JIMMA UNIVERSITY

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SCHOOL OF GRADUATE STUDIES

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

HIGHWAY ENGINEERING STREAM

Suitability of Waste Paper Ash Blended with Lime to Improve the Strength of Expansive Subgrade Soil

Thesis Submitted to the School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Highway Engineering

BY: LEBETA BAYEN CHUKO

April 2021
Jimma, Ethiopia

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SUITABILITY OF WASTE PAPER ASH BLENDED WITH LIME TO IMPROVE THE STRENGTH OF EXPANSIVE SUB-GRADE SOIL

By:

Lebeta Bayen

APPROVED BY BOARD OF EXAMINERS

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DECLARATION

I, the undersigned, declare that this thesis proposal entitled by: **"Suitability of Waste Paper Ash Blended with Lime to Improve the Strength of Expansive Sub-grade Soil"** is my original work and has not been presented by any other person for an award of a degree in this or any other University, and all sources of material used for this thesis proposal have to be duly acknowledged.

Candidate:

Mr. Lebeta Bayen Chuko ___________________ ___/ ___ /___

Signature Date

As Master's Research Advisors, we hereby certify that we have read and evaluated this MSc thesis research prepared under our guidance by **Mr. Lebeta Bayen Chuko** entitled **"Suitability of Waste Paper Ash Blended with Lime to Improve the Strength of Expansive Sub-grade Soil."**

We recommend that it can be submitted as fulfilling the MSc Thesis requirement.

Dr.Eng. Fekadu Fufa (PhD) _________________ ____/ _____/_____

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ABSTRACT

Expansive soils have the tendency to behave volume change during wetting and drying conditions which causes significant damage to structures such as buildings and pavements. This behavior of the soil cause large uplift pressure, differential settlement, and upheaval of structures built on them. Avoiding these types of unsuitable soils is mostly impractical. Hence, the aim of the study is to investigate the suitability of waste paper ash blended with lime to improve the strength of expansive sub-grade soil to be used as sub-grade materials.

The research design followed the experimental type of study which began by collecting samples. In this study moisture content, Atterberg limits, particle size distribution, soil classification, free swell index, linear shrinkage, specific gravity, compaction (moisture-density relationship) tests, CBR and CBR swell were determined. The sampling technique used for this research was a purposive sampling which is non– probability method. Two expansive soil samples were taken for the study by observation and free swell index tests at a depth of 1.5 m-2 m to remove organic matters. The data processing and analyzing were conducted by using both descriptive and analytical methods.

Laboratory test result of waste paper ash shows that specific gravity, liquid limit, plastic limit, plastic index and optimum moisture content were 1.67, 50.8%, 43.7%, 7.1% and 50.1%, respectively. The laboratory test results fulfilled the requirements of class-C fly ash according to ASTM C-618. JIT soil sample has plastic index 39 %, free swell index 88.12 %, linear shrinkage 17.73% and CBR value 2.33%. In addition, the MAR soil sample has plastic index 41.8 %, free swell index 96.83%, linear shrinkage 20.07% and CBR value 1.95%. Since both soil samples were found with high degree of expansion, stabilization was made by mix-ratio of (0, 2%HL+12%WPA, 4%HL+10%WPA, 6%HL+8%WPA, 8%HL+6WPA).

As the content of hydrated lime increases and WPA decreases in WPA: HL mix-ratios, LL, PI, MDD, FSI, CBR swell decreased whereas PL, OMC, CBR are increased. All the laboratory test results were compared with standard specifications. It was recommended to investigate the effect of additional curing time and aging effect of soil stabilized by WPA. Additional parameter like unconfined compressive strength, PH value test, volumetric shrinkage and mineralogical tests should also be performed to have more realistic test results.

Keywords: Waste paper ash, lime, soil stabilization, expansive soil.

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Acronyms

AASHTO	American Association Of Highway And Transportation Officials
ASTM	American Society For Testing And Materials
CBR	California Bearing Ratio
CAH	Calcium Aluminate Hydrate
CSA	Central Statics Agencies
CSH	Calcium Silicate Hydrate
ERA	Ethiopian Roads Authority
ES	Expansive Soil
FSI	Free Swell Index
IS	Indian Standard
JIT	Jimma Institute of Technology
Gs	Specific Gravity
HL	Hydrated Lime
LL	Liquid Limit
LS	Linear Shrinkage
MAR	Merkato-Airport Road
MC	Moisture Content
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PDM	Pavement Design Manual
PI	Plastic Index
PL	Plastic Limit
SP	Swelling Pressure
T1	Trial one
T2	Trial two
T3	Trial three
USCS	Unified Soil Classification System
USA	United States Of America
WPA	Waste Paper Ash

CHAPTER ONE

INTRODUCTION

1.1. Background

Worldwide the availability of natural construction materials within reasonable hauling distance is one of the major factors that have a direct impact on the investment cost of road projects. In areas where natural construction materials are readily available, roads can be constructed on Sound economic basis. However in some regions, natural construction materials are either not available or do not fulfill the quality requirements of road construction materials. Problems associated with these construction materials have been reported in Africa, Australia, Europe, India, and South America, the United States as well as some regions in Canada [1]. In the United States alone, expansive clays have been estimated to produce at least two billion dollars of damage annually. In many areas of the tropics especially Africa and India, tropical expansive soils often known as black cotton soils are the major problematic soils. These soils show very strong swelling and shrinkage characteristics under changing moisture conditions [1].

In addition, expansive clay soil causes major problems in the design, construction, and maintenance of pavements. It is approximated that about 40% of the country of Ethiopia is covered with expansive clay soil interrupting economic development and causing construction challenges [2]. To solve this problem, stabilization should implement with different stabilizing additives to achieve the required specification of sub-soil materials.

Soil stabilization is the alteration of one or more soil properties, by mechanical or chemical means, to create an improved soil material possessing the desired engineering properties. The process may include the blending of soils to achieve a desired gradation or mixing of commercially available additives that may alter the gradation, texture or plasticity, or act as a binder for cementation of the soil [3]. The replacement of soil stabilizer by various wastes creates a tremendous saving of energy, costs and also leads to important environmental benefit. As a result, waste materials like; ceramic waste, paper pulp, ground granulated blast furnace slag, silica fume, hypo sludge, fly ash, paper sludge, waste glass powder, waste paper ash [4]. More than 450 million tons of paper is produced worldwide per annual and it is expected that the demand for paper will reach 500 million tons per annual by the end of 2020 [4]. And also the study predicted that the demand of paper to be imported to Ethiopia by the year 2015/16 was 157,956.7 tons of paper and paper board. This demand of paper was increasing from year to year because of education expansion policies and overall economic development of the country [5]. Paper and cardboard are reported as the second largest component of domestic waste next to organic waste which contributes about 13% of the total domestic solid waste [6].

Disposable paper available in abundance throughout the world is composed mainly of short, natural, cellulose fibers and is already used in many local raw materials. Waste paper arises from several sources such as newspapers, office and printing papers etc. The chemical composition of paper ash mainly contains SiO_2 (60%), CaO (14%), Al_2O_3 (2.06%), and Fe_2O_3 (0.92%) which are the main essential compounds needed for cement hydration for strength development [7].

Due to this pozollanic property, waste paper ash has a possibility to be used as partial replacement of cement and lime. Therefore, the aim of this study is to investigate the properties of expansive sub-grade soil by using waste paper ash blended with lime in order to ascertain their suitability to be used as weak sub-grade soil stabilizing agent.

1.2. Statement of the Problem

The fact that expansive soils are major engineering problem makes their study an important aspect due to their tendency to swell in presence of moisture and shrink in moisture absence and the accruing cost involved in terms of economic loss when construction is undertaken without giving consideration to the probability of their presence.

The aerial coverage of expansive soils in Ethiopia is estimated to be 10 million hectare [43]. Currently, different construction activities are taking place in the road and building sector on these soil types. It has been noticed that construction on expansive soil face numerous problems and the causes of the problems are not investigated in depth in Ethiopia. Most of the roads constructed in Ethiopia on this type of soil fail before their expected design life, in some cases after a few months of completion [44].

When unsuitable materials are encountered measures like avoiding the route, redesigning the pavement with thicker sections, or replacing the poor soil with good quality materials are practical but it is an uneconomical option and particular problems associated with road construction over expansive soils are the seasonal volumetric change, low-bearing strength, shrinkage and crack and swell, and expansions [35].

To reduce the impact of expansive soils, improvement of their engineering properties is required. Chemical stabilizers and natural material are commonly used to improve the performance of soils with high plasticity, poor workability, and low strength and stiffness. To achieve effective soil stabilization, special attention needs to be given to proper type and concentration of the stabilizer. Besides, the effectiveness and efficiency of the stabilizer in terms of strength and durability improvement should be stated and specified [39].

In the state of current practice, even though American Association of State Highway and Transport Officials (AASHTO) and ASTM are excellent starting points for the selection of the stabilizers, they only have official sets of target strengths for lime, cement and lime-fly ash specimens. This study aimed to investigate the suitability of waste paper ash blended with lime to improve the strength of expansive sub-grade soil.

1.3. Significance of the Study

The significance of the research is to use WPA as partial replacement of expensive stabilizer for the stabilization of expansive sub grade soil. So it is beneficial to use abundantly available and environmentally friendly resources of Agro-industrial waste instead of expensive stabilizer to treat the problem due to expansiveness. Natural pozzolanas are the most alternative to replace part of lime and environmental friendly and cost effective than traditional stabilizers such as lime, cement, and manufactured fly ashes.

On the other hand, the extraction of substantial amount of non-renewable natural resources for construction projects creates significant damaging impacts on the local environment and its inhabitants. Therefore, construction techniques implemented to solve the socio-economic problems need to found not only time and cost effective but also environmentally friendly.

1.4. Research questions

1. What are the engineering properties of existing expansive soil in Jimma town?
2. What are the engineering properties of expansive sub-grade soil stabilized with different ratios of waste paper ash and lime?
3. What are the performance of waste paper ash and lime as stabilizing agent on the properties of expansive soil?
4. What amount of the stabilizing agent does the substandard soil can be used as road sub-grade material based on ERA specification?

1.5. Objective

1.5.1. General Objective

The aim of the study is to investigate the suitability of waste paper ash blended with lime to improve the strength of expansive sub-grade soil.

1.5.2. Specific Objectives

The specific objectives of the study are:

1. to identify the engineering properties of existing expansive sub-grade soil.
2. to investigate the engineering properties of expansive sub-grade soil stabilized with different ratios of waste paper ash and lime.
3. to evaluate the performance of waste paper ash and lime as stabilizing agent for expansive sub-grade soil.
4. to recommend on the appropriate percentage by weight of waste paper ash and lime, required to improve weak sub grade soils to meet the specification

1.6. Scope and Limitations of the Study

This study was supported by different types of literatures and a series of laboratory experiments. However, the findings of the research were limited to the two weak soil samples to conduct this study. The results are also specific to the percent of additives used in the experimental work. This study covered the stabilization of materials, which is not appropriate for road sub-grade construction with naturally occurring expansive soil materials through using mechanical, physical and chemical stabilization of different mix proportion of samples by conducting laboratory tests. The relevant laboratory tests conducted were grain size analysis, Atterberg limit, compaction, CBR, CBR swell, linear shrinkage and free swell test. Then the study compared the results with ERA, AASHTO and ASTM specifications likewise a recommendation was drawn and forwarded.

CHAPTER TWO

LITERATURE REVIEW

2.1. Introduction

Expansive soils are fine grained soil or decomposed rocks that show huge volume change when exposed to the fluctuations of moisture content. Swelling-shrinkage behaviour is likely to take place near ground surface where it is directly subjected to seasonal and environmental variations. The expansive soils are most likely to be unsaturated and have montmorillonite clay minerals. Most of severe damage in relation to expansive soil is depended on the amount of monovalent cations absorbed to the clay minerals. Construction of residential buildings and other civil engineering structures such as highways, bridges, airports, seaports on expansive soil is highly risky in that such soil is susceptible to cycles of drying and wetting, inducing shrinkage and swelling property under building foundations, which results in cracking to structural and none structural elements of those structures [8].

Expansive soils have a complicated behavior and are generally characterized by detrimental volume changes when subjected to moisture fluctuations. At dry state, the expansive soils are very difficult to compact since their consistency varies from hard to very hard. At wet state, they are very sticking [9]. Expansive soils expand when they get wet and cracks when they dry. The swell and volumetric change of expansive soils has increased the interest of Engineers to further study in this area. Expansive clays are different in that near to surface; clay often varies in density and moisture conditions from the wet season to the dry season. For example, near-or at-surface clays often dry out during periods of drought but then expand during the rainy season [10].

2.2. Sub-grade Soil

Sub-grade layer is the lowest layer in the pavement structure underlying the base course or surface course, depending upon the type of pavement. Generally, sub-grade consists of various locally available soil materials that sometimes might be soft and/or wet that cannot have enough strength/stiffness to support pavement loading [11]. A sound knowledge of performance of the sub-grade soil under prevailing in-situ condition is necessary prior to the construction of the pavement [12]. The better the strength/stiffness quality of the materials the better would be the long-term performance of the pavement. Hence, the design of pavement should be focused on the efficient, most economical and effective use of existing sub-grade materials to optimize their performance. In case of soft and wet sub-grades, proper treatment might be needed in order to make the sub-grade workable for overlying layers for pavement construction [13].

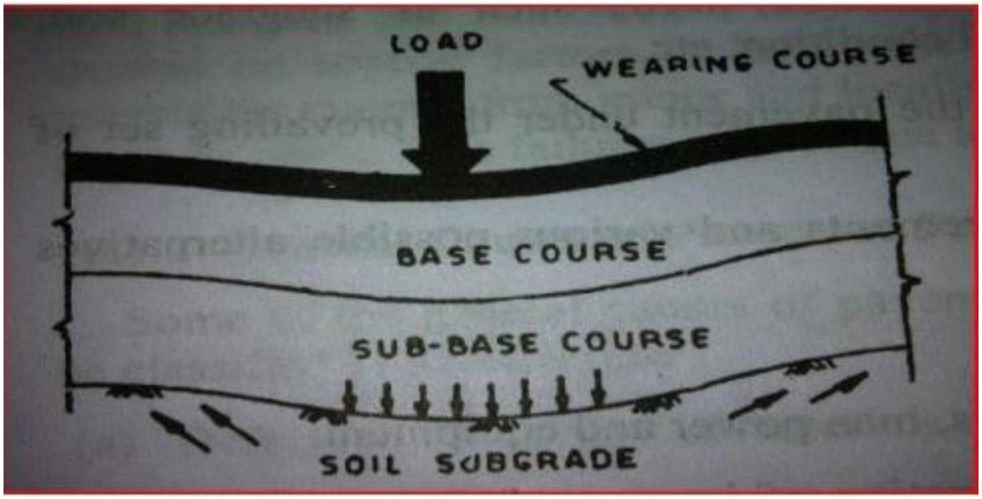

Figure 2.1: Sub-grade failures [13]

2.3. Desirable Properties of Sub-grade Soil

Sub-grade soil as highway material should be; stable, incompressible, permanent strength, minimum changes in volume due to climate, superior drainage and ease of compaction.

Table 2.1: Sub-grade strength class [50]

Sub-grade class	CBR range (%)
S1	< 3
S2	3, 4
S3	5, 6, 7
S4	8 – 14
S5	15 – 30
S6	> 30

2.4 .Soil Classification System

Soil classification is the arrangement of soil in to groups which have similar behavior [14].The main objective of any soil classification system is predicting the engineering properties and behavior of a soil based on a few simple laboratory or field tests. Based on the laboratory or field test results, identify the soil and categorized into groups with similar engineering characteristics. Although there are many classification systems like particle size, textural, AASHTO and USCS classification, the last two classification systems are more common [14].

2.4.1. AASHTO Classification System

According to AASHTO, Particle size analysis and plasticity characteristics are required to classify soil for both coarse-grained and fine-grained from A-1 to A-7 of soil classification. This classification system requires particle size analysis, plasticity index and liquid limit to group the soil with similar engineering characteristics [14].

Table 2.2: AASHTO soil classification system [31]

Soil group		Grain size % passing			LL (%)	PI (%)	Mineral type	Sub-grade range
		#10 sieve	#40 sieve	#200 sieve				
A -1	A – 1 - a	≤ 50	≤ 30	≤ 15		≤ 6	Stone, gravel sand	Excellent to good
	A – 1 - b		≤ 50	≤ 25		≤ 6		
A - 3			≥ 51	≤ 10		Non plastic	Fine sand	
A - 2	A – 2 - 4			≤ 35	≤40	≤ 10	Silty sand Clayey gravel & sand	
	A – 2 - 5			≤ 35	≥41	≤ 10		
	A – 2 - 6			≤ 35	≤40	≥ 11		
	A – 2 - 7			≤ 35	≥41	≥ 11		
A - 4				≥ 36	≤40	≤ 10	Silty soil	
A - 5				≥ 36	≥41	≤ 10	Silty soil	Fair to poor
A - 6				≥ 36	≤40	≥ 11	Clayey soil	Fair to poor
A - 7	A – 7 - 5			≥ 36	≥41	≥ 11 PI ≤ LL - 30	Clayey soil	Fair to poor
	A - 7 - 6			≥ 36	≥41	≥ 11 PI > LL - 30	Clayey soil	Fair to poor

2.4.2. USCS Classification System

Unified soil classification system (USCS) was first developed by casagrande in 1948 and modified by Bureau of reclamation and crop engineers of USA [16]. It is also accepted by American Society of Testing Materials (ASTM) and most popular classification system for all types engineering problems involving soil. This method is used to categorize the soil with similar engineering properties, including strength, permeability and compressibility which specify soil types to achieve a desired performance [16]. According to USCS, soils are classified as coarse grained or fine grained soil. The soil is classified as coarse-grained when soil sample retained on sieve #200 (0.075 mm) more than 50%. Coarse-grained soils are further classified as, gravels if 50 percent or more of the coarse fraction is retained on #4(4.75 mm) sieve and sands if 50 percent or more of the coarse fraction passes through #4 (4.75 mm) sieve. The soil is classified as fine-grained if 50 percent or more of the sample passes #200 (0.075 mm) sieve. Fine-grained soils are further classified according to whether their liquid limit is less than or greater than 50% percent.

Table 2.3: Unified Soil Classification System (ASTM D2487)

Major Group	Sub-group	Symbol	Description
Coarse-grained Soil (> 50% retained #200 sieve)	Gravels (> 50% coarse fraction retained on #4 sieve)	GW	Well-graded gravels and gravel-sand mixtures (little or no fines)
		GP	Poorly-graded gravels and gravel-sand mixtures (little or no fines)
		GM	Silty gravels (gravel-sand-silt mixtures)
		GC	Clayey gravels (gravel-sand-clay mixtures)
	Sands (≥ 50% coarse fraction pass #4 sieve)	SW	Well-graded sands and gravelly-sands mixtures ((little or no fines)
		SP	Poorly-graded sands and gravelly-sands mixtures (little or no fines)
		SM	Silty sands (sand-clay mixture)
		SC	Clayey sands (sand-clay mixture)
Fine-grained soil (≥ 50% pass through #200 sieve)	Silts & and Clays (with Liquid limit < 50)	ML	Inorganic silt (very fine sands, silty or clayey sands)
		CL	Inorganic clay of low-medium plasticity
		OL	Organic silts and silty-clay of low plasticity
	Silts and Clays (with Liquid limit ≥ 50%)	MH	Inorganic silts, elastic silts
		CH	Inorganic clay of high plasticity
		OH	Organic clay of medium-to high plasticity
		PT	Peat muck and other organic soil

2.5. Expansive Sub-grade Soil

Clay material is a natural, earthy, fine grained material which when mixed with a limited amount of water develops plastic properties and composition of crystalline minerals consisting of essentially hydrous aluminum silicates [18]. Based on the Atterberg limits value and gradation test; clay soil can be described qualitatively as having low, medium, high, or very high expansive potential. In general, these soils classified as CL or CH as per USCS as well as A-6, or A-7 as per AASHTO classification systems may be considered as expansive soil [19]. Expansive clay soils are problematic soils because of their inherent potential to undergo volume changes corresponding to changes in the moisture variation [20].

2.6. Origin of Expansive Sub-grade Soil

The parent materials that can be associated with expansive soil classified into two groups. First group comprises the basic igneous rocks such as basalts, dolerite sills, dykes and gabbro. In these soils, the feldspar and pyroxene minerals of the parent rocks have decomposed to form montmorillonite and other secondary minerals. The second group comprises the sedimentary rocks that contain montmorillonite as a constituent which breaks down physically to form expansive soils [21].

2.7. Composition of Clay Minerals

Clay soil composed of extremely small crystalline particles of one or more members of a small group of minerals. These minerals are essentially hydrous aluminum silicates, with magnesium or iron replacing wholly or partially for the aluminum. Many clay materials may contain organic material and water-soluble salts.

Table 2.4: Clay minerals [22]

Name of mineral			Structural formula
1	Kaolin group	Kaolinite	$Al_4Si_4O_{10}(OH)_8$
		Halloysite	$Al4Si_4O_6(OH)_{16}$
2	Montmorillonite group	Montmorillonite	$Al_4Si_8O_{20}(OH)\ n\ H_2O$
3	Illite group	Illite	$K_y(AlFe_2.Mg_4.Mg_6)Si_8\text{-}yAl_y(OH)O_{20}$

The three most important groups of clay minerals are montmorillonite, Illite, and kaolinite, crystalline hydrous alumina-silicates with the help of x-ray technology minerals are identified [22].

2.8. Impact of Expansive Soil

The six major world natural hazards are earthquakes, landslides, expansive soils, hurricane and flood. Among these America's most destructive natural hazards, expansive soils problem has the second place next to hurricane wind problem in terms of dollar losses to buildings. According to the study, it was projected that by the year 2000, losses due to expansive soil would exceed 4.5 billion dollars annually [23]. Ethiopia is one of the country that loss millions of budget for progressive road maintenance due to expansive soil problem.

2.9. Distribution of Expansive Soil

Expansive soils are wide spread in African continent, occurring in South Africa, Ethiopia, Kenya, Mozambique, Morocco, Ghana, Nigeria etc. In other parts of the world case of expansive soils have been widely reported in countries like USA, Australia, Canada, India, Spain, Israel, Turkey, Argentina, Venezuela etc [24]. In addition, the aerial coverage of expansive soils in Ethiopia estimated to be 24.7 million hectares (Lyon associates, 1971;as cited by Nebro,2002).They are widely spread in central part of Ethiopia following the major truck roads like Addis-Ambo, Addis-Wolliso, Addis-Debrebirhan, Addis-Gohatsion, and Addis-Modjo are covered by expansive soils. In addition, areas like Mekele and Gambella are covered by expansive soil. The distributions are shown in Figure 2.1 [25, 26].

Figure 2.2: Distribution of expansive soil in Ethiopia [25, 26]

2.10. Nature of Expansive Soils

Soil materials which have high clay content are mostly responsible for expansiveness behavior. This material becomes swell when the moisture through it increase and it becomes shrinks greatly on drying and develop cracks on the surface. These soils possess a high plasticity index and their color varies from dark grey to black [27]. Expansive soils absorb water heavily, swell, become soft and lose strength. These soils are easily compressible when wet and possesses a tendency to heave during wet condition and shrink in volume and develop cracks during dry seasons of the year. These soils characterized by extreme hardness and cracks when dry. It is also described expansiveness of soils in relation to their FSI. Soils called highly expansive when the FSI exceeds 50% and such soils undergo volumetric changes leading to pavement distortion, cracking and general unevenness due to seasonal wetting and drying [28].

2.11. Identification Methods of expansive soils

Most of the national codes of practice do not give characterization and classification of expansive soils. A simple user-friendly approach based on the free swell ratio, defined as the ratio of the sediment volume of soil in distilled water to that in carbon tetra-chloride or kerosene, is formulated considering the compatibility of the results with Odometer free swell tests and the soil clay mineralogy. Statistical illustrations are provided which clearly indicate the assessment of soil expensiveness based on index properties is an overestimation. There is a consistency in the classifications based on odometer test results and the proposed approach [29].

Expansive soils can be recognized by using mineralogical identification, indirect index property tests or direct expansion potential tests. Expansiveness of a soil is governed by the type and proportion of clay minerals it contains. Knowing the type and proportion of the clay mineral in a soil gives an indication on the swelling potential [30].

2.11.1. Mineralogical Methods

Type of clay mineral is a fundamental factor, which determines the expansive behavior of a soil. Mineralogical test is used to identify this mineral. There are different types of techniques, which are used to identify the clay mineralogy. The common types of these techniques include; x-ray diffraction, differential thermal analysis, dye adsorption, chemical analysis and electron microscope [31]. But these methods are not suitable for routine tests because of time consuming, require expensive test equipment and results can only interpret by special trained technicians.

2.11. 2.Direct Method

These methods are the most useful data for Engineers to practice. These methods offer the most useful data by direct measurement; and tests are simple to perform and do not require complicated equipment. Testing should be performed on a number of samples to avoid erroneous conclusions. The direct measurements are the most satisfactory and convenient methods to determine the swelling potential and swelling pressure of expansive clay [29].

2.11.3. Indirect Methods

This method has used to investigate the swelling potential of a soil by examining other parameters, which indirectly give information about the soil property. These include index property tests, cation exchange capacity, and potential volume change test. The liquid limit and plasticity index are useful for determining the swelling characteristics of most of the clays and prepared a chart to support the identification [32].

The classification or rating from low potential to high heaving potential usually depends on the clay content and plasticity [32].

2.12. Soil Stabilization

Soil stabilization is a process whereby increased strength and stability of the soil is attained mainly by mechanical or chemical means. The most common improvements attained through stabilization include better soil gradation, reduction of plasticity index or swelling potential, increase in durability and strength [34].

When unsuitable materials are encountered measures like avoiding the route, redesigning the pavement with thicker sections or replacing the poor soil with good quality materials are practical but increasingly expensive options. With improved technological advances and concern for reduction of non-renewable resources, improving the properties of soil using chemical additives is gaining increased popularity [35].

2.13. Advantages of Soil Stabilization

Individual project conditions dictate different reasons for treatment. These reasons have great impact on the type and percentage of additive required. Common reasons for the need of stabilization are; provide a working platform for construction, reduce shrink/swell of expansive soils, increase strength to provide long-term support for the pavement structure, reduce pavement thickness and improve durability, reduce moisture susceptibility and improves soil workability, utilize local materials and for the reduction of cost [36].

2.14. Methods of Soil Stabilization

2.14.1. Mechanical Stabilization

Mechanical stabilization can be defined as a process of improving the stability and shear strength characteristics of the soil without altering the chemical properties of the soil [37]. It is common to use both mechanical and chemical means to accomplish specified stabilization. The main methods of mechanical stabilization can be categorized into compaction, mixing or blending of two or more gradations, applying geo-reinforcement and mechanical remediation [35]. Mechanical stabilization is best suited for coarse-grained soils or aggregates at optimum or below optimum moisture contents. However, clayey soils are more effective under chemical stabilization. If the clayey soil is mixed with the specific stabilizer just enough to make it workable, better in texture and compactable regardless the strength and durability, then it is referred to as modification ; modification is restricted to the soil having AASTHO designation A - 4, A - 5, A - 6 and A - 7 [38].

2.14.2. Chemical Stabilization

Chemical stabilization is a method of improving the engineering properties of a material by adding chemical substances. Chemical stabilization is used for a wide range of purposes including: improving the bearing capacity and strength of pavement layers, dry temporary by passes during rainy periods, delay certain chemical reactions that are harmful to road soils or aggregates, dry out soil where the moisture content is too high for successful compaction, make soil less permeable where necessary, reduce the plasticity of soils used in road construction and thereby reducing the effect of moisture variations, changing clay to a more granular and workable material and reducing swelling and shrinkage properties [39]. Chemical stabilizers are classified into three groups; traditional stabilizers such as hydrated lime, portland cement and fly ash. Non-traditional stabilizers comprised of sulfonated oils, ammonium chloride, enzymes, polymers, potassium compounds and by-product stabilizers which include cement kiln dust, lime kiln dust [40].

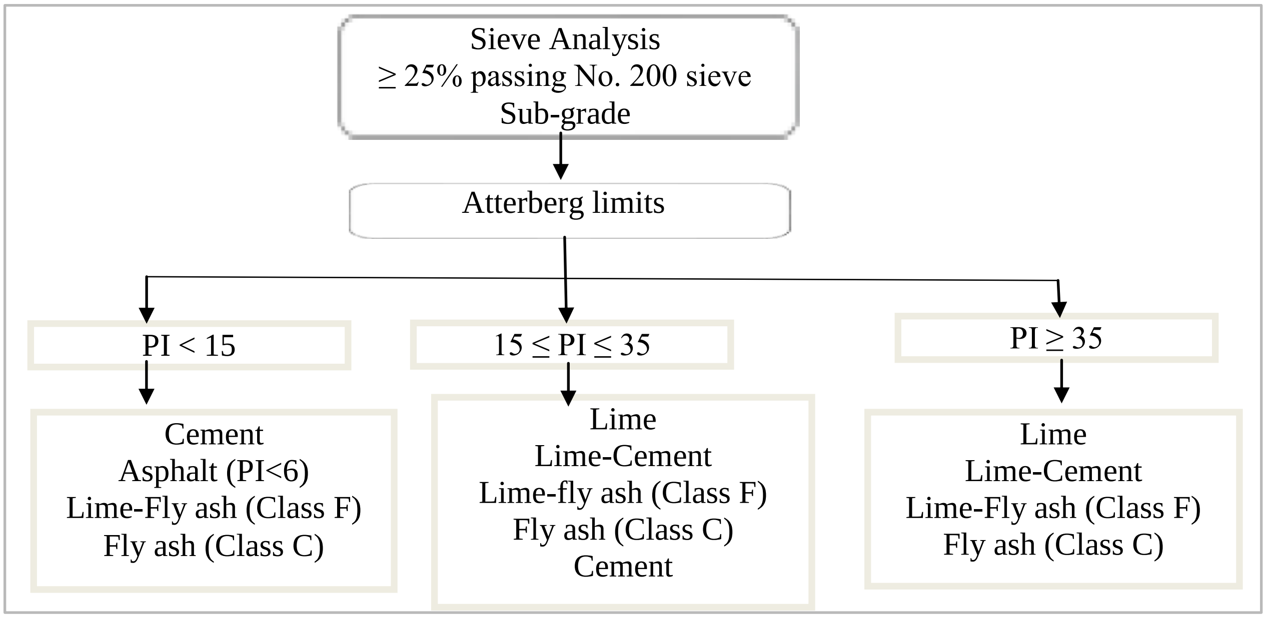

Figure 2.3: Decision tree for selecting stabilizers for sub-grade soils [41].

Soil improvement by means of chemical stabilization can be grouped into two chemical reactions; cation exchange and pozzolanic reactions.

2.14.2.1. Cation Exchange Chemical Reaction

Clay minerals have the property of absorbing certain anions and cations and retaining them in an exchangeable state. The exchangeable ions are held around the outside of the silica – alumina clay – mineral structural unit and the exchange reaction doesn't affect the structure of the silica – alumina pocket. In clay minerals, the most common exchangeable cations are Ca^{2+}, Mg^{2+} ,H^{+}, $NH4^{+}$, Na^{+}, frequently in about that order of general relative abundance [40].

The existence of such charges is indicated by the ability of clay to absorb ions from the solution. Cations (positive ions) are more readily absorbed than anions (negative ions); hence, negative charges must be predominant on the clay surface. A cation, such as Na^{+}, is readily attracted from a salt solution and attached to a clay surface. However, the absorbed Na^{+} ion is not permanently attached; it can be replaced by K^{+} ions if the clay is placed in a solution of potassium chloride (KCl). The process of replacement by excess cation is called cation exchange. Some are more strongly attracted than others, and the cations can be arranged in a series in terms of their affinity for attraction as follows:

$Al^{3+} > Ca^{2+} > Mg^{2+} > NH4^{+} > K^{+} > H^{+} > Na^{+} > Li^{+}$

This series indicates that, for example, Al^{3+} ions can replace Ca^{2+} ions, and Ca^{2+} ions can replace Na^{+} ions. The exchangeable cations may be present in the surrounding water or be gained from the stabilizers. The process is called cation exchange [40].

