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APPLICATION OF MARBLE DUST TO IMPROVE THE ENGINEERING PROPERTIES OF EXPANSIVE SOILS TO BE USED AS ROAD BEDDING MATERIAL

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A Thesis on:

Application of Marble Dust to Improve the Engineering Properties of Expansive Soils to be used as Road Bedding Material

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Declaration

I hereby declare that this thesis is my original work under the supervision of Dr: Ing Samuel Tadesse submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfilment of the Requirement for the Degree of Master of Since in Civil Engineering (Geotechnical Engineering). I further declare that this work has not been submitted to any other institutions for the award of any degree or diploma and all sources of materials used for the thesis have dully acknowledged.

Tagel Mada

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TABLE OF CONTENTS

List of Figurs

List of Tables

Abstract

Expansive soils are characterized by volume change due to variation in moisture content. The cyclic wetting and drying process causes vertical and horizontal movement in expansive soils leading to failure of civil engineering structures found on such soils. The problematic nature of such soils can be improved by employing chemical stabilization techniques such as lime, cement and enzyme stabilization. However, as these techniques are expensive to developing countries such as Ethiopia, locally available low cost materials can be applied as alternative to improve the engineering properties of expansive soils.

In the present research work, the performance of marble dust to improve the problematic nature of expansive soils is investigated. The study is carried out on a highly expansive soil section of selected road project. The natural subgrade soil obtained from the highly expansive section of the road is characterized for its engineering and strength properties and it is found to be highly expansive soil with low bearing capacity, high swell and characterized as A-7-5 according to the AASHTO classification and rated as poor (unsuitable) subgrade material.

The expansive subgrade soil is blended with increasing percentage by weight (5% to 30%) and the improvements in engineering properties are studied and it is noted that it improves with increasing percentage of marble dust and with increased periods of curing though it is not significant as compared to specification requirements of several standards.

With the higher percentage of marble dust (30%), the swelling potential of the natural soil changed from ‘High’ to ‘Medium’, LL reduced from 88% to 63%, PI reduced from 52% to 34%, CBR increased from 0.9% to 2.25% and the CBR swell reduced from 8.6% to 5.3%.

Un-soaked CBR test is conducted on the natural subgrade soil and the 30% marble dust blended sample and it has been noted that the subgrade class improves for both indicating significant reduction in project cost as result of reduction in pavement thickness.

Thus, marble dust can be recommended as an alternative stabilizing agent for expansive soils by itself or being blended with small percentage (1-3%) of effective stabilizing agents such as lime so that significant project cost reduction and minimal environmental degradation can be assured.

CHAPTER ONE

INTRODUCTION

1.1. Background

Every Civil Engineering structure is to be founded on soil. The soil on which the structure is to be built should be capable of withstanding the load to be imposed on it. However, naturally there exist problematic soils to be used as foundation or construction materials, such as expansive soils, whose engineering characteristics are mainly affected by fluctuation of moisture content.

Expansive soils originate from complex combination of conditions and processes that result in the formation of clay minerals having a particular chemical makeup which expands when it comes in contact with water. All clay soils are not expansive and the degree of expansion varies with the type of clay mineral predominantly present in the soil mass. The presence of montmorillonitecontributes to high swell-shrink potentials.

The formation of expansive soils requires specific conditions as described by Chain (1988) including:

- Suitable parent material rich in calcium and magnesium to form smectite clay minerals, commonly basic igneous rocks such as basalt or gabbro
- Seasonal dry periods to allow clay minerals to form
- High temperature and sufficient rainfall for wetting but not much enough to leach base materials from the soil (commonly a semi-arid to dry tropical or sub-tropical climate)
- Impeded drainage to slow leaching and loss of weathering products
- High PH environment

According to the Ministry of Works and Urban Development (MoWUD, 2009), expansive soils form a major soil group occurring in the high lands mostly in the western, central and southwestern part of the Ethiopia. In most cases the major clay mineral of the most parts of the country is montmorillonite indicating high expansiveness. The road sector in Ethiopia is suffering from the high swell-shrink behaviour of the expansive soils. Many damages occur each year and roads constructed on such soils exhibit serious problems including increased cost of construction and maintenance.

As a result of seasonal climatic change, expansive soils swell and shrink causing vertical and horizontal movement of the soil mass and the overlying structure. These movement leads to failure of pavement, in the form of settlement, heavy depression, cracking and unevenness. The expansive sub-grade soil when wet has a tendency to heave into the upper layers of the pavement, especially when the sub-base is of coarser materials with lots of voids. The physical properties of expansive soils vary from place to place. In general these soils have very low load carrying capacity and high swelling and shrinkage characteristics. Roads constructed over such soils as sub-grade, develop undulations at the road surface due to loss of strength of the sub-grade through softening during rainy seasons.

Fig. 1.1 Expansive soil of the project area

Fig. 1.2 Crack on office building of the project

In the absence of moisture fluctuation expansive soils are found to be good construction and/or foundation material. Their problematic nature is illustrated by swelling when they are wet and shrinking when they are dry which causes instability of structures founded on them or constructed with them.

Previous studies show that the problematic nature of such soils can be improved by employing techniques such as chemical and/or mechanical stabilization to reach at the required specification as construction/foundation material. However, these techniques are expensive to developing countries such as Ethiopia. Thus, locally available low cost materials such as marble dust (by-product of marble grinding industries) and fly ash (fine residue of call) can be applied to improve the engineering properties of expansive soils.

This research is aimed at examining the potential of marble dust to improve the problematic nature of expansive soils to be used as road bedding material and to investigate the changes in engineering properties of the soil sample when treated with marble dust.

1.2. Problem Statement

The Ethiopian roads authority (ERA) site investigation manual states that, "*during route selection the design consultant shall avoid as much as possible areas with expansive soil, flooding, landslide, etc.*" Based on this requirement, routes which can be termed as best routes may be ignored because of expansive nature of the soil in the corridor even though, the route is viable technically, economically and environmentally.

The traditional practise to solve the problematic nature of soils in areas with expansive soils is to remove the problematic soil to the required depth and replace it with selected non expansive materials. The excavation and replacement of the problematic soil may lead to increase in project cost and time as it needs excavation of the problematic soil, production and transportation of the selected material and compaction to the required specification. In addition to the above, to get selected materials, additional borrow areas need to be excavated and that contribute a lot for environmental degradation.

Thus, one has to look for appropriate and economical soil stabilization techniques to minimize/avoid the additional project cost and time required forthe removal and replacement of the problematic soil.

In this study the application of marble dust to improve the problematic nature of expansive soils to be used as road subgrade material is investigated in detail by mixing the expansive soil with increasing percentage of marble dust.

1.3. Objectives

1.3.1 General Objectives

The main objective of this research work is to investigate the performance of marble dust to improve the engineering propertiesof expansive soils to be used as road subgrade material.

1.3.2 Specific objectives

The specific objectives of this research work are;

- To characterize the subgrade soil of the selected trial road section
- To reviewpossible methods of subgrade soil stabilization techniques
- To study the production and availability of marble dust in the Country
- To compare the effectiveness of marble dust stabilization of expansive soils with other stabilization techniques
- To draw recommendation on the economic and appropriate percentage by weight of marble dust required to improve expansive soil to meet the specification requirement of subgrade material
- To conduct detailed laboratory tests on natural soil and marble dust blended samples

1.4 Approach and Methodology

This research work is done by conducting literature review on expansive soils and subgrade soil stabilization, field observation and sample collection for laboratory investigation and analysis. Laboratory tests are conducted on the natural subgrade soil and blendedsubgrade soil with different percentage by weight of the marble dust.

The literature review is conducted to have frame work and understanding regarding the engineering properties of expansive soils, to identify problems associated with such soils and to investigate possible ways of improving the problematic nature of expansive soils.

The site investigation for this research work was conducted in the month of January, 2014 which is highly dry season for the project area where expansive soils can be easily identified visually. In the first week of the site investigation and sampling, the first 30km of the project length was roughly identified; the laboratory test results conducted during design and construction phases of the project for subgrade soils were studied and sections of the project with highly expansive soils were identified.After the field investigation pits were excavated and disturbed samples were collected for the detailed laboratory tests. The laboratory tests conducted on the natural subgrade soil and blended samples are Grain size analysis,Atterberg limits, Linear shrinkage,Moisture density Relations, CBR, CBR swell and Specific gravity.

Fig.1.3 Highly cracked ground surface

Fig. 1.4 Pit excavation

1.5. Materials used

To achieve the objectives of this research work expansive soil samples are collected from Sembo- Sholagebeya road project and marble dust which is obtained from the Ethiopian Marble Processing Enterprise.

1.6. Outcome of the Study

The present research work is intended to study the previously investigated improvement/stabilization methods and particularly to investigate the performance of marble dust to improve the problematic nature of expansive soils to be used as road subgrade material. This research work can also be taken as an indicative and alternative way of improvement of expansive soils to be used as road subgrade material.

1.7. Limitations of the Study

- The study is conducted on selected section of a specific road project of a particular area and samples were taken from limited test pits for laboratory studies.
- Limited number of tests are conducted only to achieve the objectives of the present study

1.8. Scheme of Presentation

The thesiscomprises of seven chapters as summarized below;

Chapter one comprises of introduction, scope, methodology, significance, limitations and scheme of presentation of the research work.

Chapter two is about literature review comprising study on previous similar studies relevant to the present study in order to provide framework of the problem statement. This includes identification of expansive soils, problems associated with expansive soils, mechanisms of improving expansive subgrade soils,

Chapter three describes the study area according to climate, geology and geographical location

Chapter four is about subgrade soil characterization including classification, gradation, plasticity, moisture content, moisture density relationships, strength and percent swell and over all characterization of subgrade soil.

Chapter five is about subgrade soil stabilization where the available stabilization techniques are discussed and the characteristics of the proposed stabilizing agent (marble dust) are discussed in detail. The proportion of the marble dust to be blended is discussed and the results for the marble dust blended samples are presented for different laboratory tests in comparison with the natural soil.

Chapter six presents interpretation and discussion of the test results of different percentage of marble dust blended samples compared to the natural soil.

Chapter seven concludes the findings of the present research work and forward recommendations that can be helpful for further studies on the same area.

CHAPTER TWO

LITERATURE REVIEW

2.1. Review on Expansive Soils

2.1.1. Origin, Distribution and Characteristics of Expansive soils

Expansive soils are residual deposits formed from basaltic and sedimentary rocks. Expansive soils of different origin and location have common characteristics such as high clay content with appreciable plasticity, dark or graycolor and tendency to expand as result of moisture increase and shrink due to loss of moisture (Chain, 1980).

According to Nelson, 2010, the constituents of the parent material during the early and intermediate stages of the weathering process determine the type of the clay formed. The nature of the parent material is much more important during these stages than after intense weathering for long period of time. The parent materials that can be associated with expansive soils are categorized into two groups. The first group comprises the basic igneous rocks and the second group comprises sedimentary rocks that contain montmorillonite as a constituent, which breaks down physically to form expansive soils. The three most common clay minerals are illite, caolinite and montmorillonite which are crystalline alumino-silicate and montmorillonite is the major constituent of expansive soils.

The topography and climate of an area plays a significant role for the formation of expansive soils. Montmorillonite type of clay mineral requires leaching restricted conditions so that,magnesium(Mg^+), sodium(Na^+) and iron(Fe^+) cations may be accumulated in the system. Thus, the formation of montmorillonite is aided by alkaline environment, presence of magnesium(Mg^+) ions and lack of leaching. Such conditions are favourable in semi-arid regions with relatively low rainfall, particularly where evaporation exceeds precipitation. In this condition enough water is available for the alteration process, but the accumulated cataions will not be removed by flash rain.

According to Nelson, 2010, Montmorillonite is a three layered mineral having a single octahedral alumina sheet sandwiched between two silica sheets to give a 2:1 latice structure. The bonds are comparatively weak and dipolar molecules of water can easily enter between the sheets causing them to expand readily. Thus, soils containing substantial amount of montmorillonite will exhibit high shrinkage and swell characteristics owing to moisture fluctuation and are potentially expansive soils.

Potentially expansive soils can be formed almost anywhere in the world; it is widely spread throughout the five continents (United States, South America, Africa, Asia and Australia). In our country Ethiopia expansive soils are observed in areas such as central Ethiopia, following the major trunk roads of Addis-Ambo, Addis-Welliso, Addis-Tarmaber, Addis-Gohastion and Addis-Modjo. Similarly regional states such as Mekelle, Gambella, and the most southern, south-western and south eastern parts of Addis Ababa are found to be covered with expansive soils as shown in the figure below (Habtamu K. (2006)).

Fig. 2.1 Expansive soil distribution in Ethiopia (Habtamu K., 2006)

2.1.2. Mineralogy of Expansive Soils

Expansiveness of soils is due to the presence of clay minerals. Clay particles have sizes of 0.002mm or less. However, according to Chain. 1988, the grain size alone does not determine clay minerals and the most important property of fine grained soils is their mineralogical composition. Clay minerals are crystalline hydrous alumino-silicates derived from parent rock by weathering. The basic building blocks of clay minerals are the silica tetrahedron and the alumina octahedron and combine into tetrahedral and octahedral sheets to form the various types of clays. Kaolinite, Illite and Montmorillonite are the common groups of clay minerals most important in engineering studies.

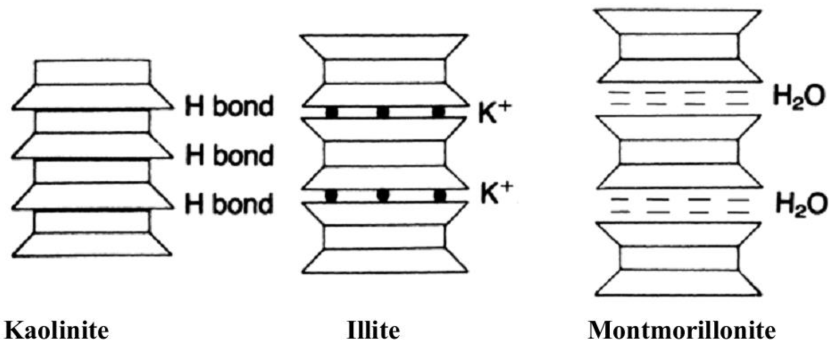


Fig. 2.2 Schematic representation of clay minerals (Craig, 1997)

Kaolinite is a typical two layered mineral having a tetrahedral and an octahedral sheet joined to form 1 to 1 layer structure held by a relatively strong hydrogen bond. Kaolinite does not absorb water and hence does not expand when it comes in contact with water. The montmorillonitegroups of clay minerals have 2 to 1 layer structure formed by an octahedron sandwiches between two tetra hedrons (Nelson, 2010). These clay groups have significant amount of magnesium and iron sandwiched into the octahedral layers. The most important aspect of the montmorilonite clay mineralogy group is the ability for water molecules to be absorbed between the layers, causing the volume of the minerals to increase when theycome in contact with water.TheIllite clay minerals have a structure similar to that of kaolinite, but are typically deficient in alkalis, with less aluminum substitution for silicon, magnesium and calcium can also sometimes substitute for potassium and illites are non-expanding type of clay minerals(Craig, 1997).

2.1.3. Identification of Expansive soils

A. Field identification

In the field expansive soils can be identified by applying several identification techniques. Some of the important field identification methods used to indicate potential of expansiveness of soils includes(AACRA-2004):

- Polygonal pattern of surface cracks during dry season. Cracks 205cm wide and over 1m depth and cracks close after rainy season
- A shiny surface when partly dray piece of the soil is polished with fingers

- When wet sticky and difficult to clean the soil from hand and when dry very hard like rock
- They usually have black or grey colour
- Cracks are observed on nearby light weight structures such as houses and fences

The typical features and common information of expansive soils identified during field reconnaissance surveys are shown below:

Table: 2.1 Typical features of expansive soils(AACRA-2004)

Description	Typical Features of Expansive Soils
Soil Type	More Clayey soils are likely to be expansive
Consistency when slightly moist to dry	Stiff to very stiff
Consistency when wet	Soft to firm and sticky
Structure	Typical cracked surface, slick-sided fissures
Color	Only a reliable indicator when combined with local knowledge

Fig. 2.3 Dry expansive soil

Fig. 2.4 Wet expansive soil

Fig. 2.5 Polygonal cracks on the ground surface during dry season of expansive soil

B. Laboratory identification

In the laboratory several methods are developed to identify expansive soils such as mineralogical identification, direct and indirect methods as described here under (Chain, 1988).

I. Mineralogical identification

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 a clue on the swelling potential.The swell shrink behaviour of expansive soils depends on the type of minerals present within the clay. The clay mineralogy of expansive soils can be identified in the laboratory by applying tests such as X-ray diffraction, electron microscope and differential thermal analysis.

II. Direct methods

The swelling pressure and volume changes of soils are measured directly using representative undisturbed samples. The swelling pressure is determined by measuring the pressure needed to prevent heaving of sample under the given condition of moisture, density and confinement. Swelling tests provide complete swelling but due to varying initial conditions of moisture, density, etc. it is difficult to assess the swelling expected in the field. The methods provide quantitative information, which are very useful for design engineers.

III. Indirect Methods

These are simple and more practical methods to identify expansive soils. The indirect tests conducted include the Atterberg limits and grain size distribution which helpdetermining the activity of clay (the ratio of plasticity index (PI) to the percentage of clay fractions finer than 2μm sieve size) present in the sample and degree of expansiveness.

According to the Ministry of Works and Urban Development of Ethiopia (MWUD, 2009), all greyish and/or brownish clays in Ethiopia with plasticity indices (PI) greater than 25% can be identified as expansive soils. The classification or rating from low potential to high heave potential usually depends on the clay content and plasticity.

2.1.4. Characteristics of Expansive Soils

Soils to be used as road sub grade material may have different characteristics such as soils with good load bearing capacity which is suitable for the use of subgrade material and on the other hand soils which are unsuitable to be used as subgrade material such as highly expansive soils. Thus, detailed investigation has to be conducted on the subgrade material to evaluate its suitability to be used as load carrying section of a pavement prior to going to the other steps of pavement design.

According to Reo (2007), 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 a year and they show extreme hardness and cracks when they are in dry condition.The seasonal change in volume of expansive soils is manifested by both horizontal and vertical movements, the horizontal movement leads to fissure opening during dry seasons and closing during wet seasons whereas the vertical movement leads to cyclic changes in levels. The magnitude of these movements decrease with depth where there is no seasonal moisture changes.

According to Seehra (2008), about 40 to 60% of expansive soils have grain sizes less than 0.001mm. These soils generally have higher liquid limit and plasticity index and extremely low CBR values.At their liquid limit, the volume change is of the order of 200 to 300% and results in swelling pressure as high as 8kg/cm^2 to 10kg/cm^2. Soaked laboratory CBR values of expansive soils are generally found to be in the range of 2 to 4%. Due to very low CBR values,highly exaggerated pavement thickness is required for designing flexible pavement which leads to extremely high project cost estimates.

2.1.5. Classification of Expansive soils

Expansive soils are classified by measuring their swelling potential which can be measured directly in the laboratory or indirectly by correlating with other test results of swell test data.

According to Bureau of reclamation method, based on direct correlation of observed volume change with colloid content, plastic index (PI) and shrinkage limit, expansive soils can be grouped according to their degree of expansiveness as shown in the table below.

Table .2.2 Classification of expansive soils based on Bureau of reclamation method(AACRA-2004)

Colloid content, %-1µm	PI, %	SL, %	Probable expansion, %	Degree of expansion
< 15	< 18	>15	<10	Low
13 – 23	15 – 28	10 – 16	10 – 20	Medium
20 – 31	25 – 41	7 - 12	20- 30	High
>28	>35	<11	>30	Very high

2.1.6. Problems Associated with Expansive Soils

According to ERA, 2002, most of the problems associated with expansive soils arise mostly from the nature of the soil itself and drainage facilities provided. As a result of their low CBR and strength, expansive soils fail to support the loads transmitted from the pavement structure and cause excessive deformation beyond permissible limits. The common problems associated with expansive soils are described below.

i. Volume change

Expansive soils have tendency to heave during wet condition, shrink and develop cracks during dry seasons which makes expansive soils a problem to road pavements. The cracks developed during dry seasons allow water to penetrate deep in to the soil during rainy seasons, hence causing considerable heave and expansion. This results in deformation of road surface constructed on expansive soils as the expansion and the subsequent heave are never uniform. Furthermore, the shrink-swell behaviour of expansive soils may lead to lateral displacements/ creep of the pavement layer on expansive soils, if the side slopes are not gentle enough. During seasonal change (rainy and dry seasons), the road edges get wet at faster rate than the surfacing of the road. This results in differential movements over the road cross section and associated crack development, first occurring in the shoulder areas and developing to carriageways.

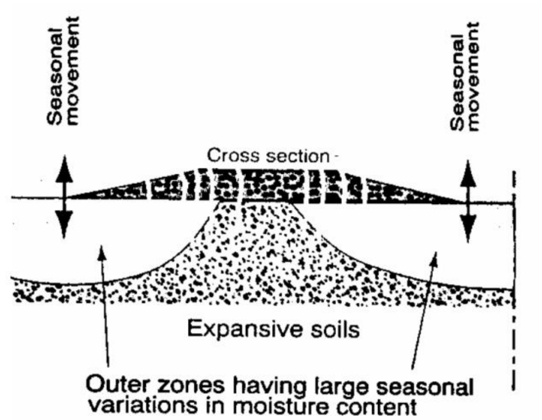


Fig. 2.6 Moisture content in Expansive Soils (ERA Site Investigation Manual-2002)

ii. Bearing capacity

When the moisture content increases, expansive soils swell and become loose and hence their bearing capacity reduces dramatically. If the soil becomes fully saturated, the CBR value reduces to a value under 2% which makes such soils unsuitable to be used as road subgrade material.

iii. Erosion

In dry state expansive soils exhibit sand like structure where they are prone to erosion to a much greater extent than that normally anticipated from their plasticity and clay content.

