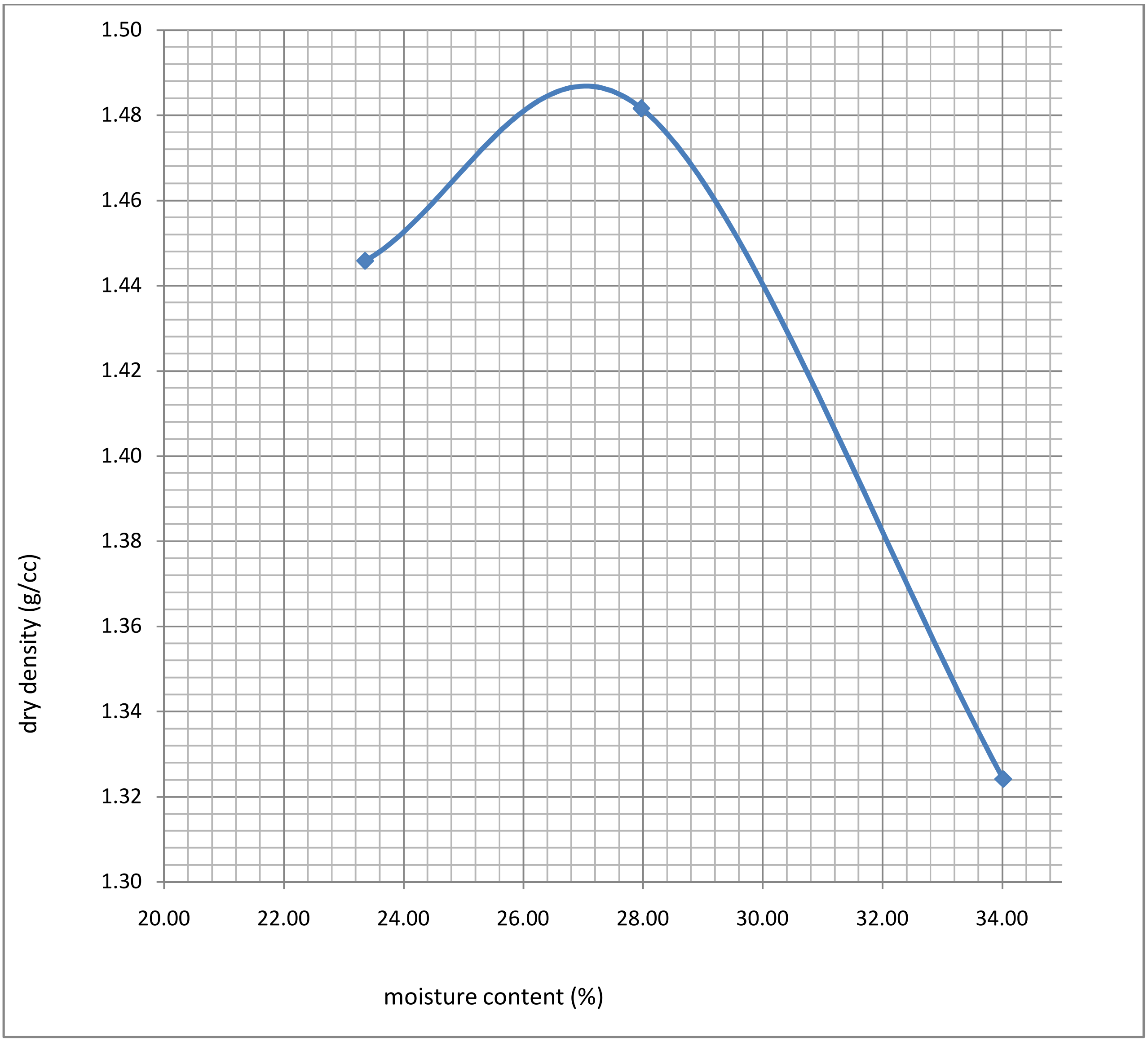
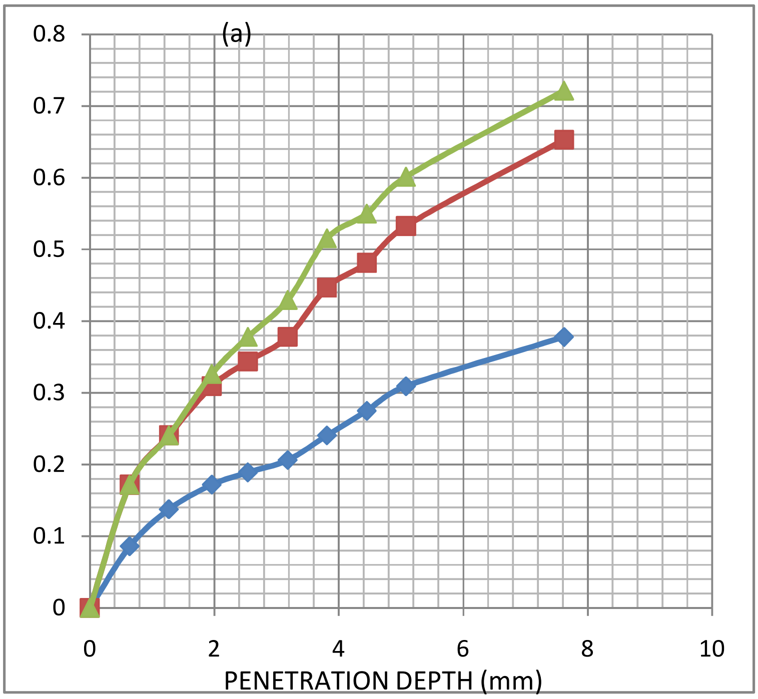