An example of the cation exchange;

Na-clay +CaCl2 → Ca-Clay + NaCl (2.1)

2.14.2.2. Pozzolanic Chemical Reaction

The pozzolanic reaction process, which can either be modest or quite substantial depending on the mineralogy of the soil, is a long term process. This is because the process can continue as long as a sufficiently high pH is maintained to solubilize silicates and aluminates from the clay matrix, and in some cases from the fine silt soil. These solubilized silicates and aluminates then react with calcium from the free lime and water to form calcium-silicate-hydrates and calcium aluminate- hydrates, which are the same type of compounds that produce strength development in the hydration of Portland cement. However, the pozzolanic reaction process is not limited to long term effects. The pozzolanic reaction progresses relatively quickly in some soils depending on the rate of dissolution from the soil matrix [41]. Pozzolanic constituents produces calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH). Rate of the pozzolanic reactions depends on time and temperature.

$Ca^{2+} + 2(OH)^{-} + SiO_2$ (Clay Silica) → CSH (2.2)

$Ca^{2+} + 2(OH)^{-} + Al_2O_3$ (Clay Alumina) →CAH (2.3)

The calcium silicate gel formed initially coats and binds lumps of clay together. The gel then crystallizes to form an interlocking structure which increases the soil strength.

2.15. Lime Stabilization

Lime provides an economical way of soil stabilization. The method of soil improvement in which lime is added to the soil to improve its properties is known as lime stabilization. The types of lime used to the soil are hydrated high calcium lime, monohydrated dolomite lime, calcite quick lime, dolomite lime. The quantity of lime is used in most soil stabilizer is in the range of 5% to 10%. Lime modification describes an increase in strength brought by cation exchange capacity rather than cementing effect brought by pozzolanic reaction. In soil modification, as clay particles flocculates, transforms natural plate like clays particles into needle like interlocking metalline structures. Clay soils turn drier and less susceptible to water content changes. Lime stabilization may refer to pozzolanic reaction in which pozzolana materials reacts with lime in presence of water to produce cementitious compounds [42]. The effect can be brought by either quicklime, CaO or hydrated lime, $Ca(OH)_2$. Slurry lime also can be used in dry oils conditions where water may be required to achieve effective compaction.

$$CaO + H_2O \rightarrow Ca(OH)_2 + \text{Heat } (65kJ / mol) \quad (2.4)$$

2.16. Waste Paper Ash Stabilization

Waste paper ash is a waste by-product from paper mill industries, which produce paper. It is estimated that about millions of tons of WPA is being produced from different paper industries in India consuming health issues and environmental hazards. The effect of WPA on plasticity, FSI, compaction, unconfined compressive strength and CBR in soft clayey soil was investigated by previous researchers. Compressive strength was increased by adding 5% WPA about 314 KN/m^2 to 496 KN/m^2 and 284 KN/m^2 to 590 KN/m^2 of 7 days 28 days curing period respectively. Furthermore UCS values increased 107.9% by using 5% of WPA in 28 days curing [45].

Geotechnical researchers presented the effect of WPA on unconfined compressive strength and CBR of sandy clay soil. WPA consist of 62.39% CaO, 23.25% of SiO_2 and 5.26% of Al_2O_3. It is concluded that increase in WPA content improves unconfined compressive strength content about 2 times by the addition of 10% WPA. The addition of 10% WPA were increased the CBR value about 1.5 times in un soaked condition and 3.6 times in soaked condition compared with untreated soil sample [46]. Disposable paper available in abundance throughout the world is composed mainly of short, natural, cellulose fibers and is already used in many local raw materials. Waste paper arises from several sources such as newspapers, office and printing papers etc. The chemical composition of paper ash mainly contains SiO_2 (60%), CaO (14%), Al_2O_3 (2.06%), and Fe_2O_3 (0.92%) which are the main essential compounds needed for cement hydration for strength development [47].

This study has investigated the effect of WPA addition to a mixture of expansive soil and lime for the improvement of the soil's engineering properties.

2.17. Laboratory tests

The samples will be collected from different source subjected to various geotechnical characterizations. The basic test such as sieve analysis, Atterberg limit, natural moisture content, compaction and CBR of materials investigated separately in order to know the natural properties of materials as per relevant code of standard.

2.17.1 .Moisture Content (AASHTO T-256)

Oven-drying method will used to determine the moisture contents of the samples. The oven-drying method, small, representative specimens obtained from large bulk samples will weighed as received, then oven-dried at 105°C for 24 hours. The sample is then reweighted, and the difference in weight is assumed to be the weight of the water driven off during drying. The difference in weight dividing by the weight of the dry soil, giving the water content on a dry weight basis as stated in Eq. (2.2).

$$\text{MC}\ (\%) = \frac{(\text{Wet weight} - \text{Dry weight}\ ) \times 100\%}{\text{Dry weight}} \qquad (2.1)$$

2.17.2. Grain Size Analysis (AASHTO T 88-93)

This test will be performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and hydrometer method is used to determine the distribution of finer particles. For this study, wet sieve analysis and hydrometer analysis done according to (AASHTO T 88-93, particle size analysis of soils), the analysis combined both wet sieve and hydrometer analysis then distribution curve was plotted.

2.17.3. Specific Gravity (ASTM D 854-00)

Values for specific gravity of the natural soil and mixture of lime with waste paper ash will be determined by placing a known weight of oven-dried soil in a flask, then filling the flask with water. The weight of displaced water will then calculated by comparing the weight of the soil and water in the flask with the weight of flask containing only water. The specific gravity will then calculated by dividing the weight of the dry soil by the weight of the displaced water.

2.17.4. Atterberg Limits (AASHTO T90)

Representative samples of each soil subjected to Atterberg limits testing to determine the consistency of the soils. An Atterberg limits device used to determine the liquid limit of each soil using the material passing through a 4.75 mm (No. 40) sieve. The liquid limit of each soil had been determined by using casagrande apparatus. The plastic limit of each soil will determined by using soil passing through a 4.75mm sieve and rolling 3-mm diameter threads of soil until they began to crack. The plasticity index then computed for each soil based on the liquid and plastic limit obtained.

2.17.5. Soil Classification (AASHTO M-145)

Soil classified using the AASHTO Soil Classification System using particle size distribution and Atterberg limits. Soil classification is the arrangement of soils into different group in order that the soils in a particular group would have similar behavior. The method of classification used in this study was the AASHTO M-145 System. The AASHTO Classification system is useful for classifying soils for high way. According to laboratory test, result the soil under study will be classified as table 2.3 shown before.

2.17.6 .Proctor compaction test (AASHTO T-180)

This test was done to determine the maximum dry density (MDD) and optimum moisture content (OMC) of the material. It is done on the natural soil and then various percentages of mixture of lime and waste paper ash added on the expansive soil and MDD and OMC were determined

2.17.7. California Bearing Ratio and CBR Swell

CBR test conducted to determine the strength of a given material and how it behaves under loading. This will be determined by measuring the relationship between force and penetration when a cylindrical plunger of cross-sectional area 1935 mm^2 is made to penetrate the soil at given rate. At any penetration value, the ratio of the force to a standard force defined as the California Bearing Ratio. Road pavement structural design has usually based on 4-days soaked CBR values, to simulate the anticipated “worst-case” soil condition on the field. It measured by placing the tripod with the dial indicator on the top of the soaked CBR mold. The compacted soil samples of the CBR mold are soaked for 96 hours in a water bath to get the CBR swell of the soil. The initial dial reading of the soil of the dial indicator on the soaked CBR of mold has taken just after soaking the sample. At the end of 96 hours, the final dial reading of the dial indicator has taken hence the swell percentage of the initial sample length.

$$\text{CBR percentage} = \frac{\text{test load on the sample} * 100\%}{\text{standard load on the crushed stone}} \qquad (2.2)$$

$$\text{CBR Swell} = \frac{\text{Change in Length in mm during soaking} \ * 100\%}{\text{initial sample length}} \qquad (2.3)$$

CHAPTER THREE

MATERIALS AND METHODOLOGY

In this section laboratory analysis of collected samples and stabilized soils were presented. Laboratory tests were done in Jimma University, Highway laboratory. The chemicals used for this study were Hydrated Lime mixed with WPA. Hydrated Lime (HL) was obtained from Sankale Lime Factory and WPA was collected from Jimma University, JIT offices.

3.1. Study area

Jimma is located at about 354 km in Southwest of Addis Ababa. It is located between 7° 38'52" and 7° 43' 14" N latitude, and between 36° 48' 00" and 36° 53' 24" E longitude. The town is found in an area of the altitude of 1718 – 2000 m above sea level. It lies in the climatic zone locally known as Woyna Daga which is considered ideal for agriculture as well as human settlement [46].

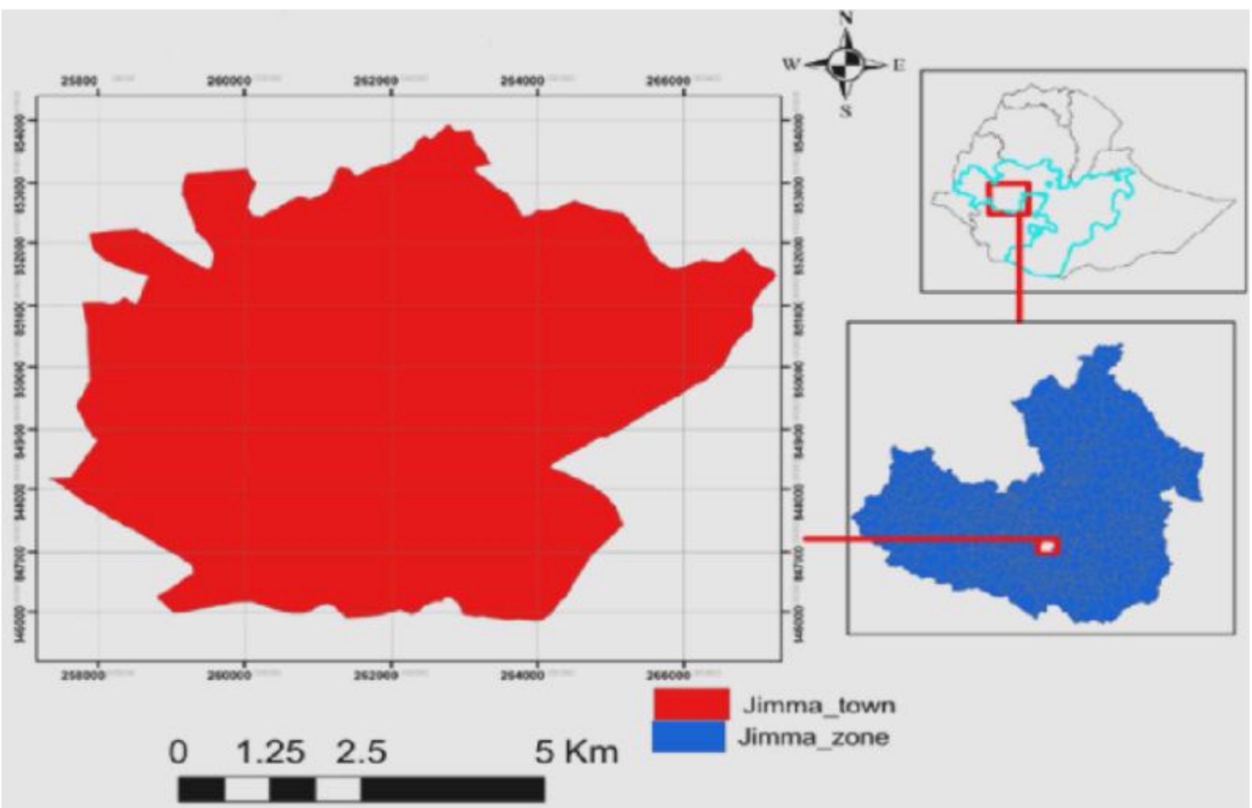

Figure 3.1: Map of the study area (http://www.earthexpoler.usgs.gov.com-using ArcGIS)

3.2. Study Design and Period

This section presents details about the experimental tests conducted on treated and untreated soil samples using WPA blended with HL. Primarily, properties of materials such as WPA, untreated expansive soil were examined. Then the effects of the stabilizers on Atterberg limits, moisture-density relation, LS, FSI, CBR and CBR swell were investigated by varying mix ratios of HL : WPA as 2% HL + 12% WPA, 4% HL + 10% WPA, 6% HL + 8% WPA, 8% HL + 6% WPA. Two representative disturbed and undisturbed soil samples were collected from JIT and MAR around Jimma town and the samples taken from 1.5–2.0 m depth. The disturbed soil samples were first air-dried and laboratory tests were conducted according to the ASTM and AASHTO soil testing procedures. The study followed the experimental type of study which begun with collecting samples, sample preparation and laboratory tests on pozzolanity properties of WPA. Then, discussions on sample collection and summary of laboratory test results were presented. The laboratory test data was analyzed and interpreted so that properties of expansive soil and additives requirement was discussed. Finally, the research findings and recommendations were forwarded based on the laboratory test results and compared with ERA and AASHTO specifications.

The overall research design has shown in Figure 3.2.

3.3. Sample Preparation Techniques

Dry preparation of disturbed soil and soil aggregate samples for laboratory test was conducted as per AASHTO T 87-86. After the required amount of stabilizer was mixed with soil, the blend should be thoroughly mixed until the color of the mixture is uniform. If necessary to achieve the desired moisture content for the batch, additional water was first blended into the soil and mixed for three to five minutes. After water addition, the appropriate amounts of stabilizer were then added to the mixture and blended thoroughly for three to five minutes. The mixing of soil and stabilizer in this research was carried out by hand mixing, and the water and stabilizer were added slowly to promote uniform blending and to prevent clumping of the soil and/or stabilizer. In order to have sufficient and reliable data for the target analysis, laboratory tests conducted on soil samples selected from six different locations of Jimma town from which detail research were done on two pits after proving they have the worst expansive potential by conducting FSI test. The representative disturbed samples were collected by manual excavation from the selected study area on the basis of visual identification of a suitable sub-grade soil and FSI test.

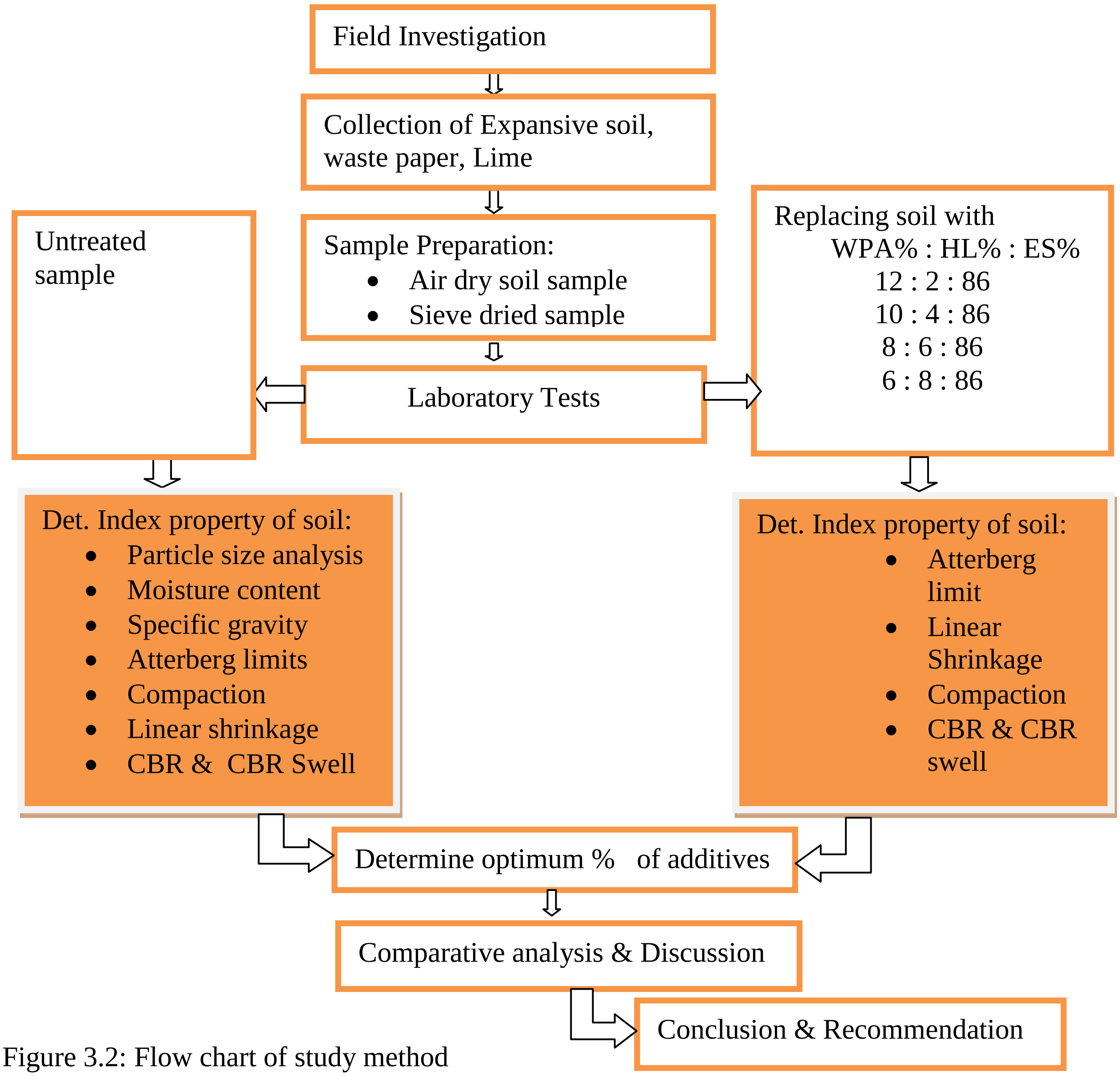

Figure 3.2: Flow chart of study method

3.4. Study Variables

The study variables are categorized into two. These are dependent and independent variables. Strength of expansive soil treated using WPA blended with HL was a dependent variable. The independent variables were physical and mechanical properties of untreated and treated soil samples, and dosage of lime-waste paper ash.

3.5. Sources of Data

Both primary and secondary data were used in this research finding. The primary sources of data for this study were all laboratory out puts and secondary data were journals and different standards i,e ERA, AASHTO, ASTM and IS.

3.6. Experimental setup and Mixing Ratios

The weak sub-grade soil was mixed with HL-WPA by percentages of the weight of soil taken for each test starting from mixture of 0-8% HL and 6-12% WPA within 2% difference for both WPA and HL. This percentage mix-ratio was fixed with some basis of observation from different scholars used by 2% difference [46]. Due to this observation the percentage mix-ratio was fixed by 2% difference, from point of saving mixing additives and starting from lowest mix- ratio. HL was first added to the pulverized, sieved and air-dried soil sample and dry mixed thoroughly. WPA added after that and wet mixing was done by sprinkling measured amount of water followed by a thorough mixing until a uniform soil-additive matrix was obtained. The the appropriate percentage by weight of WPA and HL, required to improve weak sub grade soils to meet the specification assessed and the result from laboratory test was compared with the standard and specification of AASHTO, ASTM and ERA. Finally, the research findings and recommendations were forwarded based on the laboratory test results.

Table 3.1: Percentage mix-ratios of WPA: HL by weight of soil

Percent mix- ratio by weight		
WPA (%)	HL (%)	ES (%)
12	2	86
10	4	86
8	6	86
6	8	86

3.7. Population

The populations of this research were the ES, HL and WPA of the selected study area of Jimma town.

3.8. Sample Collection for Laboratory Tests

3.8.1 .Expansive Soil

Based on physical observation and field investigation, two expansive soil samples were selected around Jimma town in JU Institute of Technology (JIT) and Hirmata Mentina Kebele along Merkato to Jimma Airport road (MAR). The representative disturbed samples were collected by manual excavation from the selected study area on the basis of visual identification and FSI test of a suitable sub-grade soil. According to AASHTO and ASTM, 300 kg disturbed sample was collected at the depth of 1.5 m to avoid the inclusion of organic matter.

Figure 3.3: Photo of soil sample collection (Taken on, 24/12/2020)

3.7.2. Hydrated Lime

A 50 kg of HL was obtained from Sanqale lime factory which is located in West Shewa, Oromia region. Chemical composition of Sanqale Lime was investigated by [33] as the composition result is presented in Table 3.1.

Table 3.2: Sanqale lime chemical composition [33]

Constituent	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	TiO_2	P_2O_5	MnO	SO_3
Percentage	6.21	2.18	3.57	59.47	3.91	0.61	0.79	0.3286	0.208	0.2785	0.58

3.7.3. Physical and Chemical Properties of Waste paper Ash

The WPA used in this study has been tested and based on ASTM C618, WPA classified as Class-C fly ash because WPA containing more than 20% lime (CaO) and possesses cementitous properties and pozzolanic properties that resulting in the self-cementing characteristics [48]. This Class-C WPA is self-cementing; in which activators such as lime or cement are not required [46]. The laboratory result for physical properties of WPA shows that, the specific gravity for WPA is about 1.65 and it can be considered as light materials. The result of particle size distribution shows, 0.08% of sand size and 99.92% of fine or silt size. It indicates that the WPA considered as a silty size particle. Atterberg limit of WPA was done by drop cone method and the test result was compiled as shown in *Appedix K* and *Appendix L*. Waste paper was collected from Highway chair office and other JIT Staff's offices in Jimma University. The collected waste paper was dried at 25^0C and manually burned on metal sheet to protect from the contamination of another organic material. To avoid the formation of crystalline ash which is less reactive to lime, the manually burned paper again re-burned using furnace at a temperature of 850°C. Burned powder was then sieved using sieve size of No.40 (0.425 mm) to remove other unnecessary material. The fraction passing through the sieve was used during testing.

Table 3.3: Chemical composition of waste paper ash [46].

Constituent	CaO	SiO_2	Al_2O_3	MgO	Fe_2O_3	SO_3	Na_2O	K_2O	L.O.I
Percentage	62.39	23.25	5.26	2.46	0.77	0.58	0.42	0.35	4.5

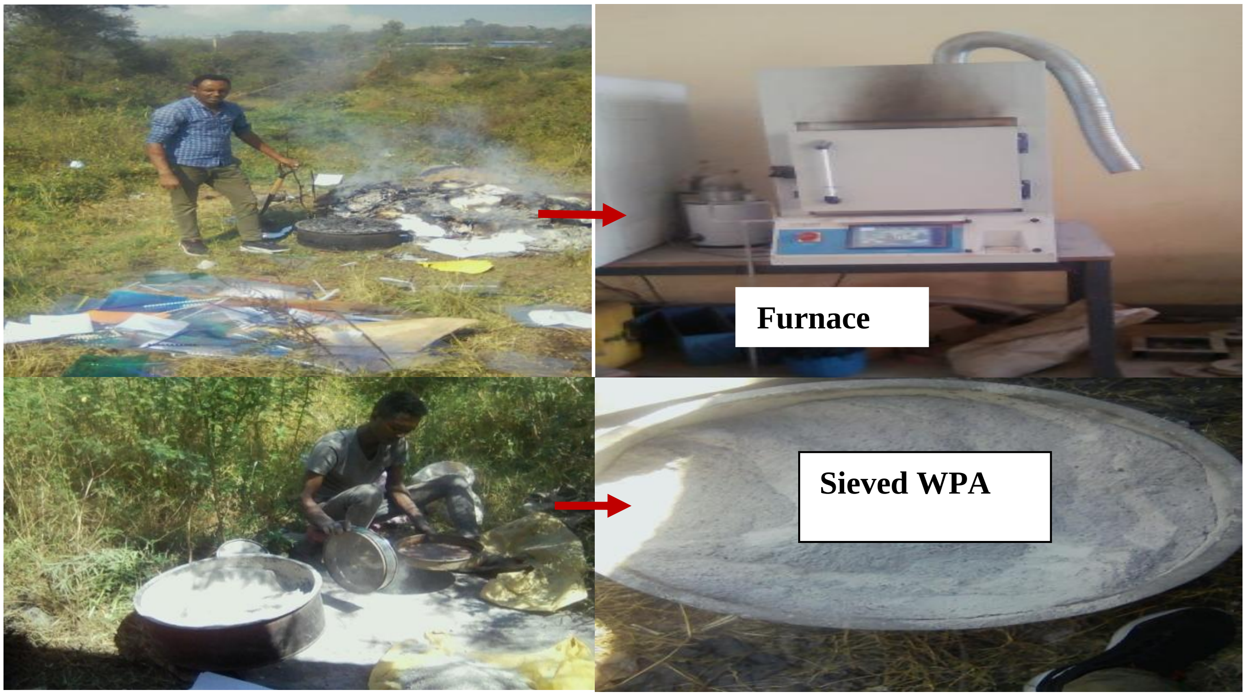

Figure 3.4: Preparation of waste paper ash (Taken on, 25/12/2020)

3.8. Software and Instruments

The following instruments and software were used for this study: meter tape, plastic bags, manual hand auger equipment, laboratory equipment, digital camera for documentation, MS word and Excel to analyze laboratory data was used in this study.

3.9. Sample preparation

The natural soil sample was placed inside the thick -gauge plastic bags to prevent moisture loss. Then breaking up the soil aggregates by a rubber covered mallet and adequately pulverized and then sieve analysis was conducted on properly pulverized natural soil. Sieving was conducted into three groups. The first team is soil samples passing #40 (4.25 mm) sieve for Atterberg limits and FSI, 2 mm sieve for G_s, and the third group is soil samples passing # 4 (4.75 mm) for compaction and CBR.

In other hands WPA sample was collected from Highway Chair office, and then was burnt under the controlled condition on metal sheet to obtain the ash form. After converted to ash then sieved through No.40 (0.425 mm) to remove other unnecessary material and to ensure it completely converted to ash. Finally, after preparation was done laboratory tests were conducted in Jimma University Institute of Technology in Highway laboratory.

3.10. Laboratory tests

The samples were collected from different sources subjected to various geotechnical characterizations which included tests for soil classification grain size analysis, FSI, G_s, and Atterberg limits. These are indicative tests that always used for identifying whether the proposed soil is expansive or not. To fully attain the objective of the research, the total conducted tests included Atterberg limits, G_s, wet sieve analysis, FSI, moisture-density relation, LS, CBR, and CBR swell. The basic test procedures were shown in Table 3.3 as per the relevant code of standard.

Table 3.4: Laboratory test as per standard

Laboratory test	Standard
Moisture content	AASHTO T 265-93
Grain size analysis	AASHTO T 88-93
Specific gravity	ASTM D 854-00
Atterberg limits	AASHTO T 89-96 and AASHTO T 90 -00
Soil classification	AASHTO M-145
Proctor compaction test	AASHTO T 180-93
California bearing ratio an CBR swell	AASHTO T 193-93 and AASHTO T-180
Free swell index	IS 1498
Linear shrinkage	ASTM D 4943

3.10.1. Grain Size Analysis Test

3.10.1.1. Wet Sieve Analysis

The wet sieve analysis test was conducted to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis was performed to determine the distribution of the coarser, larger-sized particles. About 1000 g soil sample was washed over a 0.075 mm (#200) sieve. The washing continued until the wash water is clear. The fraction retained on 0.075 mm (#200) sieve dried at a constant temperature of 110 ± 5^0C for 24 hours and sieve analysis made using 9.5 mm, 4.75 mm, 2.36 mm, 2 mm, 1.18 mm, 0.85 mm, 0.6 mm, 0.425 mm, 0.3 mm, 0.15 mm, 0.075 mm sieves with progressively smaller screen sizes to determine the percentage of sand-sized particles in the specimens as per [52]. The weight of sample retained on each sieve was weighed and percent passed through each sieve was calculated as the detail shown in *Appendix A2*.

3.10.1.2. Hydrometer Analysis

A hydrometer analysis was conducted to determine the amount of silt and clay particles in both soil samples. About 50 gm of soil sample was taken from the soil sample passed through No. 200 sieve and 2% solution of sodium hexameta phosphate with distilled water was added and immersed for 24 hours. Then the soaked soil was transferred to dispersion cup and was stirred for 15 minutes. The soil mixture was poured into the standard measuring flask and made total volume of oil suspension by 1000 cm^3. Finally, the hydrometer was calibrated and the amount of silt and clay particles was determined as per AASHTO T 88-93 [52]. The detail laboratory analysis was indicated in *Appendix G.*

Figure 3.5: Wet sieve analysis procedures (Taken on, 27/01/2021)

3.10.2. Soil Classification

Soil classification was conducted based on AASHTO and USCS soil classification methods. AASHTO classification system is recommended for classifying soil for highway. The soil sample was classified by AASHTO classification method using particle size distribution and Atterberg limits. Soil classification is the arrangement of soil into different groups in order that the soil in a particular group would have relatively similar behavior as per AASHTO M145-91 (2008), classification of soils and soil- aggregate mixtures for highway construction purpose [52].

3.10.3. Natural Moisture Content

The oven-drying method was used to determine the moisture contents of the samples. Small, representative specimens obtained from large bulk samples were weighed as received, then oven-dried at 105°C for 24 hours. The sample was then weighed, and the difference in weight was assumed to be the weight of the water driven off during drying. The difference in weight was divided by the weight of the dry soil, giving the water content on a dry weight basis as per AASHTO T 265-93, laboratory determination of moisture content of soils.

Figure 3.6: Natural moisture content determination (Taken on, 25/01/2021)

3.10.4. Specific Gravity Test

Specific gravity measures the heaviness of the soil particles. This lab is performed to determine the G_s of soil by using a pycnometer. Values for G_s of the soil solids were determined by placing a known weight of oven-dried soil in a flask, then filling the flask with water. The weight of displaced water was then calculated by comparing the weight of the soil and water in the flask with the weight of flask containing only water. The G_s was then calculated by dividing the weight of the dry soil by the weight of the displaced water as per ASTM D 854-00, test method for specific gravity of soils.

Figure 3.7: Specific gravity test (Taken on, 25/01/2021)

3.10.5. Atterberg Limits Test

A sample weighing about 250 g was taken from the mixture prepared for LL and PL test. The air dried and pulverized sample was sieved by number 40 sieve. The soil passed number 40 sieve was mixed different ratios of HL-WPA content at optimum water content and was sealed with plastic for 24 hours to give sufficient time for chemical reaction before conducting test. The sample was manually mixed with hand and the LL of each sample was determined by Casagrande apparatus. The LL was arbitrarily determined by Casagrande apparatus at which a sample of soil in a standard cup and cut by a groove of standard dimensions was flow together at the base of the groove for a distance of 13 mm when subjected to 25 shocks from the cup being dropped 10 mm in a standard LL apparatus operated at a rate of two shocks per second. The PL of each sample was determined by rolling 3 mm diameter threads of soil until it began to crack [51].

Figure 3.8: Atterberg limit test (Taken on, 26/01/2021)

3.10.6. Moisture-Density relation test

Modified proctor test was done to determine the MDD and OMC of the natural soil while using various percentages of mixture of HL and WPA added on the ES according to AASHTO T 180-93, moisture-density relations of soils using a 4.55 kg of rammer and 457 mm drop. A sufficient quantity of air dried soil were obtained in large mixing pan and pulverized the soil and run it through the No. 4 (4.75 mm) sieve and prepare representative samples each about 5000 g for a single proctor test using 15.24 cm mould.