2.1.7. Design and Construction Considerations in Expansive Soils

In road projects if expansive soils are encountered, the measures proposed to deal with them should be economically reasonable and proportionate to the risks of potential pavement damages and increased maintenance costs.

The commonly practiced measures to deal with expansive soils during design phase include (ERA, 2002);

- Avoid expansive soil area by realignment
- Excavate the expansive soil and replace it with suitable material
- Treat/stabilize the expansive soil using Lime, cement, Bitumen…
- Minimize moisture changes and potential swelling in the expansive soils

2.1.8. Mitigation Measures on Expansive Soils

Expansive soils do not meet the specification requirements of many standard including the Ethiopian Roads Authority Standard Technical Specifications(2002). Thus, expansive subgrade soilsmay need improvement to their engineering properties to be used as construction materials by physical or chemical stabilization or modification of their problematic nature.

According to ERA Site Investigation Manual -200 (Special Investigation), whenever expansive soils are encountered during the design or construction phase of a road project, the following mitigation measures are recommended.

i. Realignment

Realignment is recommended and possible if the areas covered with expansive soils is of limited extent. When the coverage of the expansive soil is of limited extent rather than going for treatment or removal of the problematic section, realignment can be effective and economical.

ii.Excavation and Replacement

This is mostly recommended measure as the problematic soils are completely removed and replaced by selected suitable material. However, these measures are only economically viable if the selected borrow material is found in the project vicinity.

It is commonly estimated that it is sufficient to excavate the expansive soil to a depth of about 1m, even if some expansive soil remains under the selected material, it will be confined and protected from moisture changes. The back fill materials should have CBR values similar to that of the over laying embankment materials (CBR >5%, i.e. subgrade strength class S3) and should not pervious in order not act as drain.

A guideline has not yet been established as to the thickness requirement for the selected fill. F H Chen reports that a minimum of 1 m should always be insisted upon, although 1.5 m is preferred. However, performance data and practices followed in other countries are available. For clays of high expansiveness, Indian case studies recommend removal of 1000mm of high expansive clay, Zimbabwean practice is to remove 700mm, Tanzanian practice is to remove 600mm, studys in Ethiopia recommends 900mm to 1200mm, Kenyan manual recommends 1000mm, Some U.S state department procedures recommends removal up to 1500mm (Nelson and Miller suggested 1200mm in 1992), Warmbaths experiment (Weston, 1980) recommended 1000mm replacement which has performed well for 20 years with some pre-wetting and horizontal membranes (Soil and Material report of Addis Ababa-Tarmaber road project).

According to Addis Ababa City Roads Authority Soil and Material Manual (AACRA-2004), the suggested treatment depth below the normal subgrade level for high potential swelling soils depends on the plastic index (PI) of the subgrade soil and the traffic class of the proposed road as shown in the table below.

Table: 2.3 Suggested treatment depths below the normal subgrade level of high swelling potential soils (AACRA-2004)

Highly Trafficked (Primary Roads)		**Lightly Trafficked (Secondary Roads)**	
Plasticity Index (PI)	Depth of Treatment (m)	Plasticity Index (PI)	Depth of Treatment (m)
10-20	0.6	10-30	0.6
20-30	0.9	30-50	0.9
30-40	1.2	*Above 50	1.2
40-50	1.5		
***Above 50**	1.8		

*Excavate and waste, replace with selected impermeable material

iii. Soil Treatment/Modification

The problematic nature of expansive soils can be improved by applying several treatment measures. Some of the treatment methods developed and being applied in road construction projects include stabilizing by using stabilizing agents such as lime, cement, bitumen and chemicals. The other treatment method is covering the whole stretch of the subgrade section of expansive soil by protective layers such as geotextile (as in the case of Dejen-Debremarkos road project). Similarly, in road projects with expansive soils and scarce suitable borrow materials, the expansive soil itself can be used as fill material by providing protective ‘’blankets’’ and applying proper drainage as which is practiced in Adura-Akobo road project under administration of the Ethiopian roads Authority (ERA).

As per specification requirements of several standards, separate treatment methods are applied on expansive subgrade soils for different road classes depending on their AADT. Accordingly, for AADT design greater than 50 of higher traffic class roads, the following treatment measures can be applied(AACRA-2004).

A) Removal of the expansive soil

   i. Where the finished road level is designed to be less than 2m above ground level, remove the expansive soil to a minimum depth of 0.6m over the full width of the road, or
   ii. Where the finished road level is designed to be greater than 2m above ground level, remove the expansive soil to a depth of 0.6m below the ground level under the un-surfaced area of the road structure, or
   iii. Where the expansive soil does not exceed 1m in depth, remove it to its full depth

B) Stockpile the excavated material on the either side of the excavation for subsequent spreading on the fill slopes so as to produce a flat slope as possible
C) The excavation formed as directed in paragraph (A) should be backfilled with a plastic non-expansive soil of CBR value of 3-4 or better, and compacted to a density of 95% of modified AASHTO density.
D) After the excavated material has been replaced with non-expansive material, in 150mm limits to 95% of modified AASHTO density, bring the road to finished level in approved materials, with a side slope of 1:2, and ensure that pavement criteria are compiled with the previously stockpiled expansive soil excavated as directed under (A) should then be spread over the slope
E) Do not construct side drains unless they are absolutely essential to stop ponding; where side drains are necessary, they should be as shallow as possible and located as far from the toe of the fill as possible

F) Ideally, construction over expansive soils should be done when the in-situ moisture content s at its highest (i.e. at the end of rainy season)

Similarly, for light traffic road classes of AADT design less than 50, ERA Manual (2002) recommends the following treatment methods to be applied on expansive soil sections of a subgrade.

A) Remove 150mm of the expansive top soil and stockpile conveniently for subsequent use on shoulder slopes
B) Shape road bed and compact to 90% of modified AASHTO density
C) The excavation formed as directed in (A) should be backfilled with a plastic non-expansive soil of CBR value of 3-4 or better, and compacted to a density of 95% of Modified AASHTO density in each 150mm layer; the subgrade material may be plastic but non expansive.

Treatment of expansive soils with hydrated lime gives good results. Previous studies shows that, addition of 4% to 6% of hydrated lime to expansive soils found to show good improvement in engineering properties of expansive soils as shown below;

- Plasticity Index (PI) reduces below 20
- Significant increase in the shrinkage limits
- Reduction of the swell to negligible values
- Increase in CBR to 10 after 7 days curing and 15 after 28 days curing
- Modification of particle size distribution (by agglomeration of the clay particles), the final grading being similar to that of silt.

Lime treatment is costly as substantial thickness of soil (at least 30cm compacted thickness) has to be prepared. This treatment can be considered advantageous only if when it is difficult to get selected backfill material near the project and when pavement savings can be made by taking advantage of the enhanced strength of the treated clay.

iv. Minimizing Moisture Changes and Consequent Movements

If there is shortage of selected borrow material in the project vicinity and if Lime treatment is found to be costly, the existing expansive soil can be used for fill and subgrade. To apply this option, the moisture change has to be controlled by employing mechanisms such as;

a) Confining expansive soils under impervious subgrades and protective blankets:-

Selected materials placed over weak subgrades reduces sub-base thickness and hence pavement thickness. Selected materials placed over expansive soils protect them from moisture changes. It is recommended that selected materials should be at least 30cm of thickness and should be relatively impervious.

Expansive soils can be used to form shallow embankments (up to about 3m), provided that a protective blanket of thickness at least 30cm is provided on the slopes. The blanketing material should be at least conducive to a subgrade strength class of S3 quality and be impermeable and resistant to erosion.

b) Surcharging Expansive Soils

When non-swelling materials are placed over them expansive soils reduce in heave. The minimum thickness required depends on the expansion pressure of the swelling soil, but it is commonly recommended to use 1-3m to have significant swell reduction. It is therefore possible to use expansive soil to form the lower part of an embankment. The total thickness of pavement plus improved subgrade should be at least 60cm thick, irrespective of the other protective measures taken.

c) Limiting the compaction of expansive clays

Expansion pressure and potential volume change increase significantly with the dry density of swelling soils. High degree of compaction may therefore be detrimental and should be avoided. It is recommended that the dry density of expansive soils in no case exceeds 95% MDD.

d) Placing expansive soils at equilibrium moisture content

If possible, the equilibrium moisture content should be measured under existing roads of the project area. Otherwise, it can be assumed that the equilibrium moisture content is near the plastic limit. This applies in areas where the mean annual rainfall exceeds 500mm and the water table is non-existent or deep (grater that 5-6m). In arid areas or in case of a water table close to the ground level, a special study may be required to determine the equilibrium moisture content.

e) Preventing moisture change under the pavement

To control moisture change in swelling soils caused whether by external water or internal variation, the following measures can be adopted;

1. Use of impervious bituminous surfacing (multiple surface treatment or asphalt concrete (AC) to make the surface relatively impermeable as possible

2. Sealing shoulders and extending their width to at least equal to the depth of the zone affected by seasonal moisture change (not less than 2m)
3. The side ditches should be dispensed with, or if impractical, they should be pocketed as far away as feasible from the pavement and they should have sufficient section and grade to ensure that no water ponding can occur

2.2. Soil Stabilization

Soil stabilization is the process of altering the properties of a soil by applying some modifiers to meet specified Engineering requirements of road pavement layers.

Soil stabilization can be taken as alternate to borrow selected materials and it has advantage that the effect to the environment is reduced and in areas where selected/granular materials are scarce, stabilization have comparative economic advantage. The presence of organic matters and sulphates affects the effectiveness of stabilizers.

In road projects with weak subgrades, it is common practice to provide capping layers between the subgrade and the sub-base. The capping layer is of granular material of less quality of the specification requirement for sub-base material. As alternative to provision of capping layer of imported granular material, subgrade soil stabilization using different stabilizers such as lime, cement and fly ash has comparative advantage with respect to environmental protection and economic advantage in areas where the granular materials are scarce.

Starting from early human history, stabilizing agents such as lime and cement were used for construction of pyramids, bridges, dames and roads being mixed with soil. Early in the 20th Century, the Americans introduced the scientific way of stabilizing soils in road projects to improve their engineering properties by application of stabilizing agents such as lime and cement (TRH, 1986).

In road construction, all the naturally available material cannot be utilized as construction material as there exists some problematic soils (such as expansive soils) and soils with limitations to meet specifications and design standards. The problematic nature and limitations of such soils can be improved by application of stabilizing agents.

The application of stabilizing agents can improve(TRH, 1986):-

- Strength (stability and bearing capacity) of the soil
- Durability and resistance to the effect of water

- Volume stability
- Permeability
- Wet soils can be dry out
- The workability of clay soils
- Load spreading capacity of pavement layers

2.2.1. Techniques of Stabilization

I. Stabilization by Compaction

Loose materials can be made more stable simply by application of compaction. Though, compaction cannot be considered as stabilization process, it plays a fundamental role in the properties of stabilized materials.

II. Mechanical Stabilization

Mechanical stabilization is a process by which the gradation of soil is improved by the incorporation of another material which affects only the physical properties of the soil. In the case of mechanical stabilization, unlike other stabilizing agents, the proportion of the stabilizing material exceeds 10% and may be as high as 50%.

III. Stabilization using stabilizing agents

Application of stabilizing agents such as lime, cement and chemical stabilizers in low amount causes significant improvement in engineering properties of expansive soils.

IV. Chemical Stabilization

Chemical stabilization of soil is mixing of soil with one or a combination of admixtures of powder, slurry, or liquid for the general objective of improving or controlling its volume stability, strength and stress-strain behavior, permanently and durability (Winterkorn and Pamukcu, 1990).

Soil improvement by means of chemical stabilization can be grouped into three chemical reactions; cataion exchange, flocculation – agglomeration, pozzolanic reactions.

a. Cation Exchange

The excess of ions of opposite charge (to that of the surface) over those of like charge present in the diffuse double layer are called exchangeable ions. These ions can be replaced by a group of different ions having the same total charge by altering the chemical composition of the equilibrium electrolyte solution (Winterkorn and Pamukcu, 1990).

Negatively charged clay particles absorb cations of specific type and amount. The ease of replacement or exchange of cations depends on several factors, primarily the valence of the cation. Higher valence cations easily replace cations of lower valence. For ions of the same valence, the size of the hydrated ions becomes important; the larger the ion the grater the replacement power. If other conditions are equal, trivalent cations are held more tightly than divalent and divalent cations are held more tightly than monovalent cations (Mitchell and Soga, 2005). A typical replicability serious is;

$$Na^{+} < Li^{+} < K^{+} < Rb^{+} < Cs^{+} < Mg^{2+} < Ca^{2+} < Ba^{2+} < Cu^{2+} < Al^{3+} < Fe^{3+} < Th^{4+}$$

The exchangeable cations may be present in the surrounding water or be gained from the stabilizers.

An example of cation exchange (Sivapullah, 2006)

$$Ca^{2+} + Na^{+} - Clay \rightarrow Ca^{2+}\ Clay + (Na^{+})$$

b. Flocculation – Agglomeration

Cation exchange reaction result in the flocculation and agglomeration of the soil particles with consequent reduction in the amount of clay=size materials and hence the soil surface area, which inevitably accounts for the reduction in plasticity. Due to change in texture, a significant reduction in the swelling of the soil occurs (Terzaghi and Peck, 1967)

c. Pozzolanic Reaction

Time depending pozzolanic reactions play a major role in the stabilization of the soil, since they are responsible for the improvement in the various soil properties. Pozzolanicconstituents produces calcium silicate hydrate (CSH) and calcium aluminate haydrate (CAH) Show et al, 2003).

$$Ca^{2+} + 2(OH)^{-} + SiO_2\ (Clay\ Silica) \rightarrow CSH$$

$$Ca^{2+} + 2(OH)^{-} + Al_2O_3\ (Clay\ Alumina) \rightarrow CAH$$

The calcium silicate gel formed initially coats and binds lumps of clay together. The gel then crystalizes to form an interlocking structure thus, strength of the soils increases (Hadi et al, 2006; Sivapullaiah, 2006)

2.2.2. Mode of Stabilization (Nelson, J.D and Miller, D.S, (1992))

i. Stabilization by Physical Reaction

It is the simplest procedure by adding water, bitumen or other bonding materials with the soil. The bitumen is added to the soil in the form of a liquid of low viscosity, which is suitably converted in to a highly viscous semi solid state by reduction in temperature or by evaporation of the solvent. As bitumen is hydrophobic it also works as water proofing agent as well.

ii. Stabilization by Reaction between Two or More Chemicals

Most of the stabilizing agents in this category are bonding agents. They form the solid by the addition of two or more chemicals, which in themselves are not stabilizing agents but react by precipitation or polymerization is the formation of calcium silicate in a soil by the reaction of sodium silicate and calcium chloride.

iii. Stabilization by Chemical Reaction between the soil and Stabilizer

Hydrated lime is the best example of a bonding agent in this group; the strength being derived from the reaction between lime and the clay fraction of the soil. Once attached, they are difficult to displace and since each particle is in effect surrounded by hydrophobic cations, the soil then is made water proof.

2.3. Previous Similar Works

In our country, Ethiopia there are no much studies and significant experience on expansive soil improvement/stabilization. Particularly the performance of marble dust to improve the engineering properties of expansive soils is not much tried yet except it was used as asphalt filler in some road projects. However, experiences of other countries such as India shows that marble dust has significant effect to improve the engineering properties of expansive soils.

According to the International Journal of Earth Science and Engineering -2011, addition of marble dust to expansive clay reduces clay content, increases the percentage of coarser particles, reduces LL, raises the SL, decrease PI, activity and swelling potential. As the percentage of the stabilizer increases, swelling percentage, free swell ratio and rate of swell decrease. Samples having marble dust reached 50% of the total swelling earlier. By curing the samples, the rate of swell and swelling percentage decreased.

BASER (2009) conducted the major laboratory tests on the natural soil and marble dust blended sample (5% to 30%) and come up with the following conclusions;

- Addition of marble dust decreasesliquid limit (LL), plasticity index (PI), Activity and shrinkage index (SI) and increasesplastic limit (PL) and shrinkage limit (SL)
- Free swell decrease with increasing percentage of marble dust
- By addition of marble dust, the swelling percentage of the soil decrease considerably
- For increase percentage of stabilizer, the t_{50} values were decreased (i.e. samples having more stabilizers reached the 50% of total swell quickly)
- Swelling percentage and rate of swell of samples decrease by curing

Habtamu(2011)has conducted the major laboratory tests on natural expansive soil and the treated samples with different percentage of hydrated lime and ANSS and come up with the following conclusions;

- The engineering properties of the expansive soil improves with increasing percentage of the stabilizers (hydrated lime and ANSS) and prolonged duration of curing. In all cases the plastic index (PI) and swelling pressure decreases and the CBR increases with increasing percentage of the stabilizers.
- Addition of hydrated lime or ANSS up to 2%does not assure the improvement of the subgrade soil to the desired engineering properties
- Addition of 4% of either chemicals has sufficiently stabilized the subgrade soil and with further increment of the curing period the subgrade soil has further improved
- The performance of hydrated lime is higher than that of ANSS

NitshitTedla (2016)has conducted the major laboratory tests on natural expansive soil and treated samples with different percentage (5%, 10% and 15%) of Rice Husk Ash (RHA) and come up with the following conclusions;

- Atterberg limits, free swell, compaction, UCS and CBRresults shows slight improvement in the geotechnical properties of the expansive soil stabilized with RHA.
- UCS and uncooked CBR values increased appreciably with increase in RHA content and curing has significant effect on the RHA stabilized expansive soil.

CHAPTER THREE

OVERVIEW OF THE STUDY AREA

3.1. Introduction

The formation of soils is influenced by the specific environmental conditions of the area. The climatic condition, the geological and physiographic condition of an area has significant role on the formation of soils. Particularly expansive soils need special conditions to be fulfilled for their formation such as;

- Suitable parent material rich in calcium and magnesium to form smectite clay minerals, commonly basic igneous rocks such as basalt or gabbro
- Seasonal dry periods to allow clay minerals to form
- High temperature and sufficient rainfall for wetting but not much enough to leach base materials from the soil (commonly a semi-arid to dry tropical or sub-tropical climate)
- Impeded drainage to slow leaching and loss of weathering products
- High PH environment

As the main objective of this research work is to study the effect of Marble Dust on engineering properties of expansive soils to be used as road subgrade material, the Sembo – Sholagebeya road project is selected for the study. The Sembo – Sholagebeya road project is selected for the study for its highly expansive subgrade soil and its proximity to the capital Addis Ababa for ease of transportation.

In this section of the research the general overview of the study area including location, climatic condition, accessibility and geology will be presented in detail.

3.2. Location, Areal extent and Accessibility

The study area is located in central Ethiopia. The road project starts at Sembo town which is 93 km from Addis Ababa and terminates at Gindeber town of northern Shewa zone.There are two alternatives to access the project area. The first option is the Sembo – Gobensa road that branches off to the right from the Addis Ababa – Dessie trunk road at Sembo, about 93km from Addis Ababa. The other option is along the Modjo – Edjere – Arerti – Gobensa road starting from Modjo town which is about 73km from Addis Ababa on the Addis – Nazareth trunk road.

3.3. Climate

According to the Ethiopian Mapping Agency (EMA, 1981), the country is classified in to five climatic zones as describe below.

Table: 3.1 Climatic Zones of Ethiopia(EMA, 1981)

S. No	Description	Altitude
1	''Kur'' (Alpine)	Above 3000m
2	''Dega'' (Temperate)	2300m to 3000m
3	''WeinaDega'' (Sub tropical)	1500m to 2300m
4	''Kola'' (Tropical)	800m to 1500m
5	''Bereha'' (Desert)	Less than 800m

The project area is located in the western highlands of Ethiopia with elevation of 2680 – 3270 meters above sea level and fall in temperate/Dega climatic zones. The mean monthly temperature of the area varies between $5^{o}C$ and $20^{o}C$ and the mean monthly rainfall between 10mm and 300mm and the mean annual rainfall is in the range of 1000 mm to 1250mm. The moisture index of the majority of the project area is greater than one, which is referred as humid.

3.4. Geology of the area

In accordance to the Geological Map of Ethiopia (1996), the project area is categorised into two geologic formations.

1. The Tarmaber - Megezez Formation (NtB) and
2. Alajae Formation (PNa)

The Tarmaber - Megezez Formation is composed of transitional and alkaline basalts and the Alajae formations consists of transitional and sub alkaline basalts with minor rhyolite and trachyte.

3.5. Soil and Material

From the soil and material report prepared by CORE Consulting Engineers during design phase of the subject project, it has been noted that the Atterberg limits and classification tests of the subgrade soil revealed that all of the tested soils are mostly clayey soils. These soils are characterized by high plasticity index [>=15%] and liquid limit [>50] and fall under A-7-5 soil group. 65% of these natural subgrade materials have four days soaked CBR values of less than 5% and/ or swell index above 2%. From the laboratory test results it can be concluded that the majority of the subgrade materials are not suitable to serve as road bed materials.

