# SUITABILITY OF WEATHERED YELLOWISH LATERITE TO STABILIZE CINDER GRAVEL FOR SUB BASE COURSE:

# A CASE STUDY FROM HUMBO AND BOSHA QUARRY SITES

**MASTER OF SCIENCE IN CIVIL ENGINEERING**

**SPECIALIZATION IN ROAD AND TRANSPORT ENGINEERING**

**TESFATSION DANIEL**

**HAWASSA UNIVERSITY HAWASSA, ETHIOPIA**

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**A THESIS SUBMITTED TO THE**

**DEPARTMENT OF ROAD AND TRASPORT ENGINEERING,**
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**IN PARTIAL FULFILLMENT OF THE**
**REQUIREMENTS FOR THE**
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**SCHOOL OF GRADUATE STUDIES**

**EXAMINERS' APPROVAL SHEET**

As members of the Board of Examiners of the Final Master's Degree Open Defense, we certify that we have read and evaluated the thesis prepared by: **Tesfatsion Daniel** under the title "**Suitability Of Weathered Yellowish Laterite To Stabilize Cinder Gravel For Sub-Base Course: A Case Study From Humbo And Bosha Quarry Sites"** and recommended that it can be accepted as fulfilling the thesis requirement for the degree of **Master's of Science** in **Civil Engineering** with Specialization in **Road and Transport Engineering**.

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# DECLARATION

I hereby declare that this thesis is my original work under the supervision of Prof. Emer Tucay Quezon submitted to the School of Graduate Studies of Hawassa University in Partial Fulfillment of the Requirement for the Degree of Master of Science in Civil Engineering, Road and Transport 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 fully acknowledged.

**Candidate:**

**Mr. Tesfatsion Daniel** ________________ __________

Signature Date

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**ADVISORS' APPROVAL SHEET**

This is to certify that the thesis entitled "Suitability of Weathered Yellowish Laterite to Stabilize Cinder Gravel for Sub-Base Course: A Case Study from Humbo And Bosha Quarry Sites" submitted in partial fulfillment of the requirements for the degree of **Master's** of Science with specialization in **Road and Transport Engineering,** the Graduate Program of the **School of Civil Engineering**, and has been carried out by **Mr. Tesfatsion Daniel,** Id. No **PGRo/020/10** under our supervision. Therefore, we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the school/department.

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# ACRONYMS

| | |
|---|---|
| AASHTO - | American Association of State Highway Officials |
| ASTM - | American Society for Testing and Materials |
| ASANRA - | Association of southern African National Roads Agencies |
| BS- | British Standard |
| CBR – | California Bearing Ratio |
| CIA - | Central Intelligence Agency |
| ERA - | Ethiopian Road Authority |
| ECWC - | Ethiopian Construction Work Corporation |
| ICT - | Information Communication Technology |
| KTC- | Kentucky transportation center |
| LL - | Liquid Limit |
| MDD – | Maximum Dry Density |
| MER- | Main Ethiopian Rift |
| MEDPG - | Mechanistic Empirical Pavement Design Guide |
| Mr - | Resilient modulus |
| OMC – | Optimum Moisture content |
| PI - | Plastic Index |
| PL - | Plastic Limit |
| PLC- | Private Limited company |
| RL CBR - | Repeated Load California Bearing Ratio |
| SD LTAP- | South Dakota Local Transportation Assistance Program |
| SE- | South East |
| S/N/N/P/R- | South Nations Nationalities People Region |
| URRAP- | Universal Rural Road Accessing Program |

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# ABSTRACT

*The road facility plays an important role to one country's economy, especially in developing countries. In Ethiopia, construction of roads and bridges are booming due to short term and long-term development plans of the government. Construction materials are becoming depleted, especially the source of good quality of aggregates for surfacing, base and sub base course materials for roads. However, there are locally available marginal materials that can be found, and improving the properties by blending with other materials, is suitable for pavement construction. One of which is Cinder gravel and weathered yellowish laterite, and using of these materials have a significant effect on the road construction. It is known that cinder materials lack fine particles and plasticity property. This research study focused to investigate the suitability of the weathered yellowish laterite to stabilize Cinder gravel for sub base course road construction. In the investigation the aforementioned materials, in this study used primary data and secondary data collection methods. The samples tested for Gradation, Atterberg Liquid Limit Tests, compaction, soaked CBR strength, shrinkage limit using ASTM manual. Likewise, the mechanistic empirical concept is essential, that can simulate traffic loading and used to know the property of the material. The two used mechanical properties are plastic or permanent deformation and elastic or resilient deformation. To study the two mechanical properties of the material, repeated load CBR is used by repeating the loads from the standard load at a different number of blows. The resilient deformation of the stabilized material going decreases in initial repeated loading phase and going into a constant as the load repetition increases. The permanent deformation going increases when the repetition of the load increases. Highly compacted material shows low permanent deformation while low compacted materials showed high permanent deformation. Highly compacted materials indicated high elastic deformation, and low compacted materials showed low elastic deformation. The results are analyzed and compared with the AASHTO and ERA Standard Specifications and found out the optimum proportion of 30% of cinder and 70% of weathered yellowish laterite. Hence, cinder gravel blended weathered yellowish laterite is suitable, and it can be used as sub base course.*

***Key words:*** *Cinder gravel, weathered yellowish laterite, Compaction, CBR, Repeated load CBR, resilient deformation, permanent deformation and sub base.*

# 1. INTRODUCTION

## 1.1 Background

Transport is a key infrastructure of a country. The rate at which a country's economy grows is very closely linked to the rate at which the transport sector grows (Subash et al., 2017). The citizens of a country need to make socio-economic relations one with another. So that citizen's need a facility to undertake the relations. There are a number of modes of transportation in the world. And thus the transportation facility needs large amount of economy investment. The road mode of transport is one mode of transportation, which it gives home to home accessibility and it can give service for all.

Physical infrastructure like road is commonly indicated as a key input to the economic process both in developed and developing countries (Subash et al., 2017). From a lot of transportation mode of choice, the road facility coverage is more in developing countries because of its initial capital and can access door to door with minimum capital than other mode of transportation. Therefore, giving more choices to a constructor (owner) of the road is the duty of professionals.

In the context of Ethiopia 90 % of the country import and export is on the road transport system. And 95 % of the mobilization of the people to a social, economical and political is on the road and transport system (S/N/N/P/R Road and Transport Bureau, 2016). From this we can recognize that more people use road transportation mode of transport for socio economic and personal cases. And as our country is a developing country there is not also more choice of selection other mode of transport.

Ethiopia has one of the lowest road densities and lowest Rural Accessibility Indexes in Sub-Saharan Africa (Atsushi et al., 2016). In Ethiopia government tries to facilitate the road facility in developing manner. As the government wants to provide the road facility to the citizens, and because of the country's economy, all roads still cannot paved during the period of the study. The economy is a great challenger to provide full facilities. Total roadway network of our country/Ethiopia/ at 2015 G.C is 110,414 km. From this road the paved portion is 14,354 km and the remaining 90,060 km is unpaved roads (CIA, 2019) . From the total road network, unpaved road covering is greater than 81.57%. The road facility is not only the problem of Ethiopia rather it is the problem of more developing countries because of the materials cost and economy of the countries.

However, the resource is scarce the road facility specifies selected and designed material for construction. Performance of the roads depends on the structural components such as sub grade, sub base, base and surface courses. Presently conventional materials required for a construction of roads are insufficient at many locations (Subash et al., 2017). To get a good performance of the road we have to design the structures very carefully. A good performance indicates the environment and the traffic of the designed area.

## 1.2 Statement of the problem

Pavement design is a structural design method for the road means to search out for the most economical combination of layer thicknesses and material types for the pavement, taking into consideration the properties of the soil, the traffic to be carried throughout the service life of the road including the climatic conditions (Araya, 2011).

The granular sub base layers need to offer the majority of the bearing capability to resist the load, particularly in developing countries (Araya, 2011). The granular sub base and gravel sporting course area are used to administer service from the load that applied at the time of service or the traffic. Smart gravel needs a definite proportion of stone to hold various vehicles, especially during the wet season, to transfer the load, a proportion of sand sized particles to fill the voids between stones and provides stability, and additionally a plastic content of plastic fines to mix along the stones and sand to create firm gravel surface to shade water (Ken et al., 2000). The particle size or gradation affects both sealed surface road and gravel road. If the particle size of a material is not well graded there is void and that it loses the load resistance capacity when the vehicle going on it. The load resistance of a material is good if the material is well graded.

To minimize the performance and economic problems, engineers should create alternative choices which are necessary to satisfy the demand. The choices must meet the economic constraints and using of locally available materials and benefits the users and the local peoples. So creating alternative options using locally available material is used to overcome the constraints arise from economy.

In spite, the scarcity of the resource, the road construction need selected and designed material. For the construction of the road projects materials come from long distance increases the cost of the project and thus affect the economy. And when using locally available materials for the construction the cost decreases but the challenge rises is the engineering qualification of the material.

From this available materials cinder is one of the accessible gravel materials. The cinder is gravel sized material but lack of sand sized particle to fill the void and plastic fines which used to give cohesion to the sand sized particle and the gravel sized particle. An advantage of cinders as a road construction material is the relative ease with which they can be dug from the quarry; a mechanical shovel or hand tools are usually adequate for their extraction although occasionally a bulldozer may be required to open up a working face (Newill, et al., 1980).

The cinder materials are lack of fine materials (ERA, 2018); thus, they cannot attain the requirements of sub base course standards of Ethiopia Road Authority standard specification. The use of cinder in the gravel surface faces a challenge in the service time of the road. The cinder material is lack of sand sized fractions, thus it has low shearing resistance when transferring the load from one particle to the other. However, the grain sized particles are far enough and it transfers the load one to another, this materials have very more void particles so they cannot stand alone. That minimizes the serviceability and the life of the road. For compaction the water holding capacity of a sand sized material is very essential but the water holding capacity of sand sized cinder is very low, therefore, compaction of cinder is impossible. Also, the material is non plastic material. But the cavities of the gravel sized cinder can hold water and that is not use for compaction.

To use the cinder as structural component of pavement in the road construction it needs to change its engineering characteristic (Newill et al., 1980). To address the problem, this research study used locally available material to the economic problem and wise use of the resource. The study suggested the material which has a good plasticity and composed of fine particles that used to increase the use of cinder in the road construction. Therefore, it should have to be stabilized with other fine materials which can improve its behavior. Therefore, by adding weathered yellowish laterite to improve the gradation requirement, compaction level, and the plasticity index and impact value of the material, which can improve its strength.

In developing countries most of the pavements are not evaluated for the mechanical behavior. That it studies the performance of the structure. The mechanistic – empirical approach is used to increase the life-time performance of the material. The empirical formulas are developed from long-time experiences, and that developed as a constant measurement. But evaluating for a particular material condition is necessary to increase the performance of the material.

The weathered yellowish laterite has good plastic, and sand sized fine materials. Therefore, by adding fine materials from weathered yellowish laterite, that can improve its property. The weathered yellowish laterite has a good plasticity character and more fine material. Thus, by adding the weathered yellowish laterite to cinder, it provides a good density and strength.

There are a lot of researches had been done to stabilize the cinder, like (Eshete, 2011) by the title blending of cinder with fine grained soil to be used as sub base materials and (Hadera, 2015) by the title of the potential use of cindergravel as a base course material when stabilizing by volcanic ash(pumice) and lime. This two and other researches focuses on changing the properties of the cinder by stabilizing with finer materials, pumice and lime. Thus, this material are good but some are not locally available materials rather they are expensive incost like lime. The researches done above are improtant to the areas with available of pumice. And also the above and other researches around cinder did not work the mechanistic-empirical concepts which is very useful to investigate the performance of the road structure. But this research tried to investigate the effect of weathered yellowish laterite in the engineering properties when blended with cinder gravel for sub-base course. Thus, the weathered yellowish laterite is a material that occurs naturally, and it is locally available.

The research also studied the mechanical characteristics of the problem that is necessary to determine the performance of the structure. The deformation character of stabilized cinder for a permanent and recoverable was studied in different compaction level.

## 1.3 Objective of the Study

### 1.3.1 General objective

Investigation of the improvement of natural cinder gravels by mechanical stabilization with weathered yellowish laterite.

### 1.3.2 Specific objectives

- ✓ To determine the index properties and classification of Cinder and weathered yellowish Laterite.
- ✓ To evaluate the density and strength property of cinder gravel by stabilizing with weathered yellowish Laterite at optimum proportion to use as sub base course and to determine the optimum level.
- ✓ To characterize the resilient and permanent deformation based on effect of compaction.