Figure 3.9: Moisture-density relation test (Taken on, 15/03/2021)

3.10.7. Linear Shrinkage

Linear shrinkage test followed a British standard (BS 1377: Part 2:1990), and covers the determination of total LS from linear measurement on a standard bar of length 140 mm with a semicircular section of diameter 25 mm, the groove filled by a soil of the fraction passing 0.425 mm sieve, oven dried for 24 hours.

$$\text{LS}\ (\%) = \frac{(\text{Initial length} - \text{Oven dried length of specimen}) \times 100\%}{\text{Initial length}} \tag{3.1}$$

Figure 3.10: Linear shrinkage test (Taken on, 06/02/2021)

3.10.8. California Bearing Ratio

The CBR test was conducted to determine the strength and shearing resistance of soil under controlled moisture and density conditions. A soil sample retained on 19 mm sieve was discarded from the test and 5000 g representative sample was prepared for each compactions. A three point CBR tests at 10, 30, and 65 blows carried out based on the standard procedure given in AASHTO T 193-93 and the CBR at 95% MDD was determined. This laboratory study involved the CBR test for soaked condition of expansive soil samples prepared with its OMC and compacted at their MDD using hand compaction tool.

$$\text{CBR}\ (\%) = \frac{\text{Applied load on sample} \times 100\%}{\text{Standard load on the crushed stone}} \tag{3.2}$$

Figure 3.11: California bearing ratio test and procedures (Taken on, 15/03/2021)

3.10.9. CBR Swell Index Test

Stabilized and un stabilized soil was compacted in CBR molds at OMC along with its respective MDD was gauged for swelling characteristics before and after soaking for 96 hours to evaluate the percent of swell. The CBR percent swell tests were determined with 4.5 kg surcharge ring in accordance with AASHTO-T193. Readings of swelling values were taken before soaking and after 4 days soaking of CBR molds for 10 blows, 30 blows and 65 blows and the swell percent value were taken as the average of the three molds.

$$\text{CBR Swell} = \frac{\text{Change in Length in mm during soaking } * 100\%}{116.30 \text{ mm}} \quad (3.3)$$

3.10.10. Free Swell Index Test

The FSI test gives a fair approximation of the degree of expansiveness of the soil sample. The procedure involves in taking two oven dried soil samples passing through the 0.425 mm sieve, 10 g each was placed separately in two 100 ml graduated soil sample. Distilled water was filled with one cylinder and kerosene in the other cylinder up to 100 ml mark. The final volume of soil is computed after 24 hours to calculate the free swell index as in Eq. (3.4).

$$\mathrm{FSI}\ (\%) = \frac{\text{Final volume of soil in water} - \text{Final volume of soil in kerosene} \ * 100\%}{\text{Final volume of soil in kerosene}} \tag{3.4}$$

Figure 3.12: Free swell index test (Taken on, 27/01/2021)

3.15. Data Quality Management

All the laboratory tests conducted throughout the research findings tripled to minimize calibration error during sample measurement. Excavation of ES conducted below 1.5 m to remove the organic materials from the soil. The collected waste paper was dried at 25^0C and manually burned on metal sheet to protect from the contamination of another organic material. To avoid the formation of crystalline ash which is less reactive to lime, the manually burned paper again re-burned using furnace at a temperature of 850°C.

3.16. Symbolization

Through this research study soil sample collected from JU, Jimma Institute of Technology, was abbreviated as JIT and sample taken from Hirmata Mentina Kebele along Merkato to Jimma Airport road was abbreviated as MAR. Additionally, hydrated lime and waste paper ash were also abbreviated as HL and WPA respectively.

CHAPTER FOUR

RESULTS AND DISCUSSION

This chapter presents test results, discussion and analysis of all experimental work that were performed on untreated and treated soils with hydrated lime blended with waste paper ash. First, the properties of materials were discussed. Then the effects of stabilizers on engineering properties of expansive soil were established by varying percentage of stabilizers and compared with untreated soil properties. Meanwhile, the effects of stabilizers on the treated soil was compared and contrasted with standard specifications such as AASHTO, ASTM, ERA and conclusions were forwarded.

4.1. Properties of Materials

4.1.1. Physical and Properties of Waste Paper Ash

Physical properties of WPA show the specific gravity of WPA is about 1.65, indicating WPA to be considered as light materials. The particle size distribution of WPA shows that 0.08% of sand size and 99.92% of fine or silt size. It indicates that the WPA considered as a silty size. Atterberg limits of WPA were given in Table 4.1. The detail of laboratory test analysis data of WPA was attached in *Appendix J* and *Appendix K*.

Table 4.1: Physical properties of waste paper ash (WPA)

Properties	Test results
G_s	1.67
LL (%)	50.8
PL (%)	43.7
PI (%)	7.1
OMC (%)	50.1
Particle size distribution: Sand (%) Silt (%) Clay (%)	 0.08 99.92 0.0
Classification (ASTM C618)	Class-C fly ash

4.1.2. Chemical Properties of Waste Paper Ash

The chemical composition of WPA was investigated by [46] and compiled as shown in Table 4.2. As observed from the table, the WPA used in this study has been tested and based on ASTM C-618, WPA classified as class-C fly ash because WPA containing more than 20% lime (CaO) and possesses cementitous properties and pozzolanic properties that resulting in the self-cementing characteristics. This Class-C WPA is self-cementing; activators such as lime or cement are not required [46]. WPA is classified as Class-C fly ash because the total combination percentage composition for major constituent components such as silicon dioxide (SiO_2), alumina (Al_2O_3), and iron oxide (Fe_2O_3) should be more than 70 percent. Instead this Class-C of WPA considered as higher of calcium fly ash of calcium carbonate or free lime content (CaO) about 62.39%. This is adequate to meet the requirement of ASTM C-618 standard for pozzolanic materials (Class-C fly ash) because of its self cementing characteristics [46].

Generally, the high content of calcium in WPA provides the pozzolanic reactants, silica and alumina, lacking in such soils. These solubilized silicates and aluminates then react with calcium from the WPA and water to form calcium-silicate-hydrates and calcium aluminate- hydrates, which are the same type of compounds that produce strength development in the hydration of weak sub-grade soil. The calcium silicate gel formed initially coats and binds lumps of clay together. The gel then crystallizes to form an interlocking structure which increases the soil strength.

Table 4.2: Chemical composition of Waste Paper Ash (WPA) [46]

Chemical Constituents	Chemical Composition of WPA (%)	Requirement ASTM C-618 (%)	Remark
CaO	62.39	$\geq$ 20	In range
SiO_2	23.25	20-60	In range
Al_2O_3	5.26		
MgO	2.46	$\leq$ 5	In range
Fe_2O_3	0.77		
SO_3	0.58	$\leq$ 5	In range
Na_2O	0.42	0-6	In range
K_2O	0.35		
L.O.I (loss on ignition)	4.5	$\leq$ 6	In range

4.1.2. Engineering Properties of Expansive Sub-grade Soil

The results of laboratory tests conducted for identification and determination of engineering properties of the untreated soil samples before mixing with waste paper ash and lime were presented in Table 4.3. Based on particle size distribution and Atterberg limit test result given in Table 4.3, both soil samples are classified as CH as per USCS and A-7-5 as per AASHTO soil classification system. From Atterberg limit test result in Table 4.3 both JIT and MAR soil samples had high percentage of LL 82.2 and 86.50%, respectively. As a result, these values indicates both soil sample are very high plastic clay and classified under inorganic clay. Therefore, the sub-grade soil shrink and swell easily with change in moisture content and cannot resist internal and external load which leads to structural and physical failure. To overcome this failure, stabilization of this sub-grade soil with different additives should be recommended.

Table 4.3: Geotechnical properties of both soil samples

Parameters	Laboratory Test Results (%)							
	JIT soil			Average	MAR soil			Average
	T1	T2	T3		T1	T2	T3	
LL (%)	80.4	82.6	83.2	82.20	85.3	86.9	87.8	86.50
PL (%)	42.5	44.2	43.0	43.20	47.0	44.2	43.0	44.7
PI (%)	37.9	38.4	40.2	39.00	38.3	42.7	44.8	41.8
LS (%)	18.64	19.35	15.21	17.73	20.07	19.93	20.21	20.07
AASHTO soil classification				A-7-5				A-7-5
USCS				CH				CH
G_s	2.72	2.78	2.77	2.76	2.73	2.74	2.70	2.73
FSI (%)	98.1	86.43	79.83	88.12	104.7	96.41	89.36	96.83
MDD (g/cm^3)	1.44	1.43	1.42	1.43	1.41	1.40	1.38	1.40
OMC (%)	27.96	27.89	28.1	27.98	30.51	30.75	30.97	30.74
Soaked CBR value (%)	2.35	2.55	2.1	2.33	1.94	2.1	1.8	1.95
CBR swell (%)	3.52	3.36	3.13	3.34	4.41	4.16	3.97	4.18
Percentage pass No.200 sieve	92.94	93.6	95.64	94.06	94.2	96.13	96.8	95.71
Color of soil				Dark gray				Dark gray

4.1.2.1. Grain Size Analysis

The particle size distribution curves of JIT and MAR are presented in figure 4.1 and the detail analysis was attached in *Appendix G*. Full grain size distribution analyses (including hydrometer) were also performed for each soil. The minimum per cent pass sieve No. 200 for JIT and MAR soil samples are 94.06 and 95.71%, respectively which in turn indicates the two soil samples are classified under fine-grained (silty-clay material) as per AASHTO T88-93. The percent passing of each test is not only used to categorize soil as coarse-grained and fine-grained but it also helps to determine the soil class together with the Atterberg limits.

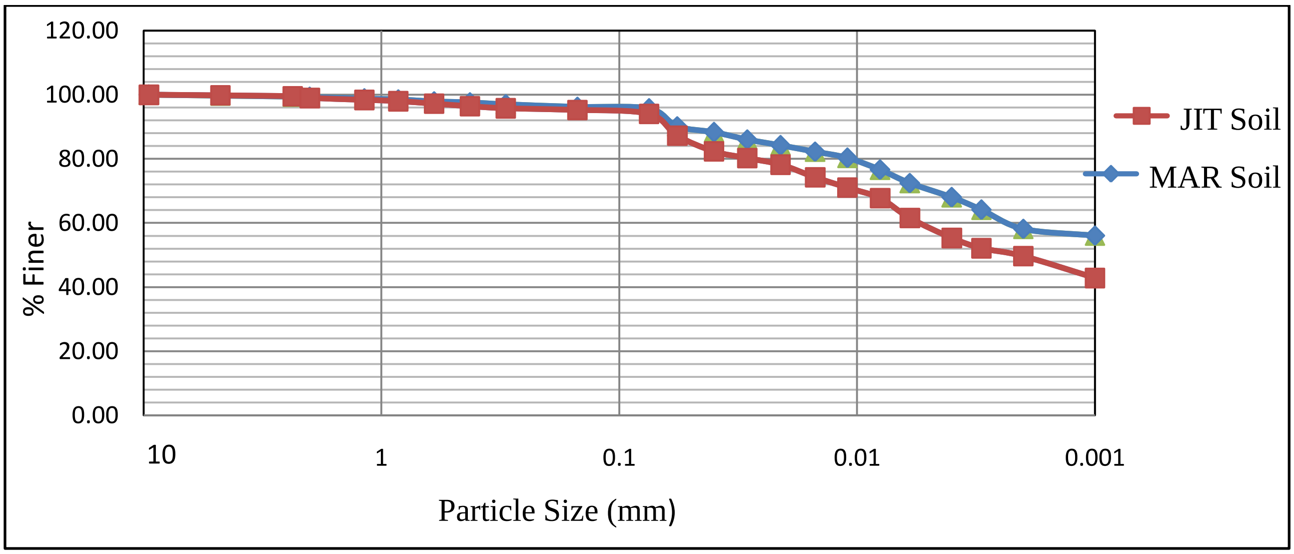

Figure 4.1: Grain size distribution curve of soil samples

4.1.2.2. Specific Gravity

Specific gravity is an important parameter in identifying the soil type, classification, and suitability as a construction material. The specific gravity of inorganic clay soil is in the range of 2.68-2.8[41]. The specific gravity values of both soil samples indicates the two soil samples are inorganic clay soils and requires treatment to serve as a roadway sub-grade material. The Gs of the soils are given in table 4.4 and its details are attached in *Appendix H* and *Appendix I.*

Table 4.4: Specific gravity of the studied soils

Sample Location	JIT			MAR		
Trial Number	T1	T2	T3	T1	T2	T3
Gs	2.72	2.78	2.77	2.73	2.74	2.70
Average Gs at 20^{o}C	2.76			2.73		

4.1.2.3. Atterberg Limit

The Atterberg limits of the soils are given in Table 4.5 and the detail analysis attached in *Appendix A* and *Appendix B.* The purpose of conducting Atterberg limit test is to know the level of plasticity property of soil and degree of cohesiveness of soil. The liquid limits (LL) of untreated soils were determined as 82.2 and 86.5%, for both JIT and MAR soil samples respectively. These values indicate the two soil samples are highly plastic and did not fulfill the ERA specifications for suitable sub-grade soil. Both soil samples are classified as inorganic clay of high plasticity (CH) and fair to poor sub-grade strength. Therefore, these soils require stabilization to improve its strength to be used as a suitable sub-grade soil.

Table 4.5: Atterberg limit test result of natural soil sample

Atterberg Limits (%)	JIT Soil			Average	MAR Soil			Average	ERA requirement
	T1	T2	T3		T1	T2	T3		
LL (%)	80.4	82.6	83.2	82.2	85.3	86.9	87.8	86.5	< 60
PL (%)	42.5	44.2	43.0	43.2	47.0	44.2	43.0	44.7	-
PI (%)	37.9	38.4	40.2	39.0	38.3	42.7	44.8	41.8	< 30

4.1.2.4. Moisture-Density Relationship

This section presents the compaction characteristic curves determined for sample soils used in the experimental work. Modified Proctor tests were performed on the raw sub-grade soils as well as the treated /stabilized soils as described in chapter 3. The detail compaction laboratory test analysis of JIT and MAR soil samples are attached in *Appendix A4* and *Appendix B4*, respectively.

The JIT soil sample had optimum moisture content of 27.98% and maximum dry density of 1.43 g/cm^3. Similarly, MAR soil sample had 30.74% optimum moisture content and 1.4 g/cm^3 maximum dry density as given in Figure 4.3. The optimum moisture content obtained from this compaction test is used as input data to prepare the CBR specimen to be tested for the soaked condition of CBR determination.

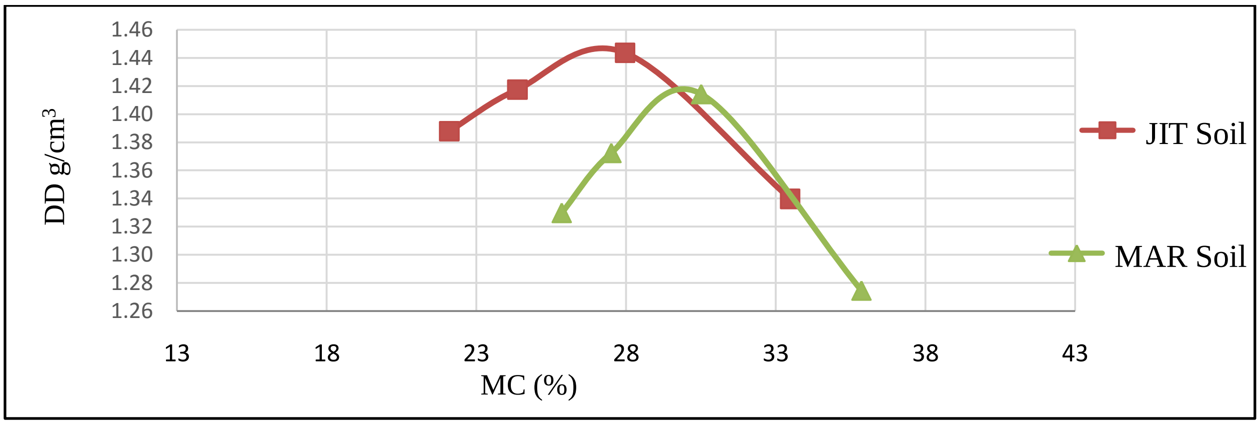

Figure 4.2: Moisture-density relationship test results for natural sub-grade soils

4.1.2.5. Linear Shrinkage

The linear shrinkages of the soils are given in Table 4.6 and the detail laboratory test data are attached in *Appendix F*. Linear shrinkage is the decrease in length of soil sample when oven dried, starting with a moisture content of the sample at the liquid limit [51]. The average linear shrinkages for JIT and MAR natural soils were 17.73 and 20.07%, respectively. These values indicate the two soil samples are highly dispersive soils that quickly swell when exposed to wet and dry upon moisture loss. Therefore, the two soils are not recommended for highway sub-grade soil without applicable stabilization.

Table 4.6: Linear shrinkage test results of untreated soil samples

Sample Location	LS (%)			Average LS (%)
	T1	T2	T3	
JIT Soil Sample	18.64	19.35	15.21	17.73
MAR Soil Sample	20.07	20.21	19.93	20.07

4.1.2.6. California Bearing Ratio

Figure 4.4 and 4.5 present the CBR values of the samples of soil. The detail test results of the samples are attached in *Appendix A3* and *Appendix B3* respectively. The CBR value is important to indicate the load bearing capacity of sub-grade soil and essential parameter in determining the thickness of the pavement layers for roadway pavement design. The CBR of JIT and MAR soil samples at MDD are 2.33 and 1.95%, respectively. These values indicate that both soil samples had low bearing capacity, which does not satisfy the ERA requirement as suitable sub-grade soil. According to [50] standard specification a CBR value of less than 3% needs special treatment to improve its workability and engineering properties.

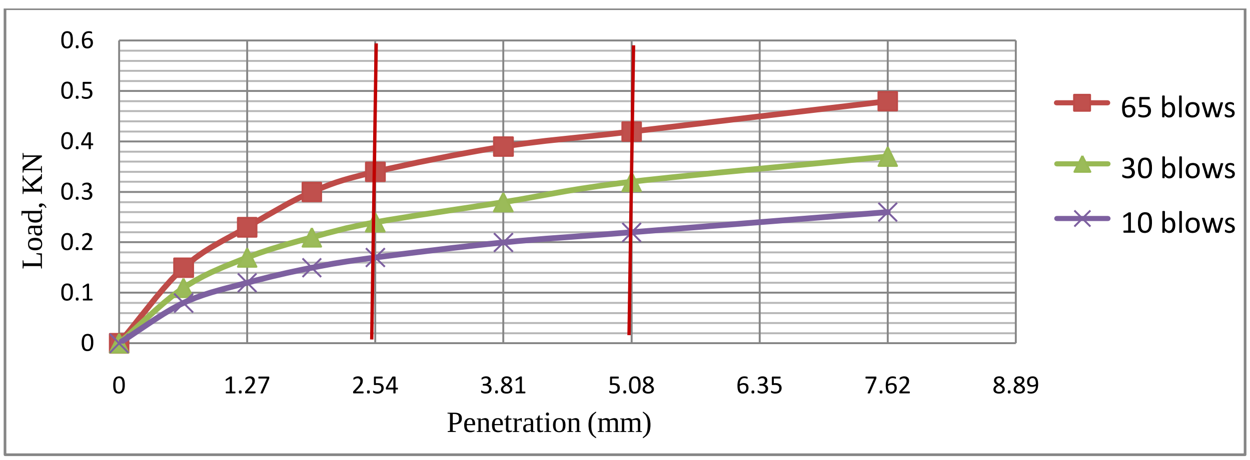

Figure 4.3: CBR laboratory test result of JIT natural soil sample

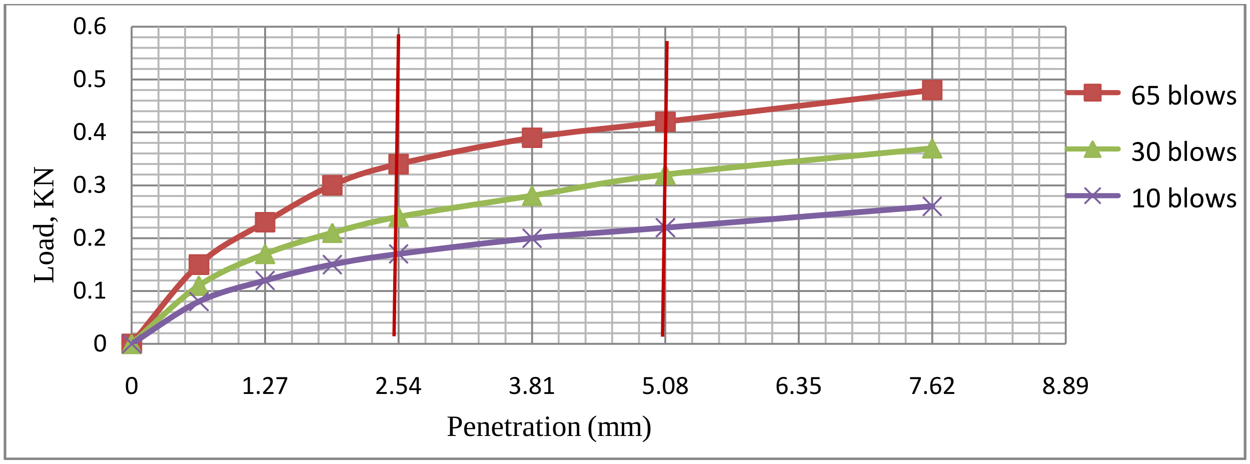

Figure 4.4: CBR laboratory test result of MAR natural soil sample

4.1.2.7. CBR Swell Index

The CBR swell indices are given in table 4.7 and the details of CBR swell test data are attached in *Appendix G*. The CBR swell values of JIT and MAR soil samples are 3.34 and 4.18%, respectively. The CBR swell indices of both soil samples exceed 2%, indicating that the soil samples are poor sub-grades with high degree of expansion. Therefore, both soil samples require stabilization to be used as a suitable sub-grade soil.

Table 4.7: CBR swell test results of natural soil samples

Sample Location	CBR swell (%)			Average CBR swell (%)	ERA requirement	Remark
	T1	T2	T3			
JIT	3.13	3.36	3.52	3.34	< 2%	Poor
MAR	3.97	4.16	4.41	4.18	< 2%	Poor

4.1.2.8. Free Swell Index

The FSI of JIT and MAR soil samples are given in Table 4.8 and the laboratory test detail are attached in *Appendix E*. Free swell index is an increase in volume of a soil, without any external constraints, on submergence in water. This test is most commonly used tests for estimating soil swelling potential. The FSI values of JIT and MAR soil samples are 88.12 and 96.83%, respectively. The values of both soil samples exceed 50% of IS standard. These values indicate the soils under study undergo high volumetric change which leads to general unevenness due to seasonal wetting and drying. Therefore, these soils are problematic soil to be used as a road sub-grade soil without stabilization.

Table 4.8: Free swell index test results of natural soil samples

Sample Location	FSI (%)			Average FSI (%)	IS 1498 Requirement
	T1	T2	T3		
JIT	98.10	86.43	79.83	88.12	FSI < 50%
MAR	104.71	96.41	89.36	96.83	FSI < 50%

4.3. Engineering Properties of Stabilized Expansive Sub-grade Soil

4.3.1. Effect of Hydrated Lime Blended with Waste Paper Ash on Atterberg Limit

The values of the LL, PL and PI of natural soil and stabilized with WPA and HL at JIT and MAR soil samples are given in Table 4.9 and the detail of the laboratory test analysis data are attached in *Appendix C* and *Appendix D*, respectively. The LL values of the JIT soil sample decreased with increasing HL percentage and decreasing WPA percentage in WPA: HL mix-ratios. The addition of *2% HL + 12% WPA, 4% HL + 10% WPA, 6% HL + 8% WPA and 8% HL + 6% WPA* diminished the LL of untreated soil by *74.20%, 64.20%, 55.10% and 52.1%* respectively. While, addition of 2*% HL + 12% WPA, 4% HL + 10% WPA, 6% HL + 8% WPA and 8% HL + 6% WPA* PL slightly increased the PL of untreated soil of JIT as percentage of HL increases and percentage of WPA decreased in WPA: HL mix-ratios by *43.50%, 44.80%, 44.70% and 45.10%*, respectively.

Similarly, The LL values of the MAR soil sample decreased with increasing HL percentage and decreasing WPA percentage in WPA: L mix-ratios. The addition of *2% HL + 12% WPA*, *4% HL + 10% WPA*, *6% HL + 8% WPA* and *8% HL + 6% WPA* reduced the LL of untreated soil by *80.40%, 69.20%, 58.50% and 54.90%* respectively. While, addition of 2*% HL + 12% WPA, 4% HL + 10% WPA, 6% HL + 8% WPA and 8% HL + 6% WPA* PL slightly increased the PL of untreated soil of MAR as percentage of HL increases and percentage of WPA decreased in WPA: HL mix-ratios by *44.90%, 45.20%, 45.80%* and *46.30%,* respectively.

PI of the samples decreased with increasing percentage of HL and decreasing percentage of WPA in WPA: HL mix-ratios. The addition of *8% HL + 6% WPA* reduced PI of the natural soil from 39 to 7% and 41.8 to 8.5%, for JIT and MAR soil samples respectively. These values satisfies the ERA specifications such that a suitable sub-grade should have LL < 60% and PI < 30% to be used as a road sub-grade materials.

Generally, the decrease in water holding capacity of the soil (LL) and PI is responsible to pozzolanic reactions between calcium from WPA in high content, hydration of HL, cation exchange between Ca^{2+} from WPA and Na^{+} found in expansive soil in the form NaCl. These solubilized silicates and aluminates react with calcium from the WPA and water to form calcium-silicate-hydrates and calcium aluminate- hydrates, which are the same type of compounds that produce strength development in the hydration of weak sub-grade soil. The calcium silicate gel formed initially coats and binds lumps of clay together. The gel then crystallizes to form an interlocking structure which increases the soil strength.

Table 4.9: Effect of hydrated lime blended with waste paper ash on Atterberg limit

Mix-Proportion (%)	Trials	LL (%)	PL (%)	PI (%)	LL (%)	PL (%)	PI (%)	ERA (2002) requirement
		JIT Soil Sample			MAR Soil Sample			
Natural Soil	T1	80.40	42.50	37.90	85.30	47.00	38.30	
	T2	82.60	44.20	38.4	86.90	44.20	42.70	
	T3	83.20	43.30	40.20	87.8	43.00	44.80	
Averages (%)		82.20	43.20	39.00	86.50	44.70	41.80	
2%HL+12%WPA	T1	70.94	43.78	29.82	78.95	42.20	33.80	
	T2	74.80	44.43	30.40	80.65	46.9	36.05	
	T3	76.86	42.29	32.18	81.60	45.60	36.65	
Averages (%)		74.20	43.5	30.80	80.40	44.90	35.50	LL < 60%
4%HL+10%WPA	T1	62.41	43.62	19.40	67.91	43.72	22.34	
	T2	64.46	44.47	19.04	68.16	45.57	23.83	PI < 30%
	T3	65.74	46.31	21.86	71.53	46.31	26.13	
Averages (%)		64.20	44.80	20.10	69.20	45.20	24.10	
6%HL+8%WPA	T1	50.52	41.38	8.42	54.68	41.74	9.26	
	T2	57.18	46.45	11.18	59.50	49.34	15.02	
	T3	57.60	46.27	11.90	61.32	46.32	13.82	
Averages (%)		55.10	44.70	10.50	58.50	45.80	12.70	
8%HL+6%WPA	T1	47.74	40.07	5.23	49.20	42.80	9.32	
	T2	55.06	47.3	7.46	56.79	47.58	9.78	
	T3	53.50	47.93	8.31	58.71	48.52	6.40	
Averages (%)		52.10	45.10	7.00	54.90	46.30	8.50	

4.3.2. Effect of Hydrated Lime Blended with Waste Paper Ash on Linear Shrinkage

The linear shrinkage values of natural and stabilized for JIT and MAR soil samples are given in Table 4.10. The linear shrinkage values of the JIT and MAR soil samples decreased with increasing HL percentage and decreasing WPA percentage in WPA: HL mix-ratios. Soils having linear shrinkage value above 8%, between 5 and 8%, and less than 5% possess critical, marginal, and non-critical degree of expansion respectively [51]. The average linear shrinkage for both JIT and MAR native soils were under critical degree of expansion with 17.73 and 20.7%, respectively. The linear shrinkage of both soil samples decrease from 17.73 - 2.24% and from 20.07 - 2.12% for JIT and MAR soil samples respectively, as percentage of HL increase and percentage of WPA decrease in WPA: HL mix-ratios. For JIT and MAR soil samples, 6*% HL + 8% WPA* and *8% HL + 6% WPA* mix proportions had improved the native soil sample into non critical degree of expansion. The rest mix-proportions were not effective to control the shrinkage behavior of the native soil which was under critical degree of expansion.

Therefore, WPA blended with HL had improved the sub-grade soil to be used as a suitable sub-grade soil at 6*% HL + 8% WPA* and *8% HL + 6% WPA* mix-ratios. The reduction in linear shrinkage values are attributed to the pozzolanic reactions between high content of calcium in WPA, and silicates and aluminates from the clay matrix that forms calcium-silicate-hydrate and calcium-aluminate-hydrate. These compounds bind and compact the matrix of clay together to form an interlocking structure which increase the soil strength. In addition, the reduction in linear shrinkage is also responsible to hydration of hydrated lime and cation exchange between Ca^{2+} from WPA and Na^{+} attracted from water and attached to clay surface.

Table 4.10: Effect of hydrated lime blended with waste paper ash on linear shrinkage

	Mix-Proportion (%)	Trials	Length of Mold (cm)	Length of dry specimen (cm)	LS (%)	Average LS (%)	Remark
JIT Soil	Natural Soil	T1	14	11.39	18.64	17.73	Critical
		T2	14	11.29	19.35		
		T3	14	11.87	15.21		
	2% HL + 12% WPA	T1	14	12.28	12.29	11.69	Critical
		T2	14	12.36	11.71		
		T3	14	12.45	11.07		
	4% HL + 10 WPA	T1	14	12.94	7.57	7.22	Marginal
		T2	14	12.98	7.29		
		T3	14	13.05	6.79		
	6% HL + 8% WPA	T1	14	13.58	3.00	2.69	Non-critical
		T2	14	13.64	2.57		
		T3	14	13.65	2.50		
	8% + 6% WPA	T1	14	13.66	2.43	2.24	Non-critical
		T2	14	13.68	2.29		
		T3	14	13.72	2.00		
MAR Soil	Natural	T1	14	11.19	20.07	20.07	Critical
		T2	14	11.21	19.93		
		T3	14	11.17	20.21		
	2% HL + 12% WPA	T1	14	12.20	12.86	12.79	Critical
		T2	14	12.25	12.50		
		T3	14	12.18	13.00		
	4% HL + 10 WPA	T1	14	12.69	9.36	8.8	Critical
		T2	14	12.78	8.71		
		T3	14	12.83	8.34		
	6% HL + 8% WPA	T1	14	13.62	2.71	2.86	Non-critical
		T2	14	13.60	2.86		
		T3	14	13.58	3.00		
	8% + 6% WPA	T1	14	13.67	2.36	2.12	Non-critical
		T2	14	13.7	2.14		
		T3	14	13.72	1.86		

4.3.3. Effect of Hydrated Lime Blended with Waste Paper Ash on Free Swell Index

The free swell index values of treated and untreated JIT and MAR soil samples are given in Table 4.11. These values decreased as percentage of HL increased and percentage of WPA decrease in WPA: HL mix-ratios. For both JIT and MAR soil samples, highest reduction in FSI attained when the sample treated with 6%L+8%WPA and 8%L+6%WPA with the average FSI of 35.18 and 26.5% for JIT soil and 39.95 and 30.72% for MAR soil sample. Sub-grade soil stabilized using

WPA mixed with HL show low degree of expansion as compared to untreated soil. As a result, the soil had a FSI within the allowable requirements. Soils called highly expansive when the free swell index exceeds 50% and such soils undergo volumetric changes leading to pavement distortion, cracking and general unevenness due to seasonal wetting and drying [28].

Generally, both soil samples had changed from high swell to low swell property. This might be due to chemical reaction and cation exchange between the soil and WPA mixed with HL. Therefore, WPA mixed with HL can be used as stabilizing additive agent for sub-grade road construction material.