CHAPTER FOUR

SUB GRADE SOIL CHARACTERIZATION

4.1. Background

Proper characterization and investigation of the existing ground surface (road subgrade material) plays a significant role for the design of the overlying sections of the road pavement. The stability and durability of pavements are largely dependent on the characteristics of the subgrade materials. The suitability of the subgrade material should be checked to avoid problems to be resulted due to problematic subgrade soils such as expansive soils. If such problematic soils are encountered, proper improvement measures need to be designed.To propose the improvement mechanisms to be adopted and to design feasible and economical subgrade, the detailed characteristics of the expansive subgrade soil need to be studied.

To meet the objectives of this research, field identification and laboratory tests were conducted to characterize the natural subgrade soil. The field investigation includes identification of soil based on observation of colour, soil texture and lithological position. The laboratory tests conducted on the natural subgrade soil include grain size analysis, Atterberg limits, linear shrinkage, moisture density-relations, CBR and percent swell of CBR.

4.2. Sub Grade Soil Survey

The site investigation for this research work was conducted in the month of January, 2014 which is highly dry season for the project area where expansive soils can be identified visually. The test results obtained during design and construction phases of the project for subgrade soils were studied and sections of the project with highly expansive soils were identified.

From the site reconnaissance survey and the laboratory tests conducted for the project work, the subgrade soil around km 23 was found to be highly expansive. Since the road bed preparation of this section of the project was already completed, samples for detailed laboratory study were collected from the road side.

4.3. Laboratory Investigation

The soil samples were first air dried and properly pulverised. Atterberg limits and linear shrinkage tests were conducted on soil samples passing #40 (4.25mm) sieve andthe other tests were conducted on soil samples passing # 4 (4.75mm) sieve.Laboratory tests conducted on the natural subgrade soilare Atterberg limits, Linear shrinkage, Grain size analysis, Moisture-Density relations (Proctor test), Californian Bearing Ratio (CBR) test and Swell from CBR test.

Note:
The test results of the natural soil discussed hereunder are only for the test pit of highly expansive soil section of the study area (the general soil characteristics of the study are is shown in appendix B and C),

4.3.1. Grain Size Distribution

a. Wet Sieve Analysis

To determine the distribution of coarser particles, 1500g of the natural subgrade soil is taken and washed on sieve size of 75µm and the results shown in Table 4.1 obtained.

Table: 4.1 Wet sieve analyses for the natural soil

Description		Natural soil	
Wt. Before wash (g)		1500	
Sieve No. (mm)	Wt. Retained (g)	% Retained	% Pass
4.75	5	0.33	99.67
2	7	0.47	99.53
0.425	33	2.20	98.67
0.075	20	1.33	97.80

b. Hydrometer Analysis

To determine the distribution of fine particles (silt and clay) 10g of air dried soil sample passing sieve 75 µm is used. The soil sample is socked in chemical solution (Sodium hexa-meta phosphate) for 24 hours. The test results are shown in Table 4.2.

Table 4.2: Hydrometer analysis results for the natural soil

Date tasted	**13/03/2014**
Hydrometer No.	152H
Specific Gravity of soil	2.7
Wt. of soil sample	50 g
Zero correction	+ 5
Meniscus correction	+ 1

The results of wet sieve analysis and haydrometer analysis are graphed in figure 4.1.

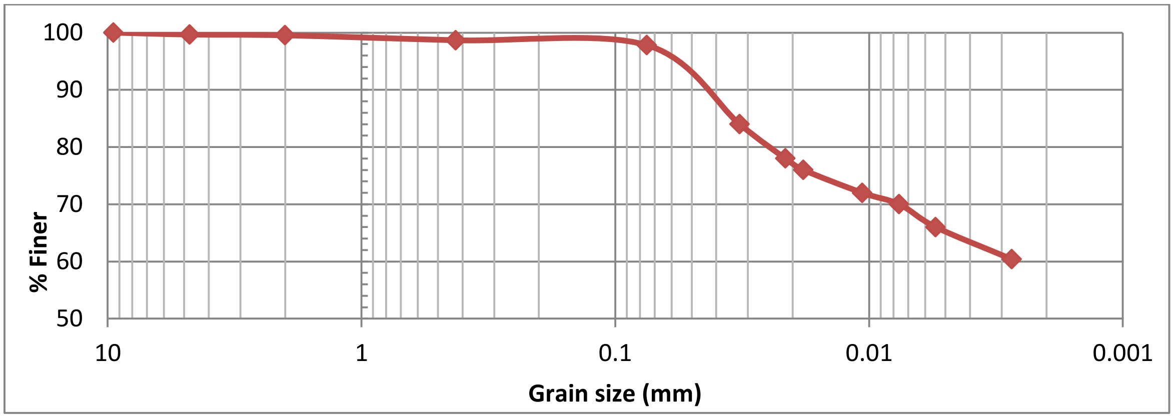


Fig. 4.1 Grain size distribution of natural sbgrade soil of the test pit (@ km 23)

4.3.2. Atterberg Limits

Fig, 4.2 Plot of Atterberg limits for the natural subgrade soil (km 23)

4.3.3. AASHTO Soil Classification

From the Atterberg limits and grain size analysis tests the natural subgrade soil of the study area has the engineering properties of LL = 88%, PI = 52 and More than 35% passing #200 sieve. As per AASHTO soil classification system, the subgrade soil falls in the category A-7-5/A-7- 6 with rating Fair-to- Poor to be used as subgrade material. Thus, the natural subgrade material is unsuitable to be used as subgrade material without employing some improvement methods.

4.3.4. Moisture Density Relationship (Procter Test)

Standard proctor test is conducted on the subgrade soil of the selected road section to determine the relationship between the moisture content and dry density for specific compaction effort according to AASHTO T99 as shown below.

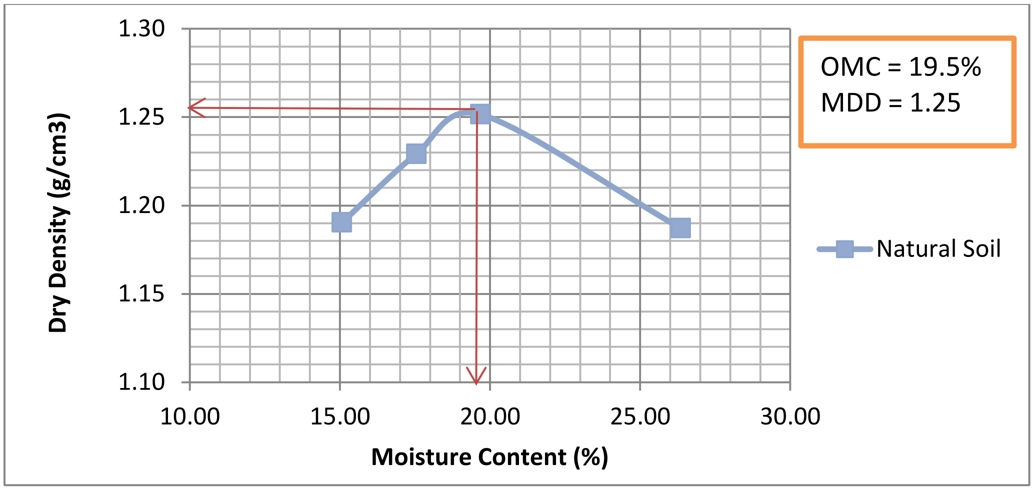


Fig. 4.3 Moisture-density relation for the natural soil (km 23)

4.3.5. Californian Bearing Ratio (CBR) Tests

One of the major laboratory tests to be conducted on subgrade and other pavement layers of a road is the CBR test. The CBR test results of a soil sample are used to evaluate the strength/ bearing capacity of a pavement layer and to rate the suitability of subgrade soils to carry the overall pavement load.

CBR test is intended for but not limited, evaluating the strength of cohesive materials having maximum particle size less than 19mm (AASHTO T193-93).

Generally about 10, 30, and 65 blows perlayer are suitable for compacting specimens 1, 2, and 3 respectively. More than 65 blows per layer are generally required to mold a CBR specimen to 100% of maximum dry density. Some laboratories may prefer to test only one specimen, which would be compacted to a maximum dry density at optimum moisture content (AASHTO T193-93). For the present study one point (one specimen) CBR test is conducted with 65 blows for each five layers.Unless specified by the authorized agency, or unless it has no effect on test results for the material being tested, all specimens shall be soaked prior to penetration (AASHTO T193-93).

Soaking of CBR specimen prior to penetration is used to simulate the field condition of the worst condition because of high rain fall and complete saturation of the entire soil mass due to presence of near ground water table. However, in our country Ethiopia in most part of the country, the ground water table is much lower than the annual moisture fluctuation depth (commonly 2-3m) and the annual rain fall intensity is not as high as the worst condition considered for CBR test. Thus, in this study the un-soaked conditions of CBR test are conducted on the natural subgrade soil and the marble dust blended samples to compare the result with the soaked case of the same.

According to ERA manual-2002, based on their CBR values earth materials to be used as different pavment layers are categorised as follows;

Table 4.3: Rating of materials based on their CBR value (ERA, 2002)

CBR (%)	General rating	To be used as:
0-3	Very poor	--
3-7	Poor to fair	Sub-grade
7-20	Fair	Sub-base
20-50	Good	Base coarse/sub-base
>50	Excellent	Base coarse

From the soaked CBR test, it was found that the natural subgrade soil has very low CBR value (1%) which is much less than the minimumrequirements for a soil to be used as subgrade material. Thus, the subgrade soil has to be removed and replaced by selected material of better strength (as done by the road project of the study area) or some methods of soil stabilization/ improvement methods need to be applied. However, for the un-soaked case the CBR value of the natural soil has shown significant valueto be used as subgrade material if other conditions such as sweel and plasticity are satisfied.

4.3.6. Potential Swell

The volume change/swell-shrink of expansive soils as result of moisture change is one of the significant identification features. The potential swell of expansive soils is important parameter to classify subgrade soils based on their expansiveness. Potential swell can be measured directly or indirectly.

i. Direct measurment of swell

The direct measurement of swell can be done by laboratory free swell method (Odometer test) and constant volume swell test.

- Swell from CBR mold

For the soaked case of CBR test, the volume change of the compacted specimen is measured befor and after soaking using dial gauge. The calculated swell value is showen in the table below;

Table: 4.4 Swell from CBR mould for the natural soil (km 23)

Dial Reading	**Before Socking (mm)**	**After Socking (mm)**
	2.40	12.30
CBR Swell (%)	8.6	

As per several specification requirements such as ERA site investigation Manual (2002), a subgrade soil should have a maximum swell of 2%. Thus, the natural soil of the subject study area has high swelling potential and cannot be used as subgrade material.

ii. Indirect measurment of swell

The indirect measurements involve the use of soil properties and classification schemes to estimate swelling potential. Some of indirect measurement methods of swell potential are described below;

a. Potential Swell based on Plasticity Chart (Casagrande method)

The plot of plasticity index (PI) against liquid limit (LL), helps to detect the swelling potential of soils by looking at where the soil samples fall in the chart. The swelling potential for any given plasticity index and liquid limit is indicated.

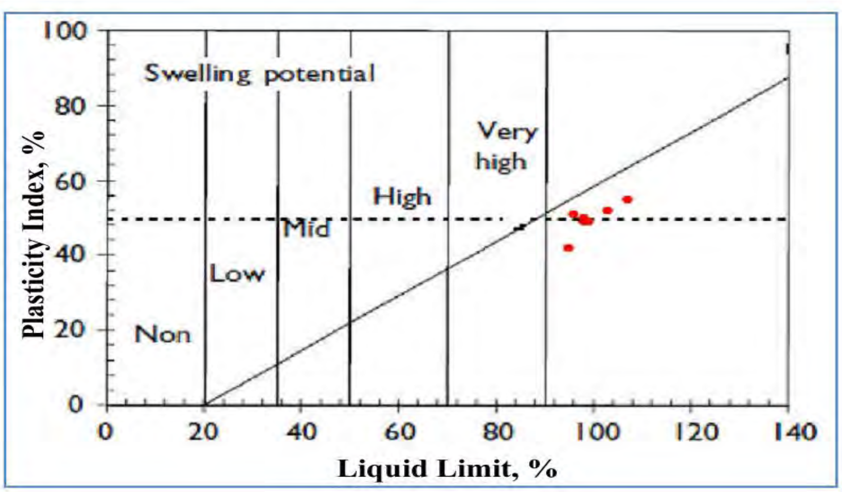


Fig. 4.4 Potential Swell of soil sample based on plasticity chart (Daksanamurthy, 1973)

From the Atterberg limit test of the present subgrade material, the LL= 88% and PI= 52. Thus, from the plasticity chart above the soil falls in the category 'inorganic clay of high plasticity' above A line, which indicates highly expansive soil.

b. Relation Between Swelling Potential and Plasticity Index (Pi)

According to F.H Chen (2008), swelling potential and PI are related as follows;

Table: 4.5 Relationship between PI and swelling potential (F.H Chen,2008)

Plasticity Index (PI)	Swelling Potential
0-15	Low
10-35	Medium
20-55	High
>55	Very high

From the Atterberg limit test conducted on the natural subgrade soil of the present study, it can be seen that the PI= 52, so the swelling potential of the soil is high.From the above results the subgrade soil of the subject project can be categorized as highly expansive soil.

4.3.7. Overall Characterization of the Subgrade Soil

From the field observation and soil survey of the study section of the project area the natural subgrade soil is found to be majorly expansive soil. The primary indications for expansive soil such as dark brown colour, widely spaced polygonal cracks were observed.The natural subgrade soil is found to have LL = 88%, PI = 52,very low load bearing capacity (CBR = 1%) and very high potential swell (8.5) far beyond the permissible limits. Such soils are potentially problematic and result in serious damages to pavements constructed over them without appropriate treatment/modification measures. Thus, one of the improvement methods mentioned in section 2.1.8 should be adopted before construction of overlaying pavement sections on such poor subgrade soils.

The present research work is aimed to investigate the performance of marble dust (by-product of marble grinding industry) to improve the poor strength and problematic nature of expansive soils to be used as road subgrade material as alternative to the methods discussed in section 2.1.8 of this paper.

CHAPTER FIVE

SUBGRADE SOIL STABILIZATION

5.1. Background

As discussed in section 2.2 of this paper, soil stabilization is the process of altering the properties of a soil by applying some modifiers to meet specified Engineering requirements. Soil stabilization can be taken as alternate to borrow selected materials and it has advantage that the effect to the environment can be reduced and in areas where selected materials are scarce, stabilization have comparative economic advantage.

In this research work the performance of marble dust as stabilizing agent to improve the engineering properties of expansive soils to be used as subgrade material will be investigated.

5.2. Common Stabilizing Agents

So far materials such as lime, cement, bitumen and other industrial chemicals have been applied to expansive soils to serve as stabilizing agents. Application of stabilizing agents such as lime, cement and chemical stabilizers in low amount shows significant improvement in engineering properties of expansive soils.

According to Habtamu Solomon, 2011, treatment of expansive soils with hydrated lime (CaO) 4% to 6% is found to gives good results in improving engineering properties of expansive soils. However, the stabilizing agents used so far are not economical as most of them arefactory products.For instance, lime treatment is costly as substantial thickness of soil (at least 30cm compacted thickness) has to be prepared. This treatment can be considered advantageous only if it is difficult to get selected backfill material nearby the project and when pavement savings can be made by taking advantage of the enhanced strength of the treated clay. Thus, low cost and easily available materials such as marble dust should be investigated to be used as alternative stabilizing agents to expansive soils.

5.3. Proposed Stabilizing Agent (Marble Dust)

5.3.1. Introduction

Marble is a metamorphosed limestone (i.e. limestone which has been fully re-crystallised and hardened under hydrothermal conditions). Marble Dust is by product of marble grinding industry.

Literature show that the need for marble products is increasingfrom time to time. According to Onur Baser (2009), marble production amount was 21.7 million tons in the year 1986in the world; however in the year 1998 this amount increased to 51 million tons showing 135% increment within 12 years. Increasing demand for marble production rises the generation of waste marble material. The production of marble discharged as waste during block production at the quarries is equal to 40-60% of the overall production volume (Celik, 1996). Large pieces of marble waste can be used as embankment or pavement material and the waste marble dust is used in cement and paper industries.

The row material is produced as block in the quarry site and transported to the factory by heavy trucks. In the industry the production process involves grinding the rock block to the required size and shape, polishing to have smooth surface and shaping to the required length and width. The processes of grinding, polishing and shaping are supplemented by application of water to reduce the formation of dust and to reduce heat produced owing to the friction between the marble and the grinding machine. The dust produced during grinding, polishing and shaping is accumulated in the form of marble mud slurry.

Fig. 5.1 Marble Block

Fig. 5.2 Grinding Machine

Fig. 5.3 Polisher Fig. 5.4 Marble Dust (Mud Slurry)

5.3.2. Source of Marble Dust

The Marble dust for this research work is obtained from Ethiopian Marble Processing Enterprise whichis located in Addis Ababa and it has three branches in the city around Gulele, Bole and Kirkos areas. The Industry was established in 1970s' by an Italian private Company and later nationalized by the government during the Dergue regime. The industry manufactures different marble products to be utilised in building constructions (floor tiles, windowsill and door siling), staircase, for decorating and siling objects to prevent them from the effect of weather such as graves and monuments.

The Ethiopian Marble Processing Enterprise is using three quarry sites for row material

1. Metekel Zone (BenishangulGumz Regional State)
2. MerabShewa (Oromiya Regional State)
3. Hakimgara (HareriRegional State)

The major by-products of marble grinding industry are marble chips and marble dust/mud slurry. The Ethiopian Marble Processing Enterprise used to dispose the by products in the previous years. However, currently the by-products specially the mud slurry is being utilized by other industries such as detergent, paint and chemical industries as additive material and as nutrient ingredient for poultry industry. The marble chips produced during grinding and shaping is being re crushed and used for production of terrazzo floor tiles in other factories.

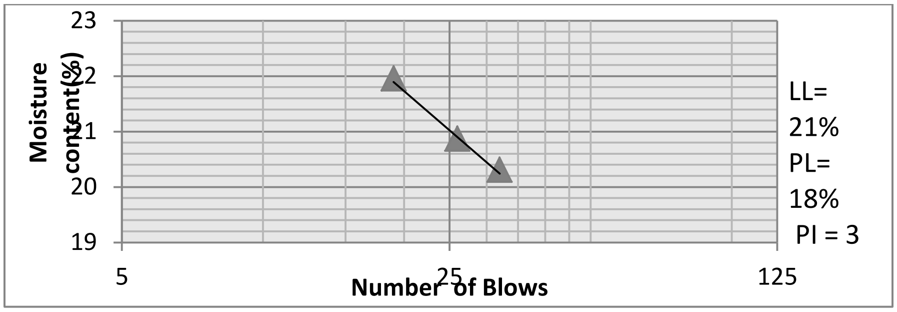
Large piecesof marble waste can be used as embankment or pavement material, and waste marble dust can be used as additives in some industries (paper, cement, ceramic etc.). However, only small portion of the waste marble products is utilized economically, most of them are stored on lands. Increasing of usage fields of waste marble products will eliminate the potentially harmful effects of them on environment and minimize the cost due to storage Onur Baser (2009).

In this research the application of marble dust (mud slurry) to improve the engineering properties of expansive soils to be used as road bedding material will be investigated.

5.3.3. Properties of Marble Dust

The marble dust used for the present study is whitish-grey in colour, fine grained and non-plastic material. The marble dust cannot be used as construction material by itself because of its fine and uniform grain sizes (poor gradation). But it can be used as filler in asphalt road projects where fine aggregate sizes are difficult to produce.

Some laboratory tests were conducted on the marble dust separately and it is noted that the specific gravity is 2.8 and it is found to be non-plastic as shown in the figure below



Fig, 5.5 Plot of Atterberg limits for the Marble Dust only

- The shrinkage limit was found to be almost zero. No shrinkage was observed after 24 hours oven drying
- The marble dust settles immediately in water and hence it is impossible to conduct hydrometers analysis

Fig, 5.6 Shrinkage test for marble dust (after oven drying)

Fig. 5.7 Atterberg Limit test on the marble dust

Fig. 5.8 Specific gravity test for marble dust

5.3.4. Percentage of Stabilizer

From different literature and research works it was noted that depending on the nature of the stabilizing agent and properties of the soil to be stabilized, the amount by present (weight or volume) of the stabilizing agent varies. For the present research the amount of marble dust to be applied on the expansive soil is from 5% to 30% by weight of the soil with 5% increment as lower ranges shows no much difference and the highest limit above 30% might not be economical.

5.4. Blending of Marble Dust and subgrade soil

Prior to blending, the natural subgrade soil and the marble dust were air dried and the subgrade soil was pulverized and sieved on 4.75mm sieve. For Atterberg limit tests, soil sample passing sieve size 4.75mm were again sieved on sieve size of 4.25mm and for hydrometer analysis they were sieved on 75µm sieve. Then each specimen was prepared by addition of marble dust with different percentage to have a sample with predetermined percentage of stabilizer varying from 5 to 30% by weight of the soil sample and well mixed by using mixing spoon.

Fig. 5.9 30% marble dust blended sample prepared for Atterberg limit tests

Fig.5.10 Marble dust blended sample prepared for Proctor and CBR tests

5.5. Laboratory Testing of Blended Material

5.5.1. Atterberg Limits

Soil sample passing #40 sieve is used to be mixed with different percentage of Marble Dust to study the changes in Atterberg limits and tests are conducted according to AASHTO T89 test procedures.

From the laboratory tests, it has been noted that the Atterberg limits (LL, PL & PI) decrease with increasing percentage of marble dust as shown in the graph below.