## 1.4 Research questions

1. What are the engineering properties of cinder and weathered yellowish laterite?
2. How does the increase in the amount of weathered yellowish laterite affect the compaction and strength when use to stabilize the cinder gravel for sub-base course?
3. How does compaction affects the resilient deformation and permanent deformation of the material?

## 1.5 Scope and limitation of the study

### 1.5.1 Scope of the study

The thesis studied the engineering properties of cinder gravel material and weathered yellowish laterite. The stabilization of material done to improve the property of the materials and the stabilized materials is evaluated for the gradation, plasticity index, compaction, shrinkage limit and CBR which is used to characterize the material for sub base. The resilient and permanent deformation characteristic was studied to characterize the mechanical behavior.

### 1.5.2 Limitation of the study

However fully characterization of the material would be done by including tri-axial test to know the values cohesion C and angle of friction Ø, but the research were not conduct the tri-axial test because of the machine in availability. And also the study focuses only on the characterization of the material and the economic analysis of the stabilized material did not carry that used the potential use of the material.

## 1.6 Significance of the study

This research is very significant to construction industries, especially countries with rich of local cinder materials. The research incorporated the suitability of cinder for road construction as sub base course. Also, it could be used as basis for the next advanced research on the use of cinder for road construction purpose. The study incorporates the use of a repeated load CBR to study the deformation properties means mechanical properties.

# 2. LITERATURE REVIEW

## 2.1 Introduction

Transportation facility is a back bone of one's country development. That there are a lot of modes of choice to travel from place to place, and that used to make socio–economic relation. From this a lot of modes of choice a road facility is one that is used to access door to door service. In all countries including developed countries, the road facility is essential for the users. According to Araya (2011) the road pavement structure needs a designed material that can perform in applied load and climatic situation. Hence, designing a material for the pavement is necessary to get a good performance. However, the materials found in local cannot meet the requirement and thus needs improvements of them. One of the locally available materials is cinder gravels. Studying about the cinder material and improving its property is very essential for the road structure.

## 2.2 Cinder gravels

### 2.2.1 Definition and background of volcanic cinder gravels

Cinder gravel is a piece of vesicular lapilli ejected from a volcanic vent during an eruption, with the appearance of cinder. Vesicular (in the context of this report): condition of lava-derived hard material containing cavities, either isolated or as part of an interconnected network. These cavities were derived from gaseous bubbles within the lava when it is solidified. Vesicularity is the degree to which the material is vesicular. Higher vesicular material (such as pumice) will have lower densities, and hence lower compressive strengths, than less vesicular material. Lapilli: gravel-sized pieces of solidified pyroclastic lava. The lapilli ejected during basaltic eruption vary considerably in its vesicularity, but densities are much greater than those of pumice materials, and usually between 1.2 and 2 gm/cc. These fragments are black or dark grey in color when initially ejected but oxidize to dark red and red-brown following contact with the atmosphere. Geologically, this material is referred to as scoria, though it is also known as cinder due to it having a similar appearance to charcoal or slag (ERA, 2018). In trained the pumice is not used in road construction because of its low density. Rather it is used in hollow concrete block work. The vesicular lapili is used in road construction because of its good weight than pumice.

### 2.2.2 Location and Distribution of volcanic cinder in Ethiopia

The opening of the Main Ethiopian Rift value as shown in Figure 2.1 commenced in the early-mid Miocene. The southern and central MER became dominated by silicic volcanic eruptions, giving rise to the creation of large-diameter calderas, such as at Hawassa see Figure 2.1 and extensive deposits of ash and pumice.

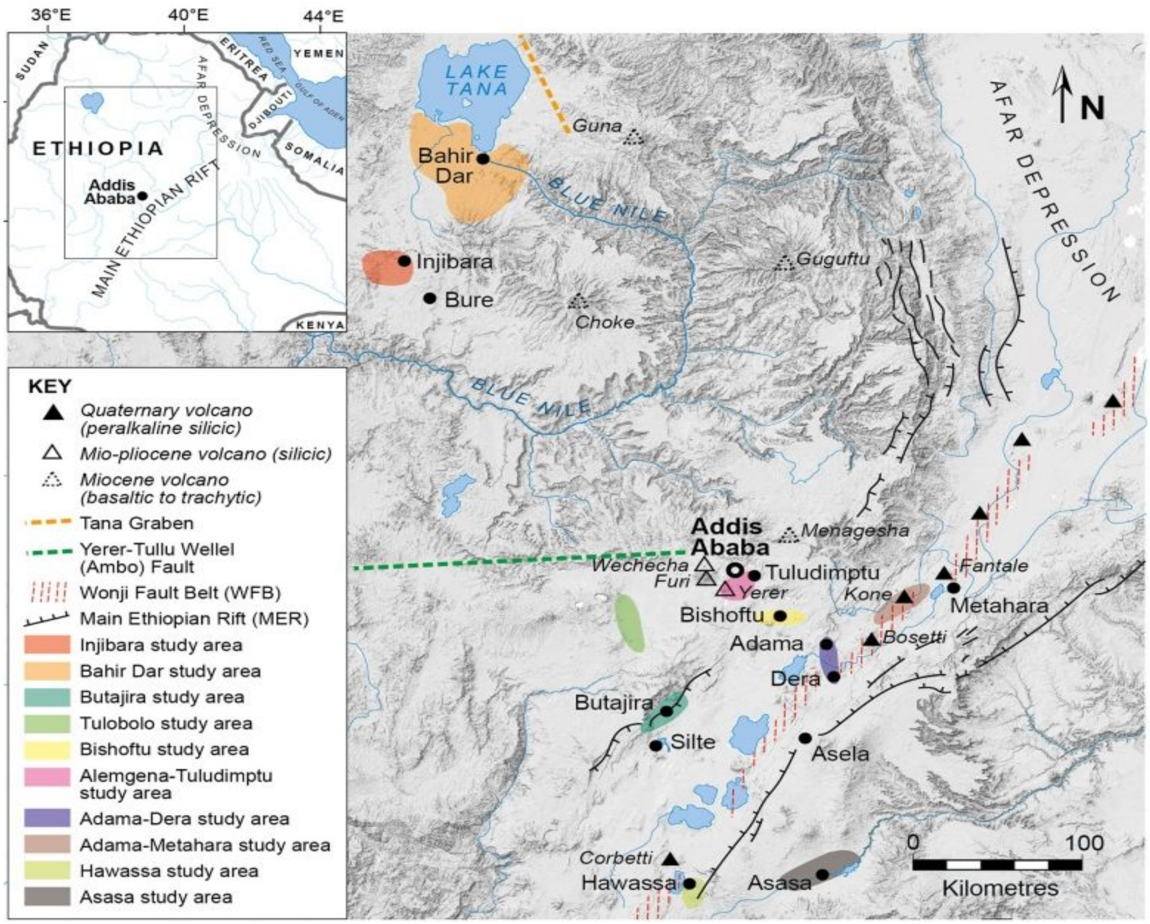



Figure.2.1 Location and Distribution of volcanic cinder in Ethiopia whereas, Tana Graben and Ambo fault taken from Berhe et al (1987) *source: (Gareth et al., 2018)*

There is also significant variability in the colour, angularity, vesicularity, and apparent strength of the materials. For example, layers of dark grey-black cinder were inter layered with red-brown oxidised material and different parts of some borrow pits were found to be predominantly one colour or the other. Vesicularity also varied significantly between deposits within the same cone, as did the apparent strength of individual class (Gareth et al., 2018).

When visually seen the color of a cinder is different from place to place. The strength of a cinder is also different from place to place. Even if the same color cinders have very different strength value.

### 2.2.3 Researches and manuals on cinder gravel of Ethiopia

There are a lot of researches under take on the cinder gravel in Ethiopia. But, the first research was under taken at 1970's. After that there are a lot of research are carried out.

Cinder gravel offers significant potential as a low-cost, naturally occurring material in low-volume rural road construction and rehabilitation. However, the variability in its engineering parameters, particularly its grading, density, porosity and strength, meant that the material often fails to meet standard specifications for road construction (Gareth et al., 2018). As this research states the cinder is natural material it has economical benefit when using as the structure of road materials. But it has its own problem in relation to its properties which can limit its use in construction. The problems of cinder are basic problems of the road structure that are grading, which can minimizes the performance, the compaction that has effect on the service and strength and also the porosity.

The high variability in vesicularity controls the engineering properties of basaltic cinder. Material of lower vesicularity will generally be denser and stronger, and therefore potentially of greater use in engineering. Therefore, the road facility providing is going high (ERA, 2018). But the problem rises from this material has less fine materials which limits it from the gradation level of road structures recommended by standard specification of ERA 2013. These materials are useful in the compaction characteristics of the material by filling the voids and giving cohesive characteristics.

It has been shown that the effect of compaction improved the grading of cinder gravels but that there was still a deficiency in the fine fraction of material less than 75 microns (Newill et al., 1987).

### 2.2.4 Using habit of the cinder materials

Volcanic cinder gravels which, although widespread in Ethiopia, had only occasionally been used for road construction, even though their use would substantially reduce road construction costs in many instances. The reason for its limited use was mainly because these gravels tend to be deficient in fine material and do not meet to the generally accepted grading specifications (Hadera, 2015). As the literature states the cinder material is very abundant available in the country that has significant effect on the road construction. As stated in the World Bank report the road density is very low in the country, it means that the accessibility of the road is at minimum level. This arises from the case of economy, because of the road facility needs large amount of money. So using locally available material uses as an alternative way to attain the road facility**.** However, in some part of they are used in construction at some level. The samples near Alemgena were obtained from the freshly dozed stoke pile to be used for the sub base construction of Alemgena - Butajira Road while the samples from Lake Chamo were obtained from the quarry site which was used for the construction of unpaved roads in Arba Minch town (Berhanu, 2009).

## 2.3 Properties of soil aggregate mixture

Gravel materials have a structure of "loose" (high void ratio or low volume ratio or low density) or "dense" (low void ratio or high density). Depending on the grain size of distribution as well as the packing or arrangement of the grains, a wide range of void ratio is possible (Robert, 1984).

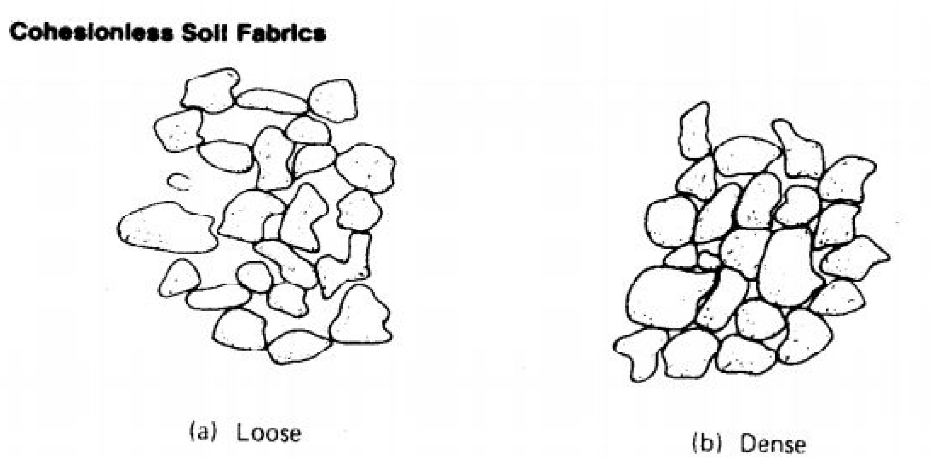



Figure 2.2 Arrangement of the grains *source: (Robert, 1984).*

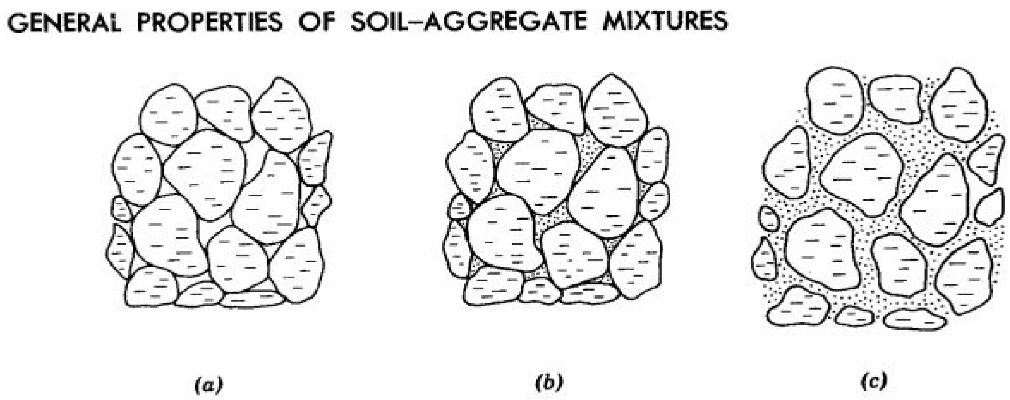



Figure 2.3 Soil – aggregate mixtures *source: (Yoder et al., 1975).*

The stability of a soil – aggregate mix depends up on particle – size distribution, particle shape, relative density, internal friction, and cohesion. A granular material designed for maximum stability should possess high internal friction to resist deformation under load. Internal friction and subsequent shearing resistance depend to a large extent up on density, particle, shape and grain size distribution. Of these factors, the size distribution of the aggregate, particularly the proportion of fine to the coarse fraction, is considered to be the most important. Figure above shows three physical states of soil-aggregate mixtures (Yoder et al., 1975).