Table 4.11: Effect of hydrated lime blended with waste paper ash on free swell index

Sample Location	Mix-proportion (%)	Trials	FSI (%)	Average FSI (%)	Reduction (%)	IS 1498 requirement	Remark
JIT	Natural	T1	98.1	88.12	0.00	FSI < 50%	Control
		T2	86.43		11.67		
		T3	79.83		18.27		
	2% HL + 12% WPA	T1	75.46	70.19	22.64		Poor
		T2	70.41		27.69		
		T3	64.72		33.38		
	4% HL + 10% WPA	T1	58.32	48.84	39.78		Poor
		T2	46.27		51.83		
		T3	41.93		56.17		
	6% + 8% WPA	T1	38.60	35.18	59.50		Satisfied
		T2	34.74		63.36		
		T3	32.21		65.89		
	8% + 6% WPA	T1	31.64	26.05	66.46		Satisfied
		T2	26.32		71.78		
		T3	20.18		77.92		
MAR	Natural	T1	104.71	96.83	0.00	FSI < 50%	Control
		T2	96.41		8.30		
		T3	89.36		15.35		
	2% HL + 12% WPA	T1	86.53	77.46	18.18		Poor
		T2	78.31		26.40		
		T3	67.53		37.18		
	4% HL + 10% WPA	T1	61.20	53.93	43.51		Poor
		T2	54.39		50.32		
		T3	46.21		58.50		
	6% + 8% WPA	T1	43.92	39.95	60.79		Satisfied
		T2	38.53		66.18		
		T3	37.41		67.30		
	8% + 6% WPA	T1	35.47	30.72	69.24		Satisfied
		T2	31.43		73.28		
		T3	25.27		79.44		

4.3.4. Effect of Hydrated Lime Blended with Waste Paper Ash on Compaction

The OMC and MDD values of natural sub-grade and stabilized soils by WPA blended with HL are presented in Table 4.12**.** The MDD of untreated soil samples were 1.43 and 1.4 g/cm^3 for JIT and MAR soil samples, respectively. Even though the compaction curve is normal the curve shifted the left upward in the case of treating the soil with WPA-HL, which also means additions of WPA-HL slightly increased the OMC and decreased the MDD for both soil samples. The MDD shows a slight decrease and OMC shows an increase in the treatment of JIT weak sub-grade soil with WPA-HL additive agents (Figure 4.5). The MDD decrease from 1.43 to 1.37 g/cm^3 and OMC increases from 27.98 to 35.72% for JIT soil sample. Similarly, for MAR soil sample, reduction in MDD from 1.4 to 1.34g/cm^3 and rise in OMC from 30.74 to 37.94% was observed. [46] observed similar results for this research on compaction characteristics of expansive clay soil treated with WPA.

The addition of WPA- HL changes the optimum moisture content and maximum dry density of expansive soils because the effects of cation exchange and short-term pozzolanic reactions between HL and the soil results in flocculation and agglomeration of clay particles leading to texture changes. In addition to this, the decrease in density of all treated soils is mainly due to the partial replacement of comparatively heavy soil particles with the light weight and fine particle size of waste paper ash.

Generally, the advantage of the increase in OMC and corresponding decrease in MDD of the soil is to allow compaction to be easily achieved with wet soil. Any adverse effect on strength due to reduction in density is unlikely to occur due to the expected substantial gain in strength of treated soils due to the pozzolanic properties of HL and WPA. The decrease in density may be related to the flocculated and agglomerated clay particles occupying larger spaces leading to a corresponding decrease in dry density, and the effect of WPA addition to soil sample on the specific gravity of soil mixed with different concentration of lime [46].

Table 4.12: Effect of hydrated lime blended with waste paper ash on compaction

Mix-Proportion (%)	Trials	JIT Soil Sample		MAR Soil Sample	
		OMC (%)	MDD (g/cm^3)	OMC (%)	MDD (g/cm^3)
Natural Soil	T1	27.96	1.44	30.51	1.41
	T2	27.89	1.43	30.75	1.40
	T3	28.1	1.42	30.97	1.38
Average		27.98	1.43	30.74	1.40
2% HL + 12% WPA	T1	28.91	1.42	31.91	1.39
	T2	28.89	1.43	32.46	1.37
	T3	28.46	1.40	32.82	1.36
Average		28.75	1.42	32.41	1.37
4% HL + 10% WPA	T1	30.81	1.40	32.74	1.38
	T2	31.32	1.39	32.86	1.36
	T3	31.96	1.38	33.65	1.35
Average		31.36	1.39	33.08	1.36
6% HL + 8% WPA	T1	32.61	1.39	34.86	1.36
	T2	32.78	1.37	34.97	1.35
	T3	34.86	1.38	35.32	1.34
Average		33.42	1.38	35.05	1.35
8% HL + 6% WPA	T1	35.18	1.38	37.73	1.35
	T2	36.20	1.36	37.83	1.34
	T3	35.78	1.37	38.27	1.33
Average		35.72	1.37	37.94	1.34

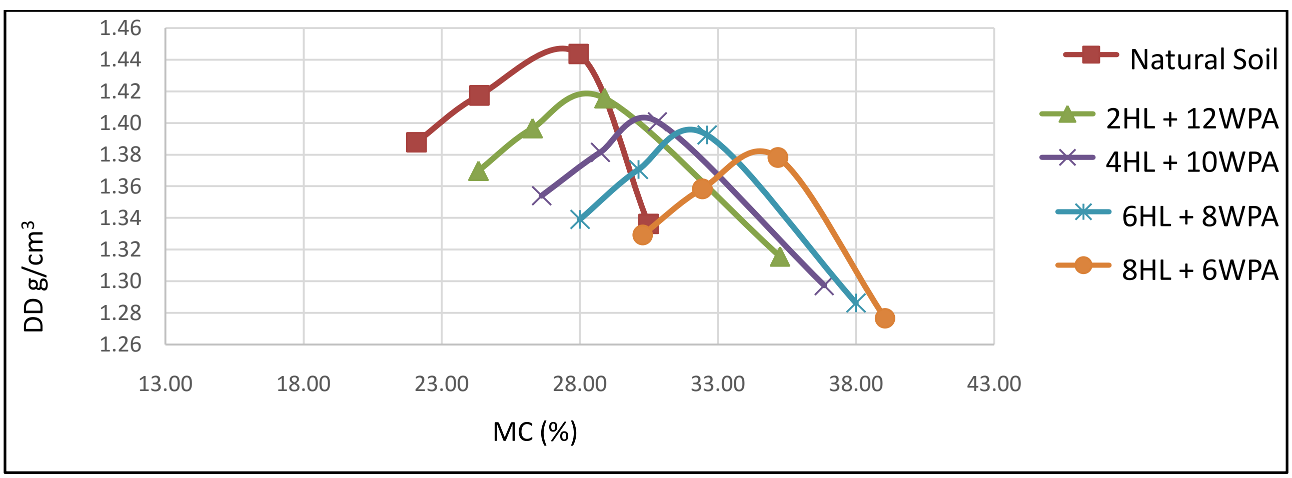

Figure 4.5: Effect of addition of WPA- HL on compaction (JIT soil sample)

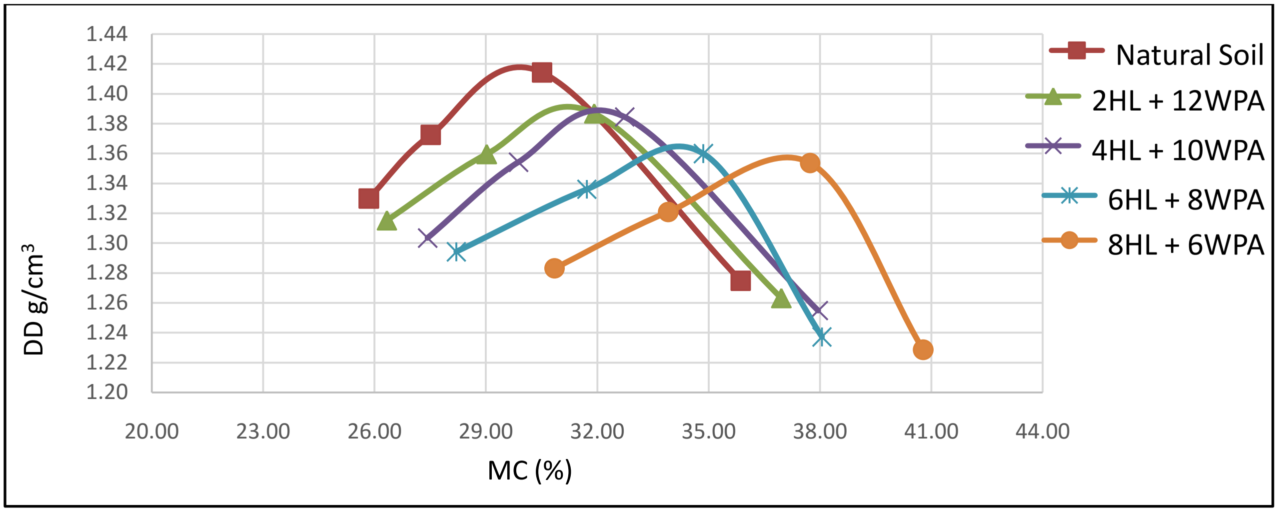

Figure 4.6: Effect of addition of WPA- HL on compaction test @ MAR soil sample

4.3.5. Effect of Lime Blended with Waste Paper Ash on CBR

The CBR values of natural and stabilized sub-grade soils by WPA blended with HL are given in Table 4.13 and the detail laboratory analysis are attached in *Appendix C3* and *Appendix D3*. The CBR values of untreated soil samples were 2.33 and 1.95% for JIT and MAR soil samples, respectively which indicates the soil samples are problematic soil that cannot be recommended for sub-grade material without improvement. The CBR values increased as percentage of HL increased and percentage of WPA decrease in WPA: HL mix-ratios. The CBR value of JIT soil samples increased from 2.33 to 10.18% and MAR soil sample increased from 1.95 to 9.32% which satisfy the ERA requirement. The highest increment was attained when the sample was treated with *4% HL + 10% WPA*, *6% HL + 8% WPA* and *8% HL + 6% WPA* for both soil samples.

The increase in the CBR values with the addition of HL and WPA could be due to the presence of adequate amount of calcium in WPA for the formation of calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) in WPA which are the major compounds responsible for strength gain due to pozzolanic reaction, cation exchange reaction and hydration of HL [46]. Therefore, WPA blended with HL is a suitable stabilizer of expansive soil.

Table 4.13: Effect of hydrated lime blended with waste paper ash on CBR

Sample Location	Mix proportion (%)	CBR (%)			Average CBR (%)	ERA requirement	Remarks
		T1	T2	T3			
JIT	Natural Soil	2.35	2.55	2.1	2.33	> 3%	Poor
	2% HL + 12% WPA	3.21	3.45	2.9	3.19		
	4% HL + 10% WPA	5.84	6.15	5.05	5.68		Satisfied
	6% HL + 8% WPA	8.68	9.22	7.8	8.57		Satisfied
	8% HL + 6% WPA	10.23	10.42	9.9	10.18		Satisfied
MAR	Natural Soil	1.94	2.1	1.8	1.95	> 3%	Poor
	2% HL + 12% WPA	2.83	3.22	2.75	2.93		
	4% HL + 10% WPA	4.76	4.9	3.9	4.52		Satisfied
	6% HL + 8% WPA	8.14	8.92	7.15	8.07		Satisfied
	8% HL + 6% WPA	9.14	9.77	9.05	9.32		Satisfied

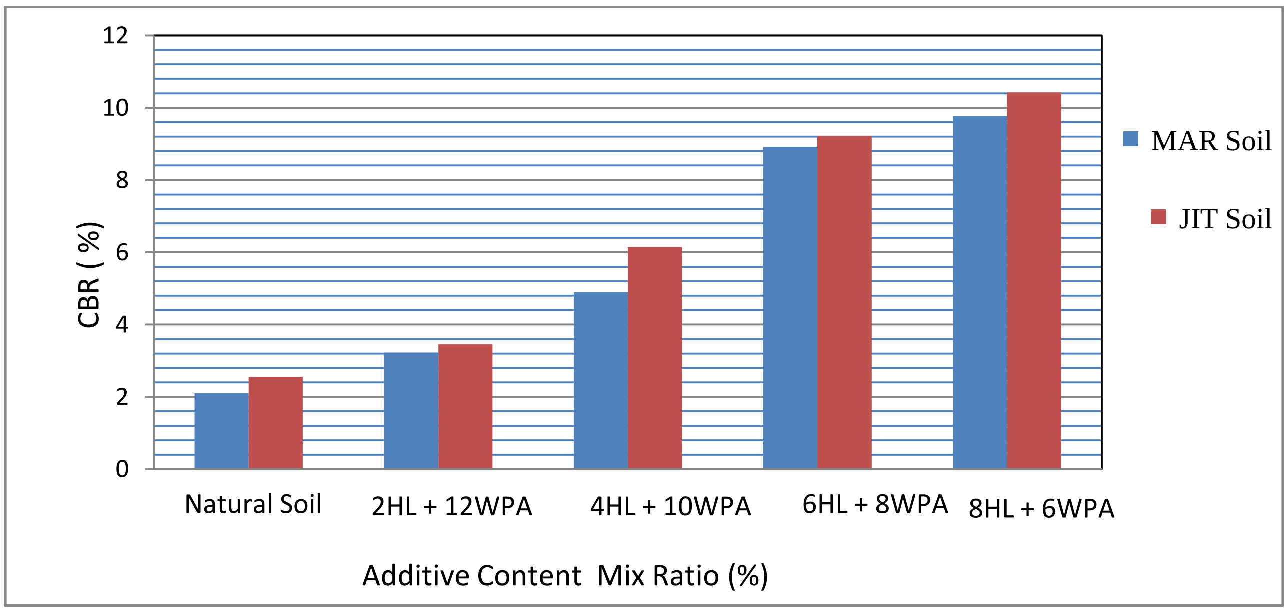

Figure 4.7: Effect of hydrated lime blended with waste paper ash on CBR

4.3.6. Effect of Lime Blended with Waste Paper Ash on CBR Swell

The CBR swell values of JIT and MAR soil samples are given in Table 4.14 and Figure 4.8.

JIT and MAR untreated soil samples have high swelling percentage of 3.34 and 4.18%, respectively. These values indicate the soils are problematic soil that cannot be recommended as sub-grade material without treatment. The highest reduction was attained when the sample was treated with *4%HL+10%WPA*, *6%HL+8%WPA* and *8%HL+6%WPA* for both soil samples. The CBR swell of the soils reduced as percentage of HL increase and percentage of WPA decrease in WPA: HL mix-ratios from 3.34 to 1.20% and 4.18 to 1.24% for JIT and MAR soil samples respectively so that it fulfill ERA requirement to be used as suitable sub-grade soil.

The reduction in CBR Swell was due to cation exchange and pozzolanic reactions between calcium from WPA and silica and alumina found in clay soil particles and variation in clay mineralogy of the expansive soils. This was happened due to replacement of some of the volume that was previously occupied by expansive clay minerals (montomorillite and illite clay minerals) by calcium obtained abundantly from WPA. Therefore, using both stabilizers improve the stability and strength of the sub-grade soils. [46] in his study on effect of waste paper ash on the strength of expansive soil found similar trend with the relationship of WPA content and swelling potential in terms CBR swell but lesser CBR swell value than this research findings. This could be due to variation in clay mineralogy of the expansive soils.

Table 4.14: Effect of hydrated lime blended with WPA on CBR swell

Sample Location	Mix-Proportion (%)		CBR Swell (%)			Average CBR swell (%)	ERA Requirement	Remark
	HL	WPA	T1	T2	T3			
JIT Soil	0	0	3.52	3.36	3.13	3.34	CBR swell < 2%	Poor
	2	12	3.15	2.93	2.72	2.93		Poor
	4	10	2.28	2.25	1.18	1.90		In range
	6	8	2.17	1.78	0.65	1.53		In range
	8	6	1.76	1.23	0.62	1.20		In range
MAR Soil	0	0	4.41	4.16	3.97	4.18	CBR swell < 2%	Poor
	2	12	3.52	3.06	2.45	3.01		Poor
	4	10	2.65	2.12	1.10	1.96		In range
	6	8	2.24	1.98	0.76	1.66		In range
	8	6	1.87	1.32	0.54	1.24		In range

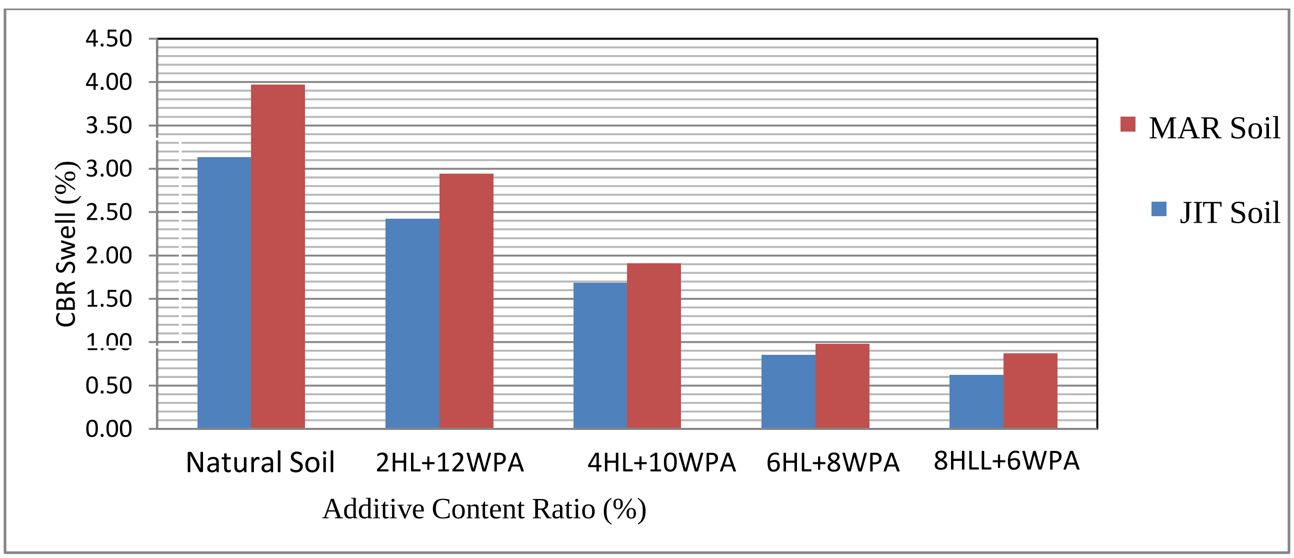

Figure 4.8: Effect of Lime Blended with Waste Paper Ash on CBR Swell

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusion

Conclusions of this study, end to end with recommendation for the future development of research along the same lines are presented under this chapter. Initially, the full thesis is briefly reviewed and then discussion is made of the objectives achieved by the research and their contribution to knowledge and practical works. Based on the laboratory test results obtained in the experimental investigation, the following conclusions have been drawn:

- The soil types were highly expansive and had high degree of expansion, high plastic index and poor strength. According to USCS and AASHTO soil classification system, both JIT and MAR soil samples are categorized as CH and A-7-5. Thus, these two natural soil were very poor in strength which is problematic to be used as a sub-grade material as per ERA (2002) specification. The engineering properties of the studied expansive soil revealed that these two soil samples are not suitable to use as a sub-grade material unless its undesirable properties are improved.
- The physical properties of waste paper ash were investigated and were found suitable for stabilization. Moreover, waste paper ash satisfies the minimum requirement of natural Pozzolanic materials to be used as a mineral admixture specified by ASTM (C 618-00) having higher percentage of calcium fly ash of calcium carbonate or free lime content (CaO) about 62.39%. This is adequate to meet the requirement of ASTM C-618 standard for pozzolanic materials (Class-C fly ash) because of its self cementing characteristics which is satisfactory to encounter as pozzolanic material.
- The liquid limit, Plastic limit, and plastic index were improved to be in the range of sub-grade material. The increment of WPA-HL content in natural soil, the value of liquid limit decrease while, plastic limit increased and plasticity index of treated soil was reduced satisfactorily. Plastic limits of both soil samples increased when the percentage of lime was higher than waste paper ash in the mix-ratio. The plasticity index of JIT soil sample reduced from 39.00 to 10.5% at *6%HL+8%WPA* mix ratio and from 39.00 to 7.00% *at 8%L+6%WPA* mix ratios that fulfils the ERA requirement. Similarly, the plasticity index of

MAR soil sample reduced from 41.80 to 12.70% at 6%HL+8%WPA mix ratio and from 41.80 to 8.50% at 8%HL+6%WPA mix ratios. This reduction is responsible to pozzolanic reactions between calcium from WPA in high content, hydration of HL, cation exchange between Ca^{2+} from WPA and Na^{+} found in expansive soil in the form of NaCl.

- The values for the maximum dry density were noted to decrease with the increase of hydrated lime content and decrease of WPA in mix-ratio and the OMC was found to increase. However, the value for the maximum dry density was noted to decrease with higher lime percentage rather than waste paper ash in the mix-ratio. The decrease in density is related to the flocculated and agglomerated clay particles occupying larger spaces leading to a corresponding decrease in dry density, and the effect of WPA addition to soil sample on the specific gravity of soil mixed with different concentration of lime.
- The addition of lime-WPA additive content improved the CBR values of both JIT and MAR soil samples. Hence combination of lime and WPA can strongly improve the strength of the expansive soil.
- Results indicated that CBR values of treated soils with lime-WPA mix increases as the quantity of lime increases from 2 to 8% and as quantity of WPA decreases from 12 to 6% in blending proportions. The CBR value of JIT soil samples increased from 2.33 to 10.18% and that of MAR soil sample increased from 1.95 to 9.32% which satisfy the ERA requirement. The increase in the CBR values with the addition of HL and WPA could be due to the presence of adequate amount of calcium in WPA for the formation of calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) in WPA which are the major compounds responsible for strength gain due to pozzolanic reaction, cation exchange reaction and hydration of HL.
- Generally, the most parameters of ERA (2013) specifications requirement were improved by hydrated lime combined with waste paper ash in different mix-proportions. The adequate mix proportion percentage in all parameters achieved at *6%HL+8%WPA* and *8%HL+6%WPA* mix-proportions.

5.2. Recommendations

Based on the findings of this research, the following recommendations were forwarded:

- Additional investigation should be recommended to investigate the effect of additional curing time and aging effect of soil stabilized by WPA.

- Comparative lime and waste paper ash expansive soil stabilization should be studied to determine the performance of waste paper ash as stabilizing agent relative to lime.

- Waste paper ash investigated in this research study was prepared manually and approved as a stabilizer of expansive soil blended with lime. Therefore, higher education sector and government entities should aware about this potential soil stabilizing material and promote its standardized production and usage.

- The current study was conducted by taking limited parameter such as consistency limit, free swell index, linear shrinkage, moisture density relation, CBR and CBR swell potential on expansive soil sample using lime as hydraulic stabilizer and that of waste paper ash as non-hydraulic stabilizer. additional test parameter like unconfined compressive strength, PH value test, volumetric shrinkage and mineralogical tests should also be performed to have more accurate test results

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APPENDIX

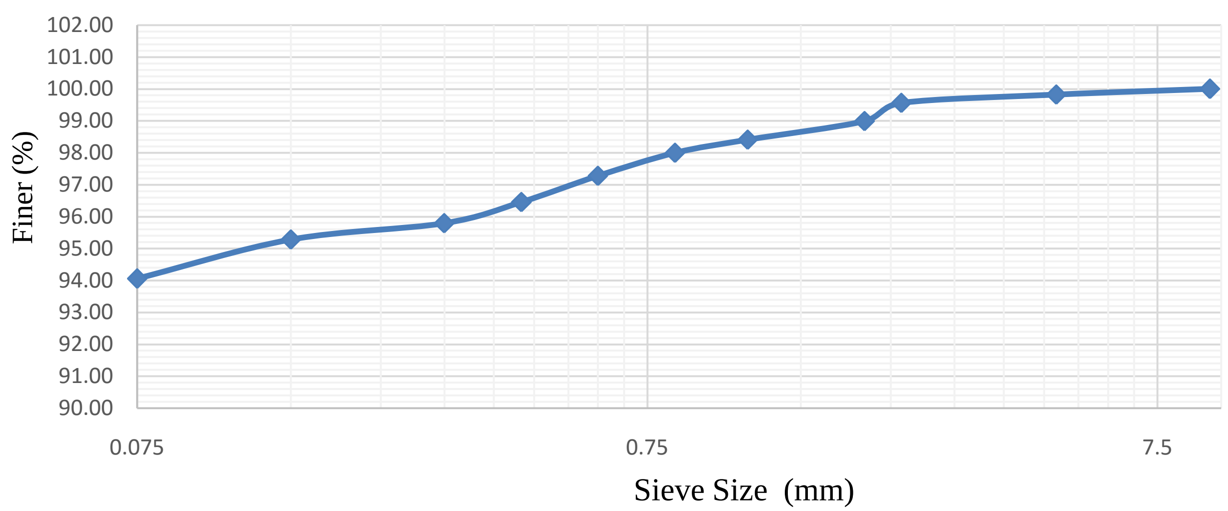
Appendix A: Laboratory Test Result of Natural JIT soil Sample

A1) Natural Moisture Content

Natural moisture Content for JIT Soil Sample	Number of Trials		
Moisture content determination	1	2	3
Container No	P-15	A	D
Mass of wet soil + container (A)	108.70	77.90	99.70
Mass of dry soil + container (B)	85.50	59.70	78.80
Mass of container (C)	33.20	19.40	34.50
Mass of moisture = A-B = (D)	23.20	18.20	20.90
Mass of dry soil = B-C =(E)	52.30	40.30	44.30
Natural Moisture content (%) = D/E*100 =(F)	44.36	45.16	47.18
Average Moisture Content (%)	**45.57**		

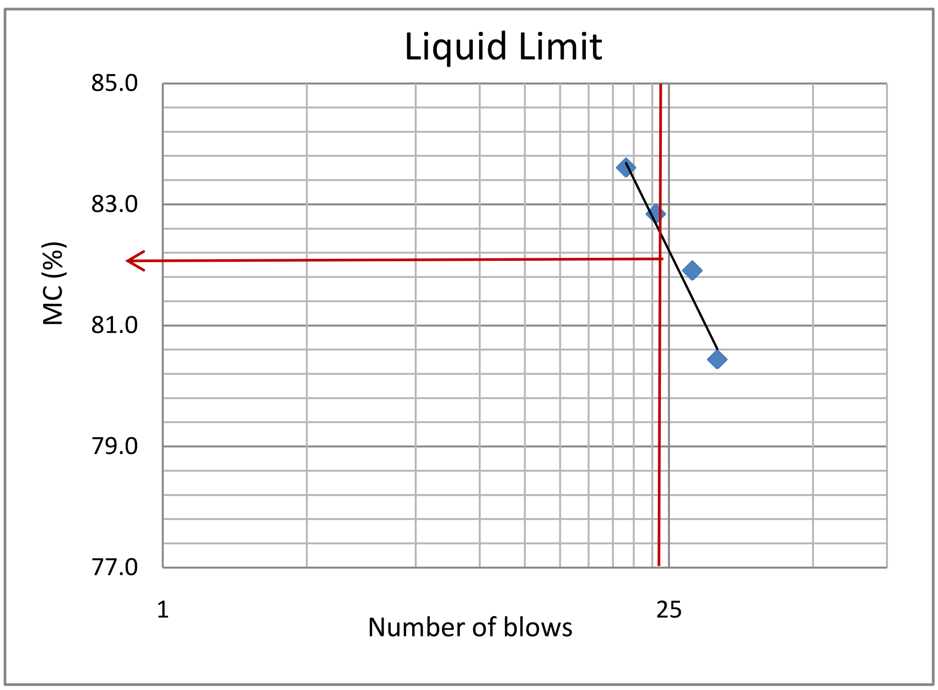
A2) Grain Size Analysis

Natural Soil Sample @JIT				
Sieve size (mm)	mass of retained soil (g)	Percent of retained soil	cumulative % of retain soil	percent of passing particle
9.5	0.00	0.00	0.00	100.00
4.75	1.75	0.18	0.18	99.83
2.36	2.62	0.26	0.44	99.56
2	5.70	0.57	1.01	98.99
1.18	5.80	0.58	1.59	98.41
0.85	4.12	0.41	2.00	98.00
0.6	7.20	0.72	2.72	97.28
0.425	8.20	0.82	3.54	96.46
0.3	6.63	0.66	4.20	95.80
0.15	5.10	0.51	4.71	95.29
0.075	12.30	1.23	5.94	94.06
pan	940.58	94.06	100.00	0.00
sum	1000.0			



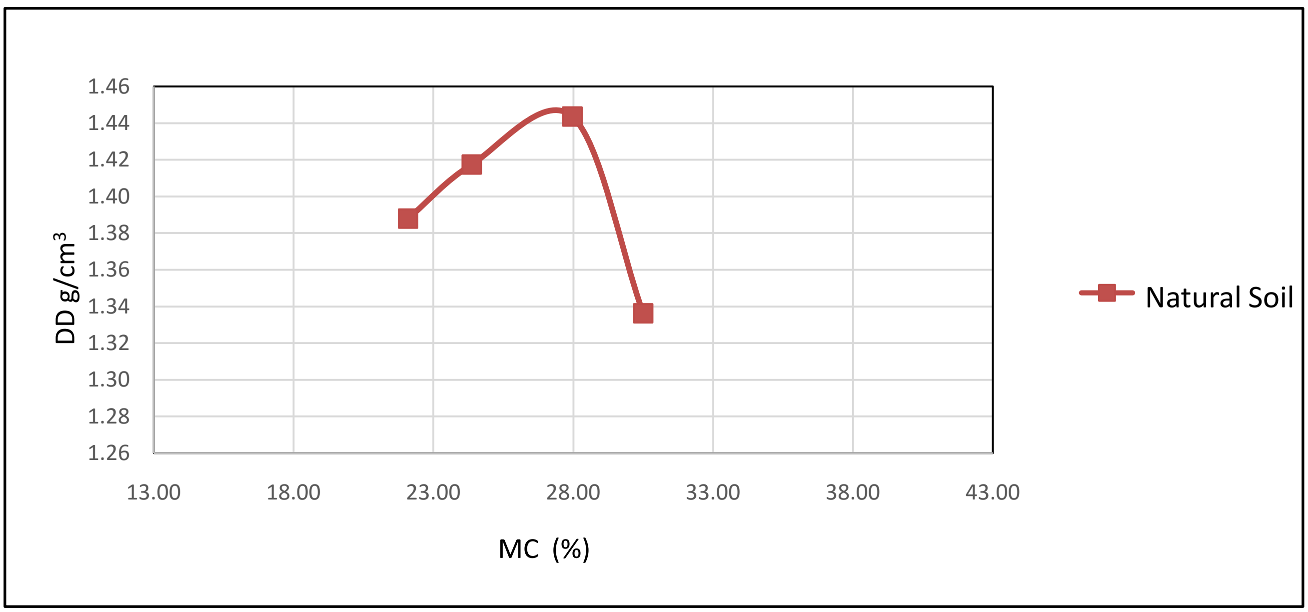
A3) Atterberg Limit Test Result of *Natural Soil Sample @ JIT*

Natural Soil Sample @ JIT							
Determination	Liquid Limit				Plastic Limit		
Number of blows	34	29	23	19			
Trial	T1	T2	T3	T4	T1	T2	T3
Container Code	A4	3L	R34	T4	A16	12L3	B9
Wt. of container + wet soil, g	17.93	34.69	23.92	21.46	21.9	12.6	35.79
Wt. of container + dry soil, g	12.79	27.9	16	14.42	20.51	10.73	31
Wt. of container, g	6.4	19.61	6.44	6	17.24	6.5	19.85
Wt. of water, g	5.14	6.79	7.92	7.04	1.4	1.87	4.79
Wt. of dry soil, g	6.39	8.29	9.56	8.42	3.27	4.23	11.15
Moisture content,%	80.4	81.9	82.8	83.6	42.5	44.2	43.0
Liquid Limit (LL),%	**82.2**						
Plastic Limit (PL),%	**43.2**						
Plasticity Index (PI), %	**39.0**						