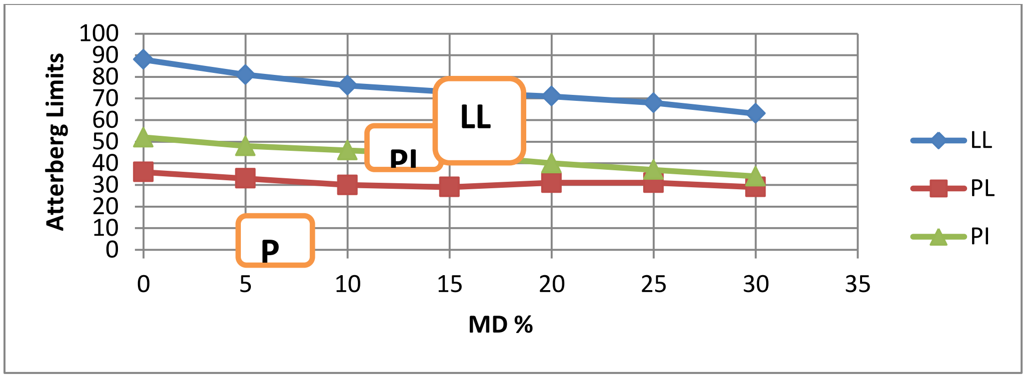


Fig.5.11 Relations between Atterberg limits and percentage of Marble Dust

5.5.2. Moisture Density Relations (Proctor Tests)

Air dried and pulverized soil sample passing #4 sieve is used to be mixed with different percentage of Marble Dust to determine moisture density relations in accordance with AASHTO T180-97. Moisture content versus dry density graph is plotted and the optimum Moisture Content (OMC) and Maximum Dry Density (MDD) are determined from the graph. From the test results it was observed that the optimum moisture content decrease and the maximum dry density increase with increasing percentage of marble dust as shown in the graph below.The variations in optimum moisture content and maximum dry density with increasing percentage of marble dust are presented in the following table;

Table :5.1 OMC and MDD with increasing percentage of marble dust

% of MD	0%	5%	10%	15%	20%	25%	30%
OMC (%)	20	22	20.5	19.5	19.5	18.3	18
MDD (g/cm3)	1.25	1.27	1.28	1.32	1.41	1.52	1.6

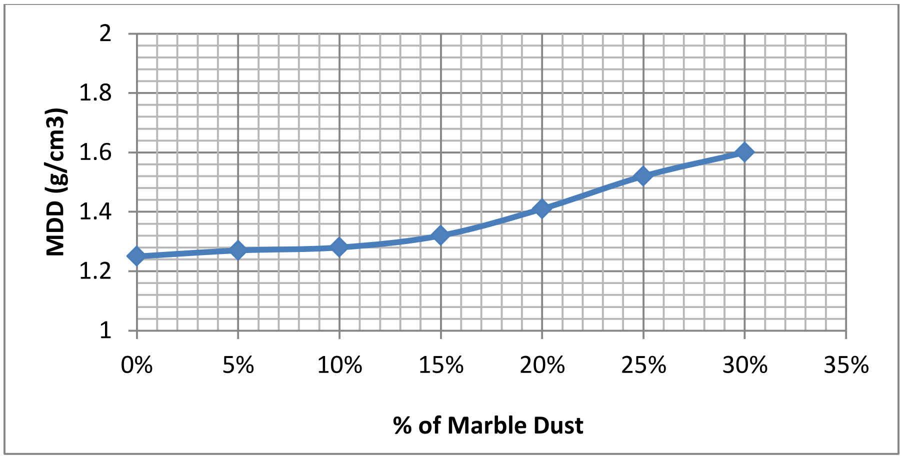


Fig. 5.12 MDDfor different percentage of marble dust

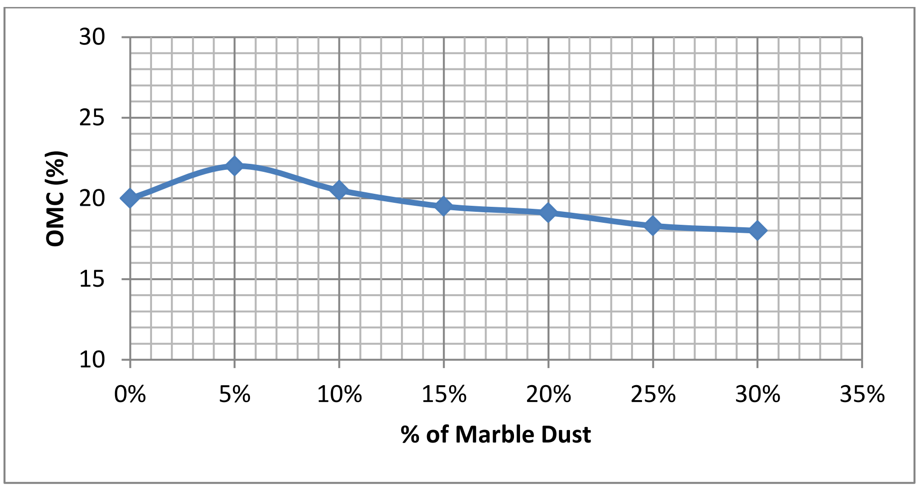


Fig. 5.13OMCfor different percentage of marble dust

The effect of curing on the engineering properties of the blended samples for 20% and 30% marble dust blended samples is observed for 7, 14 and 28 days curing. The curing was done by placing the blended sample in plastik bags for the required curing period to reduce loss of moisture.

From the test results obtained and shown in fig. 5.12 and 5.13, itisnoted that:

- Due to curing the maximum dry density (MDD) decrease and the optimum moisture content (OMC) increases for both 20% and 30% marble dust blended samples
- With increasing period of curing for both the 20% and 30%, the MDD increase and the OMC decrease but insignificantly

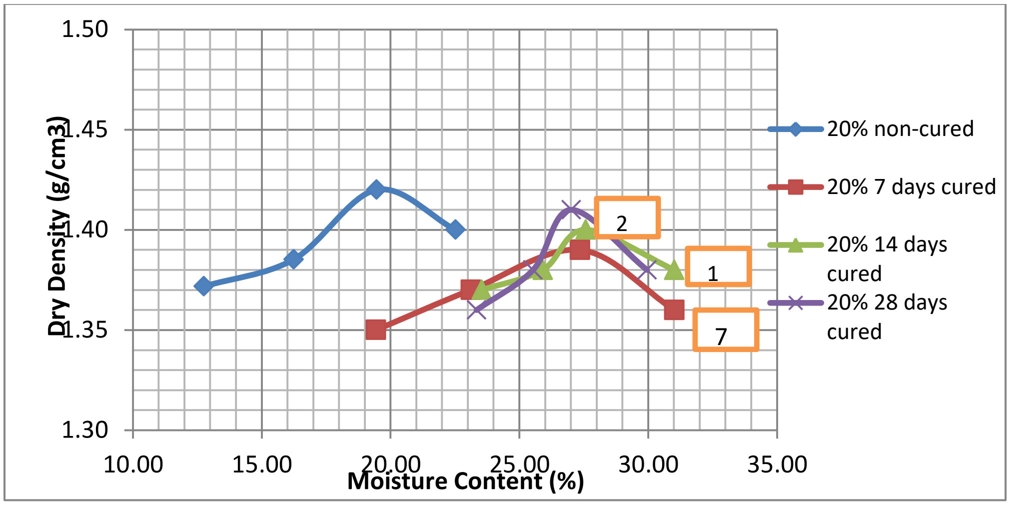


Fig. 5.14Effect of curing on 20% marble dust blended sample

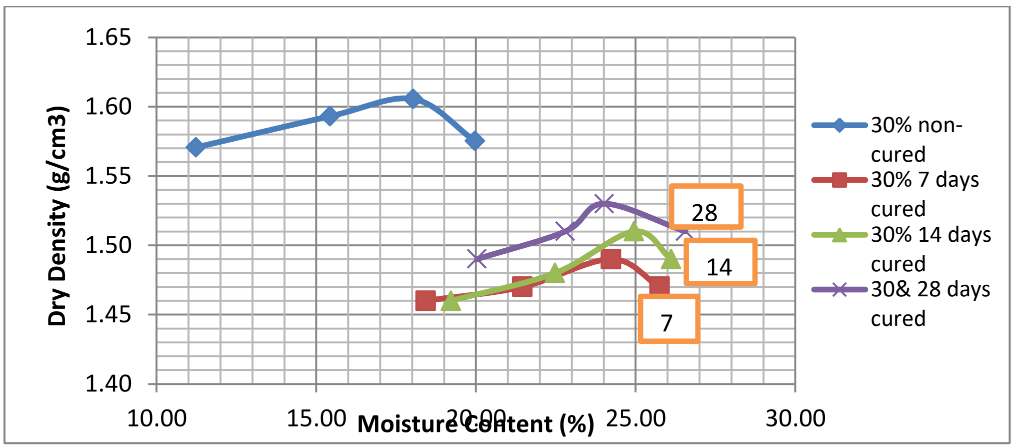


Fig. 5.15Effect of curing on 30% marble dust blended sample

5.5.3. Californian Bearing Ratio (CBR)

Air dried and well pulverized soil sample passing #4 sieve is blended with increasing percentage by weight of marble dust to study the effect of marble dust on the CBR values of the expansive subgrade soil. CBR test is intended for but not limited to evaluating the strength of cohesive materials having maximum particle size less than 19mm.Unless specified otherwise by the requiring authority, or unless it has no effect on test results for the material being tested, all specimens shall be soaked prior to penetration (AASHTO T193-93).

In this study the un-soaked conditions of CBR test are conducted on the natural subgrade soil and the 20% and 30% marble dust blended samples.For the soaked case, one point CBR with 56 blows and 4 days (96 hours) socking is conducted and CBR value at 100% MDD is determined on all specimens (0-30%). The soaked CBR test results for different percentage of marble dust are summarised in the tables below;

Table : 5.2 Soaked CBR values with increasing percentage of Marble Dust

% of Marble Dust	0%	5%	10%	15%	20%	25%	30%
CBR at **2.54**mm penetration	0.9	0.96	1.12	1.28	1.44	2.10	2.25
CBR at **5.08**mm penetration	0.85	0.95	1.06	1.27	1.38	2.01	1.91

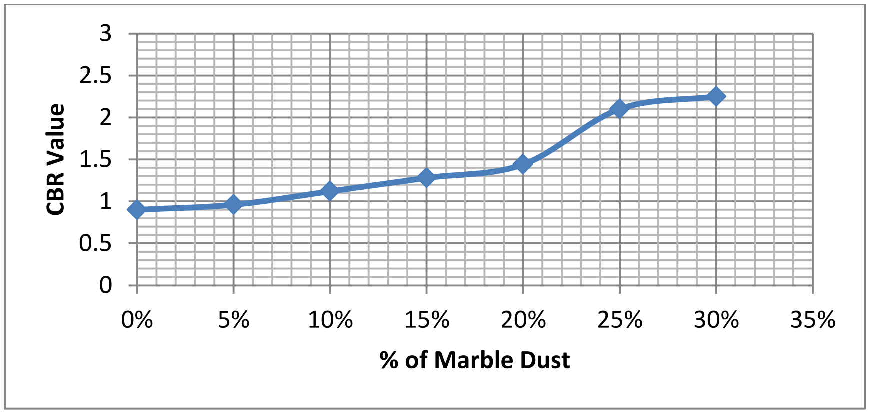


Fig. 5.16 CBR value with different percentage of Marble Dust

From the above table and graph it can be noted that though it is not significant, the soaked CBR value increases with increasing percentage of marble dust.

The effect of curing on the engineering properties of the blended samples for 20% and 30% marble dust blended samples is observed for 7, 14 and 28 days curing. The curing was done by placing the blended sample in plastic bags for the required curing period to reduce loss of moisture.

The 28 days cured 20% and 30% marble dust blended samples were also tested for both soaked and un-soaked CBR conditions.

Table: 5.3 Soaked CBR on cured samples

Days of curing	7		14		28	
% of Marble Dust	**20%**	**30%**	**20%**	**30%**	**20%**	**30%**
CBR on **2.54** mm penetration	2.09	3.53	2.57	3.85	2.89	4.50
CBR on **5.08** mm penetration	1.91	2.54	2.33	3.15	2.54	3.71

Table: 5.4 Un-soaked CBR on natural soil and 28 days cured samples

% of Marble Dust	Natural Soil	20%	30%
CBR at **2.54** mm penetration	7.22	12.04	21.19
CBR at **5.08** mm penetration	5.72	10.70	17.59

From the above tables it can be observed that, the CBR values increse with increasing percentage of marble dust and with increase period of curing.

5.5.4. Potential Swell

The potential swell of expansive soils is important parameter to classify soils based on their expansiveness. Expansive soils swell when they come in contact with moisture which will cause non uniform movement of the ground surface and instability of structures built on such soils. Potential swell can be measured directly or indirectly as discussed in section 4.3.6.

The swells of marble dust blended expansive soil samples are measured and determined from the CBR mould before and after socking. The results for natural soil, 10%, 20% and 30% marble dust blended samples are summarised in the table below. From the results it is observed that the potential swell decrease with increasing amount of marble dust and with increased period of curing the potential swell decreases.

Table :5.5 Swell from CBR test (Non cured samples)

% of Marble Dust	Natural Soil		10%		20%		30%	
Dial Reading (mm)	Before Socking	After Socking	Before Socking	After Socking	Before Socking	After Socking	Before Socking	After Socking
	2.40	12.50	2.3	9.5	2.1	8.6	1.8	8.0
% Swell	**8.6**		**6.2**		**5.6**		**5.3**	

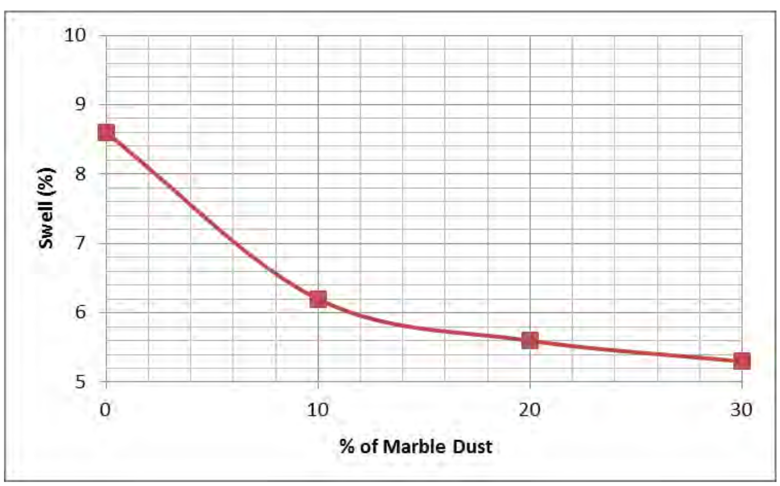


Fig. 5.17Swelling potential with increasing percentage of marble dust

5.5.4.1. Effect of Curing on Potential Swell

The 20% and 30% marble dust blended samples were cured for 7, 14 and 28 days and the potential swell from the CBR mouldsare measured and the results are summarized in the table below.

Table :5.6 Effect of curing on the potential swell of blended samples

Days of curing	7				14				28			
% of MD	20%		30%		20%		30%		20%		30%	
Dial reading (mm)	Before	After	Bef.	Aft.	Bef.	Aft.	Bef,	Aft.	Bef.	Aft.	Bef.	Aft.
	2.1	8.0	1.75	7.2	1.42	7.10	0.40	5.10	0.36	5.80	0.35	4.10
% Swell	5.15		4.5		4.88		4.04		4.67		3.22	

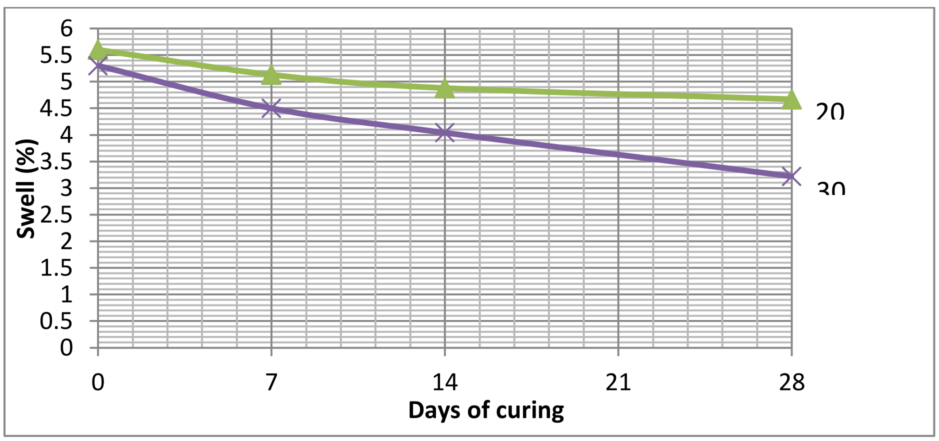


Fig. 5.18 Effect of curing on potential swell

5.5.5. Overall Characterization of Marble Dust Blended Expansive Soil

From the laboratory tests results described in section 5.5.1 to section 5.5.4 the application of marble dust has shown improvement in engineering properties and strength properties of the expansive soil.

The plasticity index (PI) reduced from 52% for the natural subgrade soil to 36% by addition of 30% by weight of marble dust and the liquid limit (LL) reduced from 88% to 65% by addition of the same amount of marble dust.

The natural subgrade soil has a CBR value of almost 1%. The application of marble dust improved the bearing capacity of the natural soil. For instance, by addition of 30% by weight of marble dust to the natural subgrade soil, the CBR value increased to 2.25 though it is not enough to characterize the soil as good subgrade material. The natural soil and the 20% and 30% marble dust blended samples are also tested for un-soaked conditions to simulate the most unsaturated conditions of the major parts of our country. The un-soaked CBR values indicated that the natural soil by itself has good bearing capacity.

From the direct and indirect swell tests it was observed that the potential swell of the soil reduced significantly by application of increasing percentage of marble dust and with increasing period of curing. For instance, the swell of the 30% marble dust blended sample has reduced to 5.3% and 3.22% for non-cured and cured case respectively from 8.6% of the natural subgrade soil.

Generally as compared to lime stabilization, marble dust is poor material to be used as stabilizing agent for expansive soils. According to Habtamu S. (2011), even though, addition of 2% hydrated lime does not assure the improvement of the subgrade soil to the desired engineering properties, 4% hydrated lime has sufficiently stabilized the subgrade soil and with further increment of the curing period the subgrade soil has further improved

CHAPTER SIX

INTERPRETATION AND DISCUSSION

6.1.Preamble

As result of their shrink-swell behavioursfollowing seasonal moisture fluctuation, expansive soils are found to be challenging material to be used as foundation or road subgrade material. The problems associated with expansive soils are more serious on light weighted structures such as road pavements and short story buildings as such structures could not counter balance the lateral and uplift pressure to be exerted by expansive soils. So far, several techniques have been developed and tried to reduce/eliminate the problematic nature of such soils due to their volume change.

6.2.Performance of Marble Dust as Stabilizing Agent

6.2.1. Improvement on Atterberg Limits

The natural subgrade soil was found to have a liquid limit of 88%, plastic limit of 36% and plasticity index of 52%. The blending of the subgrade soil with increasing percentage by weight of the marble dust has shown improvement on the values of Atterberg limits as shown below.

Table 6.1: Effect of marble dust on Atterberg limits

Marble Dust (%)	LL	PL	PI
Natural Soil	88	36	52
5	81	33	48
10	76	30	46
15	73	29	44
20	71	31	40
25	68	31	37
30	63	29	34

6.2.2. Effect of curing on the Atterberg limits

Tostudy the effect of mellowing the 30% marble dust blended sample has been cured for 28 days and tested for Atterberg limits and it has shown significant reduction of the liquid limit and plasticity index as shown in the table below.

Table: 6.2Effect of curing on Atterberg limits

Days of curing	% of marble dust	Atterberg limits	
		LL	PI
None cured	30	63	34
28 days	30	52.5	22.3

6.2.3. Improvement on Swelling Potential

The swelling potential of marble dust blended with expansive soil samples are measured and determined from the CBR mould before and after socking. The results for natural soil, 10%, 20% and 30% marble dust blended samples are summarised in the table below.

Table:6.3Potential swell of natural soil and blended samples

% of Marble Dust	Natural soil	10%	20%	30%
% Swell	8.6	6.2	5.6	5.3

6.2.4. Effect of Curing on Potential Swell

The 20% and 30% marble dust blended samples were cured for 7, 14 and 28 days and the potential swell from the CBR moulds are measured and the results are summarized in the table below.

Table:6.4Effect of curing on the potential swell of blended samples

Daysof curing	7		14		28	
% ofMarble Dust	20%	30%	20%	30%	20%	30%
% Swell	5.15	4.5	4.88	4.04	4.67	3.22

From the results obtained it can be observed that;

- The potential swell decreases with increasing amount of marble dust
- With increased period of curing, the potential swell decreases

6.2.5. Improvement on CBR

The CBR value of the natural expansive subgrade soil has shown improvement when blended with increasing percentage of marble dust as shown below.