An aggregate that contains little or no fines (Figure.2.3 a) gains its stability from grain to grain contact. An aggregate that contains no fines (is the portion of the mix which will pass a no. 200 mesh sieve.) usually has a relatively low density but is pervious and non-frost susceptible. However, this material is very difficult during construction because of its non-cohesive nature.

An aggregate that contains sufficient fines to fill all the voids between the aggregate grains will still gain its strength from grain contact but with increased shear resistance (fig.2.3.b). Its density is high, its permeability is low, and it may be frost susceptible. This material is moderately difficult to compact but is ideal from the stand point of stability, for it will have relatively high shearing resistance in either a confined or unconfined condition.

In Figure 2.3 c a material that contains a great amount of fines has no grain- to –grain contact, and the aggregate merely “floats” in the soil. Its density is low; it is particularly impervious, and it is frost susceptible. In addition, the stability of this type of mixture is greatly affected by adverse water construction and compacts quite readily (Yoder et al., 1975). It should be apparent from the above that the stability of a soil- aggregate mixture is dependent upon the size distribution of the particles. The disadvantages of having a mix with too many fines should also have structural functionality problem.

Good gravel requires a certain percentage of stone to carry and transfer loads of vehicles especially during wet season, a percentage of sand sized particles to fill the voids between stones and give good stability, and also a percent of plastic fines to combine together the stones and sand to make firm gravel surface to shade water (Ken et al., 2000). If the particle size of a material is not well graded there is voided and that it loses the load resistance capacity when the vehicle going on it. The load resistance of a material is good if the composition of the material is fair.

Cinder gravels are generally coarse and lacking in finer and sand particles (less than 4.75 mm) (ERA, 2018). Gradations that have less fine material cannot achieve a maximum cohesive effect whereas gradations that have more fine material tend to retain too much moisture, develop ruts, and become soft, instable, and slippery when wet (Gregory et al., 2001). The gradation of the aggregate must be well graded which contains all sized aggregates in a well-proportioned manner. Thus, using well graded aggregate uses in filling the voids by smaller particles and this tightness give a good strength to the road and also durability.

For unpaved roads, recommendations are made for a particle size distribution which provides a road surface that is resistant to corrugations. Improved performance can be obtained by mechanically stabilizing cinders with plastic fines (Eshete, 2011). A compaction problem is because of no or minimum plastic fine materials. The weathered yellowish laterite has amore fines and a good plasticity character. To use cinder as sub base course in this study it is used by stabilizing mechanically with locally available materials. Therefore, by blending the cinder with weathered yellowish laterite lateritic the required grading is attained.

## 2.4 Laterite

### 2.4.1 General

The selected stabilizing material, yellowish weathered material is a type of Latosolic soils. Laterite soils are highly weathered and altered residual soils formed by the in situ weathering and decomposition of rocks under tropical condition; that occur mostly in tropical and subtropical regions with hot, humid climatic condition (Bell, 2007).

### 2.4.2. Lateritic soil and its formation

Lateritic soils are highly weathered and altered residual soils formed by the in-situ weathering and decomposition of parent rocks under tropical and subtropical climatic conditions. This weathering process primarily involves the continuous chemical alteration of minerals, the release of iron and aluminum oxides, and the removal of bases and silica in the rocks. The geotechnical properties of lateritic soils are influenced by climate, drainage, geology, the nature of the parent rock and the degree of weathering or linearization of the parent rock. These factors also differentiate laterite from other soils that are developed in the temperate or cold regions. Some lateritic soils are thought to have been transported from their place of origin by wind or other action, but most of those with which is used for road construction is likely to have been formed in situ. Lateritic soils contribute to the general economy of the tropical and subtropical regions where they are in abundance because, they are widely utilized in civil engineering works as construction materials for roads, houses, landfill for foundations, embankment dams, etc. As a road construction material, they form the sub-grade of most tropical road, and can also be used as sub-base and base courses for roads that carry light traffic (Aginam et al., 2015).

### 2.4.3 Formation of weathered yellowish Laterite

They are formed under forest and tropical vegetation, they are strongly weathered. Lateritic soils are generally found in warm, humid, tropical areas of the world. The geotechnical properties of these soils are quite different from those soils developed in temperate or cold regions of the world. The properties of lateritic soils are influenced by climate, geology and the degree of weathering or laterization (Aginam, et al., 2015). In humid tropical regions, weathering of rock is more intense and extends to greater depths than in other parts of the world. Residual soils develop in place as a consequence of weathering, primarily chemical weathering (Rahardjo et al., 2004). Consequently, climate (temperature and rainfall), parent rock, water movement (drainage and topography), age and vegetation cover are responsible for the development of the soil profile.

**Tropical weathering**

In the humid, tropical regions of the world, chemical decomposition – the chemical alteration of the primary minerals into secondary and residual products - rather than physical disintegration is the dominant mode of weathering which is especially effective in the presence of water and high temperature. A schematic representation of a tropical weathering profile is shown in Figure 2-4 which indicates a gradual change from fresh rock to a completely weathered residual soil. The weathering process originates with rock exposure at or near the earth's surface in a physical environment, quite different from that in which it was originally formed. The minerals which constitute the rocks may react chemically with rainwater, ground water, and dissolved solids and gases of the new near surface environment to form new minerals which are more nearly in equilibrium with the surface conditions. The end result of these changes is to convert the upper portion of the rock into residual debris more soil-like than rock-like in character and with chemical, mineralogical and physical properties entirely different from those of the original rock (Pinard et al., 2014).

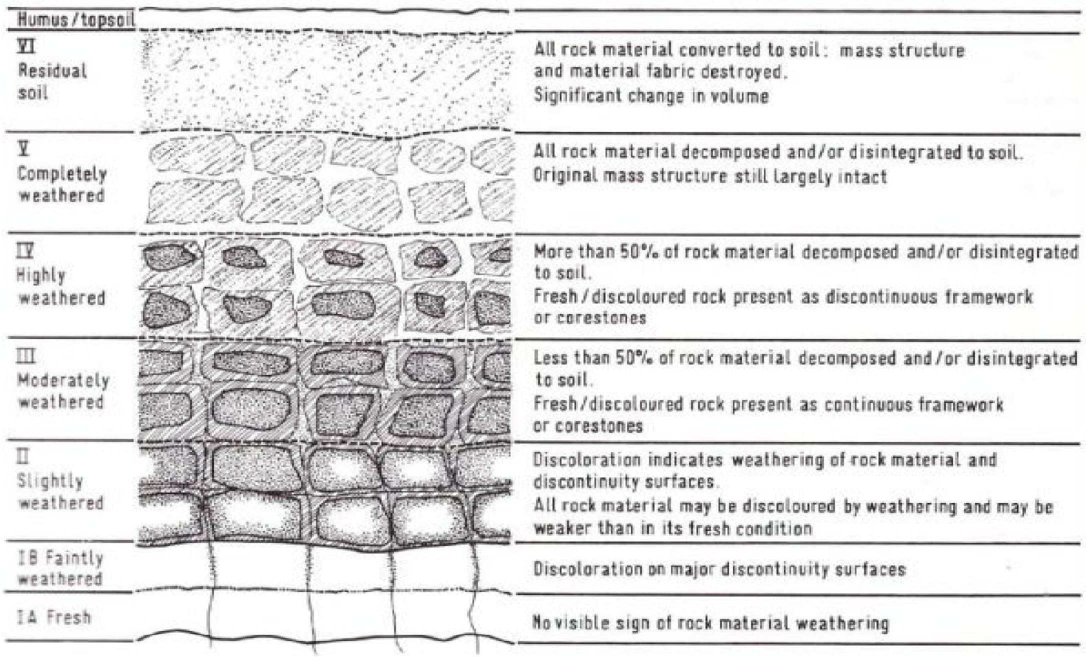



Figure 2.4 Schematic representations of tropical weathering profiles *source: (Pinard et al., 2014)*

Highly weathered materials have more than 50% of fine materials or minimum percent of rocky materials.

### 2.4.4 Occurrence of laterite

One such naturally occurring material is laterite – a type of residual soil that occurs extensively in the humid tropical and sub-tropical zones of the world, including much of central, southern and western Africa (Pinard et al., 2014).

Laterite occurs mostly in the tropical and sub-tropical regions with hot, humid climatic conditions. It has been suggested that a mean annual temperature of around 25°C is needed for their formation, and in seasonal situations there should be a coincidence of the warm and wet periods. If there is high rainfall during the cold season, laterites do not develop freely. The minimum annual rainfall required for laterite formation is generally at least 750 mm. The higher the rainfall above this value, the greater is the leaching effect, which removes free silica, reduces the silica/ sesquioxide ratio and therefore increases the degree of laterization (Gidigasu, 1976)

These soils are found in the equator belt in Africa, South America, SE Asia, SE portion of North America and the Pacific Islands.

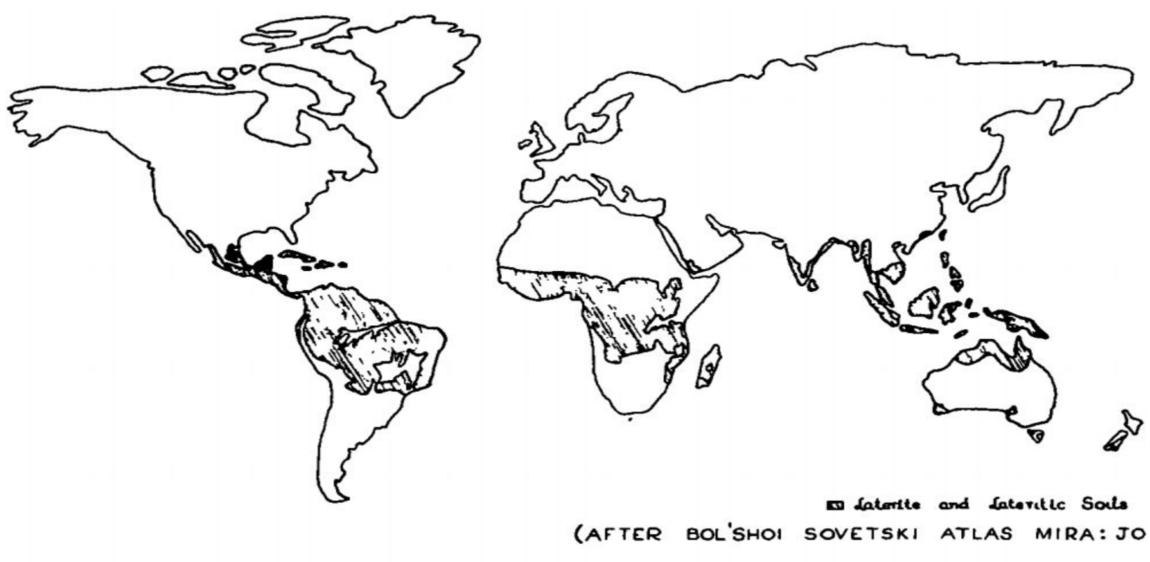



Figure.2.5 Distribution of laterite and lateritic soil in the World *source: (Gidigasu, 1976).*

From the figure 2.5 it is seen that the distribution of laterite is in some part of Ethiopia. Especially, in southern and western part of the Ethiopia. But the use of laterite in to road construction in Ethiopia is very low.

## 2.5 Stabilization

Stabilization is a process by which a soil is improved and made more stable. As stated in the properties of soil – aggregate relationship the gradation is a best to attain a good density and strength. There are 4 types of stabilization 1.Mechanical Stabilization 2. Lime Stabilization 3. Cement Stabilization 4. Bitumen Stabilization 5.Other stabilization

Mechanical stabilization has a vital role to play in the use of cinder gravels in all pavement layers. This is because cinder gravels often lack of fine particles, and where present they are, in general, non-plastic (ERA, 2018). The most widely used method of stabilization is 'mechanical stabilization', where the material is made more stable by adjusting the particle size distribution The essence of mechanically stabilized soils is the use of local soils. As Ethiopia is a full of local materials, this research cinder is stabilized by weathered yellow laterite.