A4) Compaction Laboratory Test Result of *Natural Soil Sample @JIT*

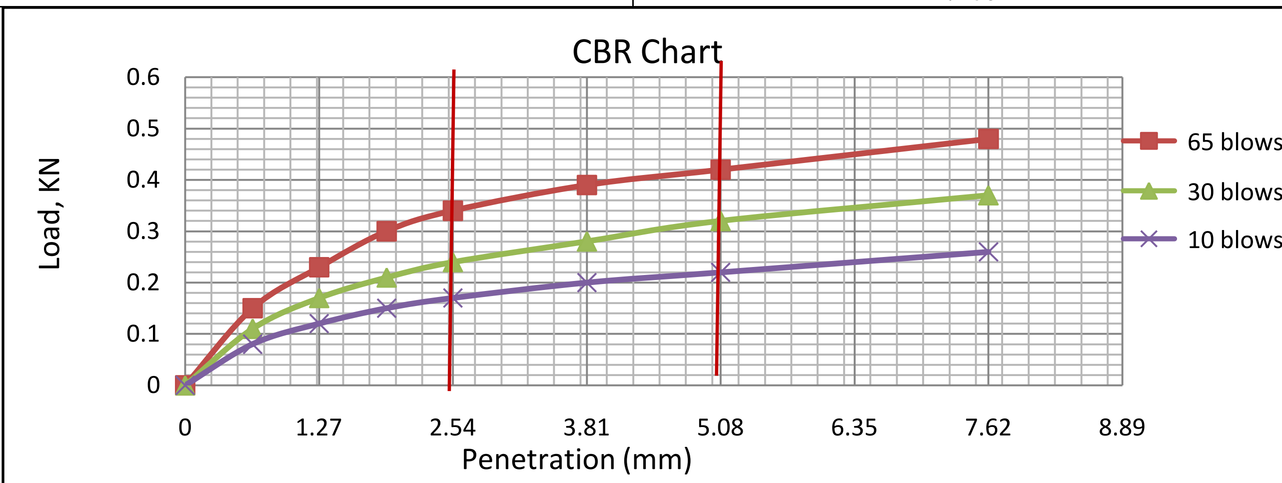
Density Determination				
Test No.	1	2	3	4
Mass of sample (g)	5000	5000	5000	5000
Water Added(cm^3)	450	650	850	1050
Mass of Mold + Wet soil(g)(A)	9764.6	9909.86	10089.2	9869.06
Mass of Mold(gm)(B)	6165.5	6165.5	6165.5	6165.5
Mass of Wet Soil(gm)A-B=C	3599.1	3744.36	3923.7	3703.56
Volume of Mold cm^3(D)	2124.00	2124.00	2124.00	2124.00
Bulk Density g/cm^3 C/D=(E)	1.69	1.76	1.85	1.74
Moisture Content Determination				
Container Code	E	ZE	T	T55
Mass of Wet soil + Container (g)(F)	198.05	205.35	190.97	105.43
Mass of dry soil + container(g)(G)	169	171.6	157	85
Mass of container(gm)(H)	37.5	33.1	35.52	18
Mass of moisture(gm)F-G=(I)	29.05	33.75	33.97	20.43
Mass of Dry soil(gm)G-H=(J)	131.5	138.5	121.48	67
Moisture content % (I/J)*100=K	22.09	24.37	27.96	30.49
Dry Density gm/cm^3 E/(100+K)*100	1.39	1.42	1.44	1.34

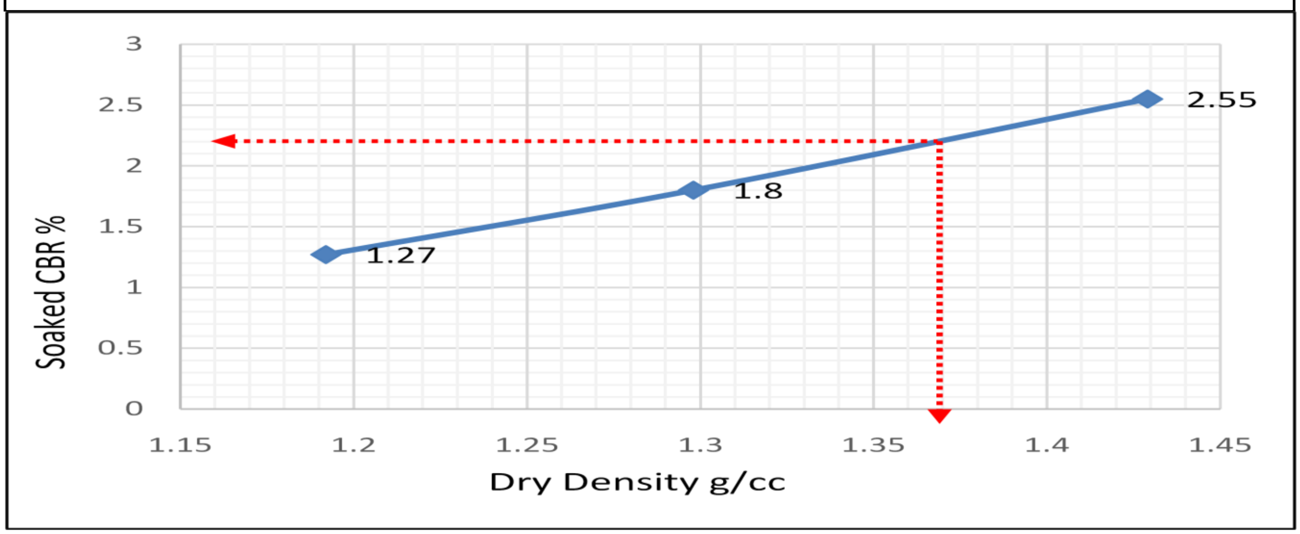


A5) CBR Laboratory Test Result Analysis Data of *Natural Soil Sample @ JIT*

PENETRATION AND LOAD DETERMINATION OF NATURAL SOIL @ JIT						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65- Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.54	0.34	2.55	0.24	1.80	0.17	1.27
5.08	0.42	2.10	0.32	1.6	0.22	1.10

CBR RESULT SUMMARY OF NATURAL SOIL @ JIT			
MMDD (g/cm^3)	1.44		
Dry Density at 95% of MDD (g/cm^3)	1.368		
No of Blows	65	30	10
CBR Values (%)	2.55	1.80	1.27
DDBS (g/cm^3)	1.192	1.298	1.429
CBR at 95% MDD	**2.2%**		





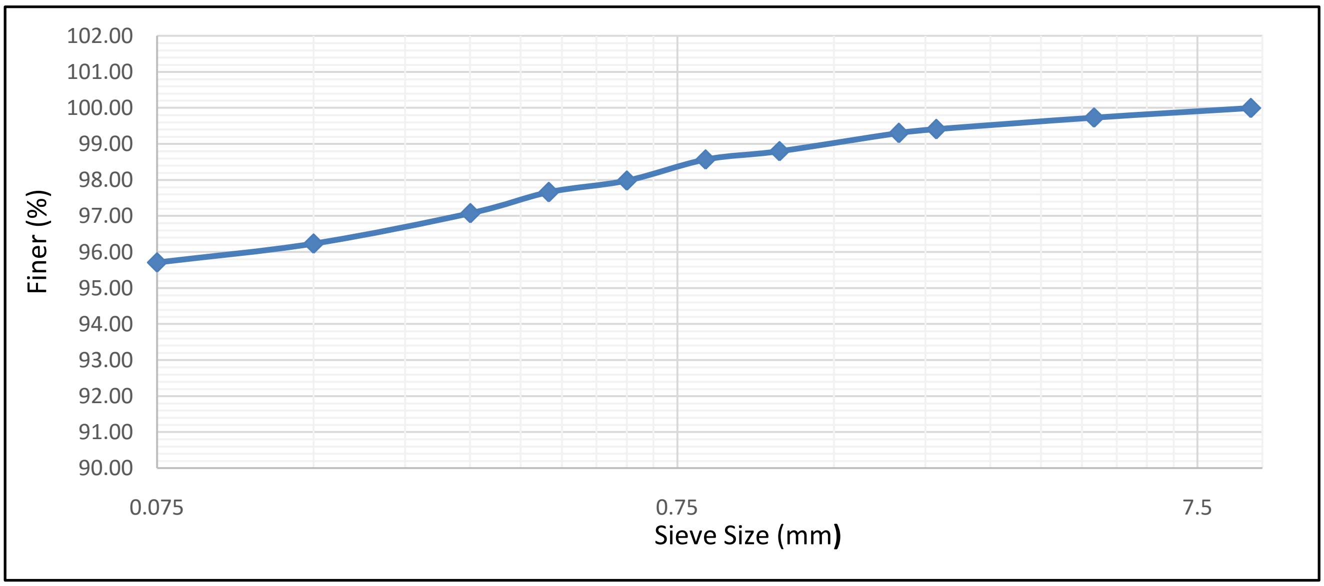
Appendix B: Laboratory Test Result *of* Natural soil Sample @ MAR

B1) Natural Moisture Content

Natural moisture Content for MAR Soil Sample	Number of Trials		
Moisture content determination	1	2	3
Container No	C	10G	P-1
Mass of wet soil + container (A)	89.00	68	81.4
Mass of dry soil + container (B)	66.2	52.1	61.4
Mass of container (C)	17.30	17.6	17.8
Mass of moisture = A-B = (D)	22.80	15.90	20.00
Mass of dry soil = B-C =(E)	48.90	34.50	43.60
Natural Moisture content (%) = D/E*100 =(F)	46.63	46.09	45.87
Average Moisture Content (%)	**46.20**		

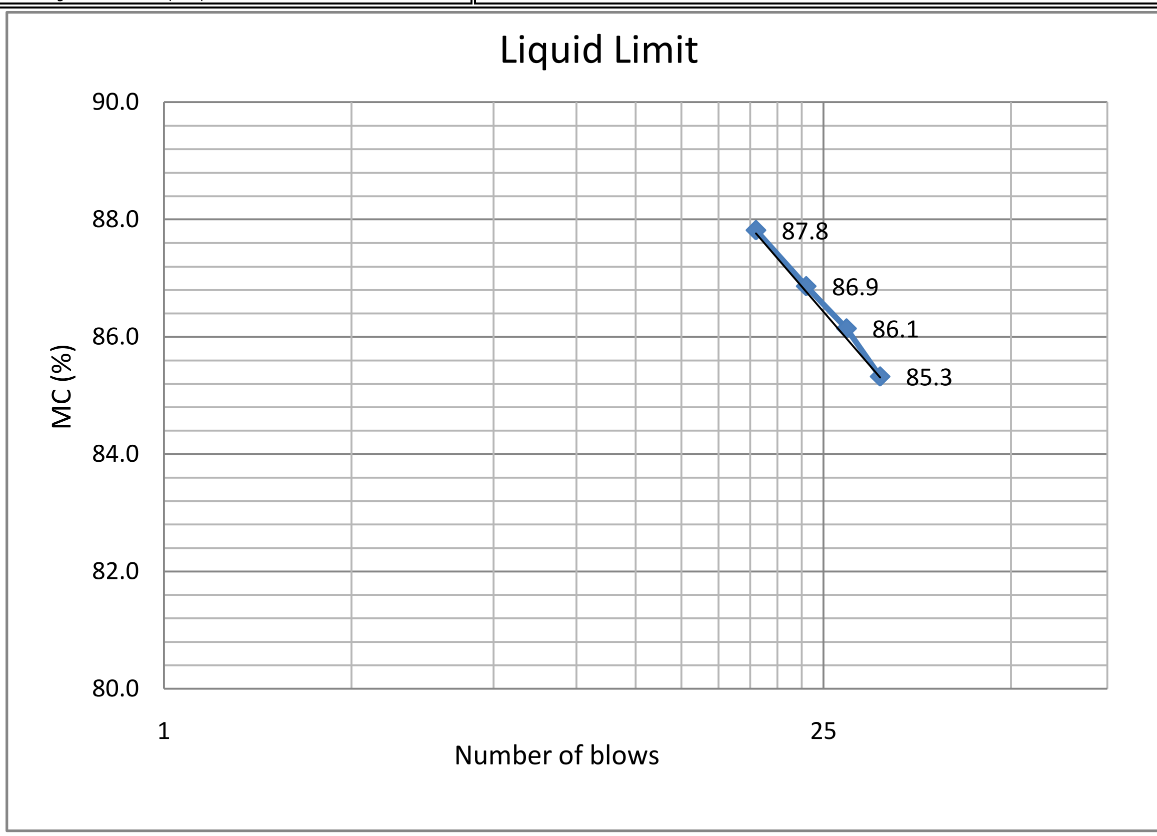
B2) Grain Size Analysis

Natural Soil Sample @MAR				
Sieve size (mm)	mass of retained soil (g)	Percent of retained soil	cumulative % of retained soil	percent of passing particle
9.5	0.00	0.00	0.00	100.00
4.75	2.67	0.27	0.27	99.73
2.36	3.26	0.33	0.59	99.41
2	1.06	0.11	0.70	99.30
1.18	5.01	0.50	1.20	98.80
0.85	2.33	0.23	1.43	98.57
0.6	5.84	0.58	2.02	97.98
0.425	3.23	0.32	2.34	97.66
0.3	5.84	0.58	2.92	97.08
0.15	8.43	0.84	3.77	96.23
0.075	5.24	0.52	4.29	95.71
Pan	957.09	95.71	100.00	0.00
Sum	1000.0			



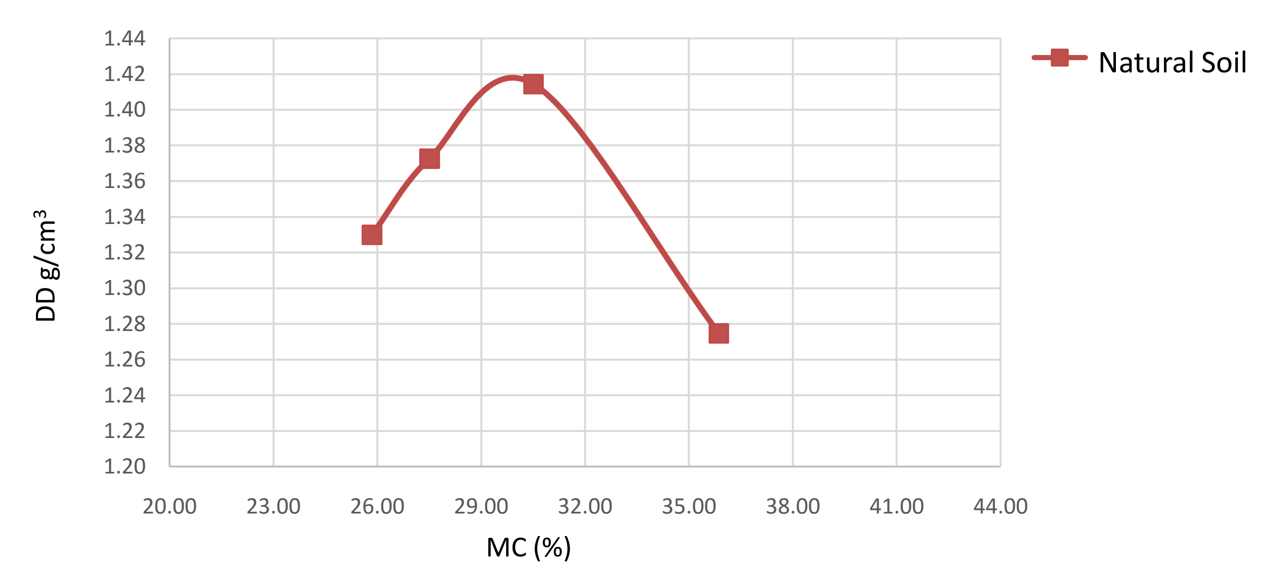
B3) Atterberg Limit Test Result of Natural Soil Sample @ MAR

Natural Soil Sample @RAM							
Determination	Liquid Limit				Plastic Limit		
Number of blows	33	28	23	18			
Trial	T1	T2	T3	T4	T1	T2	T3
Container Code	L1	A-3	H2	T1	N4	3L	G2
Wt. of container + wet soil, g	35.98	40.43	38.71	37.36	26.82	12.6	35.79
Wt. of container + dry soil, g	27.49	29.86	28.99	27.99	24.51	10.73	31
Wt. of container, g	17.54	17.59	17.8	17.32	19.59	6.5	19.85
Wt. of water, g	8.49	10.57	9.72	9.37	2.3	1.87	4.79
Wt. of dry soil, g	9.95	12.27	11.19	10.67	4.92	4.23	11.15
Moisture content, %	85.3	86.1	86.9	87.8	47.0	44.2	43.0
Liquid Limit (LL), %	86.5						
Plastic Limit (PL),%	44.7						
Plasticity Index (PI), %	41.8						



B4) Compaction Laboratory Test Result of Natural Soil Sample @ MAR

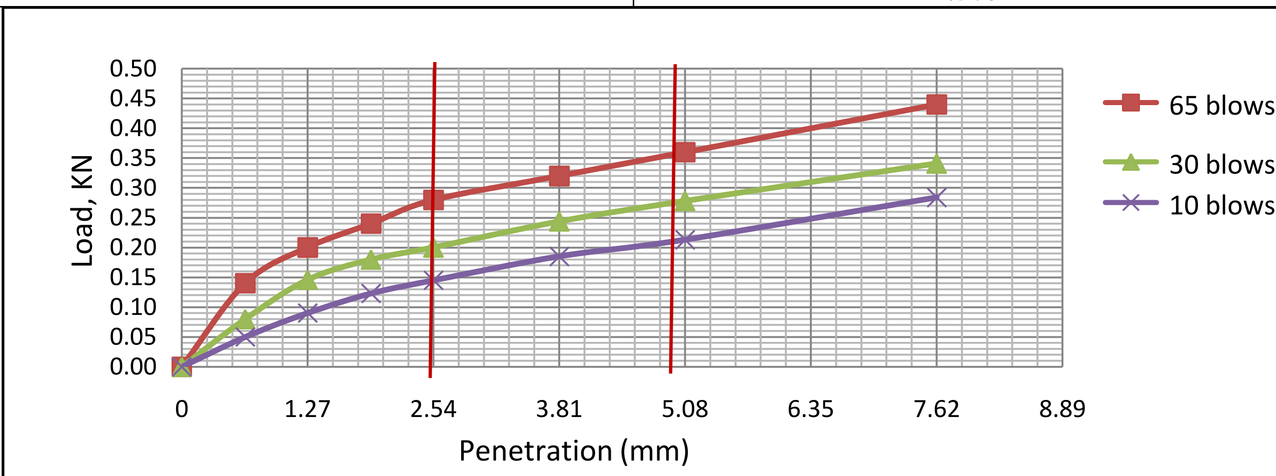
Density Determination				
Test No.	1	2	3	4
Mass of sample (g)	5000	5000	5000	5000
Water Added(cm^3)	450	650	850	1050
Mass of Mold + Wet soil(g)(A)	10131.2	10288.9	10486.2	10251.2
Mass of Mold(g)(B)	6683.3	6683.3	6683.3	6683.3
Mass of Wet Soil(g)A-B=C	3447.9	3605.6	3802.9	3567.9
Volume of Mold cm^3(D)	2060.24	2060.24	2060.24	2060.24
Bulk Density g/cm^3 C/D=(E)	1.67	1.75	1.85	1.73
Moisture Content Determination				
Container Code	A16	A13	D	C4
Mass of Wet soil + Container (g)(F)	149.91	161.82	147.77	190.99
Mass of dry soil + container (g)(G)	125.93	134.81	120.15	145.23
Mass of container(g)(H)	33.15	36.63	29.63	17.63
Mass of moisture(g)F-G=(I)	23.98	27.01	27.62	45.76
Mass of Dry soil(g)G-H=(J)	92.78	98.18	90.52	127.6
Moisture content % (I/J)*100=K	25.85	27.51	30.51	35.86
Dry Density m/cm^3 E/(100+K)*100	1.33	1.37	1.41	1.27

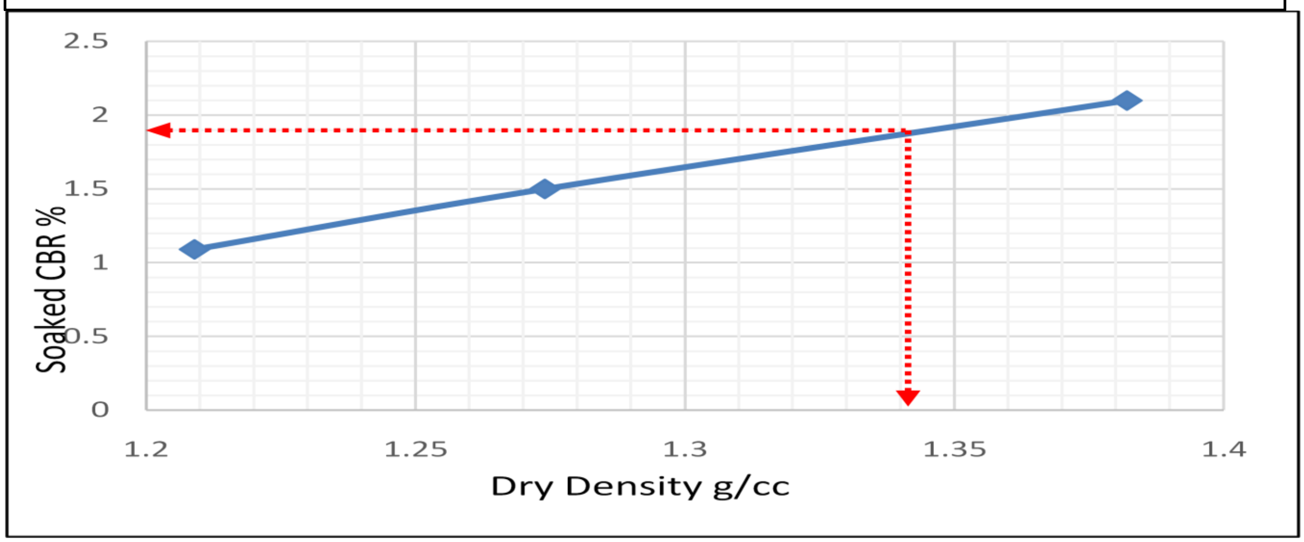


B5) CBR Laboratory Test Result of *Natural Soil Sample @ MAR*

PENETRATION AND LOAD DETERMINATION OF NATURAL SOIL @ MAR						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65- Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.54	0.28	2.10	0.20	1.50	0.15	1.09
5.08	0.36	1.80	0.28	1.39	0.21	1.07

CBR RESULT SUMMARY OF NATURAL SOIL @ JIT			
MMDD (g/cm^3)	1.41		
Dry Density at 95% of MDD (g/cm^3)	1.340		
No of Blows	65	30	10
CBR Values (%)	2.1	1.5	1.09
DDBS (g/cm^3)	1.382	1.274	1.209
CBR at 95% MDD	**1.9%**		



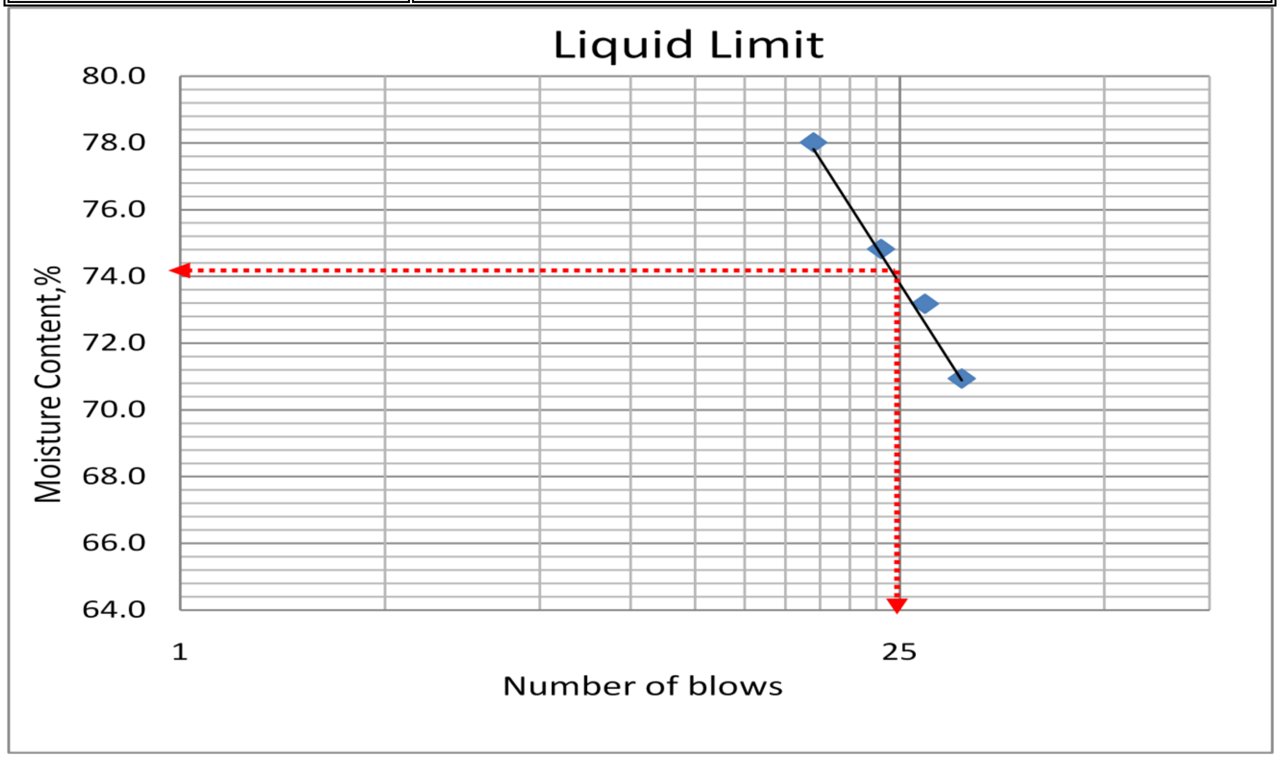


Appendix C: Laboratory Test Result of Stabilized JIT Soil Sample Using WPA Mixed with Lime

C1) Atterberg Limit Test

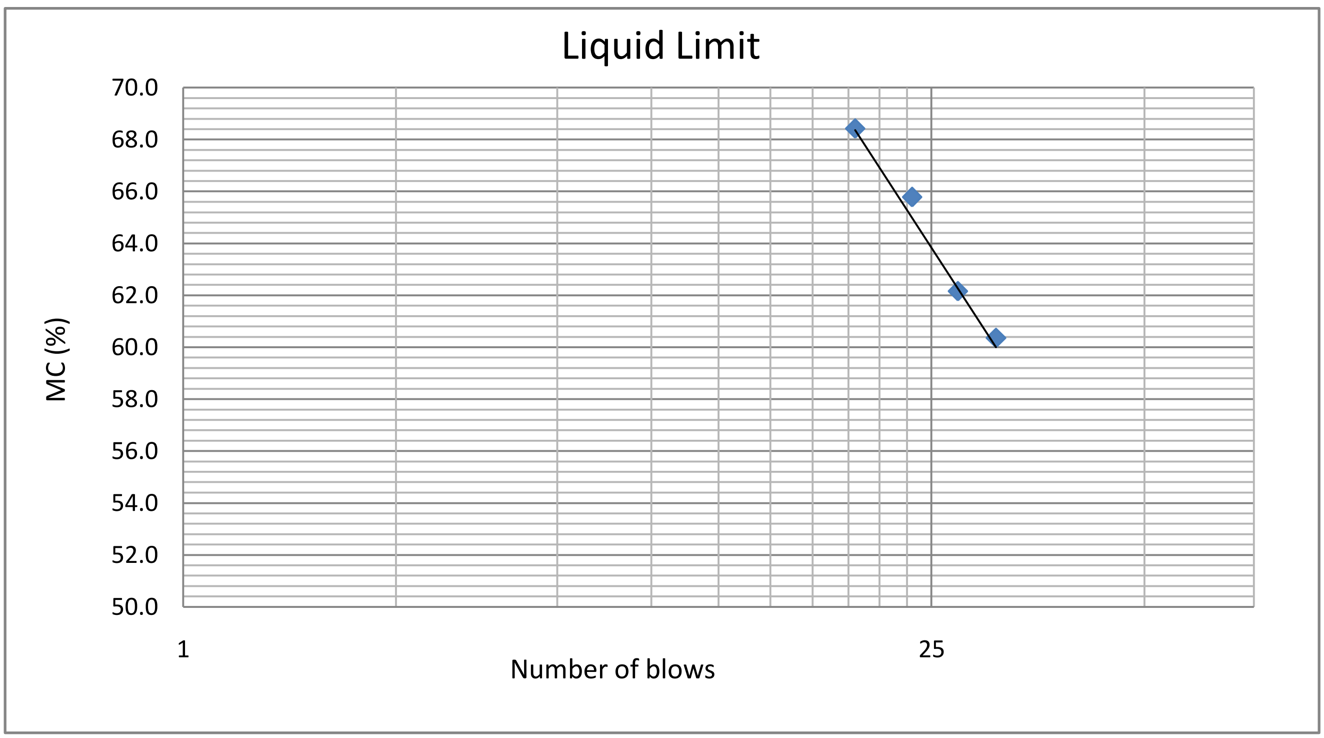
C 1.1) *2% Lime, 12% WPA*

2% lime + 12% WPA @ JIT							
Determination	Liquid Limit				Plastic Limit		
Number of blows	33	28	23	17			
Trial	T1	T2	T3	T4	T1	T2	T3
Container Code	A12	A14	4-Z	3-Z	A4	T1	A13
Wt. of container + wet soil, g	29.87	35.85	40.63	36.16	21.39	10.13	10.84
Wt. of container + dry soil, g	24.28	28.4	33.17	27.5	20.05	8.95	9.4
Wt. of container, g	16.4	18.22	23.2	16.4	16.95	6.15	6.2
Wt. of water, g	5.59	7.45	7.46	8.66	1.3	1.18	1.44
Wt. of dry soil, g	7.88	10.18	9.97	11.1	3.1	2.8	3.2
Moisture content,%	70.9	73.2	74.8	78.0	43.2	42.1	45.0
Liquid Limit (LL),%	**74.2**						
Plastic Limit (PL),%	**43.5**						
Plasticity Index (PL), %	**30.8**						



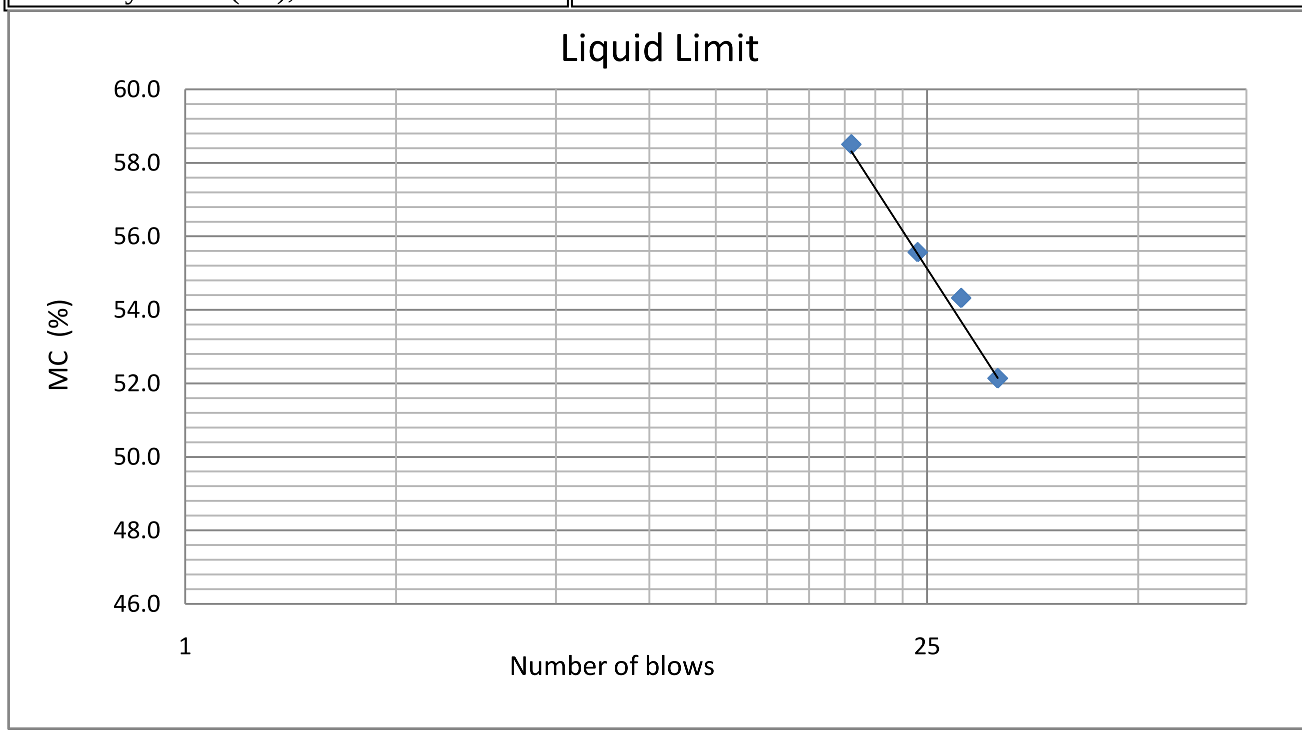
C1.2) *4% Lime, 10% WPA*

4% lime + 10% WPA @JIT							
Determination	Liquid Limit				Plastic Limit		
Number of blows	**33**	**28**	**23**	18			
Trial	T1	T2	T3	T4	T1	T2	T3
Container Code	1	A-7	B9	A36	G8	G21	A6
Wt. of container + wet soil, g	33.02	33.41	35.27	34.14	22.73	24	10.58
Wt. of container + dry soil, g	27.92	28.17	29.08	27.4	21.46	22.64	9.4
Wt. of container g	19.47	19.74	19.67	17.55	18.75	19.83	6.2
Wt. of water, g	5.1	5.24	6.19	6.74	1.3	1.36	1.18
Wt. of dry soil, g	8.45	8.43	9.41	9.85	2.71	2.81	3.2
Moisture content,%	60.4	62.2	65.8	68.4	46.9	48.4	36.9
Liquid Limit (LL),%	**64.2**						
Plastic Limit (PL),%	**44.0**						
Plasticity Index (PL), %	**20.1**						