Marble Dust (%)	**CBR Value (%)**
Natural Soil	0.90
5	0.96
10	1.12
15	1.28
20	1.44
25	2.10
30	2.25

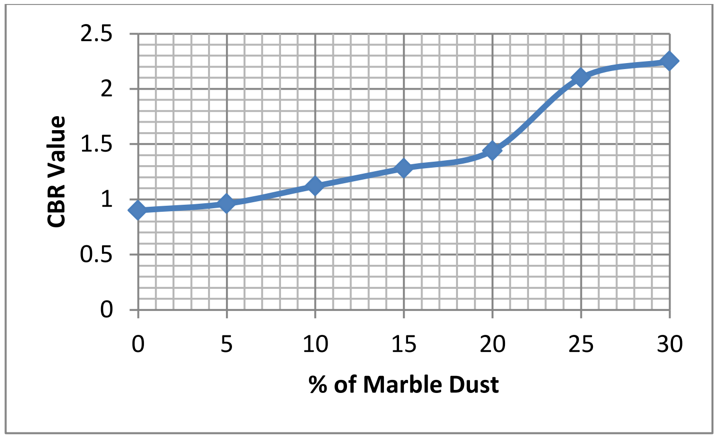


Fig.

Fig. 6.1 Improvement on CBR

6.2.6. Effect of Curing on CBR value of blended sample

To allow the blended sample to mellow and to observe if some kind of chemical reactions are happening between the expansive soil and the marble dust, the 20% and 30% marble dust blended samples were kept in plastic bags for the periods of 7, 14 and 28 days and the CBR values are investigated after the end of the respective curing periods. The results are shown in the table below

Table: 6.5Effect of curing on CBR values

Days of Curing	**% of Marble Dust**	**CBR Value**
None cured	20	1.44
	30	2.25
7	20	2.09
	30	3.53
14	20	2.57
	30	3.85
28	20	2.89
	30	4.50

The effect of curing for the 20% and 30% marble dust blended sample is shown in the graph below

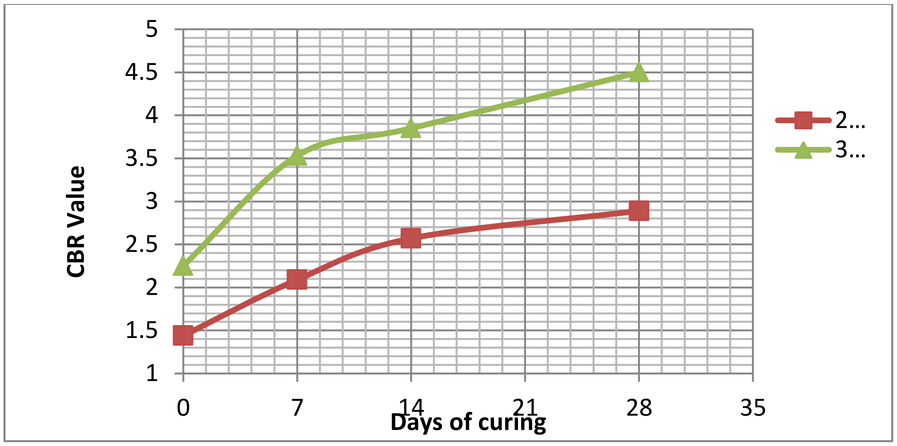


Fig. 6.2Effect of curing on CBR for 20% and 30% marble dust blended sample

6.2.7. Un-soaked CBR

According to AASHTO T193-93, unless specified otherwise by the requiring agency, or unless it has no effect on test results for the material being tested, all specimens shall be soaked prior to penetration.

Soaking of CBR specimen prior to penetration is to simulate worst field conditions because of high rain fall and complete saturation of the entire soil mass due to near ground water table. However, in Ethiopia in most part of the country, the ground water table is much lower than the annual moisture fluctuation depth, 'active zone' (commonly 2-3m) and the annual rain fall intensity is not as high as the worst condition considered for the soaked CBR test. The study area of this research work is located near Rift Valley and hence the ground water table is at lower depth.Thus, in this study un-soaked CBR tests are conducted on the natural subgrade soil and compered with CBR of blended soil using20% and 30% marble dust blended samples.

The effect of un-soaked CBR on pavement thickness is investigatedby considering an ideal section of expansive subgrade soil (attached in Appendix D).

Table: 6.6Un-soaked CBR on natural soil and 28 days cured samples

% of Marble Dust	*Natural Soil*	*20%*	*30%*
CBR at 2.54 mm penetration	*7.22*	*12.04*	*21.19*
CBR at 5.08 mm penetration	*5.72*	*10.70*	*17.59*

The subgrade class of theun-soaked natural soil has shown improvement from S1 to S3. Similarly for the 20% and 30% marble dust blended samples the subgrade class improves significantly for the un-soaked condition as summarised in the table and figure below.

Table 6.7: CBR value and subgrade class for different conditions

Specimen	Natural soil		20% Marble dust		30% Marble dust	
	CBR	Subgrade Class	CBR	Subgrade Class	CBR	Subgrade Slass
Soaked	0.9	--	2.89	S1	4.5	S2
Un-soaked	7.22	S3	12.04	S4	21.19	S5

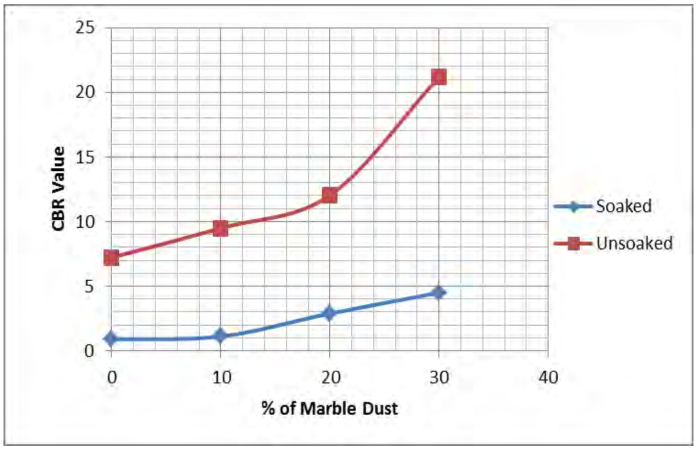


Fig. 6.3 Comparison between Soaked and un-soaked CBR

From the above result it can be noted that the 30% marble dust blended sample has a subgrade class of **S5** which is rated as '***Good***' subgrade material as specification requirements of several standards.

One of the determining factors for pavement thickness is the subgrade class besides the traffic class. If the subgrade material is found to be poor to serve as road bed, to reduce the stress to be carried by the subgrade, higher thickness of pavement is required and hence the project cost increases. For a given traffic class, the higher the subgrade class, the lower the pavement thickness.

From the design document of the study road project (Core Consulting Engineers, 2009), it has been found that the traffic class of the route is **T4**and for the highly expansive sections the subgrade class is S1. Thus, as per ERA Pavement Design Manual (2013), the recommended pavement thickness on the natural subgrade is 675mm (300mm granular capping layer, 225mm Cement/Lime stabilised road base and 150mm Granular road base). However, the un-soaked natural soil and the 30% marble dust blended subgrade soil have shown significant improvement of the subgrade class which is S3 for the natural soil and S5 for the 30% marble dust blended subgrade soil for the same traffic class and hence the pavement thickness is recommended to be 300mm and 425mm respectively (see Appendix D).

CHAPTER SEVEN

CONCLUSIONS AND RECCOMENDATIONS

7.1 Conclusions

From laboratory test results it has been observed that with increasing percentage of marble dust and with prolonged periods of curing, the expansive subgrade soil has shown improvement in engineering properties though it is not significant to meet specification requirements of several pavement design manuals. However, considering the results of un-soaked CBR tests on the natural subgrade soil and the 30% marble dust blended soil, the engineering properties improved and considerable amount of project cost reduction is observed.

The findings of the present study are summarised as follows;

- As per several specification requirements, a material to be used as road subgrade should have a liquid limit (LL) less than 60%, plasticity index (PI) less than 30%, soaked CBR greater than 5% and a potential swell less than 2%. However, the natural subgrade soil of the subject project section (LL=88%, PI=52%, CBR=1% and swell 83.6%) does not fulfil all the requirements.
- The swelling potential of the natural subgrade soil has reduced from *High* to *Medium* by application of 30% by weight of marble dust
- With increasing percentage of marble dust, the subgrade material has shown improvement in the engineering properties as summarised below;

Table: 7.1 Improvement in engineering properties by increasing amount of marble dust

% of MD	LL	PI	CBR	Swell
0	88	52	0.9	8.6
5	81	48	0.96	
10	76	46	1.12	6.2
15	73	43	1.28	
20	71	40	1.44	5.6
25	68	37	2.1	
30	63	34	2.25	5.3

- With increased period of curing, the blended samples have shown improvement in engineering properties.
- Un-soaked CBR tests are conducted on the natural subgrade soil and the 20% and 30% blended samples and found that the un-soaked CBR of the natural subgrade soil and marble

dust blended samples are significant enough to improve the unsuitable subgrade soil to be suitable to be used as subgrade material and to reduce pavement thickness and hence project cost.

7.2 Recommendations

Based on the findings of this research work and general practices in the road sector, the following recommendations are forwarded;

- In areas such as the present study area where selected materials can be found easily in the economic distance, removal of the unsuitable material and replacing it with selected borrow material will be feasible and economical rather than looking for subgrade improvement mechanisms.
- Due to geological formation and some other factors expansive soils may have different properties from pace to place. Thus, it is recommended that the performance of marble dust as stabilizing agent should be studded on expansive soils of different origins.
- In this research work it has been found that the marble dust up to 30% has not shown significant improvement of engineering properties of the expansive subgrade soil to meet specification requirements for road subgrade material. Previous similar works indicate that other stabilizing agents such as lime have effective results in improving the engineering properties of expansive soils. But lime stabilization is found to be effective for more than 4% of the lime. Thus, it is recommended that if further studies are conducted by combining marble dust and lime with lower percentage (1%-3%).
- This research is conducted by considering limited parameters of Atterberg limits, moisture density relations, CBR and percent swell of CBR on natural soil and blended samples. It is recommended that parameters of swelling potential, unconfined compressive strength and mineralogical tests (on marble dust and blended soil) shall also be conducted to have more realistic results.
- Soaked CBR is conducted to consider field condition of wetting by near ground water table or high rainfall. But, in areas where the ground water table is far away from the influence zone (such as areas in the Rift Valley) and the annual rainfall is minimal, soaked CBR test will underestimate the tested material. Thus, it is recommended that further studies should be

conducted to investigate the relation between weather condition and ground water table with the appropriate type of laboratory test

- From the un-soaked CBR test conducted on the natural soil and 30% marble dust blended soil, it is recommended that;

  - ✓ In arid areas where the ground water table is below the influence zone and the annual average rain fall is low, un-soaked CBR is recommended
  - ✓ If proper drainage is provided to prevent infiltration of surface runoff, un-soaked CBR for natural soil and 30% marble dust blended expansive subgrade soil gives significant reduction of the project cost
  - ✓ By application of marble dust, the earth work volume reduces and hence the corresponding environmental pollution and degradation is reduced
  - ✓ Additional separate and detailed research work on un-soaked CBR laboratory tests on subgrade soils especially on arid and semi-arid areas of the country is recommended

Finally, the results and findings of this research work may be considered as indicative only for further studies as these findings are based on limited parameters and small number of samples. More elaborate sampling and testing of expansive soils from different origins is recommended before concluding the performance of marble dust as stabilizing agent for expansive soils

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Annexes

Appendix A:

Material Description of the Study Area

Source: Site Investigation and Soil and Material Report Prepared by AEC and CORE Consultants during design phase of the project

Change		Material Description
From	To	
25+120	25+900	Dark clay soil
25+900	26+200	Slightly weathered Rocks
26+200	29+900	Dark clay soil
29+900	30+100	Slighlty weathered Rocks
30+100	30+600	Brown silty clay soil
30+600	30+850	Dark clay soil
30+850	31+150	Moderate to highly weathered rock
31+150	34+000	Dark clay soil underlain by moderate to highly weathered rock
34+000	34+300	Closely jointed and slight to moderately weathered rock
34+300	34+820	Dark clay soil underlain by slight to moderately weathered rock
34+820	35+080	Weathered and decomposed rock overlain by brown silty clay soil
35+080	35+540	Dark clay soil underlain by highly weathered rock
35+540	35+700	Basaltic boulders outcrop
35+700	36+150	Dark clay soil underlain by slight to moderately weathered rock
36+150	36+900	Brown silty clay soil underlain by fractured and decomposed rock
36+900	37+800	Brown to dark brown silty clay soil underlain by slight to moderately weathered rock
37+800	37+960	Dark silty clay-expansive
37+960	38+050	Reddish brown silty clay soil
38+050	38+150	Moderately weathered and fractured rock
38+150	39+100	Dark silty clay soil
39+100	39+200	Brownish gravelly silty clay soil underlain by decomposed rock
39+200	39+850	Brown to dark brown silty clay soil underlain by slight to moderately weathered rock
39+850	40+030	Dark clay soil
40+030	40+400	Dark clay underlain by moderately weathered rock
40+400	42+150	Dark clay soil
42+150	42+800	Brownish silty clay soil
42+800	43+220	Dark clay soil
43+220	43+360	Slightly weathered rock

Change		Material Description
From	To	
43+360	43+500	Dark clay soil underlain by moderately weathered rock
43+500	43+650	Dark clay soil underlain by slighlty weathered rock
43+650	43+750	Slightly weathered rock
43+750	44+040	Dark clay
44+040	44+350	Dark brown silty clay underlain by decomposed rock
44+350	44+800	Dark clay soil underlain by decomposed rock
44+800	44+900	Highly weathered and decomposed rock
44+900	45+250	Dark clay soil underlain by decomposed rock
45+250	45+500	Brownish siltly clay soil underlain by decomposed rock
45+500	45+840	Decomposed rock
45+840	45+900	Dark clay soil
45+900	46+600	Dark brown silty clay soil underlain by decomposed rock
46+600	46+850	Slightly weathered rock
46+850	47+100	Dark brown silty clay underlain by decomposed rock
47+100	47+800	Dark brown silty clay soil
47+800	48+500	Dark clay soil mixed with rock fragments
48+500	49+500	Dark clay soil underlain by decomposed rock
49+500	49+700	Moderately weathered rock
49+700	50+600	Dark clay soil
50+600	51+200	Dark clay soil mixed with rock fragments
51+200	51+900	Dark brown silty clay soil mixed with rock fragments and underlain by decomposed rock
51+900	52+200	Brownish silty clay soil
52+200	52+600	Moderate to highly weathered rock
52+600	53+800	Brownish soil mixed with cobbles and boulders and underlain by decomposed rock
53+800	55+200	Blocks, boulders and cobbles mixed with brownish soil and underlain by moderate to highly weathered rock
55+200	55+600	Blocks and boulders underlain by slightly weathered rock
55+600	57+200	Moderatately to highly weathered and decomposed trachy basalt
57+200	58+160	Brown to dark brown silty clay soil

Appendix B:

Summary of Test Results of Subgrade Material of the Study Area

Source: Project Laboratory(Sembo-Sholagebeya road project)

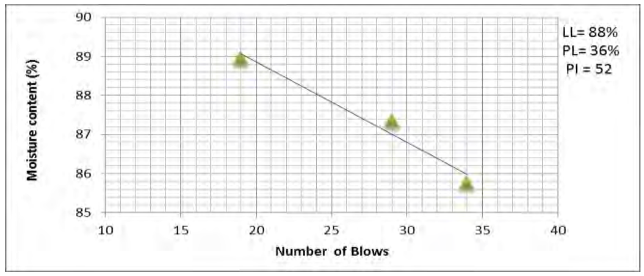
Appendix C:

Summary of Test Results of the Study

1. Natural Soil

1.1. Atterberg Limits

Description	Liquid Limit (LL)			Plastic Limit (PL)	
No. of Blows	34	29	19		
Container No.	8	29	15	43	37
Wt. of Container + Wet soil (g) = (W1)	47.5	47.4	45.1	20.41	20.46
Wt. of Container + Dry soil (g) = (W2)	33.76	33.6	32.3	19.75	19.7
Wt. of container (g)= (W3)	17.74	17.8	17.91	17.87	17.59
Wt. of moisture (g) = (W1-W2)=A	13.74	13.8	12.8	0.66	0.76
Wt. of Dry Soil (g) =(W2-W3) = B	16.02	15.8	14.39	1.88	2.11
Moisture Content (%) = (A/B)*100	85.77	87.34	88.95	35.11	36.02
			Avg. PL	35.56	



1.2. Gradation

Wet sieve analysis

Description	Natural soil at depth >60cm		
Wt. Before wash (g)		1500	
Sieve No.	Wt Retained (g)	% Retained	% Pass
4.75	5	0.33	99.67
2	7	0.47	99.53
0.425	33	2.20	97.80
0.075	20	1.33	98.67

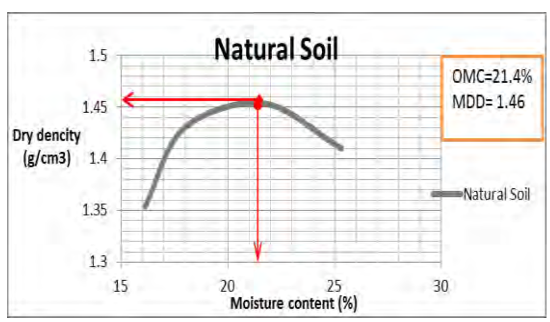
Hydrometer Analysis

Date tasted	13/03/2014
Hydrometer No.	152H
Spesific Gravity of soil	2.7
Wt. of soil sample	50 g
Zero correction	.+ 5
Miniscous correction	.+ 1

S/ No	Date	Time	Elapsed Time (min)	Actual Hydrometer Reading	Composite Correction	Hydrometer Reading with composite correction Applied	Temp. ("C)	Effective depth of Hydrometer L (cm) from Table 7.4	Value of K (from Table 7.3)	Diameter of Soil particle D (mm)	Ct From Table 3	a From Table	Corrected haydrometer Reading (Rc)	(% finer), P (%)	(Adjested % finer), Pa (%)
1	13/03/2014	03:25	0	47	1	48	20	8.4	0.0134	0.03235	0	1	42	84	
2		03:27	2	44	1	45	20	8.9	0.0134	0.02835	0	1	39	78	
3		03:30	5	43	1	44	20	9.1	0.0134	0.01813	0	1	38	76	
4		03:40	15	41	1	42	20	9.4	0.0134	0.01064	0	1	36	72	
5		03:55	30	40	1	41	20	9.6	0.0134	0.00760	0	1	35	70	
6		04:25	60	38	1	39	20	9.9	0.0134	0.00546	0	1	33	66	
7		07:25	250	35	1	36	21	10.4	0.0133	0.002741	0.2	1	30.2	60.4	
8	14/03/2014	03:25	1440	34	1	35	20	10.6	0.0134	0.001153	0	1	29	58	
9															
10															

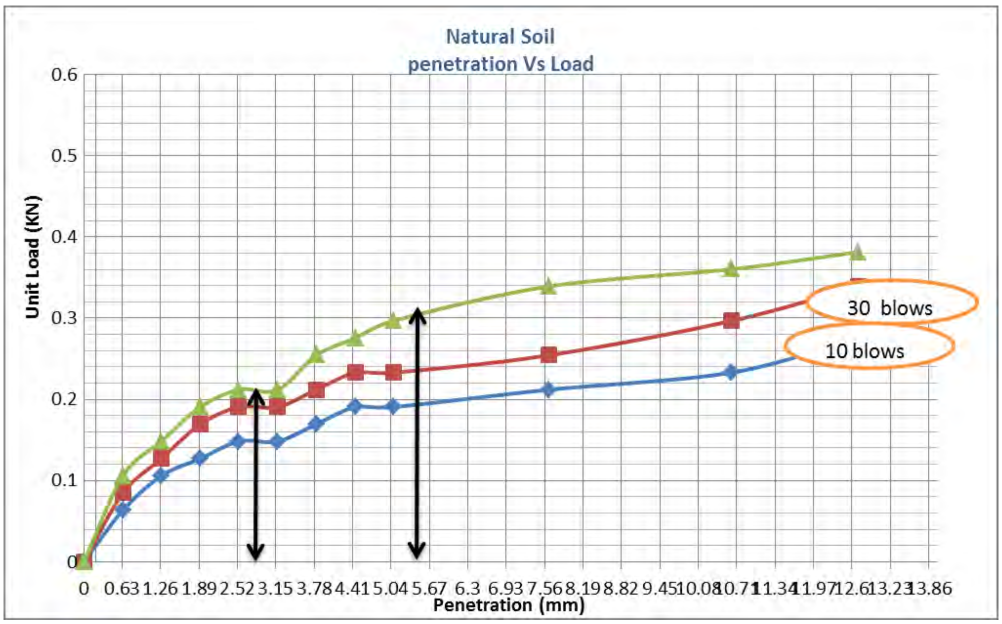
1.3. Moisture Density Relationship (Procter Test)

Target moisture content	%	8	11	14	17	
Mass of wet soil + mould	A (g)	10233.00	10470.00	10643.00	10639.00	
Mass of mould	B (g)	6953.00	6953.00	6953.00	6953.00	
Mass of wet soil	C=A-B (g)	3280.00	3517.00	3690.00	3686.00	
VOLUME OF MOULD (CC)	V	2086.00	2086.00	2086.00	2086.00	
Bulk density	C / V = W	1.57	1.69	1.77	1.77	**NMC**
Moisture determination container No.		R	Q	H	M	U
Mass of container + wet soil	a (g)	344.50	342.10	345.50	312.90	422.60
mass of container + dry soil	b (g)	307.20	301.30	297.60	264.70	397.50
Mass of container	d (g)	76	74	76.9	74.40	75.6
Mass of dry soil	b - d = e (g)	231.20	227.30	220.70	190.30	321.90
Mass of moisture	a - b = f (g)	37.30	40.80	47.90	48.20	25.10
Moisture content	f/e*100 = m (%)	16.13	17.95	21.70	25.33	7.80
Dry density	W / (100+m) *100 (Kg/m3)	1.35	1.43	1.45	1.41	
Dry density for zero air void	=[1 / (w/100) +1/Gs)]					