Blending of cinder with an optimum fine-grained soil improves its properties (Eshete, 2011). Blending cinder gravel with an optimum fine grained soil and coarse aggregate improves the CBR of cinder gravels and satisfied the specification for sub base material It means that the cinder are a full of a coarser material that needs a fine material to attain a good cohesion and to get the OMC and MDD.

**Proportioning**

Mixtures are difficult to design and build satisfactorily without laboratory control. A rough estimate of the proper proportions of available soils in the field is possible and depends on manual and visual inspection. Several trial mixtures should be made until this consistency is obtained. The proportion of each of the two soils should be carefully noted (Dennis, 1997). There are two methods of proportioning. These are: Graphical Proportioning and Arithmetical Proportioning

**Graphical Proportioning**:

In this method, the actual gradations of the two soils to be blended (soil A & B, Figure 2.6) are plotted along the left and right axes of the graph, respectively. Once plotted, a line is drawn across the graph, connecting the percent passing of material A with the percent passing of material B for each sieve size. The shaded area of the chart represents the combinations of the two materials that will meet the specified gradation requirements The boundary on the left line is fixed by the upper limit of the requirement relating to the material passing the Number 200 sieve and the right line is established by the lower limit of the requirement relative to the fraction passing the Number 40 sieve. The graphical method eliminates the need for precise blending under field conditions and the methodology requires less effort to use, its drawback is that it becomes very complex when blending more than two soils (Dennis, 1997).

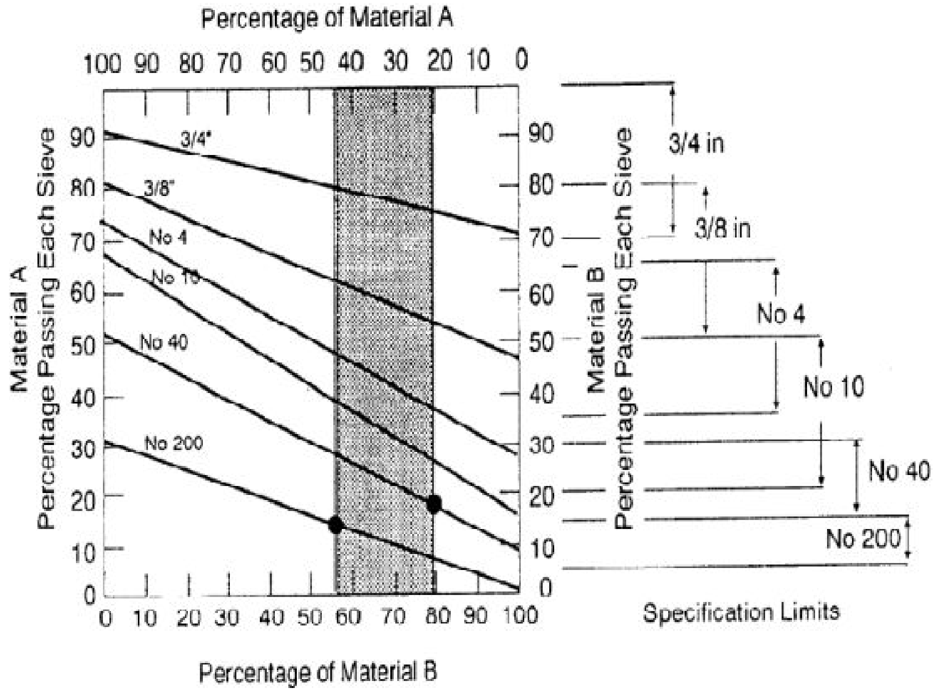



Figure 2.6 Graphical method of proportioning two soils to meet gradation requirement *source: (Dennis, 1997)*

Arithmetical Proportioning: In this method also, the actual gradation of soils A and B in their respective columns are recorded, average the gradation limits and record in the column labeled "S". Next, determine the absolute value of S-A and S-B for each sieve size and record in the columns labeled "(S-A) "and "(S-B)", respectively. Finally, sum columns (S-A) and (S-B) to determine $A$ and $B$ . To determine the percent of soil A in the final mix, use the formula (Dennis, 1997).

For soil A $\% A = \frac{\sum B}{\sum A+\sum B} * 100$…………………………………2.1

And for soil $\% B = \frac{\sum A}{\sum A+\sum B} * 100$ ………………………………...2.2

## 2.6 Mechanistic – Empirical Approach

The mechanistic - empirical process for the construction combines analytical response models with experimentation. The empirical method uses a calculation model for the specific criteria. The mechanistic approach uses experimentation that uses to relate the results with situations that can happen on the pavement. When designing the road structure for service, the performance of the material should be designed by mechanistic empirical approach.

The approach is used in two way one that this approach consists in trials on soils to assess their performance and to obtain a general design criterion in relationship with the experimental measurements and the second it consists in elaborating a calculation model in order to determine the road response under load. It can solve the problem happen in the next service time that is by simulating the applied traffic loading. Experimentation in mechanistic approach allows adjusting the mechanistic model with real conditions, because the mechanical approach is usually based on simplifying assumptions (Mickaël et al., 2016).

As it is known, the existing pavement design and analysis methods rely on empirical procedures developed through long-term experience with specific types of pavement structure and a limited number of types of pavement construction material under limited conditions. Thus it did not give more opportunity to develop and use other new material in the construction. These empirical methods have generally taken a very conservative view of the relative strength properties of granular materials used as base and sub base layers in conventional flexible pavements. The pavement designed form empirical studies of pavement performance are named as empirical methods. But using the relation of stress and strain in the pavement materials are mechanistic methods (theoretical) or analytical methods (Alex, 2001).

The analytical-mechanistic calculation method for pavement design enable the use of new pavement structures including for example recycled materials and optimization of the pavement. As the approach creates more chance to use materials marginally, therefore more accurate design methods are needed to produce life-cycle performance evidence for the contractors and owners, also after construction or rehabilitation. The traditional design methods did not give opportunities they are not able to manage new innovative materials, recycled materials or reinforcements. Obviously, more efficient and sophisticated analytical-mechanistic design methods are needed to use efficiently and safely. Analytical design methods also allow the studies needed in the implementation of the life cycle performance evaluations of the road structure (Leena, 2009).

The use of empirical models is limited to the conditions on which they are based and cannot usually account for changes in loading and environmental conditions. There has been a recent emphasis on the use of mechanistic-empirical approaches to design and analyze pavement structures, thus increase the life time service and safety for the users. In mechanistic-empirical procedures, models based on physics and engineering principles are used to predict pavement response. As in its direct result from the experiment may not directly shows the performance and its character. Therefore it should be adjusted, or calibrated, to fit observed performance, or empirical data. Understanding the behavior of pavement materials and their accurate characterization with the applied traffic loading unloading is important to the successful implementation of any mechanistic-empirical procedure (Alex, 2001).

## 2.7 Mechanical behavior of materials

The mechanical behavior of granular materials is the fundamental material properties that relate an input, such as a wheel load, to an output or pavement response, such as stress or strain on the mechanics of materials. In mechanistic flexible pavements the characterization of the material for mechanical behavior is done. Mechanical (fundamental) behavior of pavement granular layers refers to the failure (shear) strength, resilient deformation or recoverable (elastic) behavior and permanent deformation non recoverable (plastic) behavior. Resilient modulus is an important parameter in the pavement design which represents the stress dependent stiffness of the base, borrow, and sub grade materials under a certain pattern of repeated loading and confinement stress level using a tri axial set-up that can simulate the road and loading condition (Ariel, 2002) and (Jeyakaran et al., 2018). The mechanical properties shows the materials property that they response to the applied load. The materials shear strength shows the materials cohesion and angle of friction that affect the performance or the strength of the material. Different materials have different shear strength and also the deformation, recoverable and permanent deformation is different from one to another.

Cyclic load tri- axial testing is nowadays the most favored test method for the characterization of the mechanical behavior of unbound granular materials and soils. It is the best simulator to estimate the performance of the road structure in service condition. Because of the tri – axial test considers the deviator stress from the traffic and the confining stress from the adjacent confining structure of particular space that the load applied. It considers X,-Y-Z load condition and the frequency of the load applied by traffic. This testing method is, however, not readily available and not an easy test to be done especially in developing countries. Even in developed countries, tri-axial tests are mainly used for research and academic purposes. On the other hand, characterization of the unbound granular base and sub base materials is still mainly done using empirical methods such as California Bearing Ratio (CBR). But a more realistic and relatively simple testing technique is based on the widely practiced CBR test (Ariel, 2002) and (Tommy C. et al., 2007) .

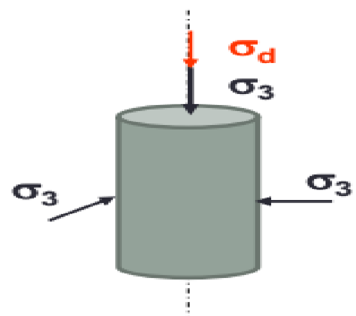



Figure 2.7 Load application in cyclic tri – axial test

The input parameters for the unbound layers include: resilient modulus, Poisson's ratio, dry density, water content, gradation, Atterberg limits, etc which considered in the design phase of the road. The resilient modulus is considered as a key input parameter that has a significant impact on the structural responses of a pavement structure in its lifetime, and thus affects its performance and design (Jeyakaran et al., 2018). The resilient modulus test provides a relationship between deformation (or strain) and stresses in pavement materials, including aggregate bases and sub grade soils, subjected to moving vehicular wheels. The strain is formed by the applied load. Hence, it is not necessarily a fixed value but varies according to the applied stresses of moving vehicles and the resulting stress level in the pavement layers. The test measures the stiffness of a cylindrical specimen of aggregate or soil that is subjected to a cyclic or repeated axial load. It provides a means of analyzing different materials and soil conditions, such as moisture density, and stress states that simulate the loading of actual wheels (Tommy C. et al., 2007).

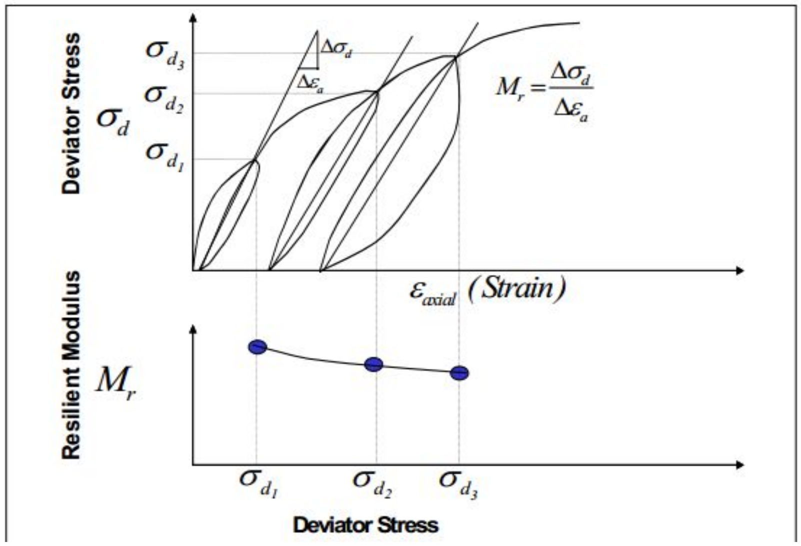



Figure 2.8 Stress – strain hysteresis loop and resilient modulus determination *source:* (Tommy C. et al., 2007).