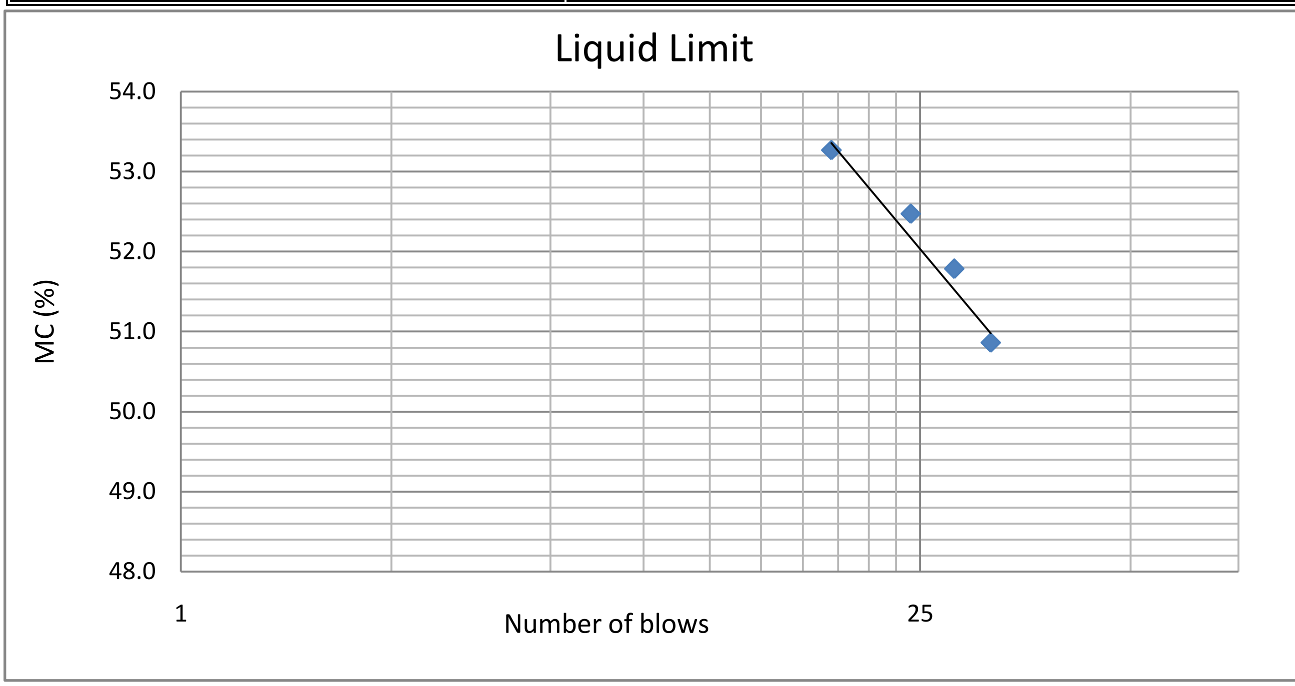
C1.3) *6% Lime, 8% WPA*

6% Lime + 8% WPA @ JIT							
Determination	Liquid Limit				Plastic Limit		
Number of blows	34	29	24	18			
Trial	T1	T2	T3	T4	T1	T2	T3
Container Code	A	T4	L3	A16	A	3L	N3
Wt. of container + wet soil, g	25.67	17.43	18.56	29.51	14.99	27.51	35
Wt. of container + dry soil, g	19.08	13.41	14.27	24.97	12.39	24.98	30.3
Wt. of container, g	6.44	6.01	6.55	17.21	6.39	19.6	19.52
Wt. of water, g	6.59	4.02	4.29	4.54	2.6	2.53	4.7
Wt. of dry soil, g	12.64	7.4	7.72	7.76	6	5.38	10.78
Moisture content,%	52.1	54.3	55.6	58.5	43.3	47.0	43.6
Liquid Limit (LL),%	**55.1**						
Plastic Limit (PL),%	**44.7**						
Plasticity Index (PL), %	**10.5**						



C1.4) *8% Lime, 6% WPA @ JIT*

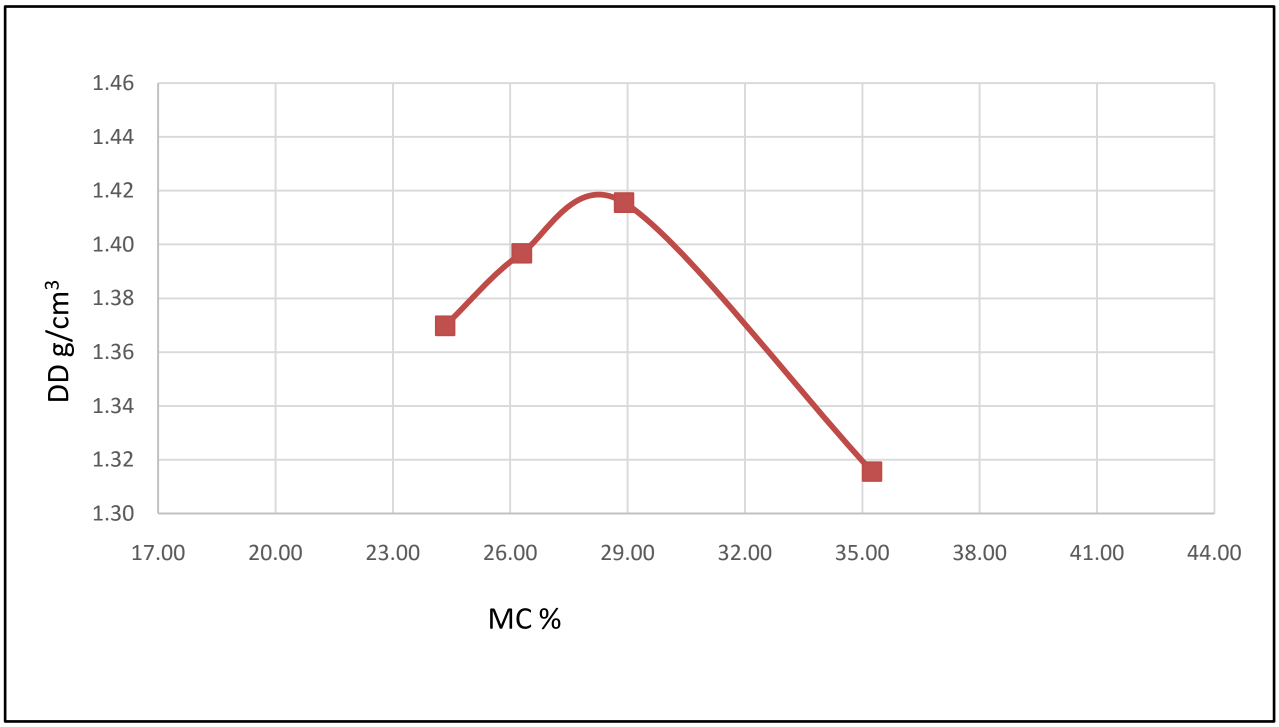
8% Lime + 6% WPA @JIT							
Determination	Liquid Limit				Plastic Limit		
Number of blows	34	29	24	17			
Trial	T1	T2	T3	T4	T1	T2	T3
Container Code	T63	P	A14	C9	I3	J1	W2
Wt. of container + wet soil, g	17.8	20.08	32.4	33.16	24.8	24.62	24.02
Wt. of container + dry soil, g	13.97	15.58	27.52	27.54	22.92	22.39	21.54
Wt. of container, g	6.44	6.89	18.22	16.99	19.14	17.95	14.52
Wt. of water, g	3.83	4.5	4.88	5.62	1.9	2.23	2.48
Wt. of dry soil, g	7.53	8.69	9.3	10.55	3.78	4.44	7.02
Moisture content,%	50.9	51.8	52.5	53.3	49.7	50.2	35.3
Liquid Limit (LL),%	**52.1**						
Plastic Limit (PL),%	**45.1**						
Plasticity Index (PL), %	**7.0**						



C2) Compaction Laboratory Test Result

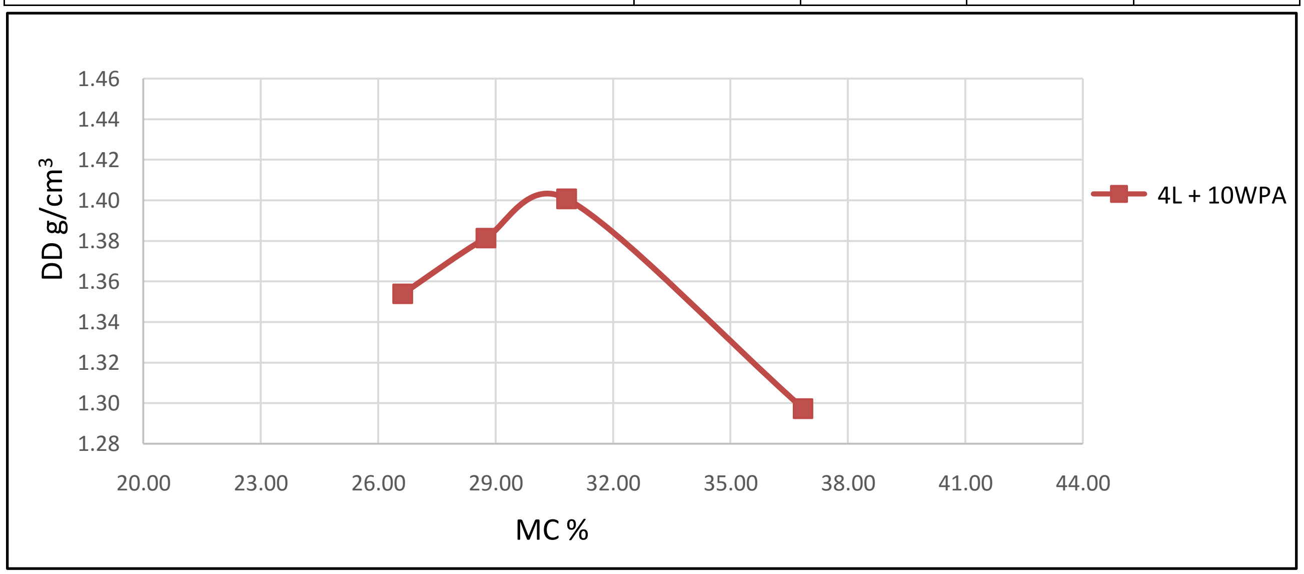
C2.1) 2% Lime, 12% WPA @JIT

Density Determination				
Test No.	1	2	3	4
Mass of sample (g)	5000	5000	5000	5000
Water Added(cm^3)	450	650	850	1050
Mass of Mold + Wet soil(g)(A)	6338.1	6467.4	6596.4	6499.9
Mass of Mold(g)(B)	2720.5	2720.5	2720.5	2720.5
Mass of Wet Soil(g)A-B=C	3617.6	3746.9	3875.9	3779.4
Volume of Mold cm^3(D)	2124.00	2124.00	2124.00	2124.00
Bulk Density g/cm^3 C/D=(E)	1.70	1.76	1.82	1.78
Moisture Content Determination				
Container Code	E	P65	G190	A-13
Mass of Wet soil + container(g)(F)	196.58	200.37	151.95	160.36
Mass of dry soil + container(g)(G)	165.5	166.5	125.5	128
Mass of container(g)(H)	37.8	37.7	34	36.2
Mass of moisture(g)F-G=(I)	31.08	33.87	26.45	32.36
Mass of Dry soil(g)G-H=(J)	127.7	128.8	91.5	91.8
Moisture content % (I/J)*100=K	24.34	26.30	28.91	35.25
Dry Density g/cm^3 E/(100+K)*100	1.37	1.40	1.42	1.32



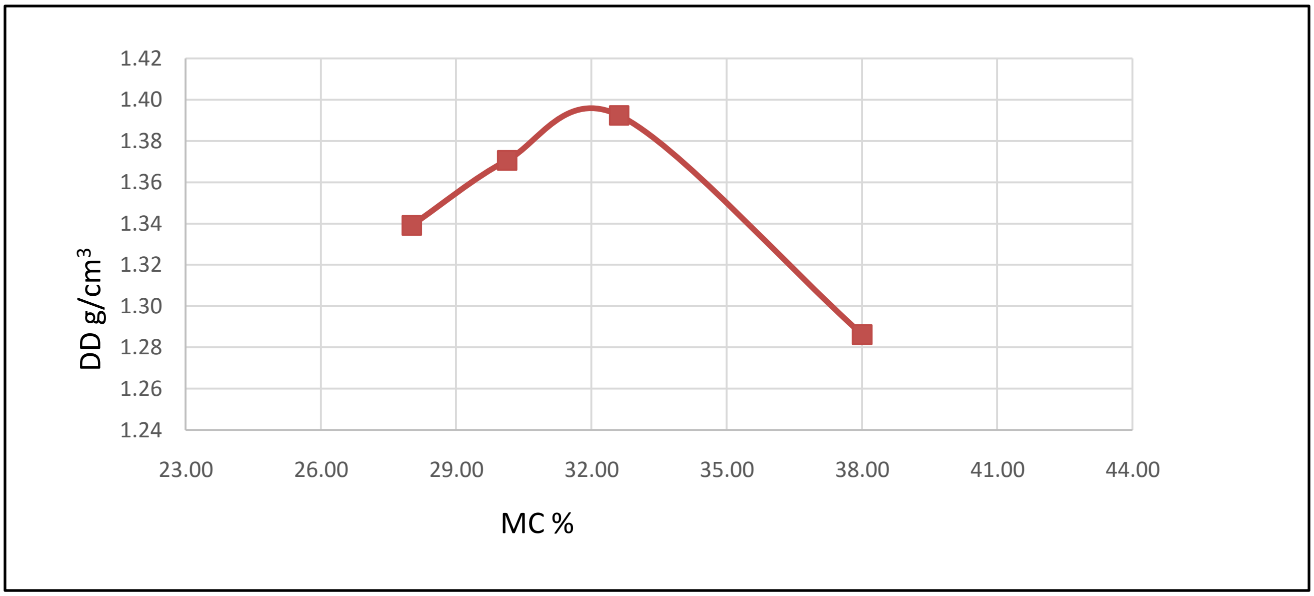
C2.2) 4% Lime, 10% WPA JIT

Density Determination				
Test No.	1	2	3	4
Mass of sample (g)	5000	5000	5000	5000
Water Added(cm^3)	450	650	850	1050
Mass of Mold + Wet soil(g)(A)	6362.1	6498.4	6612.4	6491.59
Mass of Mold(g)(B)	2720.5	2720.5	2720.5	2720.5
Mass of Wet Soil(g)A-B=C	3641.6	3777.9	3891.9	3771.09
Volume of Mold cm^3(D)	2124.00	2124.00	2124.00	2124.00
Bulk Density g/cm^3 C/D=(E)	1.71	1.78	1.83	1.78
Moisture Content Determination				
Container Code	12L	P6	T1	G22
Mass of Wet soil + container(g)(F)	210.65	143.72	148.72	161.83
Mass of dry soil + container (g)(G)	175	120	122.5	128
Mass of container(g)(H)	41.1	37.5	37.4	36.2
Mass of moisture(g)F-G=(I)	35.65	23.72	26.22	33.83
Mass of Dry soil(g)G-H=(J)	133.9	82.5	85.1	91.8
Moisture content % (I/J)*100=K	26.62	28.75	30.81	36.85
Dry Density g/cm^3 E/(100+K)*100	1.35	1.38	1.40	1.30



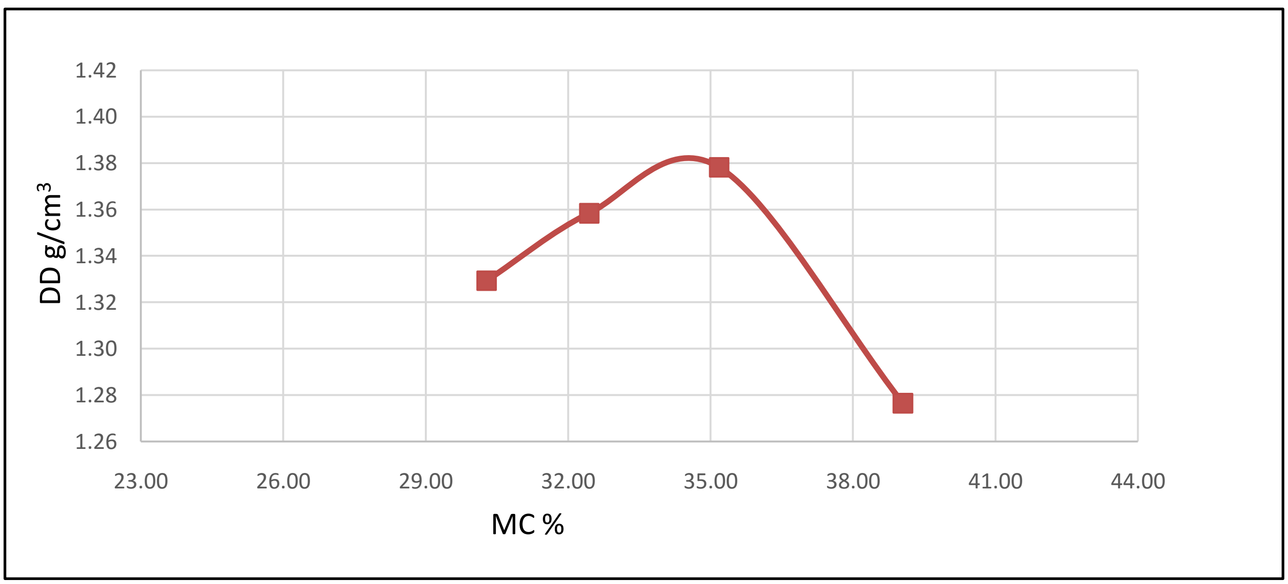
C2.3*) 6% Lime, 8% WPA @JIT*

Density Determination				
Test No.	1	2	3	4
Mass of sample (g)	5000	5000	5000	5000
Water Added(cm^3)	450	650	850	1050
Mass of Mold + Wet soil(g)(A)	6361.6	6508.9	6642.6	6490.8
Mass of Mold(g)(B)	2720.5	2720.5	2720.5	2720.5
Mass of Wet Soil(g)A-B=C	3641.1	3788.4	3922.1	3770.3
Volume of Mold cm^3(D)	2124.00	2124.00	2124.00	2124.00
Bulk Density g/cm^3 C/D=(E)	1.71	1.78	1.85	1.78
Moisture Content Determination				
Container Code	D-5	P15	G3T3	A2
Mass of Wet soil + container(g)(F)	212.92	151.35	156.82	158.63
Mass of dry soil + container(g)(G)	175.32	124.99	127.5	121.83
Mass of container(g)(H)	41.1	37.5	37.6	25
Mass of moisture(g)F-G=(I)	37.6	26.36	29.32	36.8
Mass of Dry soil(g)G-H=(J)	134.22	87.49	89.9	96.83
Moisture content % (I/J)*100=K	28.01	30.13	32.61	38.00
Dry Density g/cm^3 E/(100+K)*100	1.34	1.37	1.39	1.29



C2.4) *8% Lime, 6% WPA @JIT*

Density Determination				
Test No.	1	2	3	4
Mass of sample (g)	5000	5000	5000	5000
Water Added(cm^3)	450	650	850	1050
Mass of Mold + Wet soil(g)(A)	6398.9	6541.8	6677.6	6490.8
Mass of Mold(g)(B)	2720.5	2720.5	2720.5	2720.5
Mass of Wet Soil(g)A-B=C	3678.4	3821.3	3957.1	3770.3
Volume of Mold cm^3(D)	2124.00	2124.00	2124.00	2124.00
Bulk Density g/cm^3 C/D=(E)	1.73	1.80	1.86	1.78
Moisture Content Determination				
Container Code	B2	A4	G4	H-5
Mass of Wet soil + container(g)(F)	207.52	143.95	150.12	150.23
Mass of dry soil + container (g)(G)	169.12	118.1	121.1	115.43
Mass of container(g)(H)	42.3	38.42	38.6	26.31
Mass of moisture(g)F-G=(I)	38.4	25.85	29.02	34.8
Mass of Dry soil(g)G-H=(J)	126.82	79.68	82.5	89.12
Moisture content % (I/J)*100=K	30.28	32.44	35.18	39.05
Dry Density g/cm^3 E/(100+K)*100	1.33	1.36	1.38	1.28

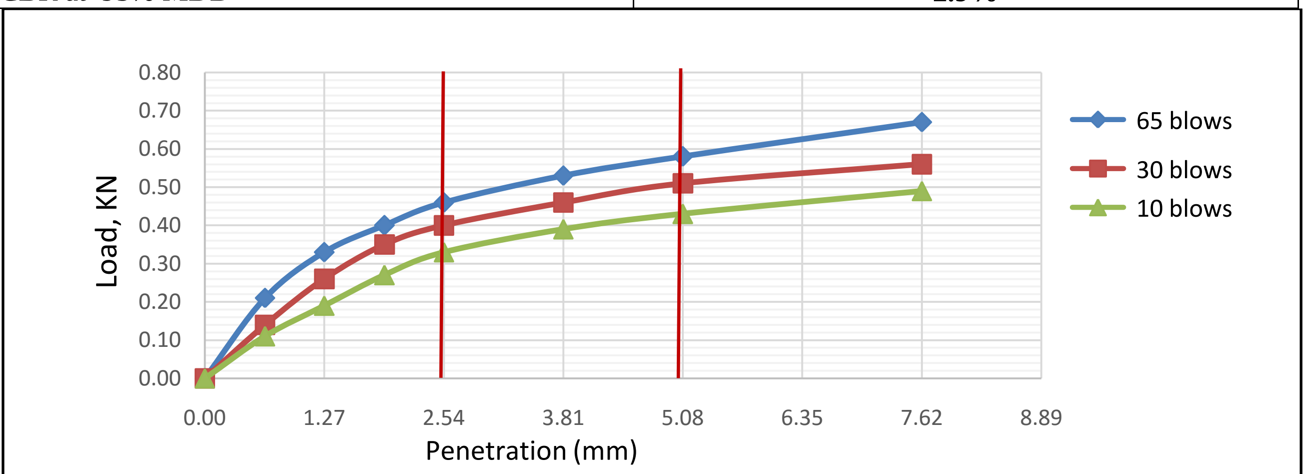


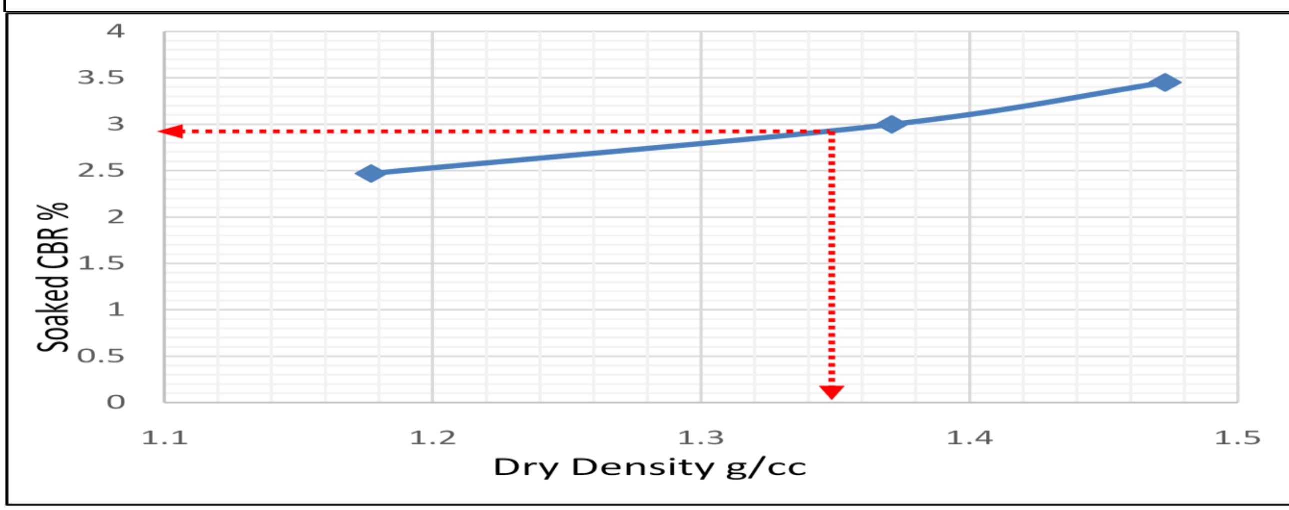
C 3) CBR Laboratory Test Results *of* Stabilized Soil Sample @ JIT

C 3.1*) 2% Lime, 12% WPA*

PENETRATION AND LOAD DETERMINATION OF NATURAL SOIL @ JIT						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65- Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.54	0.46	3.45	0.4	3.00	0.33	2.47
5.08	0.58	2.9	0.51	2.25	0.43	2.15

CBR RESULT SUMMARY OF NATURAL SOIL @ JIT			
MMDD (g/cm^3)	1.42		
Dry Density at 95% of MDD (g/cm^3)	1.349		
No of Blows	65	30	10
CBR Values (%)	3.45	3.00	2.47
DDBS (g/cm^3)	1.473	1.371	1.177
CBR at 95% MDD	**2.9%**		

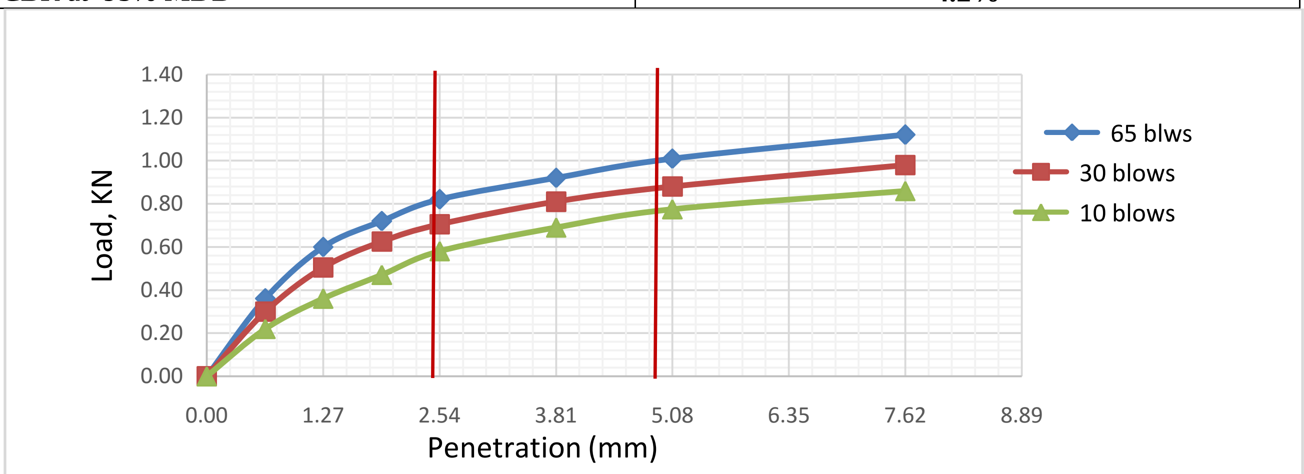


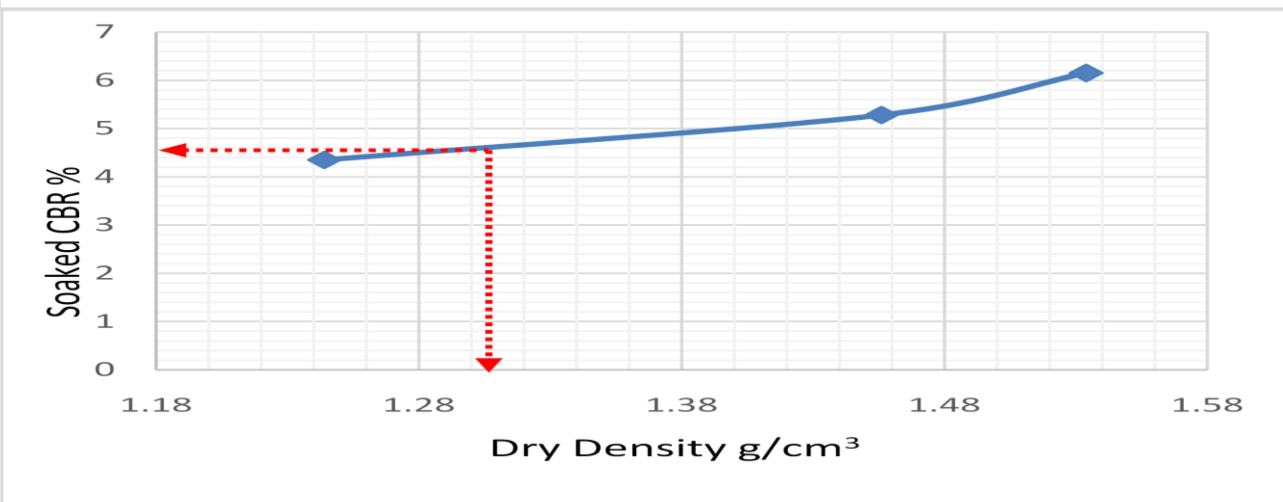


C3.2) *4% Lime, 6% WPA @JIT*

PENETRATION AND LOAD DETERMINATION OF NATURAL SOIL @ JIT						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65- Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.54	0.82	6.15	0.71	5.28	0.58	4.35
5.08	1.01	5.05	0.88	4.40	0.78	3.88

CBR RESULT SUMMARY OF NATURAL SOIL @ JIT			
MMDD (g/cm^3)	1.400		
Dry Density at 95% of MDD (g/cm^3)	1.330		
No of Blows	65	30	10
CBR Values (%)	6.2	5.3	4.3
DDBS (g/cm^3)	1.534	1.456	1.344
CBR at 95% MDD	**4.2%**		