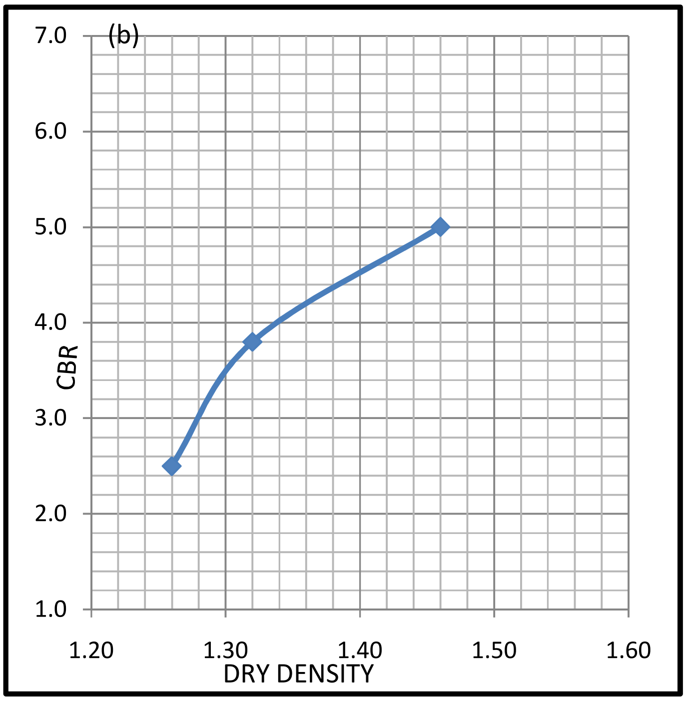