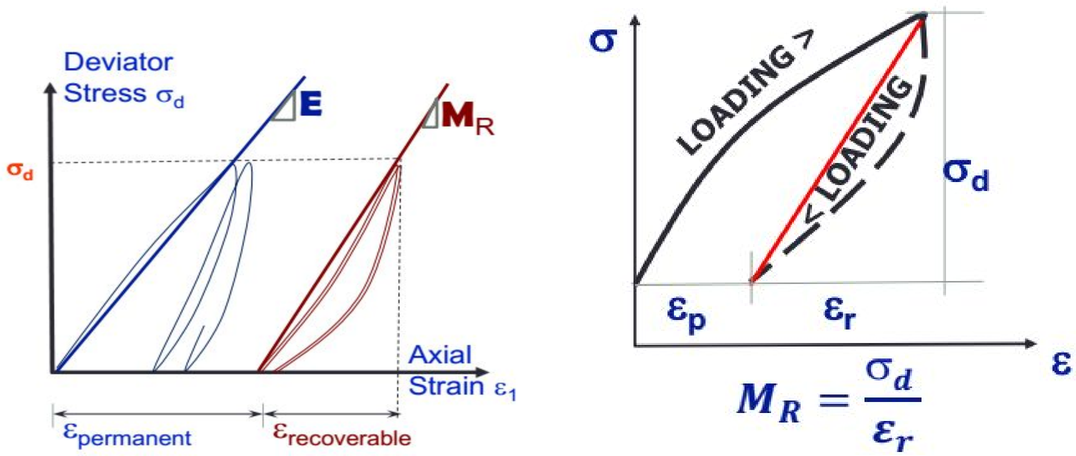



Figure 2.9 Deviator stress to strain graph of cyclic Tri – axial test *source:* (Jan, 2011)

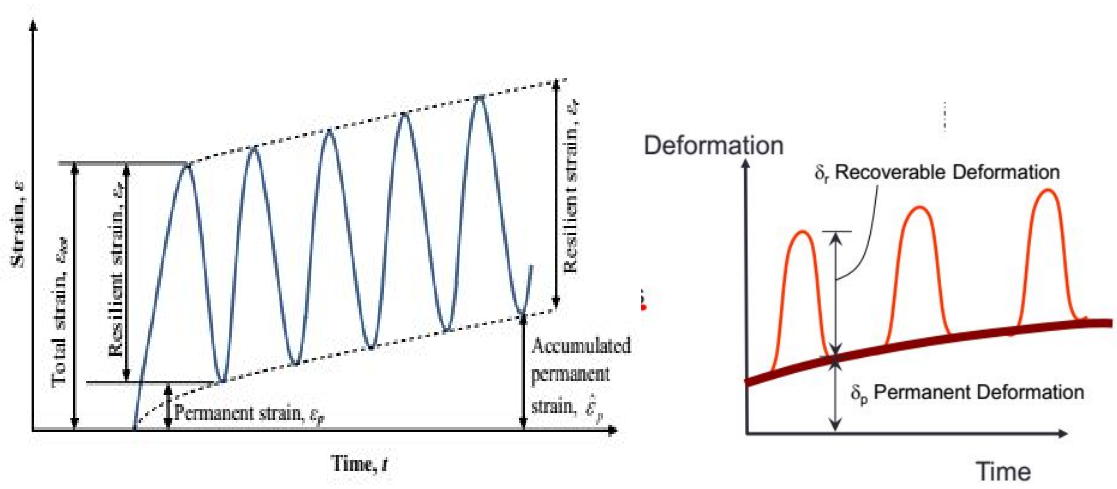



Figure 2.10 Deformation characteristic of material with time *source: (Mohammad, 2015)*

**Repeated load CBR**

To study the property of the specimen repeated load CBR uses to characterize it. The repeated load CBR test uses the standard CBR test setup. The response of the material from the repeated load CBR test, evaluated in the form of modulus was compared with the resilient modulus of the material as obtained from the cyclic resilient modulus test (Nagula et al., 2017).

The repeated load CBR test provides a realistic estimate of the stress dependent resilient modulus of unbound granular materials and soils, which can be used for mechanical design, analysis of pavements in which the bearing capacity is mainly determined by the granular layers and sub grade soil (Araya et al., 2011).

The repeated load CBR test is a good simulator for the repetition of the load and the confinement of the specimen. The test tries to simulate the repetition of load and the deformation formed by the repetition of the load.

The molds and the preparation of the specimen for the test is based on the ASTM standard. That gives the limits for fine and coarse gravel material. The sieves used for the preparation of the sample are the same as standard CBR but the application of the load used is in the 2.54 mm deformation.

**Resilient modulus**

The resilient modulus is to express the recoverable deformation that formed when the structure is subjected to the load. At the time of transferring the load to the sub grade the material exhibits some elastic character. Therefore resilient modulus has been proposed as a means of characterizing the elastic properties of pavement materials. It is expressed as the ratio of deviator stress applied to elastic deformation on the pavement layers. And the resilient axial deformation recovered after release of the deviator stress (AASHTO, 1993).

The road structure is intended to be exposed to the external load applied by traffic that is named as deviatoric load. And this load is repeated in its life time. To resist the load that coming from the traffic the material should go elasticity character.

The resilient modulus (Mr) is a performance based parameter used to characterize the material. It is a parameter which describes the mechanical behavior of unbound granular materials subjected to the traffic. Thus it is used as input parameter in pavement analysis and design and has influence in pavement performance and maintenance.

**Permanent deformation**

Granular materials exhibit two types of deformation when subjected to repeated loading: resilient deformation and permanent deformation. During each load repetition, the permanent deformation is normally just a fraction of the total deformation produced. However, the gradual accumulation of these small permanent deformation increments with number of load cycles could lead to an eventual failure of the pavement due to excessive rutting (Fredrick et al., 1997).

Energy is put into a material when it is loaded. The resilient energy is that part of the applied energy that can be recovered when the material is unloaded. The rest of the energy that is not recovered is capable of doing work on the material. In unbound aggregates, most of the work goes into permanent strain that accumulates with repeated loading and unloading. It is this accumulating permanent strain in an aggregate base course that creates rutting (Alex, 2001).

Permanent deformation in unbound granular materials accumulates with the number of load applications. Although, for each load cycle, permanent deformation is small compared to RD, it may accumulate to a significantly large value leading to rutting and eventual failure of the pavement. Development of permanent deformation is very much dependent on stress levels. It is directly related to deviator stress and inversely related to confining pressure (Mohammad, 2015). Highly confined materials have high strength and it can resist applied stress by recovering and minimum deformation.

# 3. MATERIALS AND METHODS

## 3.1 Introduction

This chapter presents and describes the approaches and techniques the researcher used to collect data and investigate the research problem. It includes the research design, study population, sample size and selection, sampling techniques and procedure, data collection methods, procedure of data collection, data analysis. It states the approach to attain the appropriate specific objective of the study.

## 3.2 Description of the Study Area

For the case study the Cinder samples taken from near Humbo town quary site and the weathered yellowish laterite taken from the quarry site of bosha, near to Shinshicho town. Both areas are found in S/N/N/P/R/of Ethiopia.

Humbo town is founded at Wolayita zone in the road of Sodo to Arbaminch. Its location is 6º44'45"N to 6º43'51"N and 37º45'51"E to 37º45'52"E at average height of 5,840 ft.

The study woreda Kacha Birra is located in Kambata Tembaro zone, SNNPR of Ethiopia. The woreda lies between 07°9'0" - 07°18'0" N and 37°42'48"– 37°50'30.6"E degree north and east longitude in Kembata Tembaro zone of SNNPRS. The woreda capital is found 327 Kms away from the country capital Addis Ababa and 117 km away from the regional capital, Hawassa.

Bosha quarry site founded at Kembata Tembaro Zone near to Shinshicho town. Its location is 7º11'23"N to 7º11'35"N and 37º45'17"E to 37º45'21"E at average height of 5810 ft.

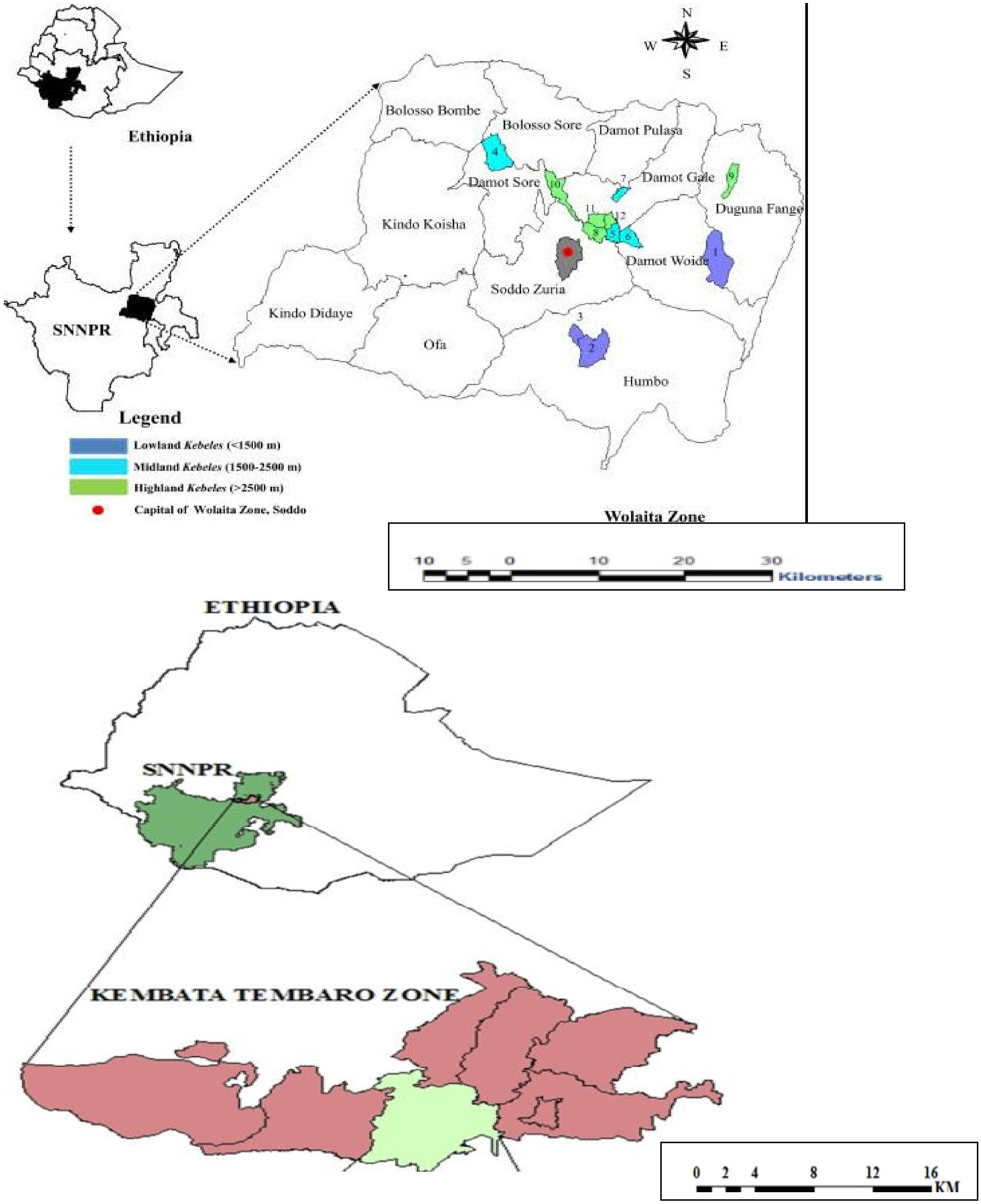



Figure 3.1 Wolaita zone and Kembata Tembaro zone *source : (Olango et al., 2014)and Tadesse et al., 2017)*

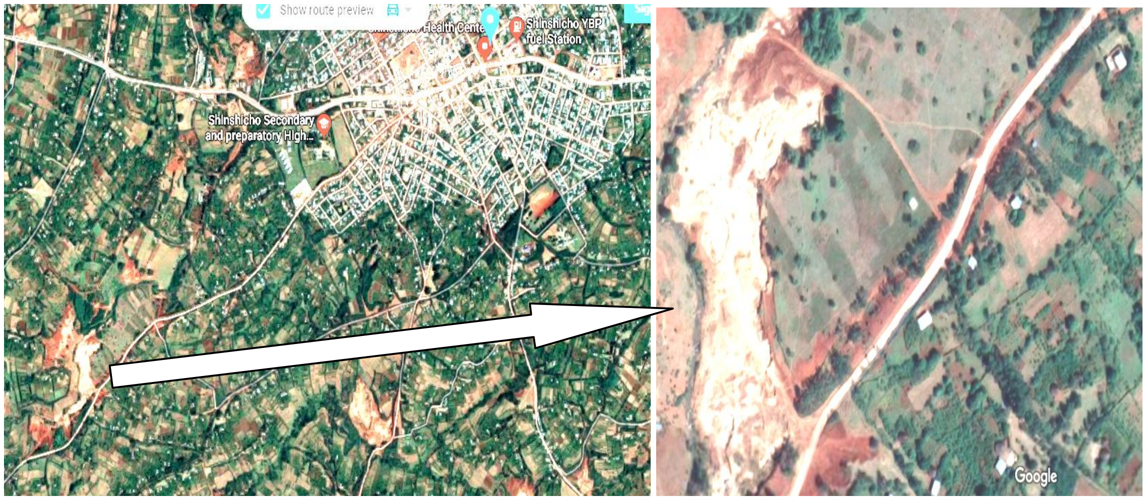



Figure.3.2 Weathered yellowish Laterite Bosha quarry site *source: (Google Earth, 2018)*


Figure 3.3: Cinder site around Humbo town *source: (Google Earth, 2018)*

## 3.3 Research Design

The research is designed to attain the specific objective of the study. The research started with purposive sampling selection process in terms of which a representative sample of both the cinder and weathered yellowish laterite as they used in the construction from quarry site. Three samples were collected from each borrow pit to obtain representative samples. The selection of borrow pits was based on different criteria. First criteria based on abundance of gravel not greater than 72 mm, because there is much larger materials like boulders and how the materials from the borrow pit are used for local roads and cobblestone sites. The information on the frequency use of the respective borrow pits was collected from the infrastructure department specifically the road construction and maintenance crew. The second criteria based on the perceived quality of the material on the borrow pit due to its past performance, although there was no any information of laboratory tests done before (Michael, 2017). The research conducted by using Experimental methods. Which means that the methodology used in the research is a laboratory analysis of sample data. After comprehensively, organizing a literature review of different previous published researches, on the performance of the cinder gravel material then the appropriate or best choose able blending material is selected.