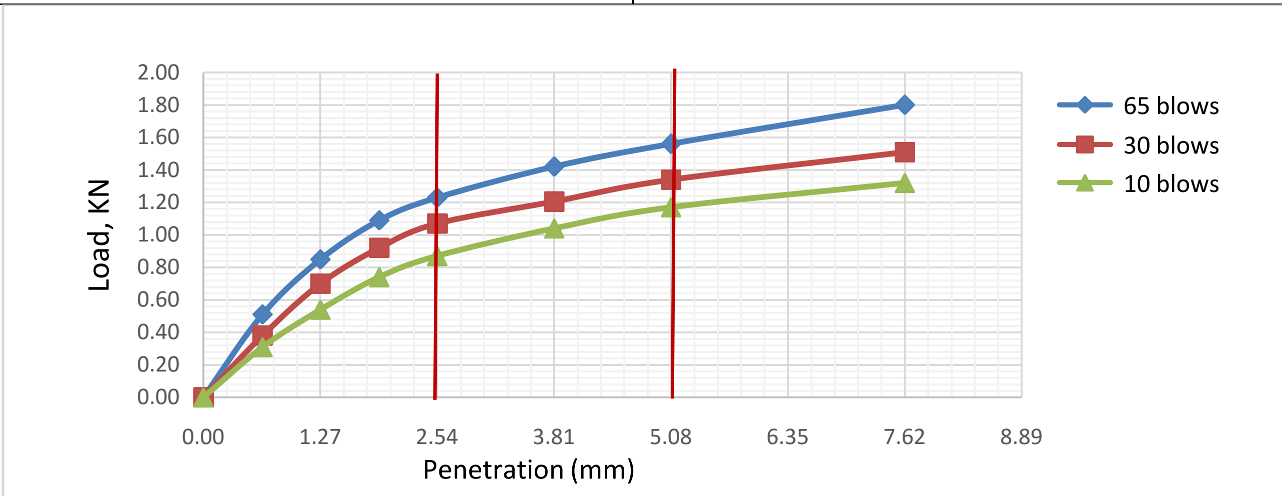


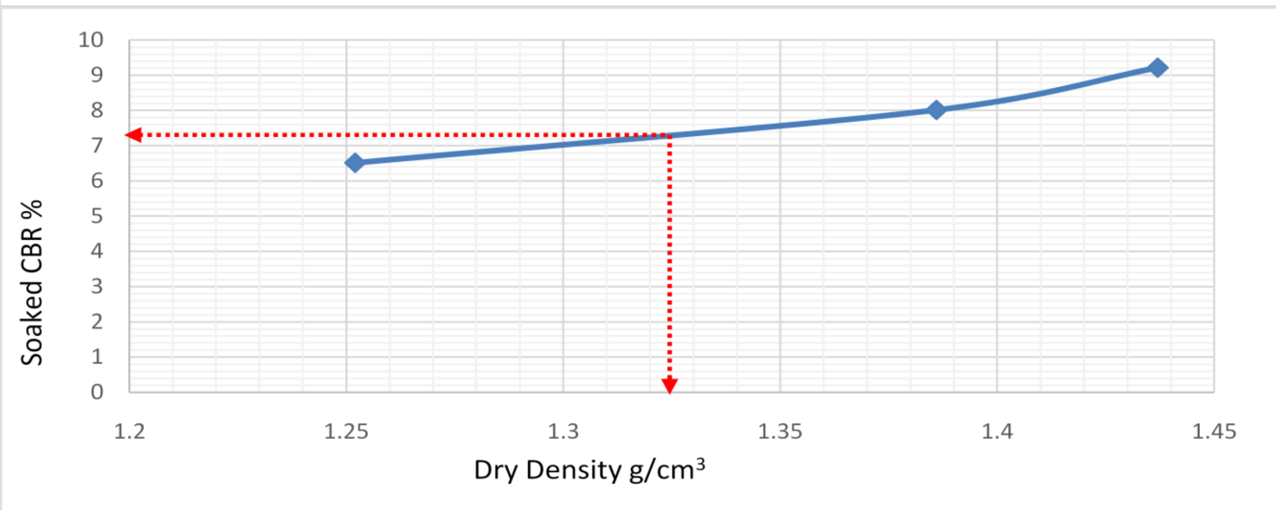


C3.3) *6% Lime, 8% WPA @JIT*

PENETRATION AND LOAD DETERMINATION OF NATURAL SOIL @ JIT						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65- Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.54	1.23	9.22	1.07	8.02	0.87	6.52
5.08	1.56	7.80	1.34	6.70	1.17	5.85

CBR RESULT SUMMARY OF NATURAL SOIL @ JIT			
MMDD (g/cm^3)	1.390		
Dry Density at 95% of MDD (g/cm^3)	1.321		
No of Blows	65	30	10
CBR Values (%)	9.2	8.0	6.5
DDBS (g/cm^3)	1.437	1.386	1.252
CBR at 95% MDD	**7.3%**		

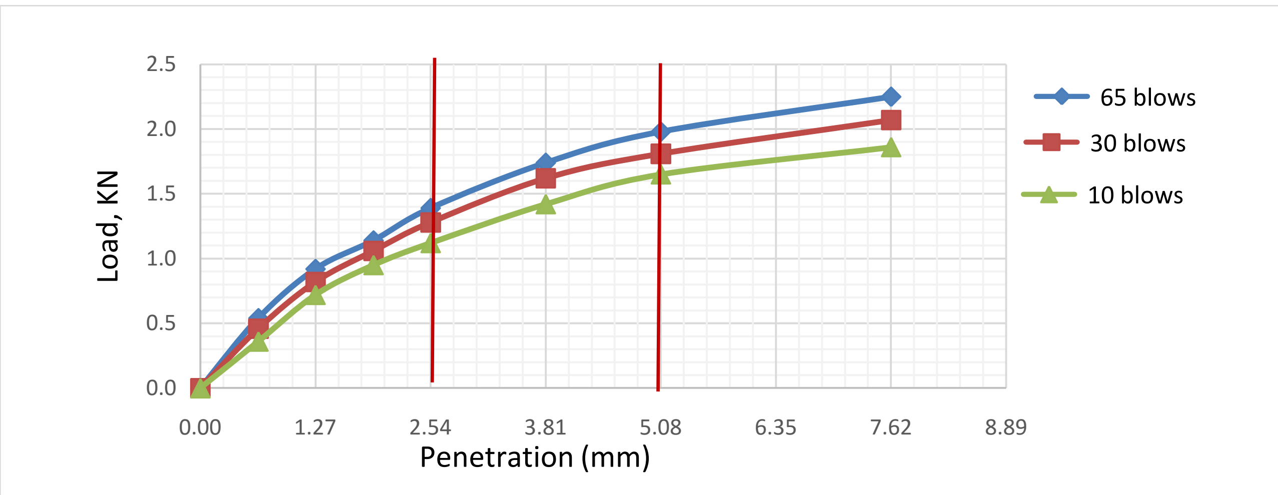


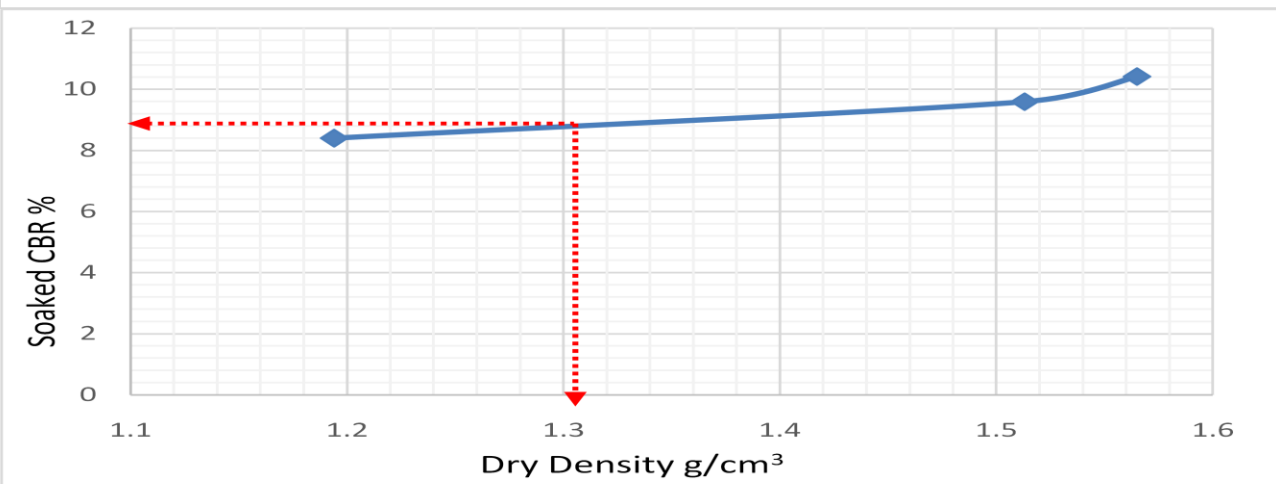


C3.4) 8% Lime, 6% WPA JIT

PENETRATION AND LOAD DETERMINATION OF NATURAL SOIL @ JIT						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65- Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.54	1.39	10.42	1.28	9.60	1.12	8.40
5.08	1.98	9.90	1.81	9.05	1.65	8.25

CBR RESULT SUMMARY OF NATURAL SOIL @ JIT			
MMDD (g/cm^3)	1.380		
Dry Density at 95% of MDD (g/cm^3)	1.311		
No of Blows	65	30	10
CBR Values (%)	10.5	9.6	8.4
DDBS (g/cm^3)	1.565	1.513	1.405
CBR at 95% MDD	**8.7%**		



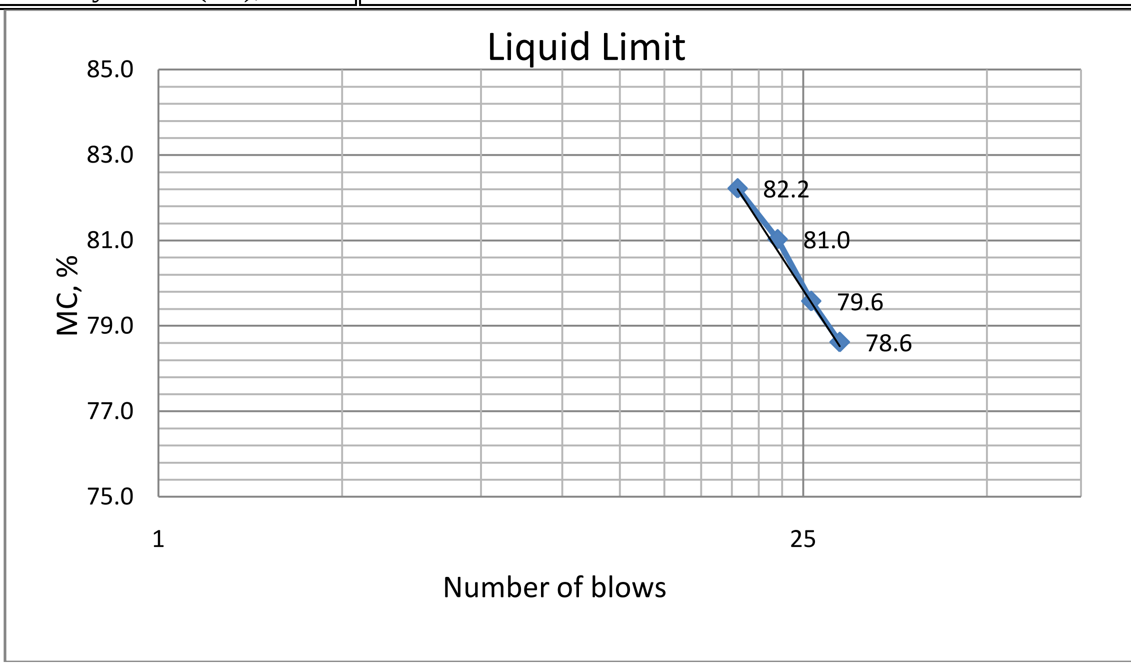


Appendix D: Laboratory Test Results of Stabilizing MAR Soil Sample Using WPA Mixed with Lime

D1) Atterberg Limit Test Results

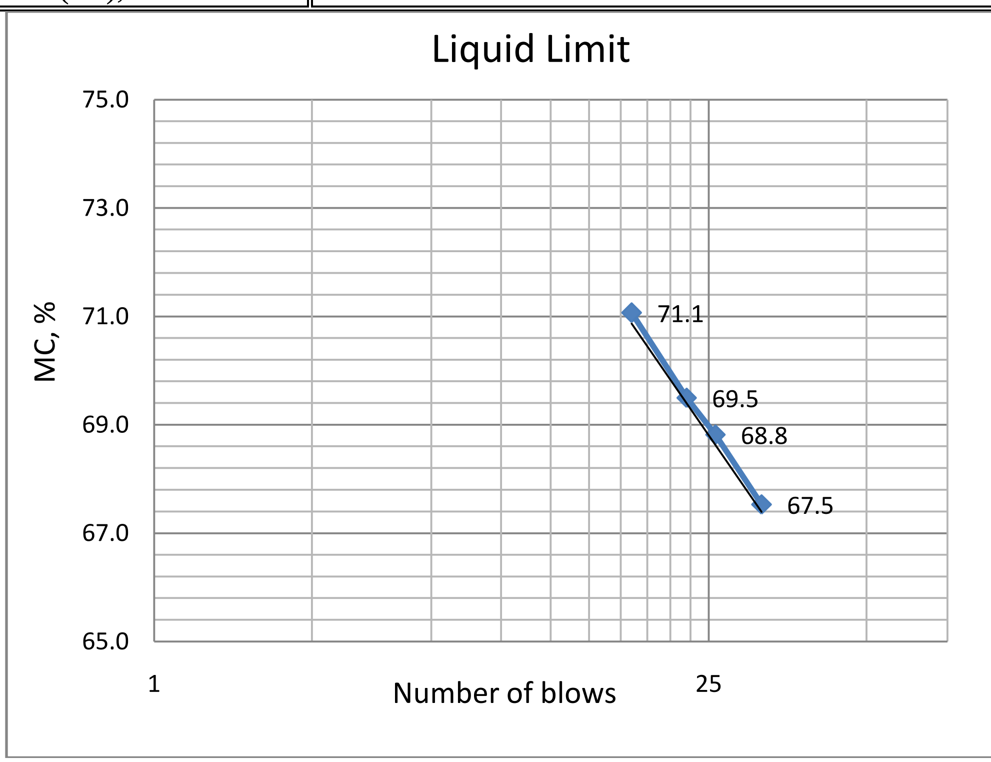
D1.1) *2% Lime, 12% WPA*

2% Lime + 12% WPA @RAM							
Determination	Liquid Limit				Plastic Limit		
Number of blows	30	26	22	18			
Trial	T1	T2	T3	T4	T1	T2	T3
Container Code	A6	M12	Z2	B4	L2	W1	F12
Wt. of container + wet soil, g	58.27	73.85	56.79	59.46	21.51	26.11	27.15
Wt. of container + dry soil, g	47.9	64.77	45.94	47.76	20.05	23.88	24.83
Wt. of container, g	34.71	53.36	32.55	33.53	16.95	18.74	19.58
Wt. of water, g	10.37	9.08	10.85	11.7	1.5	2.23	2.32
Wt. of dry soil, g	13.19	11.41	13.39	14.23	3.1	5.14	5.25
Moisture content,%	78.6	79.6	81.0	82.2	47.1	43.4	44.2
Liquid Limit (LL),%	**80.4**						
Plastic Limit (PL),%	**44.9**						
Plasticity Index (PL), %	**35.5**						



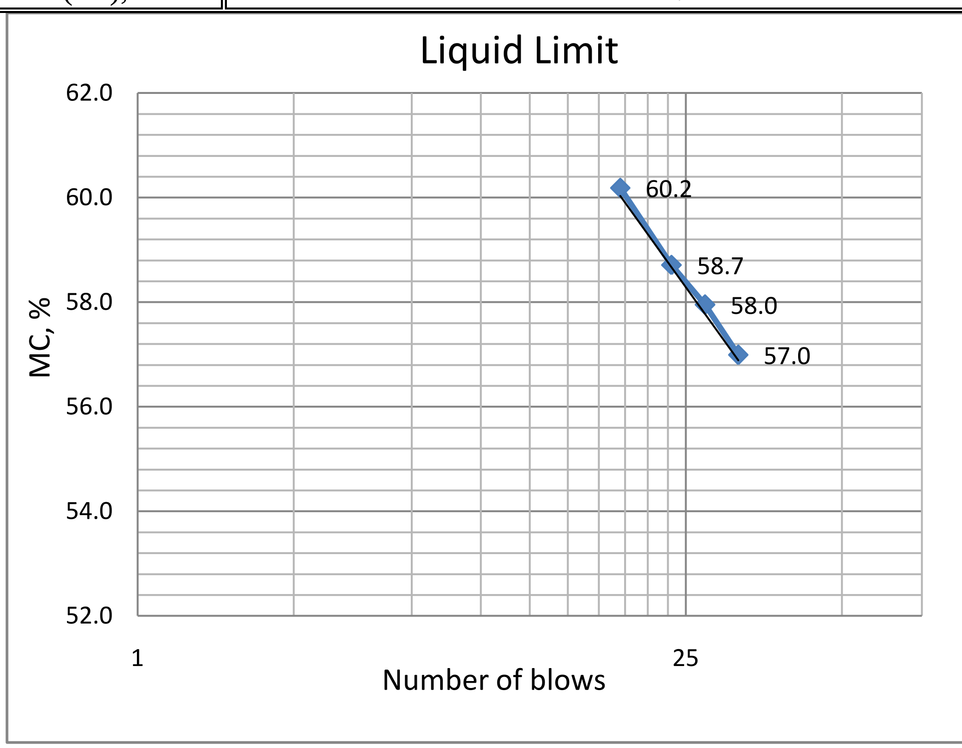
D1.2) *4% Lime, 10% WPA*

4% Lime + 10% WPA @MAR							
Determination	Liquid Limit				Plastic Limit		
Number of blows	34	26	22	16			
Trial	T1	T2	T3	T4	T1	T2	T3
Container Code	H3	D2	G19	A6	W1	W-21	E-3
Wt. of container + wet soil, g	38.62	39.52	42.11	34.4	27.66	25.96	26.31
Wt. of container + dry soil, g	30.53	30.56	32.04	27.4	24.39	22.89	23.51
Wt. of container, g	18.55	17.54	17.55	17.55	16.97	16.57	16.98
Wt. of water, g	8.09	8.96	10.07	7	3.3	3.07	2.8
Wt. of dry soil, g	11.98	13.02	14.49	9.85	7.42	6.32	6.53
Moisture content,%	67.5	68.8	69.5	71.1	44.1	48.6	42.9
Liquid Limit (LL),%	**69.2**						
Plastic Limit (PL),%	**45.2**						
Plasticity Index (PL), %	**24.1**						



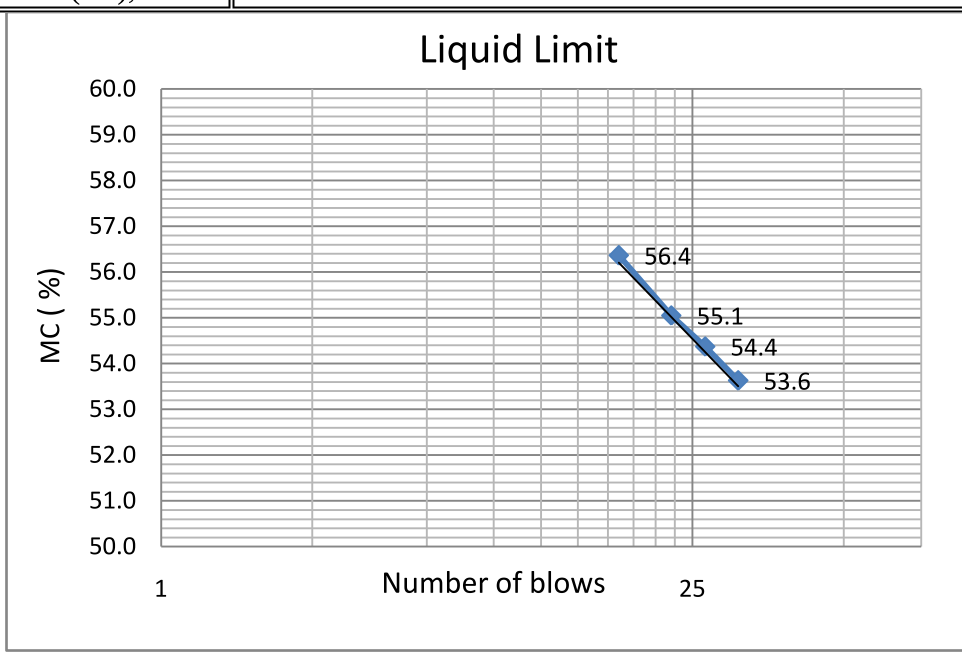
D1.3*) 6% Lime, 8% WPA*

6% Lime + 8% WPA @MAR							
Determination	Liquid Limit				Plastic Limit		
Number of blows	34	28	23	17			
Trial	T1	T2	T3	T4	T1	T2	T3
Container Code	N6	Z4	W-5	A12	G4	G8	U-2
Wt. of container + wet soil, g	60.86	64.12	56.07	55.63	25.25	27.11	26.9
Wt. of container + dry soil, g	52.34	54.39	47.24	48.45	22.93	24.62	24.72
Wt. of container, g	37.39	37.6	32.2	36.52	17.88	19.59	19.52
Wt. of water, g	8.52	9.73	8.83	7.18	2.3	2.49	2.18
Wt. of dry soil, g	14.95	16.79	15.04	11.93	5.05	5.03	5.2
Moisture content,%	57.0	58.0	58.7	60.2	45.9	49.5	41.9
Liquid Limit (LL),%	**58.5**						
Plastic Limit (PL),%	**45.8**						
Plasticity Index (PL), %	**12.7**						



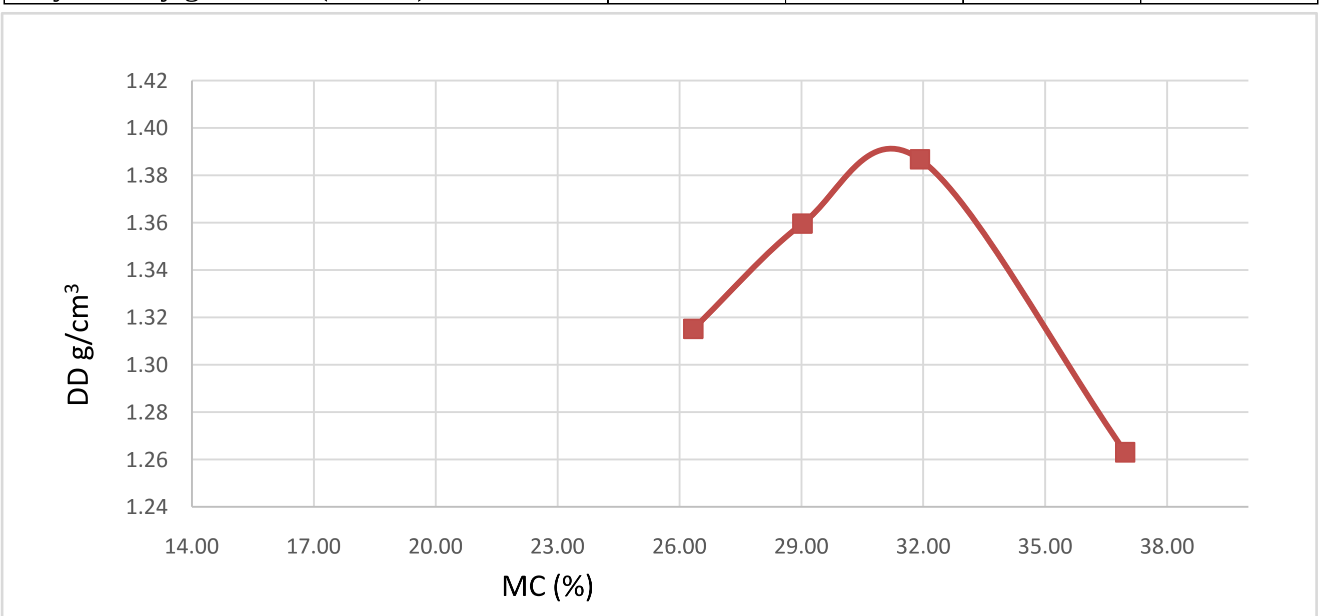
D1.4) *8% Lime, 6% WPA*

8% Lime + 6% WPA @MAR							
Determination	Liquid Limit				Plastic Limit		
Number of blows	33	27	22	16			
Trial	T1	T2	T3	T4	T1	T2	T3
Container Code	E-5	1-Z	H34	C3	K4	Y-6	B3
Wt. of container + wet soil, g	61.62	52.74	52.14	56	26.28	25.93	24.91
Wt. of container + dry soil, g	52.9	46.15	44.73	47.54	24.15	23.88	21.54
Wt. of container, g	36.64	34.03	31.27	32.53	19.43	19.41	14.52
Wt. of water, g	8.72	6.59	7.41	8.46	2.1	2.05	3.37
Wt. of dry soil, g	16.26	12.12	13.46	15.01	4.72	4.47	7.02
Moisture content,%	53.6	54.4	55.1	56.4	45.1	45.9	48.0
Liquid Limit (LL),%	**54.9**						
Plastic Limit (PL),%	**46.3**						
Plasticity Index (PL), %	**8.5**						



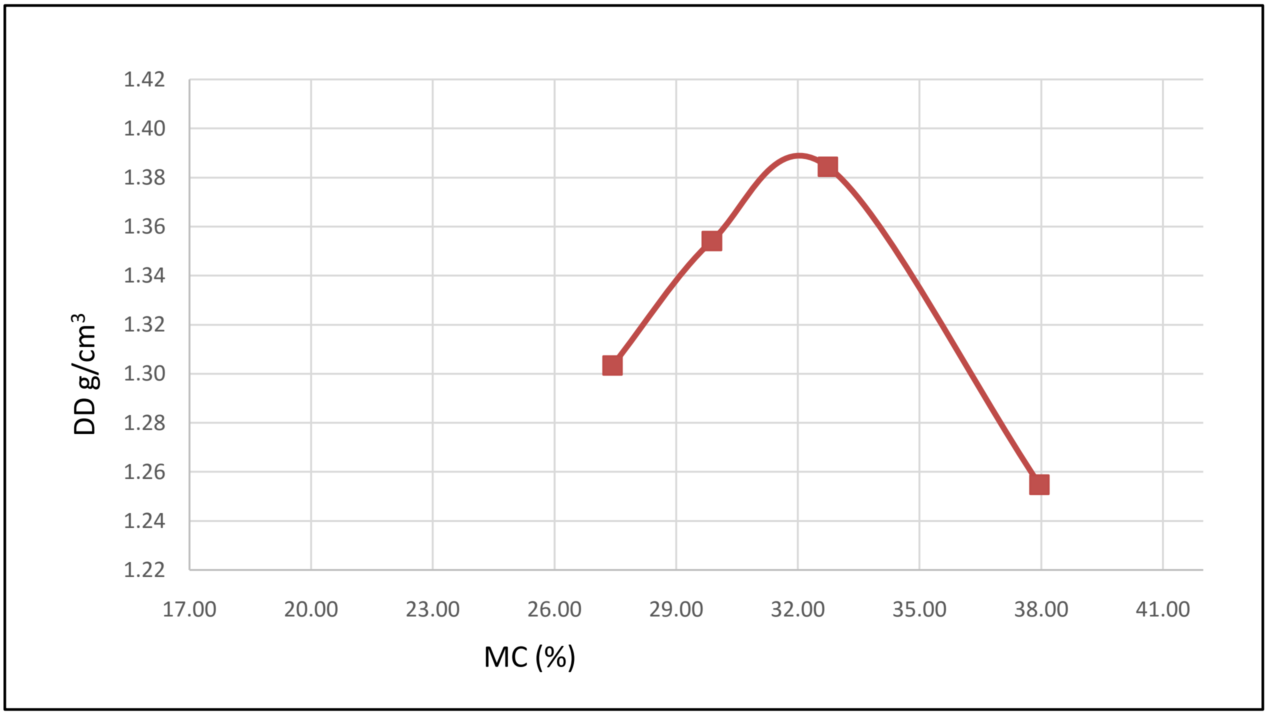
D2) Compaction Laboratory Test Result for *MAR Soil Sample*
D 2.1) *2% Lime, 12% WPA @MAR*

Density Determination				
Test No.	1	2	3	4
Mass of sample (g)	5000	5000	5000	5000
Water Added(cm^3)	450	650	850	1050
Mass of Mold + Wet soil(g)(A)	10106.45	10297.62	10452.31	10247.54
Mass of Mold(g)(B)	6683.3	6683.3	6683.3	6683.3
Mass of Wet Soil(g)A-B=C	3423.15	3614.32	3769.01	3564.24
Volume of Mold cm^3(D)	2060.24	2060.24	2060.24	2060.24
Bulk Density g/cm^3 C/D=(E)	1.66	1.75	1.83	1.73
Moisture Content Determination				
Container Code	E	P65	G190	A-13
Mass of Wet soil + Container(g)(F)	159.61	174.92	148.99	163.37
Mass of dry soil + container (g)(G)	136.72	144.56	121.41	129.32
Mass of container(g)(H)	49.8	39.96	34.99	37.2
Mass of moisture(g)F-G=(I)	22.89	30.36	27.58	34.05
Mass of Dry soil(g)G-H=(J)	86.92	104.6	86.42	92.12
Moisture content % (I/J)*100=K	26.33	29.02	31.91	36.96
Dry Density gm/cm^3 E/(100+K)*100	1.32	1.36	1.39	1.26



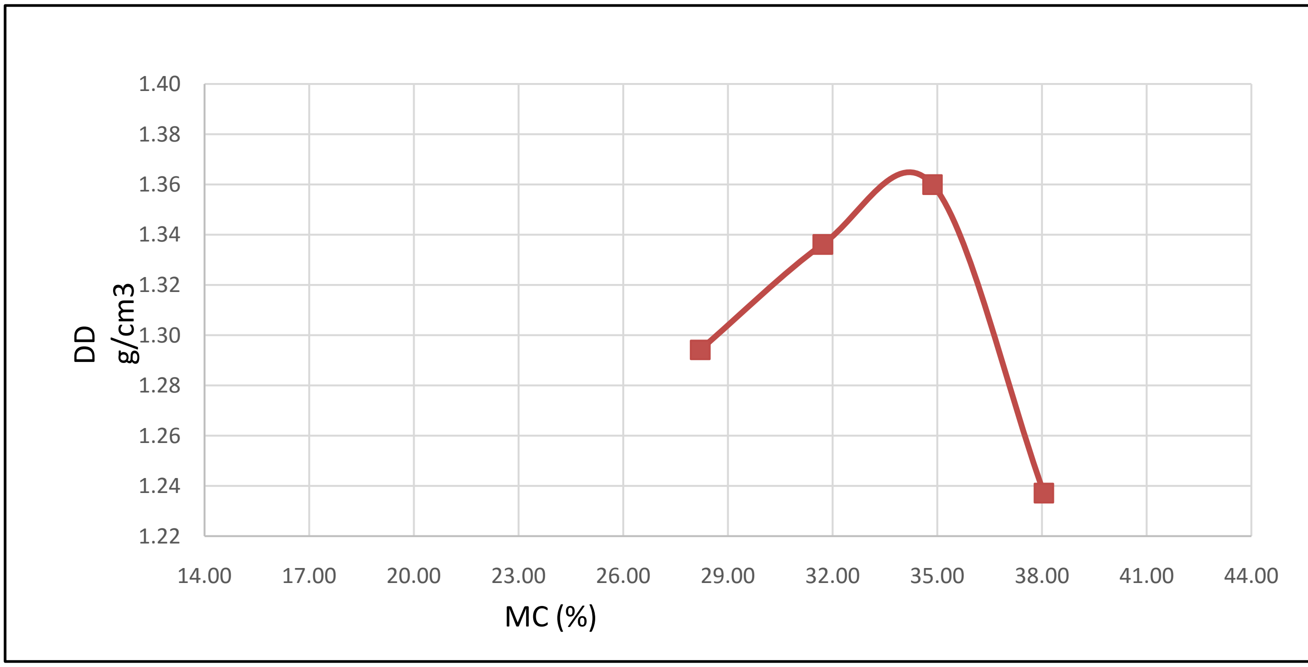
D 2.2) *4% Lime, 10% WPA @MAR*

Density Determination				
Test No.	1	2	3	4
Mass of sample (g)	5000	5000	5000	5000
Water Added(cm^3)	450	650	850	1050
Mass of Mold + Wet soil (g)(A)	10105.13	10306.71	10469.32	10249.59
Mass of Mold(g)(B)	6683.3	6683.3	6683.3	6683.3
Mass of Wet Soil(g)A-B=C	3421.83	3623.41	3786.02	3566.29
Volume of Mold cm^3(D)	2060.24	2060.24	2060.24	2060.24
Bulk Density g/cm^3 C/D=(E)	1.66	1.76	1.84	1.73
Moisture Content Determination				
Container Code	12L	P6	T1	G22
Mass of Wet soil + Container (g)(F)	170.99	186.42	156.48	170.87
Mass of dry soil + container (g)(G)	144.56	152.62	127.11	133.82
Mass of container(g)(H)	48.21	39.5	37.4	36.2
Mass of moisture(g)F-G=(I)	26.43	33.8	29.37	37.05
Mass of Dry soil(g)G-H=(J)	96.35	113.12	89.71	97.62
Moisture content % (I/J)*100=K	27.43	29.88	32.74	37.95
Dry Density g/cm^3 E/(100+K)*100	1.30	1.35	1.38	1.25