1.4. Californian Bearing Ratio (CBR) Tests

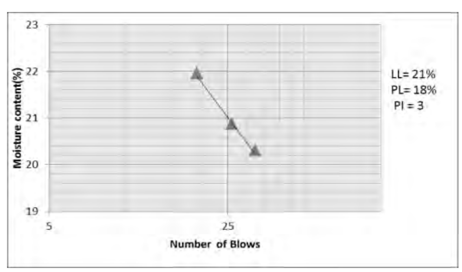
Ring Factor:	21.190	N/Divistion							
	10 Blows			30 Blows			65 blows		
PLUNGER PENT.(mm)	DIAL	LOAD(KN)	CBR %	DIAL	LOAD(KN)	CBR%	DIAL	LOAD(KN)	CBR %
0	0	0.00		0	0.00		0	0.00	
0.64	3	0.06		4	0.08		5	0.11	
1.27	5	0.11		6	0.13		7	0.15	
1.91	6	0.13		8	0.17		9	0.19	
2.54	7	0.15	1.11	9	0.19	1.43	10	0.21	1.59
3.18	7	0.15		9	0.19		10	0.21	
3.81	8	0.17		10	0.21		12	0.25	
4.45	9	0.19		11	0.23		13	0.28	
5.08	9	0.19	0.96	11	0.23	1.17	14	0.30	1.49
7.62	10	0.21		12	0.25		16	0.34	
10.62	11	0.23		14	0.30		17	0.36	
12.7	13	0.28		16	0.34		18	0.38	



2. Marble Dust

2.1. Atterberg Limits

Description	Liquid Limit (LL)			Plastic Limit (PL)	
No. of Blows	32	26	19		
Container No.	8	29	15	43	37
Wt. of Container + Wet soil (g) = (W1)	52.3	48.75	51.32	29.03	28.92
Wt. of Container + Dry soil (g) = (W2)	46.8	43.85	45.7	27.78	27.57
Wt. of container (g)= (W3)	19.72	20.37	20.1	20.78	19.87
Wt. of moisture (g) = (W1-W2)=A	5.5	4.9	5.62	1.25	1.35
Wt. of Dry Soil (g) =(W2-W3) = B	27.08	23.48	25.6	7	7.7
Moisture Content (%) = (A/B)*100	20.31	20.87	21.95	17.86	17.53
			Avg. PL	17.69	

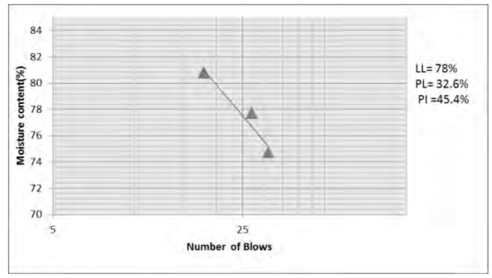


3. Marble dust Blended Samples

3.1. Atterberg Limits

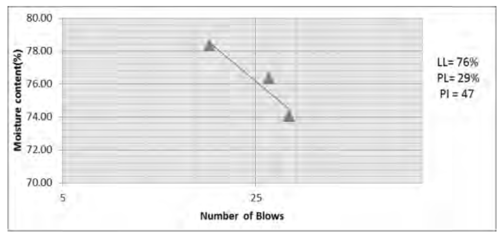
5%

Description	Liquid Limit (LL)			Plastic Limit (PL)	
No. of Blows	31	27	18		
Container No.	9	21	27	12	34
Wt. of Container + Wet soil (g) = (W1)	42.56	39.16	41.59	22.39	22.41
Wt. of Container + Dry soil (g) = (W2)	32.1	29.82	30.96	21.27	21.27
Wt. of container (g)= (W3)	18.1	17.8	17.8	17.8	17.8
Wt. of moisture (g) = (W1-W2)=A	10.46	9.34	10.63	1.12	1.14
Wt. of Dry Soil (g) =(W2-W3) = B	14	12.02	13.16	3.47	3.47
Moisture Content (%) = (A/B)*100	74.71	77.70	80.78	32.28	32.85
			Avg. PL	32.56	



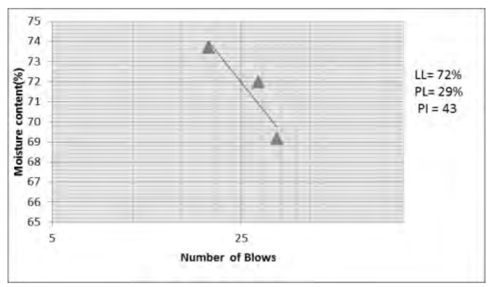
10%

Description	Liquid Limit (LL)			Plastic Limit (PL)	
No. of Blows	33	28	17		
Container No.	4	10	11	15	29
Wt. of Container + Wet soil (g) = (W1)	43.03	38.58	41.51	22.39	22.31
Wt. of Container + Dry soil (g) = (W2)	32.35	29.6	31.1	21.38	21.3
Wt. of container (g)= (W3)	17.93	17.84	17.81	17.91	17.79
Wt. of moisture (g) = (W1-W2)=A	10.68	8.98	10.41	1.01	1.01
Wt. of Dry Soil (g) =(W2-W3) = B	14.42	11.76	13.29	3.47	3.51
Moisture Content (%) = (A/B)*100	74.06	76.36	78.33	29.11	28.77
			Avg. PL	28.94	



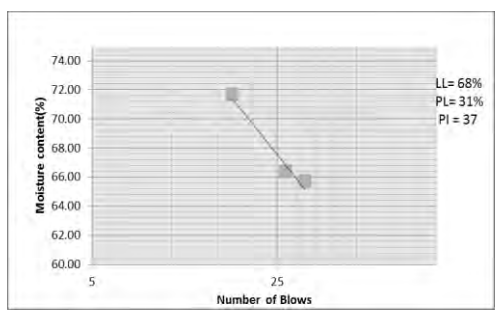
15%

Description	Liquid Limit (LL)			Plastic Limit (PL)	
No. of Blows	34	29	19		
Container No.	8	22	38	43	49
Wt. of Container + Wet soil (g) = (W1)	46.02	47.49	46.64	23.35	23.51
Wt. of Container + Dry soil (g) = (W2)	34.46	35.05	34.47	22.13	22.36
Wt. of container (g)= (W3)	17.75	17.77	17.96	17.87	18.36
Wt. of moisture (g) = (W1-W2)=A	11.56	12.44	12.17	1.22	1.15
Wt. of Dry Soil (g) =(W2-W3) = B	16.71	17.28	16.51	4.26	4
Moisture Content (%) = (A/B)*100	69.18	71.99	73.71	28.64	28.75
			Avg. PL	28.69	



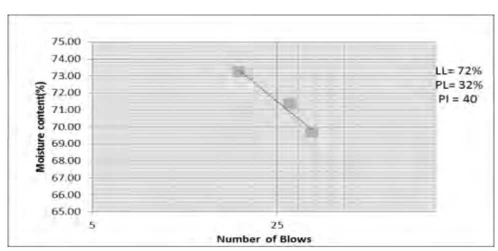
20%

Description	Liquid Limit (LL)			Plastic Limit (PL)	
No. of Blows	32	27	17		
Container No.	1	14	30	35	39
Wt. of Container + Wet soil (g) = (W1)	50.03	50.78	49.82	22.8	22.28
Wt. of Container + Dry soil (g) = (W2)	37.24	37.65	36.67	21.58	21.14
Wt. of container (g)= (W3)	17.79	17.88	18.33	17.61	17.52
Wt. of moisture (g) = (W1-W2)=A	12.79	13.13	13.15	1.22	1.14
Wt. of Dry Soil (g) =(W2-W3) = B	19.45	19.77	18.34	3.97	3.62
Moisture Content (%) = (A/B)*100	65.76	66.41	71.70	30.73	31.49
			Avg. PL	31.11	



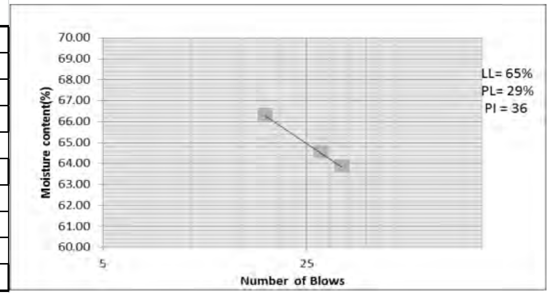
25%

Description	Liquid Limit (LL)			Plastic Limit (PL)	
No. of Blows	34	28	18		
Container No.	3	6	19	40	44
Wt. of Container + Wet soil (g) = (W1)	44.66	47.63	45.3	22.82	22.55
Wt. of Container + Dry soil (g) = (W2)	33.67	35.1	33.64	21.6	21.37
Wt. of container (g)= (W3)	17.9	17.55	17.72	17.75	17.68
Wt. of moisture (g) = (W1-W2)=A	10.99	12.53	11.66	1.22	1.18
Wt. of Dry Soil (g) =(W2-W3) = B	15.77	17.55	15.92	3.85	3.69
Moisture Content (%) = (A/B)*100	69.69	71.40	73.24	31.69	31.98
			Avg. PL	31.83	



30%

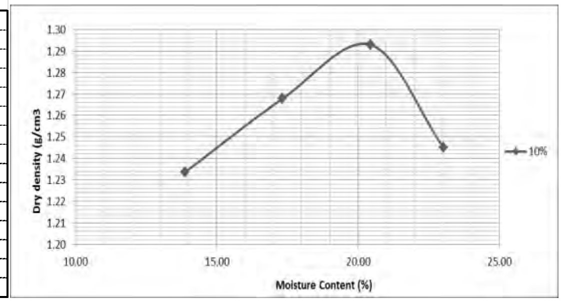
Description	Liquid Limit (LL)			Plastic Limit (PL)	
No. of Blows	33	28	18		
Container No.	29	30	37	22	28
Wt. of Container + Wet soil (g) = (W1)	50.02	50	51.34	23.04	22.94
Wt. of Container + Dry soil (g) = (W2)	37.45	37.57	37.88	21.82	21.85
Wt. of container (g)= (W3)	17.78	18.32	17.59	17.76	17.88
Wt. of moisture (g) = (W1-W2)=A	12.57	12.43	13.46	1.22	1.09
Wt. of Dry Soil (g) =(W2-W3) = B	19.67	19.25	20.29	4.06	3.97
Moisture Content (%) = (A/B)*100	63.90	64.57	66.34	30.05	27.46
			Avg. PL	28.75	



3.2. Moisture Density Relationship (Procter Test)

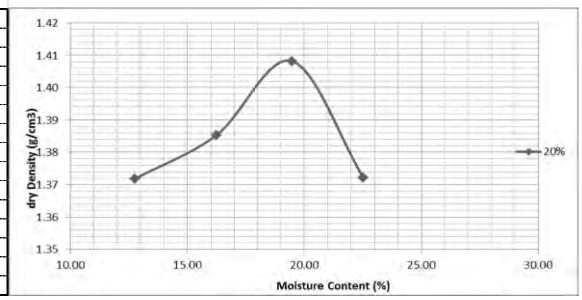
10%

Description						
Target moisture content	%	5	8	11	14	
Mass of wet soil + mould	A (g)	5246.00	5324.00	5390.00	5366.00	
Mass of mould	B (g)	3920.00	3920.00	3920.00	3920.00	
Mass of wet soil	C=A-B (g)	1326.00	1404.00	1470.00	1446.00	
VOLUME OF MOULD (CC)	V	944.00	944.00	944.00	944.00	
Bulk density	C / V = W	1.40	1.49	1.56	1.53	NMC
Moisture determination container No.		O	K	W	R	G
Mass of container + wet soil	a (g)	369.90	286.20	337.80	330.80	330.50
mass of container + dry soil	b (g)	334.10	255.20	293.20	283.10	311.10
Mass of container	d (g)	76	76.2	75	75.90	77.3
Mass of dry soil	b - d = e (g)	258.10	179.00	218.20	207.20	233.80
Mass of moisture	a - b = f (g)	35.80	31.00	44.60	47.70	19.40
Moisture content	f/e*100 = m (%)	13.87	17.32	20.44	23.02	8.30
Dry density	W / (100+m) *100 (Kg/m3)	1.23	1.27	1.29	1.25	
Dry density for zero air void =[1 / (w/100) +1/Gs)]						



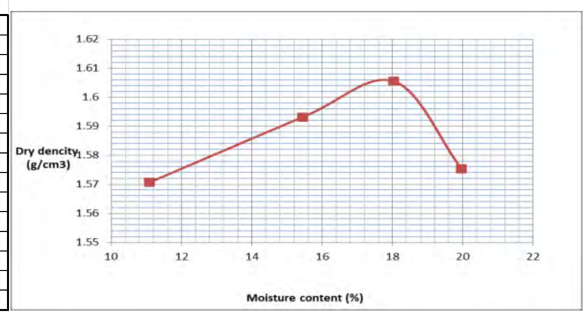
20%

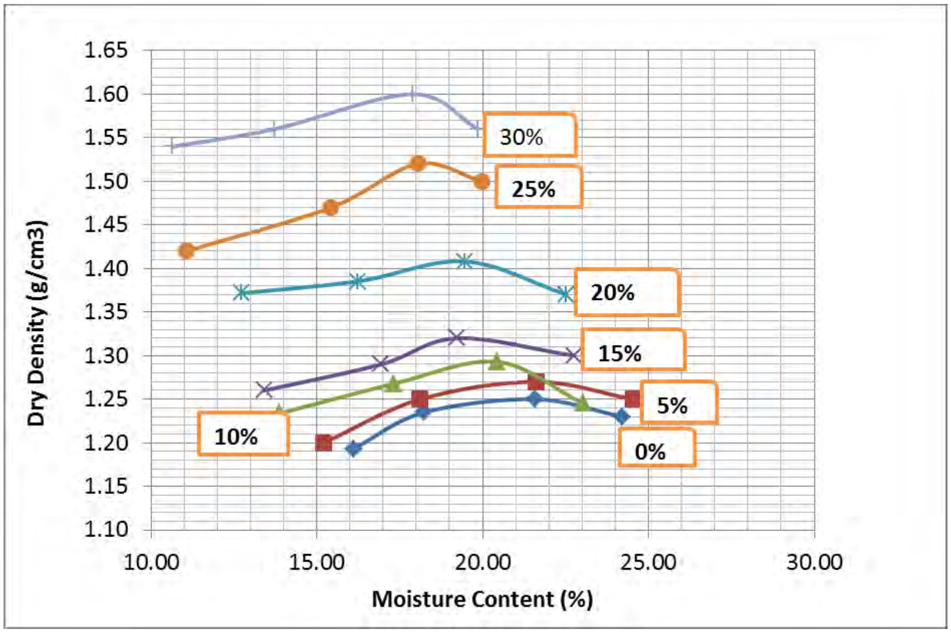
Description						
Target moisture content	%	5	8	11	14	
Mass of wet soil + mould	A (g)	5370.00	5430.00	5498.00	5497.00	
Mass of mould	B (g)	3910.00	3910.00	3910.00	3910.00	
Mass of wet soil	C=A-B (g)	1460.00	1520.00	1588.00	1587.00	
VOLUME OF MOULD (CC)	V	944.00	944.00	944.00	944.00	
Bulk density	C / V = W	1.55	1.61	1.68	1.68	NMC
Moisture determination container No.		K	4	6	O	U
Mass of container + wet soil	a (g)	226.40	257.70	284.60	194.60	192.20
mass of container + dry soil	b (g)	206.40	228.70	246.50	168.20	181.80
Mass of container	d (g)	49.5	50.1	50.7	51.00	50.9
Mass of dry soil	b - d = e (g)	156.90	178.60	195.80	117.20	130.90
Mass of moisture	a - b = f (g)	20.00	29.00	38.10	26.40	10.40
Moisture content	f/e*100 = m (%)	12.75	16.24	19.46	22.53	7.94
Dry density	W / (100+m) *100 (Kg/m3)	1.37	1.39	1.41	1.37	
Dry density for zero air void =[1 / (w/100) +1/Gs)]						



30%

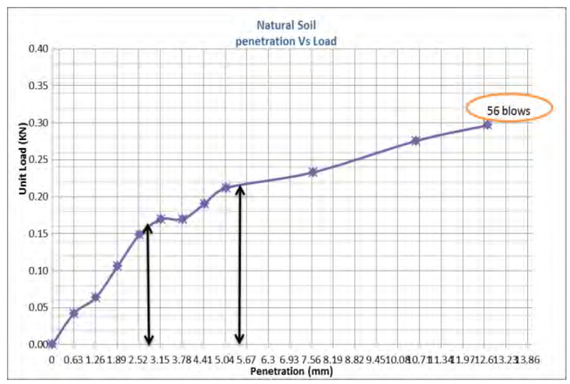
Description						
Target moisture content	%	6	8	11	14	
Mass of wet soil + mould	A (g)	5553.00	5642.00	5695.00	5690.00	
Mass of mould	B (g)	3906.00	3906.00	3906.00	3906.00	
Mass of wet soil	C=A-B (g)	1647.00	1736.00	1789.00	1784.00	
VOLUME OF MOULD (CC)	V	944.00	944.00	944.00	944.00	
Bulk density	C / V = W	1.74	1.84	1.90	1.89	NMC
Moisture determination container No.		R	D	H	T	K
Mass of container + wet soil	a (g)	289.30	239.70	282.80	306.80	354.60
mass of container + dry soil	b (g)	268.00	218.00	251.30	270.40	338.30
Mass of container	d (g)	75.8	77.4	76.7	88.10	76.4
Mass of dry soil	b - d = e (g)	192.20	140.60	174.60	182.30	261.90
Mass of moisture	a - b = f (g)	21.30	21.70	31.50	36.40	16.30
Moisture content	f/e*100 = m (%)	11.08	15.43	18.04	19.97	6.22
Dry density	W / (100+m) *100 (Kg/m3)	1.57	1.59	1.61	1.58	
Dry density for zero air void =[1 / (w/100) +1/Gs)]						



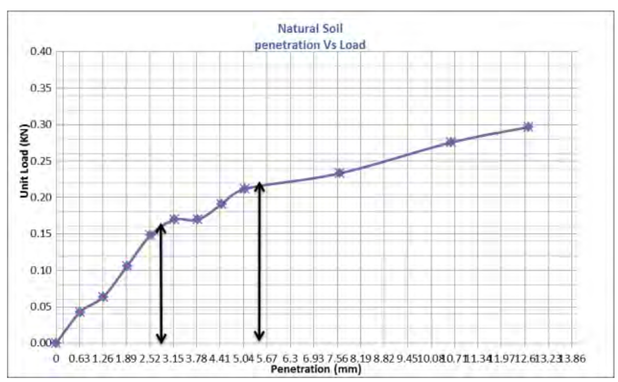


3.3. Californian Bearing Ratio (CBR) Tests

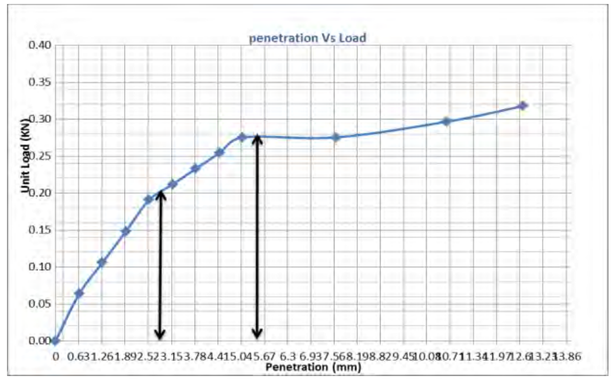
Ring Factor:	21.190	N/Divistion	
5%			
	56 Blows		
PLUNGER PENT.(mm)	DIAL	LOAD(KN)	CBR %
0	0	0.00	
0.64	2	0.04	
1.27	3	0.06	
1.91	4	0.08	
2.54	6	0.13	0.96
3.18	7	0.15	
3.81	7	0.15	
4.45	8	0.17	
5.08	9	0.19	0.95
7.62	11	0.23	
10.62	13	0.28	
12.7	14	0.30	



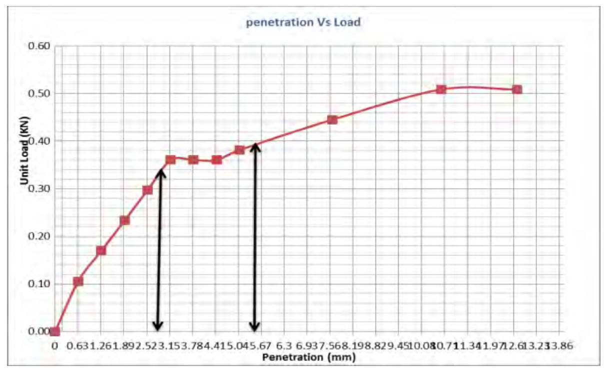
Ring Factor:	21.190	N/Divistion	
10%			
	56 Blows		
PLUNGER PENT.(mm)	DIAL	LOAD(KN)	CBR %
0	0	0.00	
0.64	2	0.04	
1.27	3	0.06	
1.91	5	0.11	
2.54	7	0.15	1.12
3.18	8	0.17	
3.81	8	0.17	
4.45	9	0.19	
5.08	10	0.21	1.06
7.62	11	0.23	
10.62	13	0.28	
12.7	14	0.30	