In this phase of study, collection of previous works on the study area and literature related to cinder gravel investigation and usage of it and as well as the investigation and using trend of the weathered yellowish laterite. The necessary data that are collected includes,

- Topographic map of the study area to identify the size and shape of the study area and distributions of features on the land surface.
- Geological map in order to understand local and regional geological formations and geological structures in the study area.
- Sampling techniques and conducting laboratory tests.

To study the blended material suitability for sub base layer the ASTM laboratory procedure were conducted. The Characteristic/Engineering properties of the cinder, weathered yellowish laterite at Appendix A and blending proportion at Appendix B and types of tests are at Appendix C and repeated load CBR test at Appendix D.

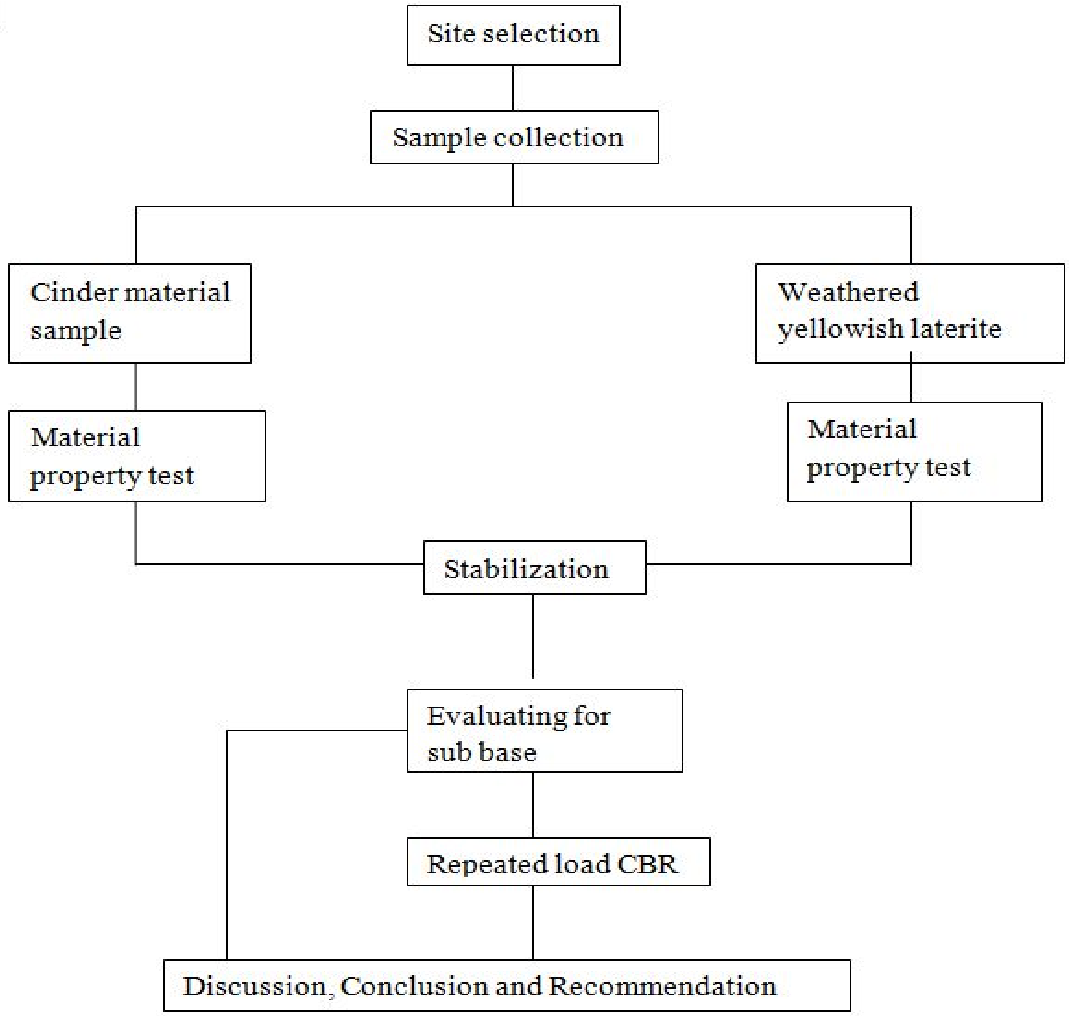



Figure 3.4 Schematic research design

## 3.4 Population

### 3.4.1 Sample Size and Selection

Sampling of 3 bags or 150 kilograms of cinder gravels from potentially used area of Humbo quarry site (ERA, 2018). The sample of 3 bags or 150 kilograms of weathered yellowish laterite are from Bosha quarry site. Three samples were collected from each borrow pit to obtain representative samples (Michael, 2017). Thus, the samples are taken from upper part, from middle part and the bottom of the collected material.

### 3.4.2 Sampling Technique and procedure

The samples collected from the existing borrow pits. From the quarry sites, the sample will take from potentially used area for cobblestone base course road construction that can represent real condition of the area in the road construction (Berhanu, 2009). After the selection of the site, the samples are taken from the site, and the materials were quartered to work the test on the representative samples. For the stabilization process graphical proportioning method is used and the cinder is plotted at right and labeled as material B where as weathered yellowish laterite material is plotted at left labeled as material A. For both materials the percentage passing of the material is plotted at left and right for weathered yellowish laterite and cinder material, respectively. The proportion at the bottom shows the percentage to be used in material B or cinder. The proportion at the top of the graph shows the percentage to be in material A or weathered yellowish laterite.

Figure 3.5 Quartering the sample

## 3.5 Study Variables

The variables studied in the research, the suitability of the blended material quality for sub base layer.

### 3.5.1 Independent variables

The independent variables in the study are Gradation, Liquid limit, plastic limit, CBR, OMC &MDD, Shrinkage limit.

### 3.5.2 Dependent variables

The dependent variables are plasticity, density, strength, permanent and resilient deformation.

## 3.6 Software and instruments

The instruments and software used for this study: plastic bags, manual hand auger equipment, laboratory equipments, and field test instrument, Digital Camera for documentation, Google earth, geographical information system, MS word and Excel into analysis laboratory data and display research data were used in this study.

## 3.7 Data Collection Process

Data collection is very crucial and important for conducting the research.

### 3.7.1 Field work

- Verified the geological data/characteristics that developed from review of data analysis and interpretation of literatures.
- Cinder samples were from Abela, near Humbo town obtained from the quarry site which was used for the construction of base course of cobblestone roads in Sodo and nearby towns.

Figure 3.6 Sampling cinder from Humbo (Abela) site

Figure 3.7 Using the cinder for the construction of for cobble stone roads


Figure 3.8 Sampled cinders from the humbo (Abela) site.

- Weathered yellowish laterite was obtained from Bosha quarry site, which used for unpaved roads in Shinshicho town and in URRAP roads.

### 3.7.2 Experimental or Laboratory Tests

In conducting laboratory tests, the tests are carried by ordered step. The carried tests are particle size distribution, the index properties that mean plastic limit and liquid limit, the proctor compaction test, CBR tests, linear shrinkage, based on ASTM standard.

The first test carried is particle size distribution and conducting laboratory test of particle size distribution for the sub base course is carried by wet sieving. Thus the laboratory experiments were started by drying the samples and weighing the dried sample. Then the samples washed with pure water to remove the dust particles or silt and clay particles. Those are very fine particles which can pass 0.075 mm sieve when washed by pure water.

Then the sample dried by an oven to weigh the washed sample and to sieve in the standard sieve sizes by ASTM and ERA specification.

Figure 3.9 Sieving the cinder materials

Table 3.1 Particle size distribution of cinder material

| | Test method AASHTO T-27<br>Cinder /red ash/ | | | | | | |
|---|---|---|---|---|---|---|---|
| | trial 1 | | | trial 2 | | | |
| | total weight before washing (gm) 7398.00 | | | total weight before washing (gm) 7400.00 | | | |
| | total weight after washing (gm) 7302.00 | | | total weight after washing (gm) 7283.00 | | | |
| sieve size | weight retained (gm) | % retained | % pass | weight retained | % retained | % pass | Specification for sub base |
| 63.0 | 0.000 | 0.000 | 100.000 | 0.000 | 0.000 | 100.000 | 100.000 |
| 37.5 | 143.000 | 1.933 | 98.067 | 556.000 | 7.514 | 92.486 | 70-100 |
| 20.0 | 2189.000 | 29.589 | 68.478 | 1580.000 | 21.351 | 71.135 | 50-100 |
| 4.75 | 3774.000 | 51.014 | 17.464 | 3612.000 | 48.811 | 22.324 | 30-100 |
| 2.0 | 684.000 | 9.246 | 8.218 | 915.000 | 12.365 | 9.959 | 17-75 |
| 0.425 | 375.000 | 5.069 | 3.149 | 472.000 | 6.378 | 3.581 | 11-56 |
| 0.075 | 116.000 | 1.568 | 1.582 | 130.000 | 1.757 | 1.824 | 5-25 |
| Total | 7281.000 | | | 7265.000 | | | |

From the gradation test the property of the material is founded and other engineering characteristic of the materials were tested. The atterberg limit test conducted to get the plastic limit. Modified proctor compaction did to derive the optimum moisture content and maximum dry density. The CBR of the material were founded from the standard CBR for three different compaction levels by ASTM manuals. Then the repeated load CBR used to investigate the deformation were conducted steps given (Araya, 2011).

**Repeated load CBR laboratory tests to characterizing deformation**

According to Araya (2011) To apply the test method in a standard CBR test machine in routine road project tests the standard CBR loading rate i.e. 1.27 mm/min is adopted for both loading and unloading and the following procedure is used:

- The specimen is loaded, at the standard CBR test rate (1.27 mm/min), to a predetermined load or deformation level (for e.g. 2.54 mm). The load is recorded and the specimen is unloaded to a minimum contact load of 0.1 to 0.3 MPa.
- The specimen is re-loaded to the same load at the same rate of loading 1.27 mm/min, and released once more to the minimum contact load. The load level for each cycle is therefore kept constant.
- These cycles are repeated for about 60 – 100 load cycles at which the permanent deformation due to the last 5 loading cycles will be less than 2% of the total permanent deformation at that point.