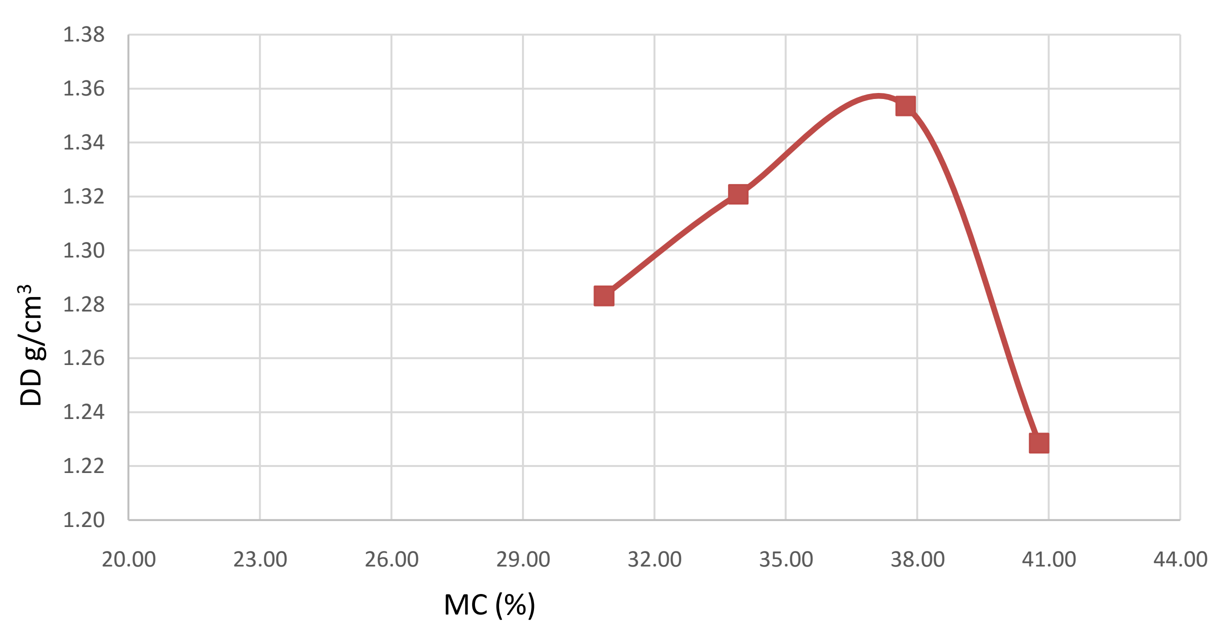
D 2.3) *6% Lime, 8% WPA @MAR*

Density Determination				
Test No.	1	2	3	4
Mass of sample (g)	5000	5000	5000	5000
Water Added(cm^3)	450	650	850	1050
Mass of Mold + Wet soil (g)(A)	10101.48	10309.06	10461.67	10201.94
Mass of Mold(g)(B)	6683.3	6683.3	6683.3	6683.3
Mass of Wet Soil(g)A-B=C	3418.18	3625.76	3778.37	3518.64
Volume of Mold cm^3(D)	2060.24	2060.24	2060.24	2060.24
Bulk Density g/cm^3 C/D=(E)	1.66	1.76	1.83	1.71
Moisture Content Determination				
Container Code	F-12	D3	L12	C4
Mass of Wet soil + container (g)(F)	185.16	198.97	173.73	159.52
Mass of dry soil + container (g)(G)	152.71	159.37	138.86	126.43
Mass of container(g)(H)	37.65	34.52	38.83	39.47
Mass of moisture(g)F-G=(I)	32.45	39.6	34.87	33.09
Mass of Dry soil(g)G-H=(J)	115.06	124.85	100.03	86.96
Moisture content % (I/J)*100=K	28.20	31.72	34.86	38.05
Dry Density g/cm^3 E/(100+K)*100	1.29	1.34	1.36	1.24



D 2.4) *8% Lime, 6% WPA @MAR*

Density Determination				
Test No.	1	2	3	4
Mass of sample (g)	5000	5000	5000	5000
Water Added(cm^3)	450	650	850	1050
Mass of Mold + Wet soil(g)(A)	10142.45	10327.43	10524.17	10246.53
Mass of Mold(g)(B)	6683.3	6683.3	6683.3	6683.3
Mass of Wet Soil(g)A-B=C	3459.15	3644.13	3840.87	3563.23
Volume of Mold cm^3(D)	2060.24	2060.24	2060.24	2060.24
Bulk Density g/cm^3 C/D=(E)	1.68	1.77	1.86	1.73
Moisture Content Determination				
Container Code	G4	J8	S-2	Z3
Mass of Wet soil + container(g)(F)	201.82	214.27	193.89	182.02
Mass of dry soil + container (g)(G)	164.21	169.99	151.36	137.93
Mass of container(g)(H)	42.3	39.42	38.65	29.81
Mass of moisture(g)F-G=(I)	37.61	44.28	42.53	44.09
Mass of Dry soil(g)G-H=(J)	121.91	130.57	112.71	108.12
Moisture content % (I/J)*100=K	30.85	33.91	37.73	40.78
Dry Density g/cm^3 E/(100+K)*100	1.28	1.32	1.35	1.23

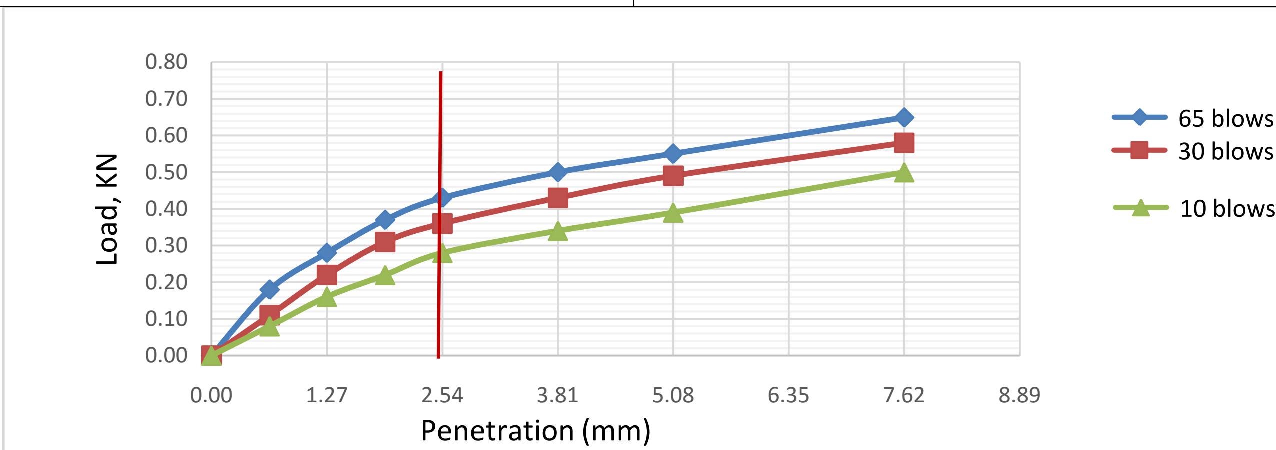


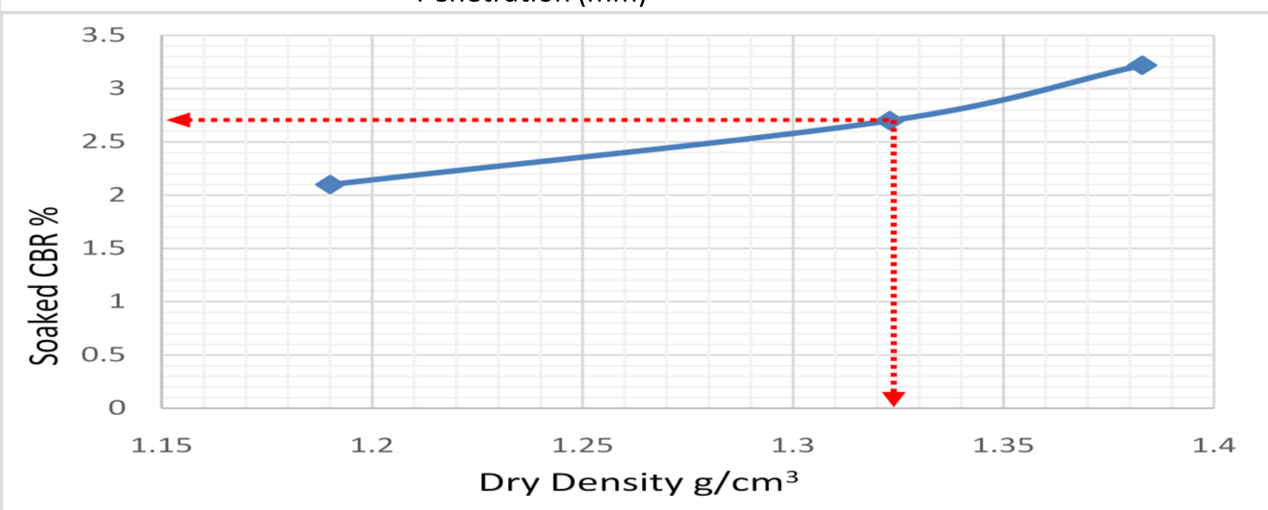
D 3) CBR Laboratory Test Results Of Soil Sample @MAR

3.1) *2% Lime + 12% WPA @MAR*

PENETRATION AND LOAD DETERMINATION OF NATURAL SOIL @ MAR						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65- Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.54	0.43	3.22	0.36	2.70	0.28	2.10
5.08	0.55	2.75	0.49	2.45	0.39	1.95

CBR RESULT SUMMARY OF NATURAL SOIL @ JIT			
MMDD (g/cm^3)	1.390		
Dry Density at 95% of MDD (g/cm^3)	1.321		
No of Blows	65	30	10
CBR Values (%)	3.2	2.7	2.1
DDBS (g/cm^3)	1.383	1.323	1.190
CBR at 95% MDD	**2.7%**		

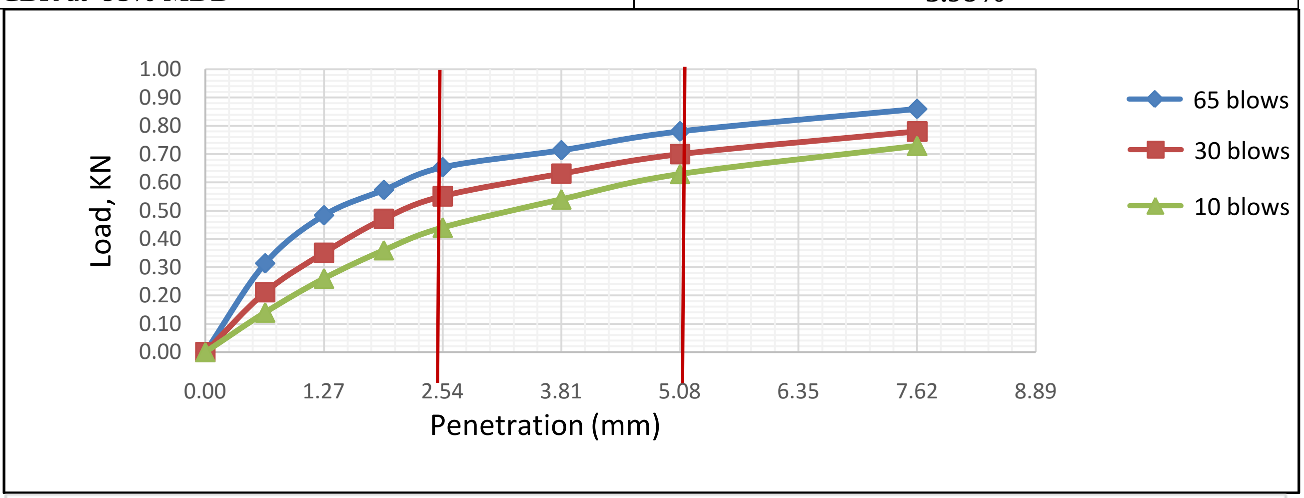


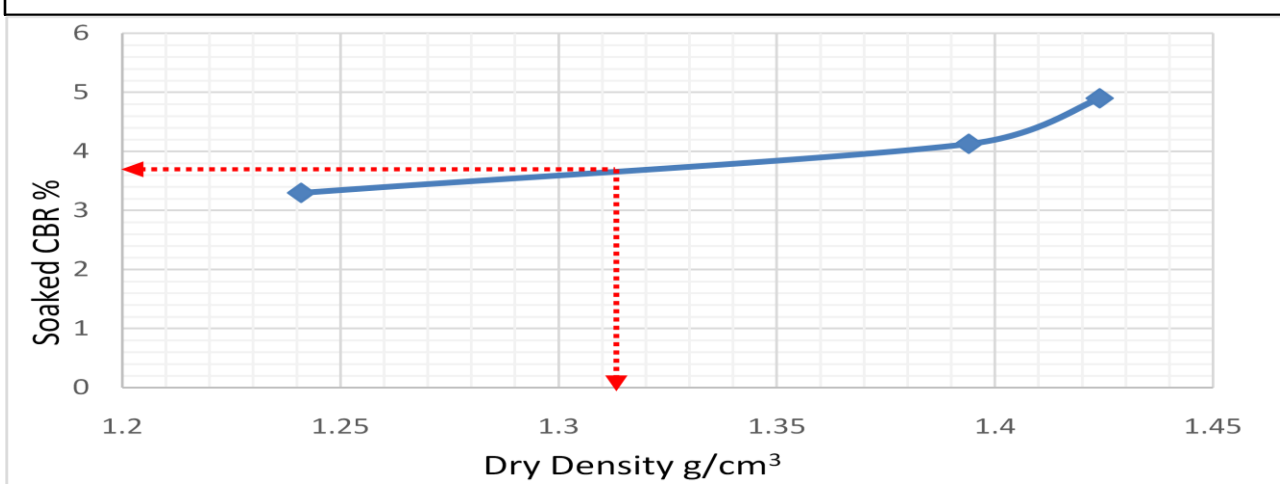


D3.2) *4% Lime, 10% WPA @ MAR*

PENETRATION AND LOAD DETERMINATION OF NATURAL SOIL @ MAR						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65- Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.54	0.65	4..9	0.55	4.13	0.44	3.30
5.08	0.78	3.90	0.70	3.50	0.63	3.15

CBR RESULT SUMMARY OF NATURAL SOIL @ MAR			
MMDD (g/cm^3)	1.380		
Dry Density at 95% of MDD (g/cm^3)	1.311		
No of Blows	65	30	10
CBR Values (%)	4.9	4.1	3.3
DDBS (g/cm^3)	1.424	1.394	1.241
CBR at 95% MDD	**3.98%**		

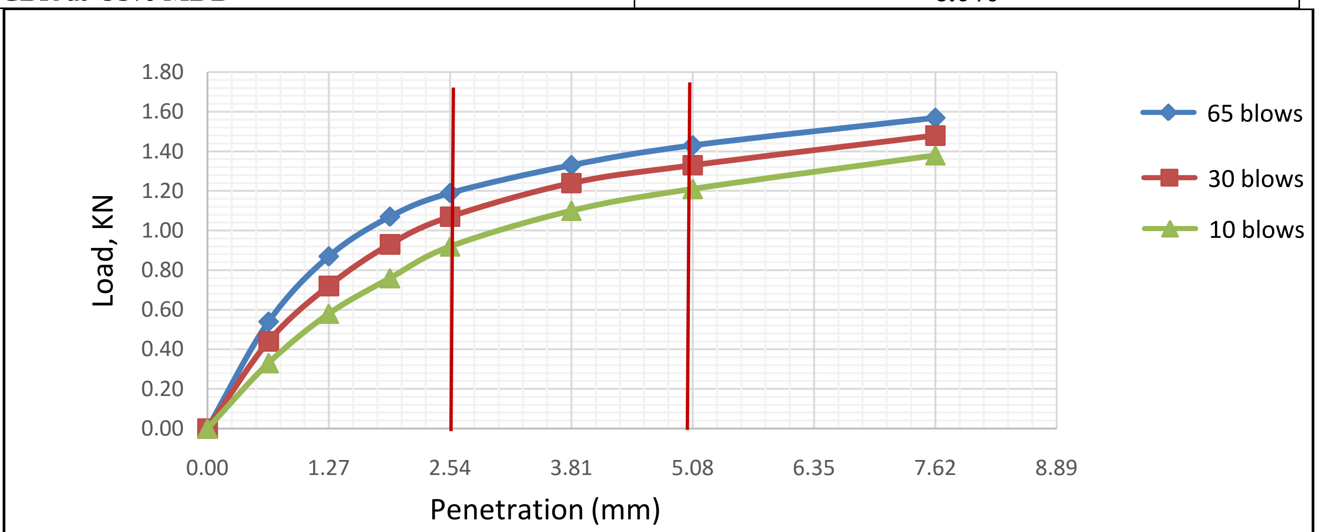


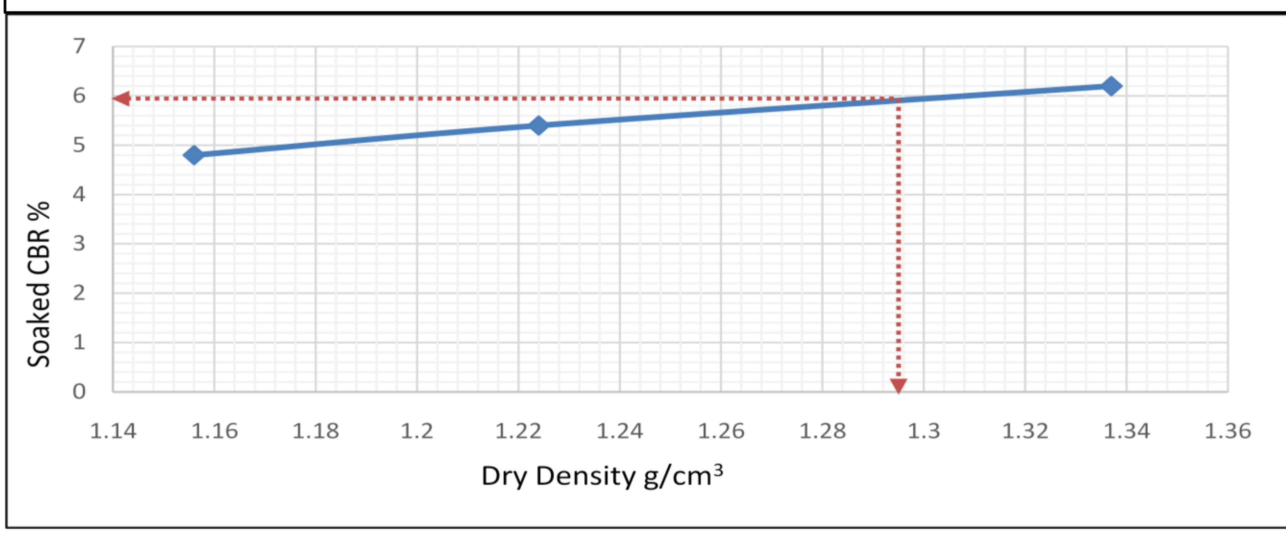


D3.3) *6% Lime, 8% WPA @MAR*

PENETRATION AND LOAD DETERMINATION OF NATURAL SOIL @ MAR						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65- Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.54	1.19	8.92	1.07	8.02	0.92	6.90
5.08	1.43	7.15	1.33	6.65	1.21	6.05

CBR RESULT SUMMARY OF NATURAL SOIL @ MAR			
MMDD (g/cm^3)	1.360		
Dry Density at 95% of MDD (g/cm^3)	1.292		
No of Blows	65	30	10
CBR Values (%)	8.9	8.0	6.9
DDBS (g/cm^3)	1.337	1.224	1.156
CBR at 95% MDD	**6.0%**		

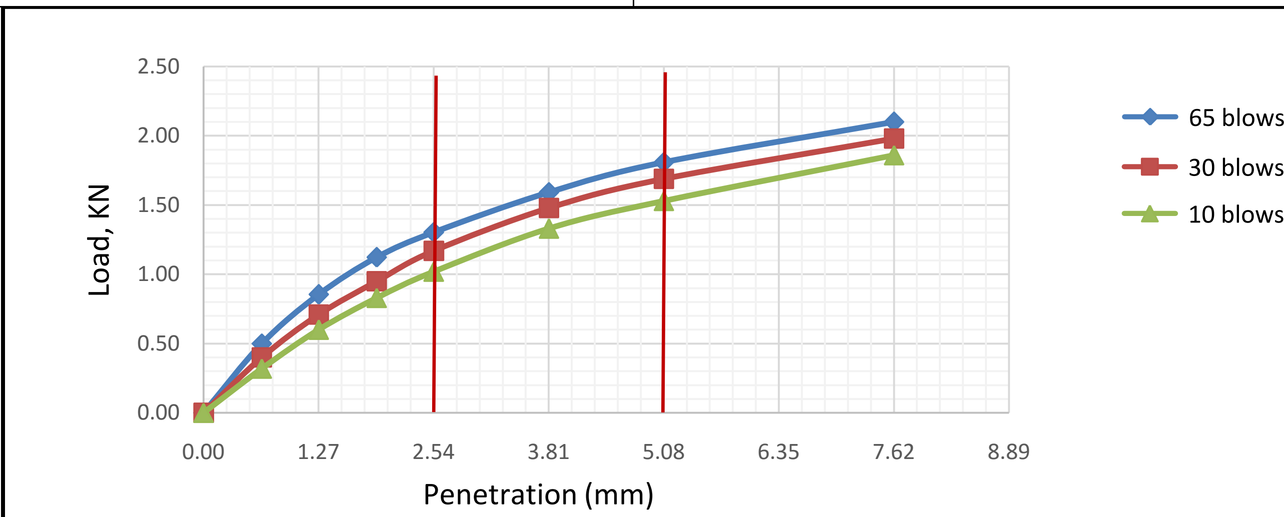


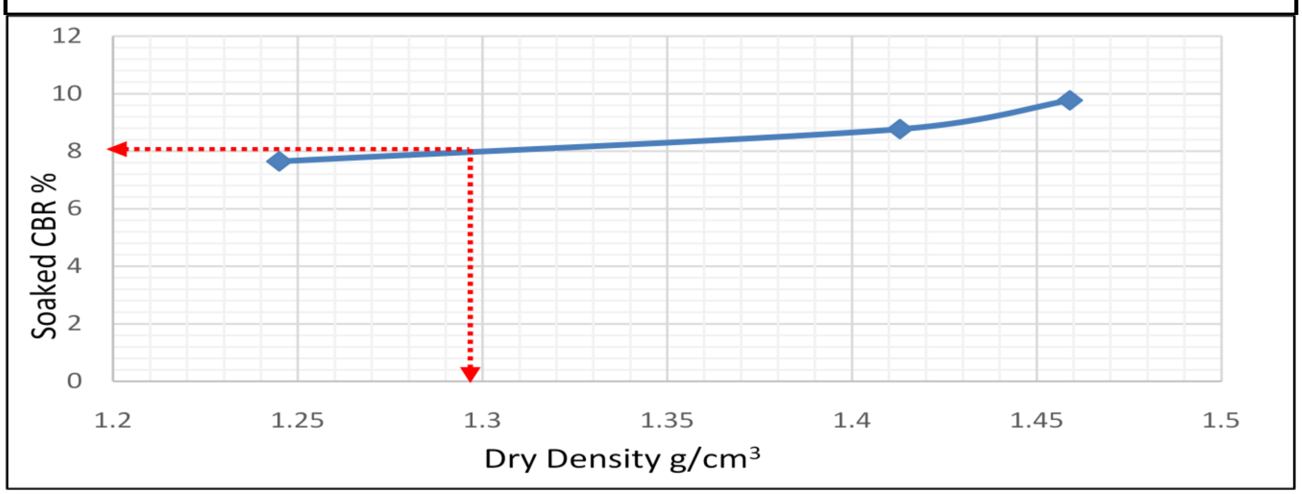


D3.4) *8% Lime, 6% WPA @MAR*

PENETRATION AND LOAD DETERMINATION OF NATURAL SOIL @ MAR						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65- Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.54	1.30	9.77	1.17	8.77	1.02	7.65
5.08	1.81	9.05	1.69	8.45	1.53	7.64

CBR RESULT SUMMARY OF NATURAL SOIL @ MAR			
MMDD (g/cm^3)	1.350		
Dry Density at 95% of MDD (g/cm^3)	1.283		
No of Blows	65	30	10
CBR Values (%)	9.8	8.8	7.7
DDBS (g/cm^3)	1.459	1.413	1.375
CBR at 95% MDD	**8.0%**		





Appendix E: Free Swell Index Test Result for Unstabilized and Stabilized soil of both JIT and MAR

E 1) Free Swell Index Test Result

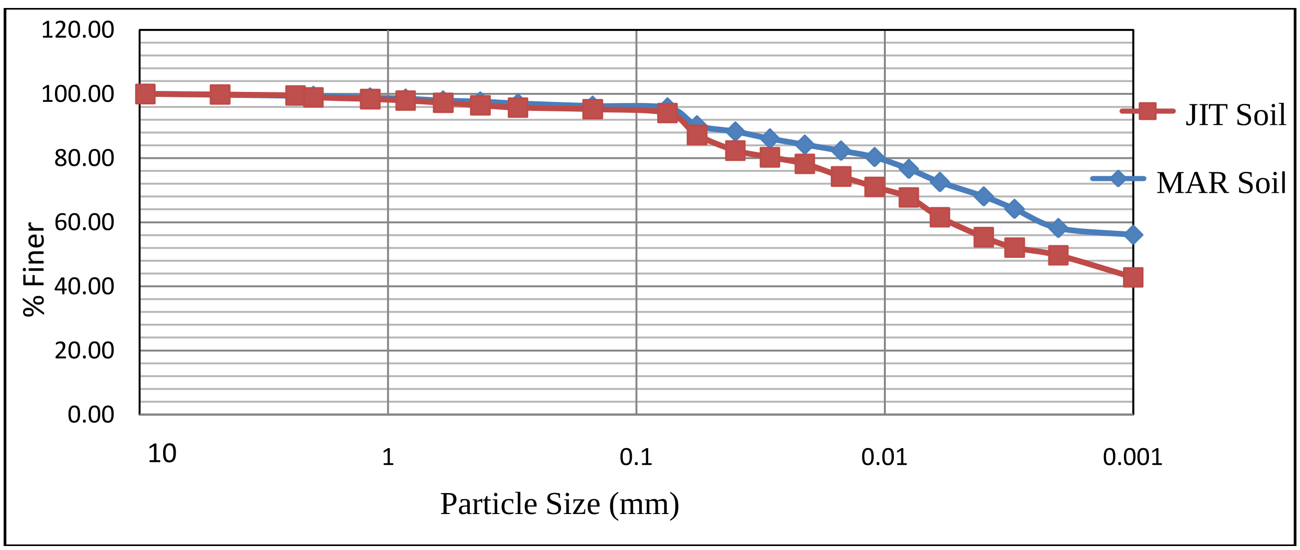
Sample Location	Mix-Proportion (%)		FSI (%)	Percentage of Reduction (%)	IS1498 requirement	Status of Test result
	Lime	WPA				
JIT	0	0	98.10	0.00	FSI < 50%	Control
	2	12	75.46	22.64		Poor
	4	10	58.32	39.78		Poor
	6	8	38..60	59.5		Satisfied
	8	6	31.64	66.46		Satisfied
MAR	0	0	104.71	0.00	FSI < 50%	Control
	2	12	86.53	18.18		Poor
	4	10	61.20	43.51		Poor
	6	8	39.92	64.79		Satisfied
	8	6	35.47	69.24		Satisfied

Appendix F: Linear Shrinkage Test Analysis for Both Soil Sample

Sample Location	Mix-Proportion (%)		Length of Mold (cm)	Length of dry specimen (cm)	Linear Shrinkage (%)	Degree of Expansion
	Lime	WPA				
JIT	0	0	14	11.39	18.64	Critical
	2	12	14	12.28	12.29	Critical
	4	10	14	12.94	7.57	Critical
	6	8	14	13.58	3.00	Non-critical
	8	6	14	13.66	2.43	Non-critical
MAR	0	0	14	11.19	20.07	Critical
	2	12	14	12.20	12.86	Critical
	4	10	14	12.69	9.36	Critical
	6	8	14	13.41	4.21	Non-critical
	8	6	14	13.56	3.14	Non-critical

Appendix G: Combined Wet Sieve Analysis and Hydrometer Analysis for Both Soil Samples

Sieve Opening (mm)	JIT	MAR
	Percent passing	Percent passing
9.5	100.00	100.00
4.75	99.83	99.73
2.36	99.56	99.41
2	98.99	99.30
1.18	98.41	98.80
0.85	98.00	98.57
0.6	97.28	97.98
0.425	96.46	97.66
0.3	95.80	97.08
0.15	95.29	96.23
0.075	94.06	95.71
0.057	87.25	90.14
0.040	82.37	88.32
0.029	80.27	86.04
0.021	78.21	84.21
0.015	74.28	82.27
0.011	71.06	80.31
0.008	67.78	76.64
0.006	61.56	72.51
0.004	55.35	68.03
0.003	52.13	64.14
0.002	49.71	58.20
0.001	42.84	56.12



Appendix H: Specific Gravity Test Data for Natural Soil @ JIT

Determination Code		L	N	K
Mass of dry, clean Calibrated pycnometer, Mp, in g		19.23	20.05	19.56
A. Mass of oven dry sample(g)		10	10	10
B. Mass of Pycnometer + water(g)		45.65	46.3	43.55
C. Mass of Pycnometer + water + sample(g)		51.97	52.68	49.91
Observed temperature of water,Ti		23	23	22
Water Temperature (^oC)				
^oC		23	22	22
K		1.0005	1.0070	1.0070
Temperature of contents of pycnometer when Mpsw was taken, Tx, in ^oC		23	22	22
K for Tx		1.0005	1.0070	1.0070
Specific gravity at 20^oC, Gs	Gs=A*k/(A+B-C)	2.72	2.78	2.77
Average Specific gravity at 20^0C, Gs		**2.76**		

Appendix I: Specific Gravity Test Data for Natural Soil @ MAR

Determination Code		F2	B6	M
Mass of dry, clean Calibrated pycnometer, Mp, in g		18.52	19.43	18.92
A. Mass of oven dry sample(g)		10	10	10
B. Mass of Pycnometer + water(g)		43.58	46.27	45.68
C. Mass of Pycnometer + water + sample(gm)		49.92	51.68	51.95
Observed temperature of water,Ti		22	23	23
Water Temperature (^oC)				
^oC		22	23	23
K		1.0005	1.0070	1.0070
Temperature of contents of pycnometer when Mpsw was taken, Tx, in ^oC		23	22	23
K for Tx		1.0005	1.0070	1.0070
Gs at 20^oC, Gs	Gs=A*k/(A+B-C)	2.73	2.74	2.70
Average Gs at 20^0C, Gs		**2.73**		

Appendix J: Specific Gravity Test Data for Waste paper Ash (WPA)

Determination Code		J2	G1	E3
Mass of dry, clean Calibrated pycnometer, Mp, in g		30.76	31.34	31.34
A. Mass of oven dry sample(gm)		35.41	45.39	56.43
B. Mass of Pycnometer + water(g)		126.56	123.01	118
C. Mass of Pycnometer + water + sample(g)		141.2	140.21	141.23
Observed temperature of water,Ti		20	20	20
Water Temperature (^oC)				
^oC		22	22	22
K		1.0070	1.0070	1.0070
Temperature of contents of pycnometer when Mpsw was taken, Tx, in ^oC		22	22	22
K for Tx		1.0070	1.0070	1.0070
Specific gravity at 20^oC, Gs	Gs=A*k/(A+B-C)	1.72	1.62	1.66
Average Specific gravity at 20^0C, Gs		**1.67**		

Appendix K: Atterberg Limit Test Data for Waste Paper Ash (WPA)

Determination	Liquid Limit				Plastic Limit		
Number of penetration, mm	24.57	22.97	18.52	15.54			
Trial	1	2	3	4	1	2	3
Container Code	A12	A7	G21	3-Z	A-36	I	4-Z
Wt. of container + wet soil, g	33.51	36.83	44.7	36.09	40.19	36.91	48.24
Wt. of container + dry soil, g	27.42	30.96	36.4	29.73	33.38	31.54	40.67
Wt. of container, g	16.04	19.75	19.84	16.26	17.56	19.48	23.32
Wt. of water, g	6.09	5.87	8.3	6.36	6.8	5.37	7.57
Wt. of dry soil, g	11.38	11.21	16.56	13.47	15.82	12.06	17.35
Moisture content,%	53.5	52.4	50.1	47.2	43.0	44.5	43.6
Liquid Limit (LL),%	**50.8**						
Plastic Limit (PL),%	**43.7**						
Plasticity Index (PL), %	**7.1**						