Ring Factor:	21.190	N/Divistion	
20%			
	56 Blows		
PLUNGER PENT.(mm)	DIAL	LOAD(KN)	CBR %
0	0	0.00	
0.64	3	0.06	
1.27	5	0.11	
1.91	7	0.15	
2.54	9	0.19	1.44
3.18	10	0.21	
3.81	11	0.23	
4.45	12	0.25	
5.08	13	0.28	1.38
7.62	13	0.28	
10.62	14	0.30	
12.7	15	0.32	



Ring Factor:	21.190	N/Divistion	
30%			
	56 Blows		
PLUNGER PENT.(mm)	DIAL	LOAD(KN)	CBR %
0	0	0.00	
0.64	5	0.11	
1.27	8	0.17	
1.91	11	0.23	
2.54	14	0.30	2.25
3.18	17	0.36	
3.81	17	0.36	
4.45	17	0.36	
5.08	18	0.38	1.91
7.62	21	0.44	
10.62	24	0.51	
12.7	24	0.51	



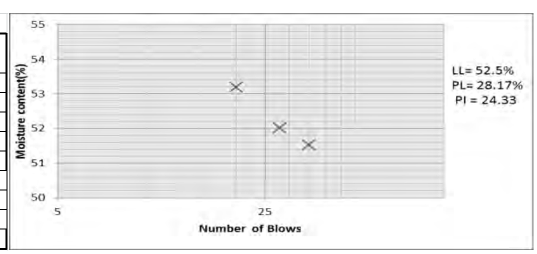
3.4. Summary of Test Results

Amount of Marble Dust (%)	Atterberg Limits		Proctor		1 point CBR	Swell from CBR
	LL	PI	OMC	MDD		
Natural Soil	88	52	21.4	1.46	0.9	8.6
5%	81	48	22	1.27	0.96	
10%	76	46	20.5	1.28	1.12	6.2
15%	73	44	19.5	1.32	1.28	
20%	71	40	19.5	1.41	1.44	5.6
25%	68	37	18.3	1.52	2.10	
30%	63	34	18.0	1.60	2.25	5.3

4. Effect of Curing on Marble Dust Blended Samples

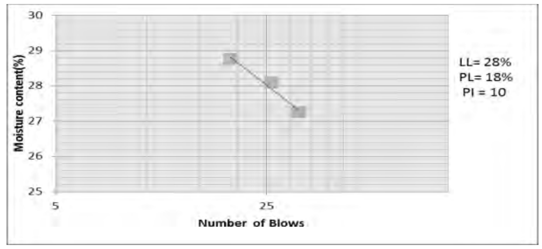
4.1. Atterberg Limits

30% and 7 days cured					
Description	Liquid Limit (LL)			Plastic Limit (PL)	
No. of Blows	35	28	20		
Container No.	105	C52	C49	D	C27
Wt. of Container + Wet soil (g) = (W1)	46.28	47.68	47.67	30.88	30.84
Wt. of Container + Dry soil (g) = (W2)	37.31	38.63	38.24	28.37	28.4
Wt. of container (g)= (W3)	19.9	21.23	20.51	19.5	19.7
Wt. of moisture (g) = (W1-W2)=A	8.97	9.05	9.43	2.51	2.44
Wt. of Dry Soil (g) =(W2-W3) = B	17.41	17.4	17.73	8.87	8.7
Moisture Content (%) = (A/B)*100	51.52	52.01	53.19	28.30	28.05
			Avg. PL	28.17	



30% and 28 days cured

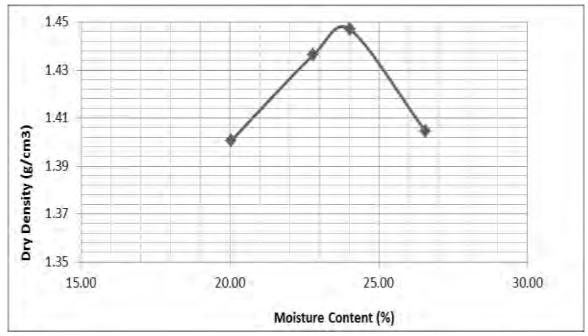
Description	Liquid Limit (LL)			Plastic Limit (PL)	
No. of Blows	32	26	19		
Container No.	105	C52	C49	D	C27
Wt. of Container + Wet soil (g) = (W1)	52.66	51.89	49.71	29.78	27.67
Wt. of Container + Dry soil (g) = (W2)	45.79	44.9	43.22	28.33	26.54
Wt. of container (g)= (W3)	20.59	20.02	20.66	20.08	20.65
Wt. of moisture (g) = (W1-W2)=A	6.87	6.99	6.49	1.45	1.13
Wt. of Dry Soil (g) =(W2-W3) = B	25.2	24.88	22.56	8.25	5.89
Moisture Content (%) = (A/B)*100	27.26	28.09	28.77	17.58	19.19
			Avg. PL	18.38	



4.2. Moisture Density Relationships

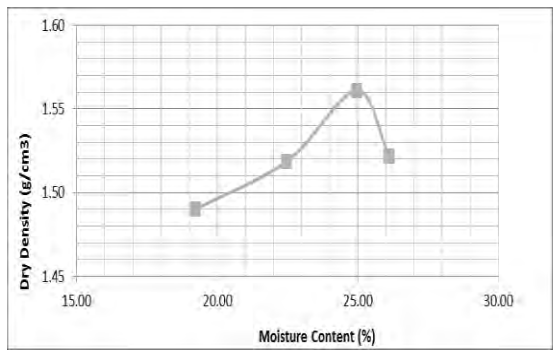
30% 7 Days cured

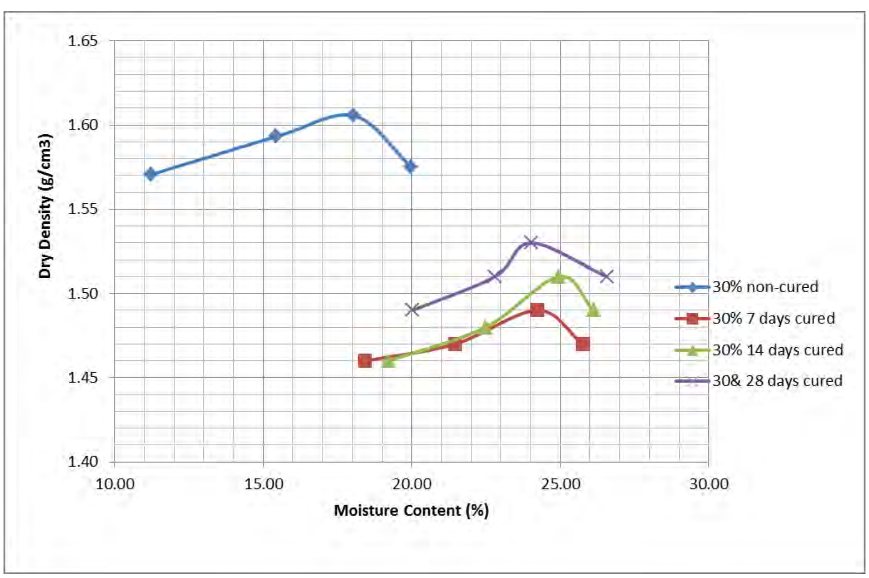
Target moisture content %	5	8	11	14	
Mass of wet soil + mould A (g)	5510.00	5589.00	5618.00	5602.00	
Mass of mould B (g)	3923.00	3924.00	3924.00	3924.00	
Mass of wet soil C=A-B (g)	1587.00	1665.00	1694.00	1678.00	
VOLUME OF MOULD (CC) V	944.00	944.00	944.00	944.00	
Bulk density C / V = W	1.68	1.76	1.79	1.78	NMC
Moisture determination container No.	Q	L	C	U	D
Mass of container + wet soil a (g)	192.00	180.60	180.80	154.00	176.70
mass of container + dry soil b (g)	168.30	156.30	155.70	132.30	160.50
Mass of container d (g)	50	49.7	51.2	50.60	51.5
Mass of dry soil b - d = e (g)	118.30	106.60	104.50	81.70	109.00
Mass of moisture a - b = f (g)	23.70	24.30	25.10	21.70	16.20
Moisture content f/e*100 = m (%)	20.03	22.80	24.02	26.56	14.86
Dry density W / (100+m) *100 (Kg/m3)	1.40	1.44	1.45	1.40	
Dry density for zero air void =[1 / (w/100) +1/Gs)]					



30% 28 days cored

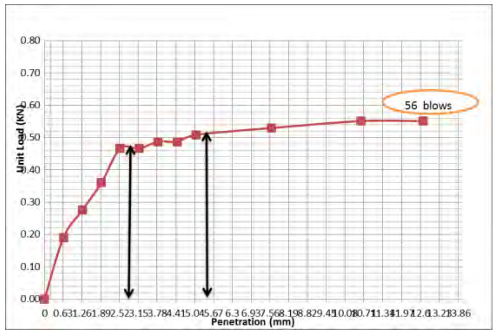
Target moisture content %	5	8	11	14	
Mass of wet soil + mould A (g)	5600.00	5679.00	5764.00	5735.00	
Mass of mould B (g)	3923.00	3923.00	3923.00	3923.00	
Mass of wet soil C=A-B (g)	1677.00	1756.00	1841.00	1812.00	
VOLUME OF MOULD (CC) V	944.00	944.00	944.00	944.00	
Bulk density C / V = W	1.78	1.86	1.95	1.92	NMC
Moisture determination container No.	Q	L	C	U	D
Mass of container + wet soil a (g)	206.60	204.70	229.30	204.20	273.30
mass of container + dry soil b (g)	180.00	176.40	193.50	172.60	243.10
Mass of container d (g)	41.6	50.5	50	51.60	51.1
Mass of dry soil b - d = e (g)	138.40	125.90	143.50	121.00	192.00
Mass of moisture a - b = f (g)	26.60	28.30	35.80	31.60	30.20
Moisture content f/e*100 = m (%)	19.22	22.48	24.95	26.12	15.73
Dry density W / (100+m) *100 (Kg/m3)	1.49	1.52	1.56	1.52	
Dry density for zero air void =[1 / (w/100) +1/Gs)]					



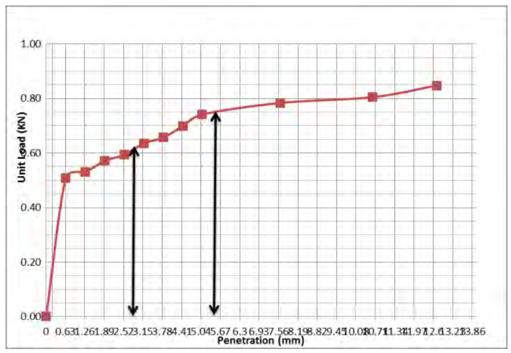


4.3. Californian Bearing Ratio (CBR) Tests

Ring Factor:	21.190	N/Division	
30% 7 Days cured			
	56 Blows		
PLUNGER			
PENT.(mm)	DIAL	LOAD(KN)	CBR %
0	0	0.00	
0.64	9	0.19	
1.27	13	0.28	
1.91	17	0.36	
2.54	22	0.47	3.53
3.18	22	0.47	
3.81	23	0.49	
4.45	23	0.49	
5.08	24	0.51	2.54
7.62	25	0.53	
10.62	26	0.55	
12.7	26	0.55	

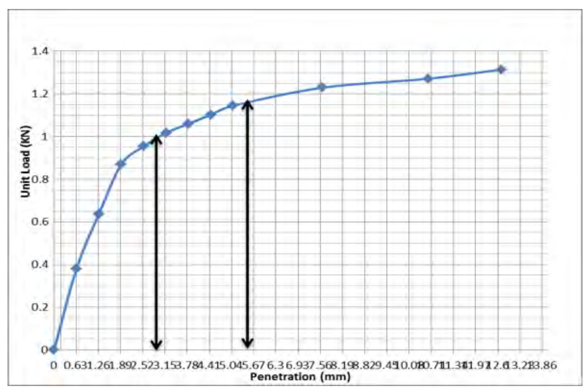


Ring Factor:	21.190	N/Division	
30% 2/ Days Cured			
	56 Blows		
PLUNGER			
PENT.(mm)	DIAL	LOAD(KN)	CBR %
0	0	0.00	
0.64	24	0.51	
1.27	25	0.53	
1.91	27	0.57	
2.54	28	0.59	4.49
3.18	30	0.64	
3.81	31	0.66	
4.45	33	0.70	
5.08	35	0.74	3.71
7.62	37	0.78	
10.62	38	0.81	
12.7	40	0.85	

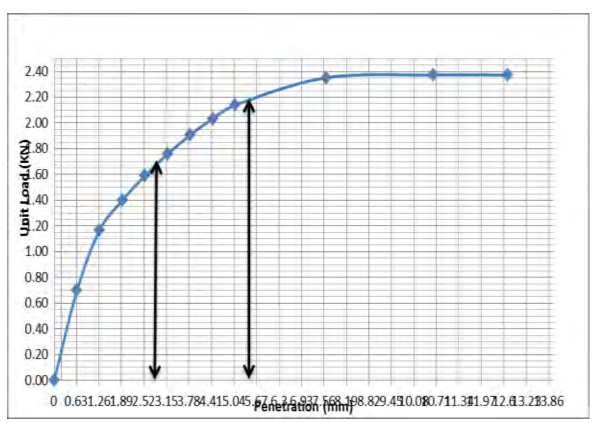


5. Unsoaked CBR

Ring Factor:	21.190	N/Division	
Natural Soil	Unsoaked		
	56 Blows		
PLUNGER			
PENT.(mm)	DIAL	LOAD(KN)	CBR %
0		0.00	
0.64	55	1.17	
1.27	90	1.91	
132	114	2.42	
2.54	132	2.80	21.19
3.18	147	3.11	
3.81	152	3.22	
4.45	160	3.39	
5.08	166	3.52	17.59
7.62		0.00	
10.62		0.00	
12.7		0.00	



Ring Factor:	21.190	N/Division	
30% 28 Days Cured (Unsoaked)			
	56 Blows		
PLUNGER			
PENT.(mm)	DIAL	LOAD(KN)	CBR %
0	0	0.00	
0.64	33	0.70	
1.27	55	1.17	
1.91	66	1.40	
2.54	75	1.59	12.04
3.18	83	1.76	
3.81	90	1.91	
4.45	96	2.03	
5.08	101	2.14	10.70
7.62	111	2.35	
10.62	112	2.37	
12.7	112	2.37	



Appendix D:

Comparison between Soaked and Un-soaked CBR

1. Natural Soil

A. Soaked CBR **(CBR=1)**

As per several specification requirements the natural subgrade soil is classified as S1 of subgrade class and it is unsuitable to be used as subgrade material. Thus, replacement of the natural subgrade soil with selected fill material is recommended.

For the removal and replacement of the expansive soil, the following assumptions are made for cost computation purpose.

Assumptions:

- ✓ 100m expansive soil section is considered
- ✓ 10m road width
- ✓ 0.8m undercut depth
- ✓ 100 Birr/m^3 is considered for improved subgrade

Table D1: Cost for improved subgrade (Soaked)

Description	**Unit**	**Qty**	**Unit price**	**Amount**
Improved Subgrade	m^3	800	100	80,000.00

B. Uncooked CBR **(CBR=7.2)**

As per specification requirements the natural subgrade soil for un-soaked condition is classified as S3 of subgrade class and it can be considered as suitable material to serve as road subgrade with small improvement.

For the un-soaked condition the following assumptions are made for cost computation purpose

Assumptions:

- ✓ 100m expansive soil section is considered
- ✓ 10m road width
- ✓ 0.3m undercut depth
- ✓ 100 Birr/m^3 is considered for improved subgrade

Table D2: Cost for improved subgrade (Un-Soaked)

Description	**Unit**	**Qty.**	**Unit price**	**Amount**
Improved Subgrade	m3	300	100	30,000.00

From the above tables it can be noted that, by applying un-soaked CBR for the ideal improved subgrade preparation, the earth work volume reduces and the cost of subgrade preparation reduced by 62.5%.

A. Pavement Thickness Determination for the Natural Soil (Soaked and Un-Soaked)

Tthickness of different layers of road pavement are determined based on the subgrade class and the traffic class. Depending on CBR values, road subgrades are classified into six classes (S1 to S6) as shown below:

Table D3: CBR value Vs Sub grade class

CBR Range (%)	Subgrade Class
<3	S1
3-4	S2
5-7	S3
8-14	S4
15-30	S5
>30	S6

Similarly, depending on the traffic volume, a given road project can be categorised into a specific traffic class (T1 to T10).From the design documents of the study road project, it has been found that the traffic class of the route is **T4.**

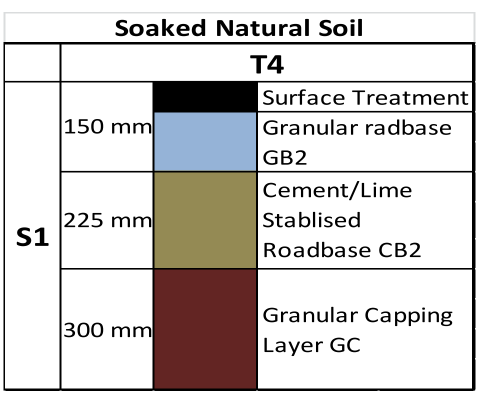
Chart A2 of ERA Flexible Pavement Design Manual (2013) is considered for pavement thickness determination as described below.

I. Soaked Natural Soil

- Subgrade class = S1
- Traffic Class = T4

II. Un-Soaked Natural Soil

Subgrade class = S3
Traffic Class = T4



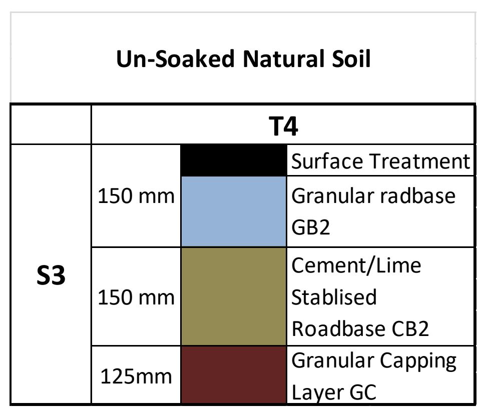


Fig.D1 Pavement thickness determination for natural soil

From the above charts it can be noted that, by application of un-soaked CBR on the natural subgrade soil, the pavement thickness reduced by 37%.

2. Blended Sample (30%)

A. Soaked Condition (CBR = 4.5%)

The 30% marble dust blended sample is found to be in S2 subgrade class and hence unsuitable to be used as road bedding material. Thus, it is recommended to remove and replace the subgrade soil by selected fill material as stated in section 6.4.7.1 above rather than blending with 30% marble dust.

B. Un-Soaked Condition (CBR = 21.2%)

For the un-soaked case the 30% marble dust blended sample is found in S5 subgrade class and is suitable to be used as subgrade material. Thus considering the same assumptions stated for the natural soil and considering additional cost for purchasing and transportation of the marble dust the cost estimate for the stabilized subgrade is summarized below.

Assumptions:

- ✓ 100m expansive soil section is considered
- ✓ 10m road width
- ✓ 0.3m stabilization depth
- ✓ 75 Birr/m^3 is considered for stabilized subgrade (including purchasing and transportation cost of marble dust)

Table D4: Cost for Stabilized subgrade (30% Un-Soaked)

Description	Unit	Qty.	Unit price	Amount
stabilized Subgrade	m3	300	75	22,500.00

From the above results it can be noted that by application of un-soaked CBR on the 30% marble dust blended subgrade soil, cost for subgrade preparation reduces by 72% as compared to soaked condition of the natural subgrade soil.

A. Pavement Thickness Determination for the 30% Marble Dust blended Soil (Soaked and Un-Soaked)

Soaked 30% MD blended Soil	Un-Soaked 30% MD blended Soil
• Subgrade class = S2	Subgrade class = S5
• Traffic Class = T4	Traffic Class = T4

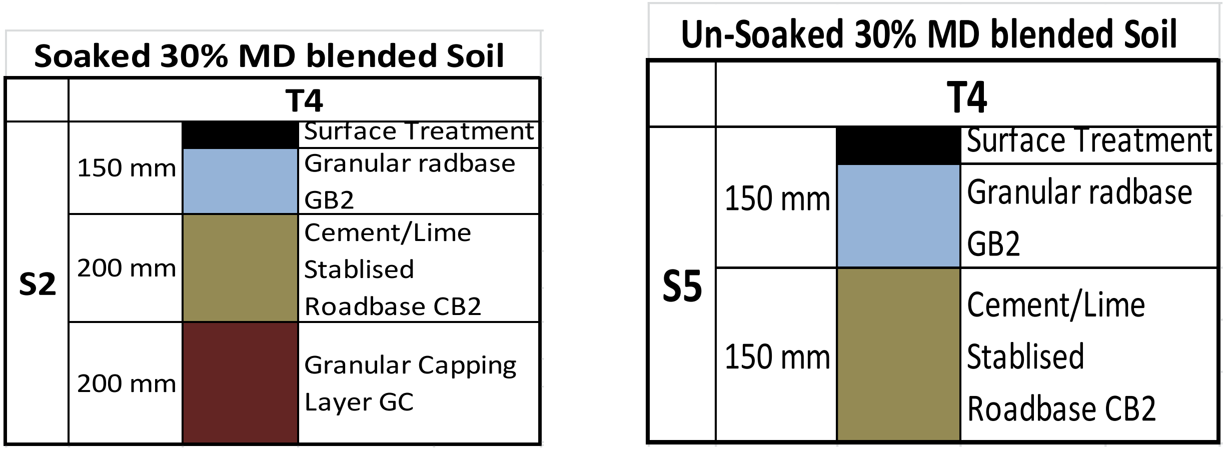


Fig.D2 Pavement Thickness determination for 30% marble dust blended soil

From the above charts it can be noted that, by application of un-soaked CBR on the 30% marble dust blended subgrade soil, the pavement thickness reduced by 45%.