Fig 3.4: moisture content vs. Dry density Graph.

CBR value at standard loads and CBR Test summary.

project: ECX-Mikael junction Station 0+000 RHS orT1





(c)

Blow	**LOAD (KN)**		**CBR(%)**		**Swell %**
	2.54mm	5.08mm	2.54mm	5.08mm	
10	0.19	0.31	1.42	2.7	1.95
30	0.34	0.53	2.55	4.0	
65	0.38	0.60	2.83	5.2	

(d)

Blow	**Dry density**	**CBR%**
10	1.26	2.5
30	1.32	3.8
65	1.46	5.0
CBR at 95 %		**4.6**

Figure 3.5: Typical Penetration Load vs. Penetration Depth Graph

In the same manner of the above typical sample of soil test the following summary of table filled for each soil sample individually eight tests carried out.

3.2.2 Summary of Laboratory Test Results

Table 3.4: SUMMRY OF TEST RESULT FOR EXISTING SUB-RADE FROM KM 0+000 RHS

<table>
<tr><th>Sample No</th><th>Location</th><th>off set</th><th>material type</th><th colspan="3">Type Of Test</th><th>Test result</th><th>Specific limit</th><th>Remark</th></tr>
<tr><td rowspan="11">1</td><td rowspan="11">0+000</td><td rowspan="11">RHS</td><td rowspan="11">Existing Sub grade</td><td colspan="3">Group Classification</td><td rowspan="4">A-7-6 {3}</td><td rowspan="4">Embankment fill</td><td rowspan="11"></td></tr>
<tr><td colspan="3">% Passing sieve (mm)</td></tr>
<tr><td>2</td><td>0.425</td><td>0.075</td></tr>
<tr><td>92</td><td>67.8</td><td>32.3</td></tr>
<tr><td colspan="3">PL</td><td>36</td><td rowspan="7"></td></tr>
<tr><td colspan="3">LL</td><td>71</td></tr>
<tr><td colspan="3">PI</td><td>35</td></tr>
<tr><td colspan="3">Swell</td><td>1.95</td></tr>
<tr><td colspan="3">MDD</td><td>1.495</td></tr>
<tr><td colspan="3">OMC</td><td>26.8</td></tr>
<tr><td colspan="3">CBR at 95%</td><td>4.6</td></tr>
</table>

Table 3.5: SUMMRY OF TEST RESULT FOR EXISTING SUB-RADE FROM KM 0+100 LHS

Sample No	Location	off set	material type	Type Of Test			Test result	Specific limit	Remark
2	0+100	LHS	Existing Sub grade	Group Classification			A-7-6 {17}	Embankment fill	
				% Passing sieve (mm)					
				2	0.425	0.075			
				98	85.9	66.8			
				PL			36		
				LL			61		
				PI			25		
				Swell			4.07		
				MDD			1.535		
				OMC			22.6		
				CBR at 95%			2		

Table 3.6: SUMMRY OF TEST RESULT FOR EXISTING SUB-RADE FROM KM 0+200 RHS

Sample No	Location	off set	material type	Type Of Test			Test result	Specific limit	Remark
3	0+200	RHS	Existing Sub grade	Group Classification			A-7-6 {22}	Embankment fill	
				% Passing sieve (mm)					
				2	0.425	0.075			
				91	92	76.4			
				PL			36		
				LL			61		
				PI			25		
				Swell			4.96		
				MDD			1.461		
				OMC			26.5		
				CBR at 95%			0.1		

Table 3.7: SUMMRY OF TEST RESULT FOR EXISTING SUB-RADE FROM KM 0+300 LHS

Sample No	Location	off set	material type	Type Of Test			Test result	Specific limit	Remark
4	0+300	LHS	Existing Sub grade	Group Classification			A-7-6 {25}	Embankment fill	
				% Passing sieve (mm)					
				2	0.425	0.075			
				78	69.7	67.8			
				PL			43		
				LL			77		
				PI			34		
				Swell			7		
				MDD			1.56		
				OMC			20.8		
				CBR at 95%			0		

Table 3.8: SUMMRY OF TEST RESULT FOR EXISTING SUB-RADE FROM KM 0+400 RHS

Sample No	Location	off set	material type	Type Of Test			Test result	Specific limit	Remark
5	0+400	RHS	Existing Sub grade	Group Classification			A-7-6 {16}	Embankment fill	
				% Passing sieve (mm)					
				2	0.425	0.075			
				96	81.5	62.2			
				PL			41		
				LL			79		
				PI			38		
				Swell			3.87		
				MDD			1.385		
				OMC			32.3		
				CBR at 95%			7.8		