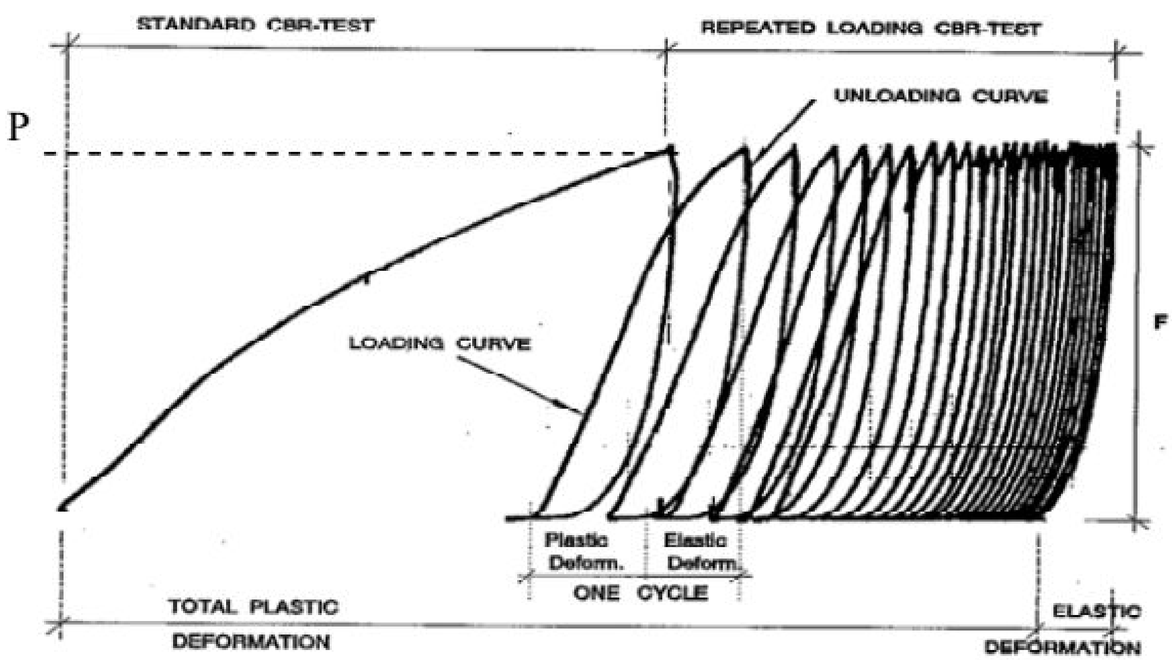


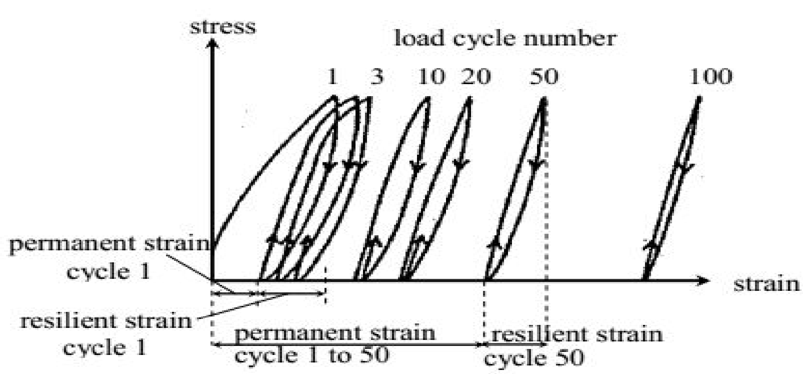

Figure 3.10 Loading and deformation *source:* (Araya, 2011)

## 3.8 Data processing and analysis

In data processing and analysis the initial step is studying the engineering properties of the cinder material and weathered yellowish laterite. In studying the engineering properties of a cinder and weathered yellowish laterite, the gradation and the index properties are studied. The gradation properties of the material are the important variable that evaluated in the first for the sub base course. The liquid limit and plastic limit are processed by the ASTM manual and these engineering properties are evaluated by ERA 2013 standard specifications.

After evaluating the gradation of the material, the material stabilization did based on the proportioning method of the graphical proportioning method. Then study the engineering properties of the stabilized materials. The gradation, liquid limit, plastic limit, proctor compaction test, CBR test and the shrinkage limit are processed by the ASTM manual and the values are evaluated by ERA 2013 standard specification for sub base course.

Then the deformation is studied from the repeated load CBR test. The repeated load CBR did for the 3 specimens for the 65 blows, 30 blows and 10 blows. From the test permanent or plastic deformation and elastic deformation are recorded. The elastic and permanent deformation was studied as in figure 3.11.




.Figure 3.11 Stress–strain behavior for granular materials under repeated CBR loading *source:* (Ariel, 2002).

The results of repeated load CBR are processed by the MS – Excel to get the permanent deformation and resilient deformation and the data processing and analysis the descriptive statistical technique in the form of graphical and tables that can relate the results with the other scientifically results.

# 4. RESULTS AND DISCUSSION

The road construction requires standard materials which can resist, distribute and transfer the load to the next layer. Therefore, studying the engineering properties of the materials is very necessary to use the materials in a construction site. The cinder material has its own engineering properties that help to use the materials widely.

## 4.1 Engineering properties of cinder materials and weathered yellowish laterite

Studying the engineering properties of a cinder and the weathered yellowish laterite is essential to use the materials as sub base course in road construction.

### 4.1.1 Engineering properties of cinder materials

#### 4.1.1.1 General

The cinder material is naturally occur material. Locally the material is used as a base course for the cobblestone site.

Figure 4.1 Occurrence of the cinder material

The color of the cinder is different from the site to the site as discussed in the literature. And the material strength is differing from site to site. The used cinder material in this thesis is red in color and locally the material is called as red ash.

#### 4.1.1.2 Gradation

Table 4.1 Gradation of cinder

| trial 1 | | | | trial 2 | | | |
|---|---|---|---|---|---|---|---|
| total weight before washing 7612.000 | | | | total weight before washing 7598.000 | | | |
| sieve size | weight retained (gm) | % retained | % pass | sieve size | weight retained (gm) | % retained | % pass |
| 63.000 | 0.000 | 0.000 | 100.000 | 63.000 | 0.000 | 0.000 | 100.000 |
| 37.500 | 143.000 | 1.879 | 98.121 | 37.500 | 253.000 | 3.330 | 96.670 |
| 26.500 | 214.000 | 2.811 | 95.310 | 26.500 | 501.000 | 6.594 | 90.076 |
| 20.000 | 2189.000 | 28.757 | 66.553 | 20.000 | 1580.000 | 20.795 | 69.281 |
| 4.750 | 3774.000 | 49.580 | 16.973 | 4.750 | 3612.000 | 47.539 | 21.743 |
| 2.000 | 684.000 | 8.986 | 7.987 | 2.000 | 915.000 | 12.043 | 9.700 |
| 0.425 | 375.000 | 4.926 | 3.061 | 0.425 | 472.000 | 6.212 | 3.488 |
| 0.075 | 116.000 | 1.524 | 1.537 | 0.075 | 130.000 | 1.711 | 1.777 |
| pan | 0.000 | 0.000 | | Pan | 0.000 | 0.000 | 1.777 |
| Total | 7495.000 | | | | 7463.000 | | |

The cinder materials have a good gradation in upper sieves and that it has some part of coarse gravel 75 mm to 20 mm and medium gravel 20 mm to 6 mm and fine gravel 6 mm to 2mm. The material with a good composition of gravel fraction is used in pavement to give the strength and increase the serviceability of the road.

The cinder materials gradation or the percent pass starting decreases in the 4.75 sieve. Thus show the cinder material is lack of the sand fraction whose range is in between 2.0 mm and 0.075 mm. The coarse sand ranges from 2 mm to 0.6 mm. Medium sand range from 0.6 mm to

0.2 mm and fine sands range from 0.2 mm to 0.075 mm. The grain to grain contact of the material in road structure increases the load carrying capacity of the layer. Thus the grain to grain contact of material used to transfer the load and the sand fraction helps the gravel fraction by holding them in place when transferring the load to the next layer. As discussed in the literature the sand fraction in the material increases the shear resistance of the material. The shear strength of a material increases when the material has a good angle of friction and cohesive resistance. The material has a good angle of friction if the grain material stays in their original place and do not show more deflection. Therefore, the shear resistance character of the cinder material is affected by the sand fraction. The angle of friction of a material has direct effect on the material mechanical behavior.

The sand friction in the gravel material is filled in the voids between the gravel materials. The fill in the void part of the gravel material makes the material to be stable. And also it is uses in the sub base course in the worst or rainy seasons. The water can simply percolate the gravel material with void material and have effect of frost action. Thus, it causes the formation of high deformation and minimizes the performance of the road structure. Therefore, the sand fraction can minimize the percolation of the water when the structure compacted well.

#### 4.1.1.3 Plasticity characteristic

The cinder material is non plastic material. However, there are some proportion of fine materials the cinder by its nature has not the plasticity characteristics. That means its liquid limit cannot be determined because the fine materials cannot stay on the Atterberg Device That means the material has not cohesive property by nature. The cinder material has not the ability to stick together. Not only in laboratory tests, checking the material by the hand.

By confining the cinder gravel, it can increase the shear strength of the material. The strength of a material is enough to resist the coming stress from the applied load. (Newill et al., 1980)

#### 4.1.1.4 Loss of cinder material

Table 4.2 shows the loss in cinder material. Thus, cinder material has a minimum percentage of silt –clay fractions. Thus, the silt clay fraction contains the particles less than 0.075 mm.

Table 4.2 Loss of cinder sample

| | trial 1 | trial 2 |
|---|---|---|
| Total weight before washing (gm) | 7612 | 7598 |
| Total weight after washing (gm) | 7495 | 7463 |
| Total lost (gm) | 7612 - 7495 = 117 | 7598 - 7463= 135 |
| Loss by % | (117/7612)*100 = 1.54% | (135/7598) = 1.77% |

The coarse gravel material has a porosity of its surface. But the gravel less than coarse gravel low porosity. The porosity character of the material affects the performance of the material in the field at rain season. The porous part of the material has the ability to hold the water in the porous and thus it has a direct effect on the performance of the material.

#### 4.1.1.5 Material classification

The material classification is based on AASHTO classification of soils and soil aggregates.

| General classification | Granular materials (35% or less of total sample passing No. 200) | | | | | | |
|---|---|---|---|---|---|---|---|
| | A-1 | | | A-2 | | | |
| Group classification | A-1-a | A-1-b | A-3 | A-2-4 | A-2-5 | A-2-6 | A-2-7 |
| Sieve analysis (percentage passing) | | | | | | | |
| No. 10 | 50 max. | | | | | | |
| No. 40 | 30 max. | 50 max. | 51 min. | | | | |
| No. 200 | 15 max. | 25 max. | 10 max. | 35 max. | 35 max. | 35 max. | 35 max. |
| Characteristics of fraction passing No. 40 | | | | | | | |
| Liquid limit | | | | 40 max. | 41 min. | 40 max. | 41 min. |
| Plasticity index | 6 max. | | NP | 10 max. | 10 max. | 11 min. | 11 min. |
| Usual types of significant constituent materials | Stone fragments, gravel, and sand | | Fine sand | Silty or clayey gravel and sand | | | |
| General subgrade rating | Excellent to good | | | | | | |

Figure 4.2 AASHTO soil classifications *source: (Arora, 2004)*

To classify the material first it was checked the percentage passing of No. 200 sieve or 0.075 mm. In two trials, the ratio that passed no 200 sieve is:

In trial 1 (117/7612)*100 = 1. 54% and in trial 2 (135/7598) = 1. 77%

Therefore the material is generally classified as granular materials that 35% or less of total sample passing No. 200 sieve. And in group classification it has no plasticity index or zero and in both case the percentage passing in No. 200 is less than 15, in No. 40 percentage passing is less than 30; hence: the material is classified as **A-1-a.** Thus, the material is rated as excellent. That the cinder material has a very good strength and a material is appreciable in strength.

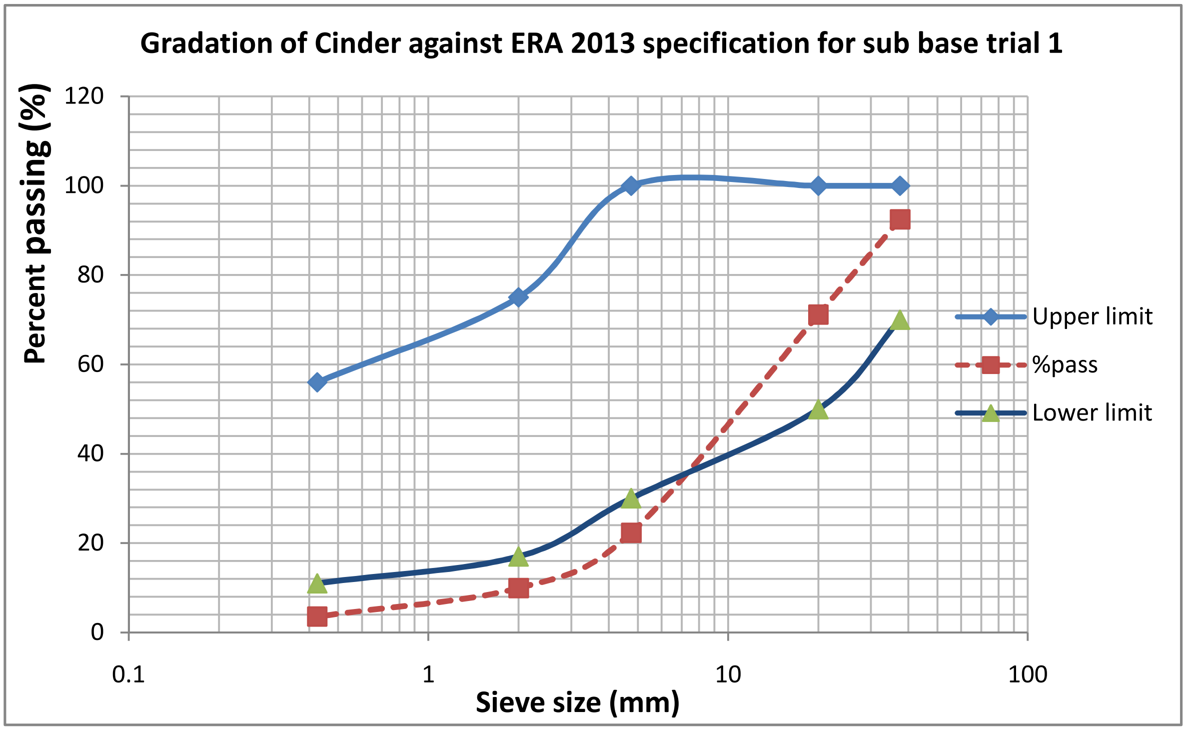
#### 4.1.1.6 Evaluating cinder gravel for sub base

To use the material for the pavement structure it is necessary to check it for the specification given by manuals and researches. Thus, the specifications are developed from the experiences, and a lot of trials. To use the material for the road structure additional criteria developed that is the mechanistic approach. Therefore, testing the material mechanistic empirical character increases its performance.