The percentage decrease in pavement thickness for soaked and un-soaked condition is summarised in the table and graph below.

Table D5Effect of un-soaked CBR on Pavement thickness

Specimen	Pavement Thickness (mm)		% decrease in pavement thickness
	Soaked	Un-soaked	
Natural Soil	675	425	37.04
30% MD	550	300	47.83

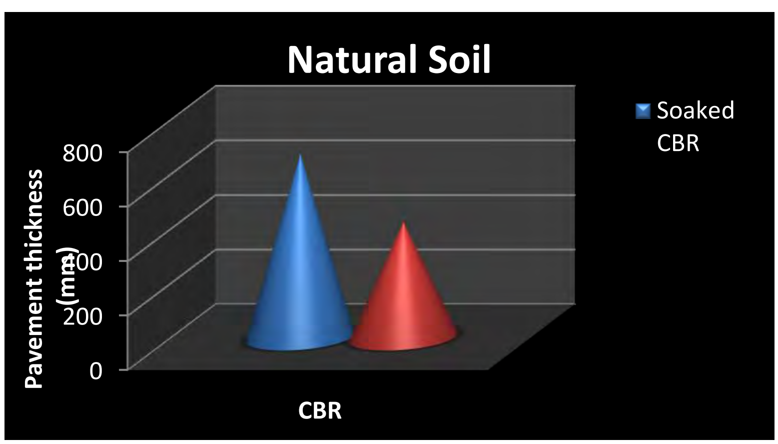


Fig.D3 Pavement Thickness VS CBR Condition for Natural Soil

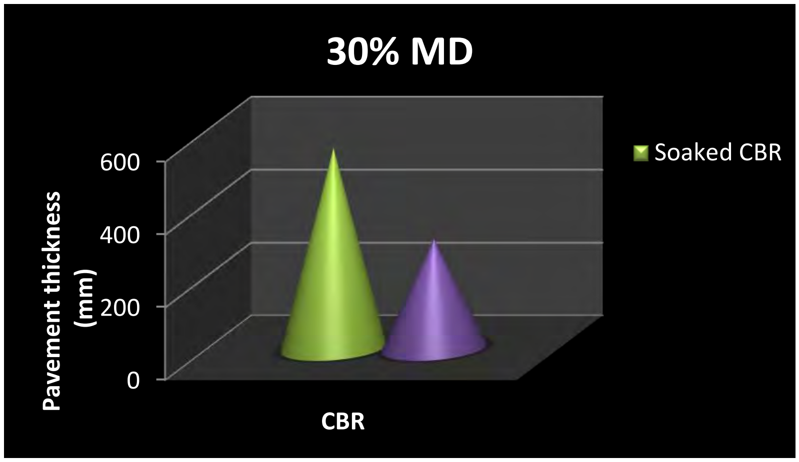


Fig.D4 Pavement Thickness VS CBR Condition for 30% Marble Dust Blended Sample

From the above results considering un-soaked CBR, it can be concluded that;

- ✓ If proper drainage is provided to prevent infiltration of surface runoff, un-soaked CBR for natural soil and 30% marble dust blended expansive subgrade soil gives significant reduction of the project cost
- ✓ In arid areas where the ground water table is below the influence zone and the annual average rain fall is low (not more than 250 mm), un-soaked CBR is recommended
- ✓ By application of marble dust, the earth work volume reduces and hence the corresponding environmental pollution and degradation is reduced
- ✓ Additional separate and detailed research work on un-soaked CBR laboratory tests on subgrade soils especially on arid and semi-arid areas of the country is recommended

Sembo-Shola-Gebeya-Gorfo- Gindeber Road Project
Summary of Laboratory test results of Sub grade

Sr. No	Station	Represented station	Depth (cm)	Soil descreption	Soil Clasification	Sieve Analysis (% passing)			LL (%)	PL (%)	PI	MDD (%) g/cm^3	OMC (%)	Soaked CBR at 95% MDD (%)	Swell (%)
					Specefecation Requirement	Sieve opening(mm)			≤60		≤30	_	_	6	≤2.5
						2	0.425	0.075							
1	0+500		20 - 60	Dark Brown Silty Clay Soil	A-7-5(30)	98	93.3	90	60	32	28	1.57	17.3	2	5.09
2	1+000		"	"	A-7-5(39)	98.3	94.7	90	73	38	35	1.51	22	1	6.2
3	1+500		"	Light brown silty clay soil with some gravel	A-7-5(18)	79	73.7	68.3	61	36	25	1.74	14.3	2	4.98
4	2+000		"	Dark Brown Silty Clay Soil	A-7-5(29)	96	90	82	71	43	28	1.5	24	2	5.26
5	2+500		"	"	A-7-5(42)	98	93.3	86	82	42	40	1.59	16.6	2	5.02
6	3+000		"	"	A-7-5(40)	96.3	93	89	73	36	37	1.53	20.8	2	5.96
7	3+500		"	"	A-7-5(31)	97	92.3	86.3	69	41	28	1.57	16.2	2	5.17
8	4+000		"	"	A-7-5(44)	100	96	89.3	74	32	42	1.5	20.4	2	4.92
9	4+500		"	"	A-7-5(34)	99.3	96	87.3	69	37	32	1.55	16.9	1	6.6
10	5+000		"	"	A-7-5(34)	99.3	96.7	90	72	43	29	1.47	16.6	1	6.56
11	5+500		"	"	A-7-5(20)	98.7	90	79	57	35	22	1.6	18.6	2	6.77
12	6+000		"	"	A-7-5(39)	99	96.3	92.3	70	35	35	1.57	19.4	2	5.05
13	6+500		"	"	A-7-5(15)	98.3	90.7	80	49	33	16	1.64	17.9	10	1
14	7+000		"	"	A-7-5(43)	99.3	94.7	86.7	78	36	42	1.65	16.9	2	5.55
15	7+500		"	"	A-7-5(38)	98.7	95.3	87.7	67	29	38	1.59	21.9	2	6.6
16	8+000		"	Light Brown silty clay soil	A-7-6(18)	94	81.3	68.7	54	28	26	1.67	17.8	2	6.43
17	8+500		"	Dark Brown Silty Clay Soil	A-7-5(34)	98.3	95.3	90.3	65	34	31	1.55	20.2	1	6.15
18	9+000			Light brown silty clay soil	A-7-6(22)	98.3	91	80.7	50	30	20	1.59	18.5	9	2.02
19	9+500		"	Dark Brown Silty Clay Soil	A-7-5(35)	99	95.5	91.9	74	46	28	1.5	20.2	2	5.3
21	10+000		20~60	Dark Brown Silty Clay Soil	A-7-5(31)	97.3	92.7	86	70	41	29	1.51	21.2	2	3.62
22	10+500		20-60	Dark Brown Silty Clay Soil	A-7-5(29)	99	91.3	82.7	68	38	29	1.55	25.4	2	4.91
23	11+000		20-60	Dark Brown Silty Clay Soil	A-7-5(30)	99.3	88	81.7	67	35	32	1.47	26.1	2	4.21
24	11+500		20-60	Dark Brown Silty Clay Soil	A-7-5(30)	97.7	91.7	91.7	66	34	32	1.50	25.2	3	3.42
25	12+000		20~60	Light Brown Silty Clay With Some gravel	A-2-7(1)	48.6	40.8	34	44	29	15	1.77	16.7	8	2.5
26	12+500		20~60	Dark Brown Silty Clay Soil	A-7-5(23)	95.7	81	70.3	68	40	28	1.63	20.4	2	6.46
27	13+000		20~60	Light Brown Silty Clay	A-7-5(15)	96	82	67.3	57	36	21	1.48	23.3	2	3.58
28	13+500		20~60	Light Brown Silty Clay Mix With Some gravel	A-2-7(0)	38.4	32.2	29	43	26	17	1.86	13.4	7	2.16
29	14+000		20~60	Dark Brown Silty Clay Soil	A-7-5(15)	93.7	82	71.3	54	35	19	1.58	19.2	3	3.57
30	17+100		20~60	Dark Brown Silty Clay	A-7-5(37)	89.3	85	81.5	78	39	39	1.62	19.1	1	7.18
31	17+500		60~100	Weatherd gravel With Sandy Silt Soil	A-2-7(0)	23.4	16.9	11.9	50	30	20	1.84	14.1	33	0.27
32	17+500		20~60	Weatherd gravel With Sandy Silt Soil	A-2-7(0)	29.8	23.4	18	58	36	22	1.74	15.3	40	0.65
33	22+500		20~60	Dark Brown Silty Clay	A-2-7(0)	37.3	29.7	24	52	30	22	1.70	20.2	2	6.46
34	22+500		20~100	Dark Brown Silty Clay	A-7-6(19)	89.2	82.9	75.3	53	28	24	1.67	18.2	2	6.42
35	23+000		20~60	Dark Brown Silty Clay	A-7-5(24)	97.3	94	91.2	61	42	19	1.63	22.3	2	8.51
36	25+000		20~100	Light Brown Silty Clay	A-7-6(14)	77.4	73.6	67.5	50	29	21	1.63	18.2	5	3.51
37	14+500		20-60	Light Brown silty clay	A-7-5(18)	90.7	81.7	74.9	55	33	22	1.71	20.2	3	2.83
38	16+000		20-60		A-7-6(25)						0				

39	16+000	20-60	Light Brown silty clay	A-7-6(26)	91.8	83.3	77.8	57	26	31	1.69	19.8	2	4.6
40	17+101	20-60								0				
41	17+100	20-60	Dark Brown silty clay	A-7-5(37)	96	85	81.5	78	39	39	1.62	19.1	1	7.18
42	17+101	60-101	Light Brown silty clay	A-7-5(53)						0				
43	17+102	60-102	Light Brown silty clay	A-7-5(53)						0				
44	17+103	60-103	Light Brown silty clay	A-7-5(53)						0				
45	17+104	60-104	Light Brown silty clay	A-7-5(53)						0				
46	17+500	20-60	Light Brown silty clay	A-7-5(33)	96.0	89.7	83.6	71	38	33	1.68	19.5	6	2.65
47	17+500	60-100	Highly weathered gravel with sandy soil	A-7-5(14)	90.8	83	70.2	50	31	19	1.62	22.4	2	8.8
48	17+500	20-60	Weathered gravely sand Sandy Silty Clay	A-2-7(0)	29.79	23.43	18.04	58	36	21	1.74	15.3	40	0.65
49	17+500	60-100	Weathered gravel With Sandy Silty Clay	A-2-7(0)	23.4	16.92	11.87	49.8	30.1	19.7	1.83	14.1	33	0.27
50	18+000	20-100	Dark Brown silty clay	A-7-6(13)	96.3	88.2	77.4	50	29	21	1.60	22	6	1.84
51	18+001	20-61	Light Brown silty clay	A-7-6(13)						0				
52	18+002	20-62	Light Brown silty clay	A-7-6(13)						0				
53	18+003	20-63	Light Brown silty clay	A-7-6(13)						0				
54	18+004	20-64	Light Brown silty clay	A-7-6(13)						0				
55	18+005	20-65	Light Brown silty clay	A-7-6(13)						0				
56	18+006	20-66	Light Brown silty clay	A-7-6(13)						0				
57	18+600	20-100	Dark Brown silty clay	A-7-6(17)	91.6	81.1	70.9	52	29	23	1.70	18.2	4	5.22
58	18+600	20-60	Dark Brown silty clay	A-7-5(22)	93.3	85.5	78.8	56	31	25	1.68	21.8	5	2.06
59	19+000	20-100	Light Brown silty clay	A-2-7(0)	65.7	19.3	4.1	47	28	19	1.66	18	6	2.63
60	19+000	20-60	Light Brown silty clay	A-7-6(15)	97.1	85.7	73	49	29	20	1.53	22.4	16	1.03
61	19+500	20-60	Light Brown silty clay with some gravel	A-7-5(8)	88.7	62.5	48.1	58	35	23	1.59	19	2	7.58
62	19+500	60-100	Highly weathered gravel mix with sandy soil	A-7-5(15)	88	73.9	63.2	63	41	22	1.37	32	5	0.6
63	20+500	20-60	Light Brown silty clay	A-7-6(20)	94.4	90.1	86.5	49	29	20	1.55	19.7	24	1.46
64	20+500	20-60	Light Brown silty clay with some gravel	A-7-5(6)	69.6	57.3	50.5	49	33	16	1.78	15.2	11	2.52
65	21+000	20-60	Light Brown silty clay with some gravel	A-7-5(7)	69.2	60.1	54.1	46	31	15	1.73	14.5	9	2.4
66	21+000	20-60	Dark Brown silty clay	A-7-5(31)	96	91.3	87.2	68	39	29	1.55	20.2	2	9.83
67	21+500	20-100	Weathered Gravel	A-7-6(7)	77.6	63.4	56.2	44	29	15	1.63	20.8	12	1.88
68	21+500	20-60	Light Brown silty clay with some gravel	A-7-5(34)	61.6	52.3	46.1	51	33	18	1.63	21.6	12	3.15

69	22+000		20-100	Light Brown silty clay with some gravel	A-7-5(18)	80.9	76.8	66.6	68	43	25	1.53	20	4	2.18
70	22+000		20-60	Yellowish sandy gravel soil	A-7-5(1)	97.9	90.6	37.6	44	31	13	1.41	23	9	3.33
71	22+500		20-100	Dark Brown silty clay	A-7-6(19)	89.15	82.86	75.27	52	28	24	1.67	18.2	2	6.42
72	22+500		20-60	Dark brown silty clay with few gravel	A-2-7(0)	37.27	29.65	24.04	52	30	22	1.70	20.2	2	6.46
73	23+000		20-100	Dark Brown silty clay	A-7-5(37)	95.7	91.8	87.7	71	36	35	1.52	25.1	2	8.3
74	23+001		20-101	Dark Brown silty clay	A-7-5(37)	95.7	91.8	87.7	71	36	35	1.52	25.1	2	8.3
75	23+000		20-60	Dark Brown silty clay	A-7-5(24)	97.28	93.96	91.24	61	42	19	1.62	22.3	2	8.51
76	23+500		20-100	Dark Brown silty clay	A-7-5(28)	95.5	90.3	83.2	60	30	30	1.67	19.6	3	6.09
77	23+500		20-60	Dark Brown silty clay	A-7-6(17)	92	85.3	77.9	49	28	21	1.72	16.8	2	5.05
78	24+500		20-60	Dark brown silty clay with few gravel	A-7-6(17)	94.4	86.3	79.1	49	29	20	1.63	19.7	3	7.04
79	25+000		20-100	Light Brown silty clay	A-2-6(14)	77.38	73.64	67.47	50	29	21	1.62	18.2	5	3.51
80	31+500		20~60	Light Brown Silty Clay Soil	A-7-5(16)	63.7	60.4	54.1	68	34	34	1.60	22.2	2	7.77
81	32+000		20~60	Light Brown Silty Clay Soil	A-7-5(17)	98	88.3	66.3	60	36	24	1.41	33.7	5	0.89
82	32+500		20~60	Dark Brown Silty Clay Soil	A-7-5(23)	98.3	87	74.7	69	43	26	1.51	21.2	1	4.1
83	33+000		20~60	Red to Brown Silty Clay Mix With Some gravel	A-7-5(7)	91.8	75	59.5	44	30	14	1.49	23	9	2.4
84	33+500		20~60	Brown Silty Clay	A-7-5(29)	97.7	84.3	78.7	68	36	32	1.56	26.7	2	2.18
85	34+000		20~60	Dark Brown Silty Clay Soil	A-7-5(19)	96.7	89	79.3	59	40	20	1.55	20.2	1	8.18
86	34+500		20~60	Dark Brown Silty Clay Soil	A-7-5(19)	97	87.5	82.5	63	47	16	1.52	23.2	2	8.14
87	35+000		20~60	Dark Silty Clay Soil	A-7-5(31)	99.3	92	81.3	75	44	31	1.50	19.6	2	5.62
88	35+500		20~60	Light Brown Silty Clay Soil	A-7-5(15)	95.5	86.3	75.3	61	46	15	1.75	20.7	11	1.16
89	36+000		20~60	Redish Brown Silty Clay Soil	A-7-6(22)	98.5	92.3	78.8	52	26	26	1.62	20.2	4	3.94
90	37+100		40 - 85	Dark Silty Clay Soil	A-7-5(24)	92.6	86.5	79.7	61	35	26	1.45	24.2	2	4.62
91					A-7-6(22)										
92	37+800		20 - 60	Light brown silty clay soil with some gravel	A-2-6(0)	31.2	26.8	21.7	38	22	16	1.78	14.2	15	1.52
93	38+300		50 - 90	"	A-2-7(3)	42.9	39.4	34.4	62	35	27	1.77	16.6	3	4.5
94	39+120		20 - 60	"	A-2-7(0)	23.4	19.2	13.5	41	25	17	1.87	15.3	21	0.62
95	39+500		20 - 100	Dark silty Clay Soil with some gravel	A-7-5(15)	59.9	54	47.7	76	36	39	1.72	16.7	2	4.8
96	40+060		60 - 80	Brown Silty Clay with some gravel	A-7-5(23)	74.8	68.6	64.8	72	38	34	1.56	18	2	4.68
97	40+500		20 - 60	Dark silty clay soil	A-7-5(27)	97.3	92	83.3	64	37	27	1.60	19.4	2	4.74
98	40+980		"	Dark Brown Silty Clay Soil	A-7-5(36)	98.7	96.4	91.3	67	35	32	1.56	15.8	1	4.8
99					A-7-6(22)										
100	41+500		50 - 85	Brown silty clay soil with some gravel	A-7-5(17)	75.4	67.5	62.7	62	34	28	1.62	19.2	2	5.72
101	41+900		20 - 60	Dark Brown Silty Clay Soil	A-7-5(34)	99	91.5	83.6	69	35	35	1.52	21.4	2	4.6
102					A-7-6(22)										
103					A-7-6(22)										
104					A-7-6(22)										

105					A-7-6(22)										
106	42+400		"	Brown silty clay soil with some gravel	A-2-7(0)	47.4	31.7	23	61	30	30	1.56	15.9	2	5.33
107	43+500		20 - 80	Brown silty clay soil	A-7-5(30)	98.6	93.7	81	63	32	31	1.67	17.3	2	3.92
108	44+020		"	Dark silty clay soil	A-7-5(27)	97.6	91.3	84.3	62	35	27	1.56	17.3	1	4.62
109	44+700		20 - 60	Dark Brown Silty Clay Soil	A-7-5(38)	96.7	91.9	87	73	37	36	1.58	14.4	2	3.76
110	45+400		"	Light brown silty clay with some gravel	A-2-7(0)	32.8	27.8	22.9	73	36	36	1.56	19.5	2	3.69
111	46+300		"	Black cotton soil	A-7-5(32)	98.3	91.2	79.1	76	42	33	1.49	24	2	3.68
112	46+900		"	Yellowish sandy silt soil	A-2-6(0)	70.4	48.1	33.1	36	24	12	1.64	17	16	0.72
113	47+420		"	Light brown silty bclay soil	A-7-6(34)	100	97.5	91	58	24	34	1.61	21.8	2	5.86
114	47+980		"	Reddish silty clay soil	A-7-5(15)	87	71	58.5	61	33	27	1.39	18.8	3	4.22
115	48+800		"	Highly weatherd light yellowish gravelly soil	A-2-7(0)	65.4	38.3	24.1	50	26	24	1.63	19.9	3	2.76
116	49+480		"	Dark Brown Silty Clay Soil	A-7-6(32)	93.5	89.5	83	63	28	35	1.50	22.4	1	5.35
117	50+000		"	"	A-7-5(42)	98.3	90.4	83.4	75	30	45	1.53	24.7	2	4.88
118	50+600		"	"	A-7-6(35)	98.5	93.8	82.3	67	29	38	1.53	19.6	2	4.77
119	51+400		"	"	A-7-5(27)	100	88.5	74.5	66	33	33	1.42	22.4	2	5.93
120	58+940		0.20~100	Light brown Silty Clay	A-7-5(19)	98.7	93.8	78.6	54	32	22	1.56	21.9	3	
121															
122															
123															
124															
125															
126															
127	56+940		0.20~100	Dark brown Silty Clay	A-7-6(13)	98.8	89.6	81	42	28	14	1.66	18.2	10	
128	55+940		0.20~100	Light brown Silty Clay	A-7-5-(20)	98.8	89.5	73.1	60	35	25	1.58	23.2	3	
129	54+940		0.20~100	Light brown Silty Clay	A-7-5-(19)	98.6	89.6	79.5	53	31	22	1.65	19.8	4	
130	53+940		0.20~100	Dark brown Silty Clay With Some gravel	A-7-5(18)	93.1	83.4	77.1	52	31	21	1.65	19.9	2	
131	52+940		0.20~100	Redish brown Silty Clay With few Sand	A-7-5(3)	90	56.3	37.3	59	39	21	1.60	26.1	15	
132	51+940		0.20~100	Dark brown Silty Clay	A-7-5(18)	94.3	84.2	71.7	54	30	24	1.63	23.8	6	