Table 3.9: SUMMRY OF TEST RESULT FOR EXISTING SUB-RADE FROM KM 0+500 LHS

Sample No	Location	off set	material type	Type Of Test			Test result	Specific limit	Remark
6	0+500	LHS	Existing Sub grade	Group Classification			A-7-6 {18}	Embankment fill	
				% Passing sieve (mm)					
				2	0.425	0.075			
				100	97.1	91			
				PL			38		
				LL			63		
				PI			25		
				Swell			4.84		
				MDD			1.45		
				OMC			28		
				CBR at 95%			5.9		

Table 10: SUMMRY OF TEST RESULT FOR EXISTING SUB-RADE FROM KM 0+600 LHS

Sample No	Location	off set	material type	Type Of Test			Test result	Specific limit	Remark
7	0+600	LHS	Existing Sub grade	Group Classification			A-7-6 {65}	Embankment fill	
				% Passing sieve (mm)					
				2	0.425	0.075			
				98	94	90.9			
				PL			36		
				LL			97		
				PI			61		
				Swell			0		
				MDD			1.568		
				OMC			13.9		
				CBR at 95%			0		

Table 3.11: SUMMRY OF TEST RESULT FOR EXISTING SUB-RADE FROM KM 0+700 RHS

Sample No	Location	off set	material type	Type Of Test			Test result	Specific limit	Remark
8	0+700	RHS	Existing Sub grade	Group Classification			A-7-6 {18}	Embankment fill	
				% Passing sieve (mm)					
				2	0.425	0.075			
				98	89.9	86.8			
				PL			23		
				LL			43		
				PI			20		
				Swell			4.67		
				MDD			1.706		
				OMC			14.7		
				CBR at 95%			2		

CHAPTER FOUR

4.1 Regression Analysis of CBR and Correlations with index properties

Regression analysis is a statistical technique that is very useful in the field of engineering and science in modeling and investigating relationships between two or more variables. The method of regression analysis is used to develop the line or curve which provides the best fit through a set of data points. This basic approach is applicable in situations ranging from single linear regression to more sophisticate nonlinear multiple regressions. The best fit model could be in the form of linear, parabolic or logarithmic trend. A linear relationship is usually practiced in solving different engineering problems because of its simplicity.

Fitting a regression model requires several assumptions. The method of least squares is used in order to choose the best fitting line for a set of data. Estimation of the model parameters requires the assumption that, the residuals (actual values less estimated values) corresponding to different observations are uncorrelated random variables with zero mean and constant variance (α^2) In most practical situation, the variance (α^2) of the random error (α^2) will be unknown and must be estimated from the sample data [17]. The standard error of an estimate gives some idea about the precision of an estimate. During modeling, a variable that shows the least standard error of estimates is the one to be chosen.

In the subject study, the California Bearing Ratio is taken as the dependent variable where as the percent passing 0.075mm sieve size, liquid limit, plastic limit, plasticity index, maximum dry density and optimum moisture content are independent variables.

4.2 Regression Analysis

In this research work, an attempt is made to apply single linear regression model and multiple linear regression models to characterize the strength of sub grade soil from soil index parameters using a statistical approach. The general representation of a probabilistic single and multiple linear regression models are presented in the following forms:

$Y = \beta 0 + \beta 1 x + \varepsilon$ ....(4.1)

$Y = \alpha 0 + \alpha 1 x1 + \alpha 2 x2 \ldots + \alpha n xn + \varepsilon$ .... (4.2)

Where, the slope ($\beta 1$) and intercept ($\beta 0$) of the single linear regression model are called regression coefficients. Similarly, coefficients $\alpha 0$, $\alpha 1$, $\alpha 2$ and αn are termed multiple regression coefficients. The appropriate way to generalize this to a probabilistic linear model is to assume that the actual value of Y is determined by the mean value function (the linear model) plus the random error term, ε [17]. The basic assumption to estimate the regression coefficients of the single and multiple regression models is based on the least square method.

Specific to this research, a statistical package for social science software (SPSS) is employed to investigate the significance of individual regressor variables. Accordingly, the eight laboratory test results of the independent and dependent variables are used in the following regression analysis. The statistical information's of the test results are presented in Table 4.1&4.2

Computational Analysis of Regression Analysis

Table 4.1 presents the summary of the computation of linear regression equations for different categories of test results while Table 4.2 presents the summary of the experimental and calculated values of the California Bearing Ratio, compaction parameters as a function of other index values.

Table 4.1. Linear regression equation for different categories of tests

Description	Linear regression
CBR (unsoaked) vs. Liquid limit	CBR = 0.031 LL + 83.19
CBR (unsoaked) vs. Plastic limit	CBR = 0.8 P.L + 65.31
CBR (unsoaked) vs. Specific gravity	CBR = 10.43 S.G + 56.19
CBR (unsoaked) vs. MDD	CBR = 8.66 MDD + 65.88
CBR (soaked) vs. Liquid limit	CBR = 0.22 L.L + 28.87
CBR (soaked) vs. Plastic limit	CBR = 1.04 P.L + 13.56
CBR (soaked) vs. Specific gravity	CBR = 9.42 S.G + 10.91
CBR (soaked) vs. MDD	CBR = 50.28 MDD – 70.22

*CBR = -21.734 - 0.003*LL - 0.137*PI + 20.244*MDD*

is the representative formula of all to calculate average CBR values.

Table 4.2 Summary table for experimental and calculated values

sample no.	1	2	3	4	5	6	7	8
PL	36	36	36	43	41	38	36	23
LL	71	61	61	77	79	63	97	43
PI	35	25	25	34	38	25	61	20
Swell	1.95	4.07	4.96	7	3.87	4.84	0	2
MDD	1.495	1.535	1.461	1.56	1.385	1.45	1.568	1.568
OMC	26.8	22.6	26.5	20.8	32.3	28	13.9	13.9
CBR at 95%	4.6	2	0.1	0	7.8	5.9	0	0

According to specification of ERA manual (2002), the quality of materials to be fulfilled to be used as sub grade material for road construction:

- CBR value shall be greater than 5 at 95% of AASHTO (T180)
- The plasticity index shall be not exceeding 20%
- The maximum swell value of 1.5%

According to the test result: even though, T1,T5 and T6 seems satisfying CBR but does not satisfy PI value and/or Swell limit when compared with ERA specification manual with stabilization or replacement of material must done.

CHAPTER-FIVE

Conclusion and Recommendation

5.1 Conclusions

The case study was conducted to find Suitability of localized existing soil material to use as sub grade layer Accordingly, the required laboratory tests were conducted on samples retrieved from different from different stations of ECX -Mikael junction of Jimma town. Using the obtained eight test results a single linear regressions were analyzed to find CBR value ,LL, PL, PI, MDD and OMC.

The suitability of the existed soil material is evaluated by utilizing a separate control test results. From the results of this study the following conclusions are drawn:

- For preliminary design purpose the above estimation might be used to estimate suitability of sub grade material, without a detailed laboratory test at Jimma town.
- As test result showed us the soil is erratic even every 100m , this result indicates may did represent every Jimma town.
- CBR, is the most basic to decide suitability of soil as sub grade material even though soil index is may not satisfy specification.
- According to ERA manual of specification of suitable sub grade materials T1,T5 and T6 are good . as a result may have quarter of the stretch may not want to be stabilized.
- Unsuitable sub-grade material, those CBR values at 95% of MDD is less than 5%,maximum plasticity index of 30%, Maximum CBR swell values of 2%
- Non plastic materials, whose plasticity index is 0, non swell and CBR values less(more) than the minimum required 5%

5.2 Recommendations

- For unsuitable sub-grade soils, whose CBR values at 95% of MDD is below 5%,maximum CBR Swell values of 2% and maximum plasticity index of 30%, it is recommended to excavate down up to 600 mm depth and replace it with a plastic no expansive soil having a minimum CBR of 5%, 2% swell and PI between maximum 20% which is equivalent to an S3 type material indicated as per ERA, 2002. The work shall be executed in three layers of equal thickness and must be compacted to the required minimum density.
- In addition, the road bed of the expansive clays should be kept moist during road bed preparation and should be covered by the appropriate fill/improved sub-grade without undue delays; culverts and drainage pipes shall not be directly laid on expansive soils; trees should not be planted and allowed to grow near the road.

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