In the selection of the materials for sub base there are a lot of choices are given to the pavement designers. But it needs characterizing and evaluating the materials with the standard given. To get a good performance the material should have characterized for developed by the experienced workers and professionals. Ethiopia road authority gives the material for the road sub base construction are natural gravel, scoria (cinder gravel), and weathered rock, crushed gravel, crushed rock or crushed boulders, recycled pavement material and any other granular material complying with the requirements. Therefore, the standard helps us to use Scoria or cinder gravels material for the road sub base.

**Gradation**

The gradation or particle size distribution and shape are the factor for the sub base material, which in turn affects another engineering property of the material. ERA, 2018 recommends well - graded material with a smooth continuous grading within the limits given by blue color in figure 4.4. For natural gravel material the standard gives wide grading range. The well graded materials have all sized particles in appropriate ratio. Thus uses to compact and fill the voids and to give a good performance.



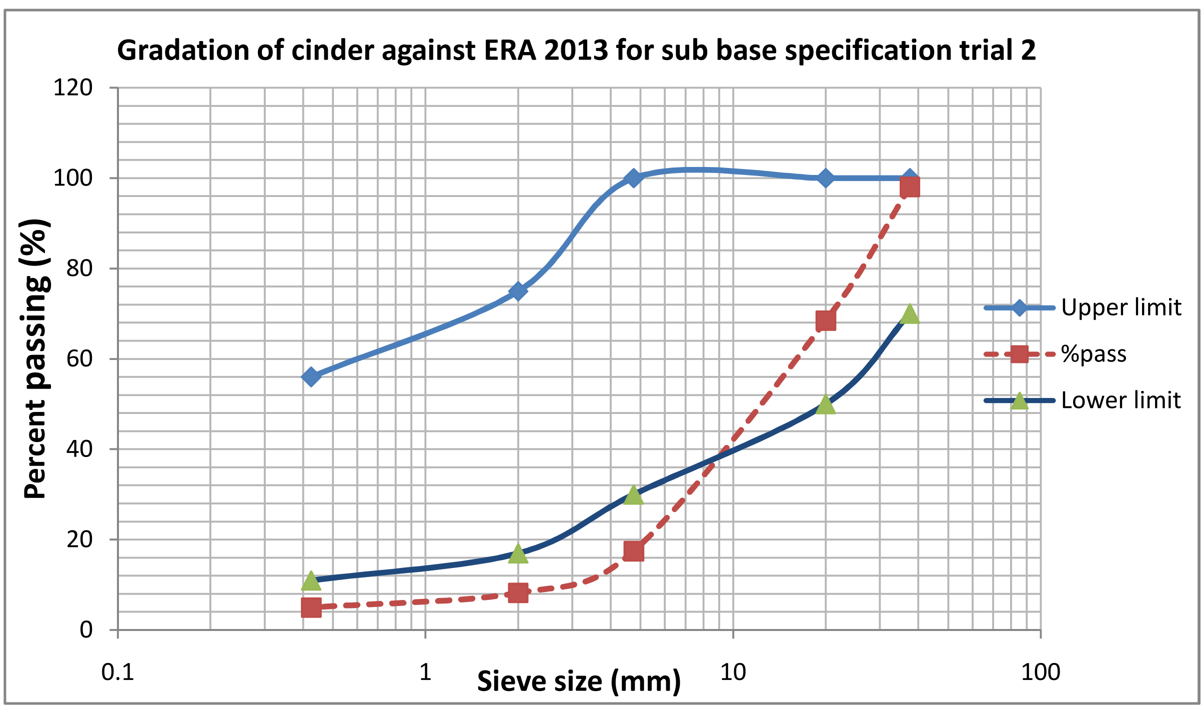




Figure 4.3 Gradation curve for the cinder gravel material for trial 1 and 2, respectively

Figure 4.3 show that the material cannot attain the requirement of the sub base specification standard. That means it is lack of fine gravel that less than 6 mm and sand fractions between 2 mm to 0.06 mm. And the material is not well graded and it is gap graded. That the curve is steep and the slope is very high.

From the material gradation and literatures before these material is lack of sand fraction. But According to Yoder et al., (1975) an aggregate that contains no fines usually has a relatively low density and in sufficient fines to fill all the voids. Thus, its density is low and its permeability is high. An aggregate that contains sufficient fines to fill the voids between the aggregate will still gain its strength from grain contact but with increased shear resistance.

The cinder materials is very hard to compact because the sticking property of soil particles more closely together and impossible to increase the dry density of the soil. It is cohesion less material that limits the use of the material for road construction. As the gradation of cinder cannot satisfied the gradation requirement of sub base, therefore another specification requirement cannot undergo.

Therefore, to use the cinder gravel for road construction as sub base, it needs improving the engineering characteristics of materials. As this material occurs naturally and locally in many parts of the country it has to be improved and marginalizing it is necessary. To overcome the problem of gradation, and plasticity characteristics the material with good plastic and highly composed of fine material is needed. The weathered yellowish laterite has good plasticity characteristics and fine materials; therefore, it can improve the problem.

### 4.1.2 Engineering characteristics of weathered yellowish laterite

#### 4.1.2.1 General

Laterite is the product of a humid tropical weathering process, current or past, which has the following effects: The parent material is chemically enriched with iron and aluminum oxides and hydroxides (sesquioxides), the clay mineral component is largely kaolinitic and the silica content is reduced.

The above processes usually produce yellow, brown, red, or purple materials. The weathered yellowish laterite is naturally occurring material. It is a soft material that any iron instruments cuts, dig up and cut into the shape wanted with a trowel, or large knife.

The visual color of the material is yellow, and therefore, it is named as weathered yellowish laterite. According to Aginam C.H et al, (2015) the Lateritic soil is weathered materials and altered residual soils formed by the in situ weathering and decomposition of parent rocks, and also the materials may have red, yellow or brown color. The mineral composition is affected by the parent rocks. It is defined as laterite is a highly weathered tropical soil rich in secondary oxides of combination of iron, aluminum and manganese.

Pinard et al., (2014) discussed the variety of soils and their uses for road construction. In use of laterite for road construction, all types of laterite formed by weathering can not be used. As reviewed in the review of specifications for the use of Laterite in road pavements lateritic soil confines the material to a fairly small group of red soils that harden irreversibly on exposure (described as red clay used for air dried brick production) to any variety of reddish, iron rich, tropical residual soils. That means the material cannot undergo the recoverable deformation so that the material cannot meet engineering requirement.

The engineering properties of weathered yellowish laterite incourages to use the material in road construction. Thus, this research study did focus on the weathered yellowish laterite.

#### 4.1.2.2 Occurrence and schematics of weathered yellowish laterite

According to the review of Pinard et al., (2014) in specification for the use of laterite in road pavement states that laterite materials is a type of residual soil. And that the laterite material occurs extensively in the humid tropical and subtropical zones of the world. Ethiopia found in the humid tropical zone. The weathering of laterite is carried by the climatic condition. Parent rocks weathered and decomposed under tropical and subtropical climatic condition

The specific area of the study that kacha birra woreda lie in between $7^{o}9'0''$ - $7^{o}18'0''$N and $37^{o}42'48''$ - $37^{o}50'30.6''$E. That is in between $25^{o}$ which is used to classify the place as tropical area. The study area is characterized by mind land (1500-2300 m.a.s.l) were considered, for they are important regarding the area coverage and population size. Average temperature ($^{o}$c) and annual rainfall of the area range between 23 $^{o}$c to 16 $^{o}$c and 800 mm to 1200 mm rainfall (Tadesse et al., 2017). According to Gidigasu, (1976) the climatic condition of the steady area means the rainfall and temperature shows the laterite is formed in the zone.

Figure 4.4 Natural occurrence of weathered yellowish laterite

The weathered yellowish laterite has some percentage of rocky materials that remained in the weathering process. This shows that < 50% of the sample has rock, therefore, the material is highly weathered laterite (Pinard et al., 2014). When the rock is exposed to high weathering process, percent of fine material increase. This occurs because of the original rock material is disintegrated to fractions. The visible rocks are materials retain on the number 20 sieve.

Figure 4.5 Rocky materials in weathered yellowish material

Laterite is a soil and rock type that rich in iron and aluminum and occur mostly in tropical and sub-tropical regions with hot, humid climatic conditions (Deboch, 2018).

Figure 4.6 Schematic occurrence of yellowish weathered laterite above moderately weathered rock

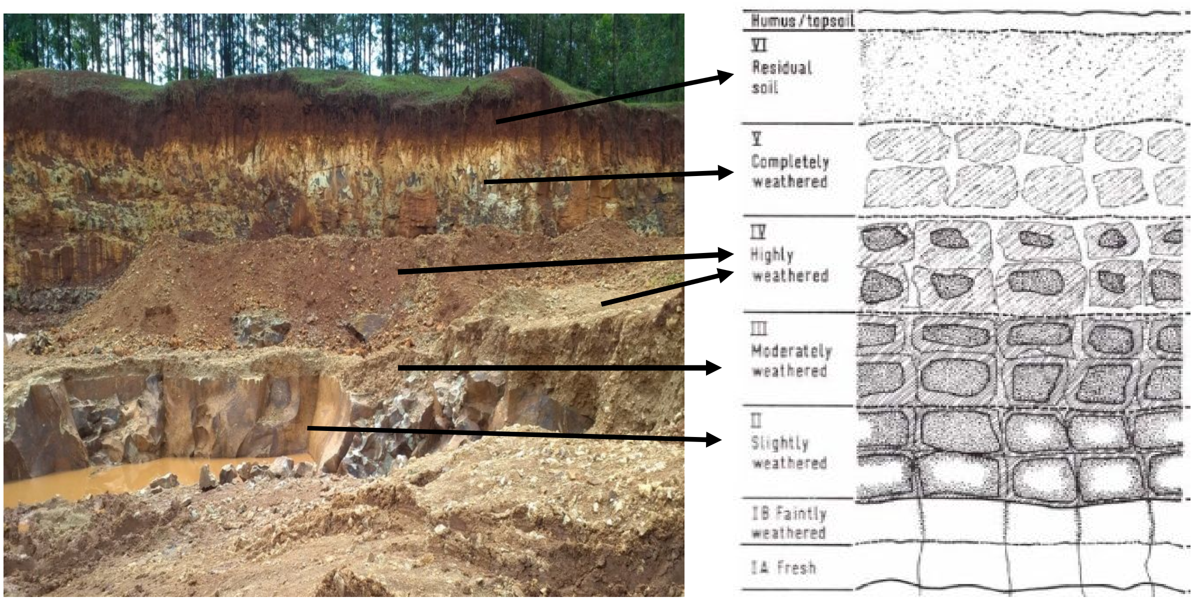



Figure 4.7 Schematic occurrence of weathered yellowish laterite

Figure 4.7 shows the highly weathered materials that found above the moderately weathered materials. According to Pinard et al., (2014) the laterite are the process of weathering and its schematic representation shows that the weathered yellowish laterite is highly weathered. The weathered yellowish laterite occurred in layer form, thus it is not very complicated to differentiate.

#### 4.1.2.3 Plasticity character

The plasticity character of the weathered yellowish laterite is the deformation characteristics of the material. That it is the range of water content for which a soil will behave as a plastic material (deformation without cracking).

Plasticity Index = Liquid limit – Plastic limit ……………………………………Equation 1

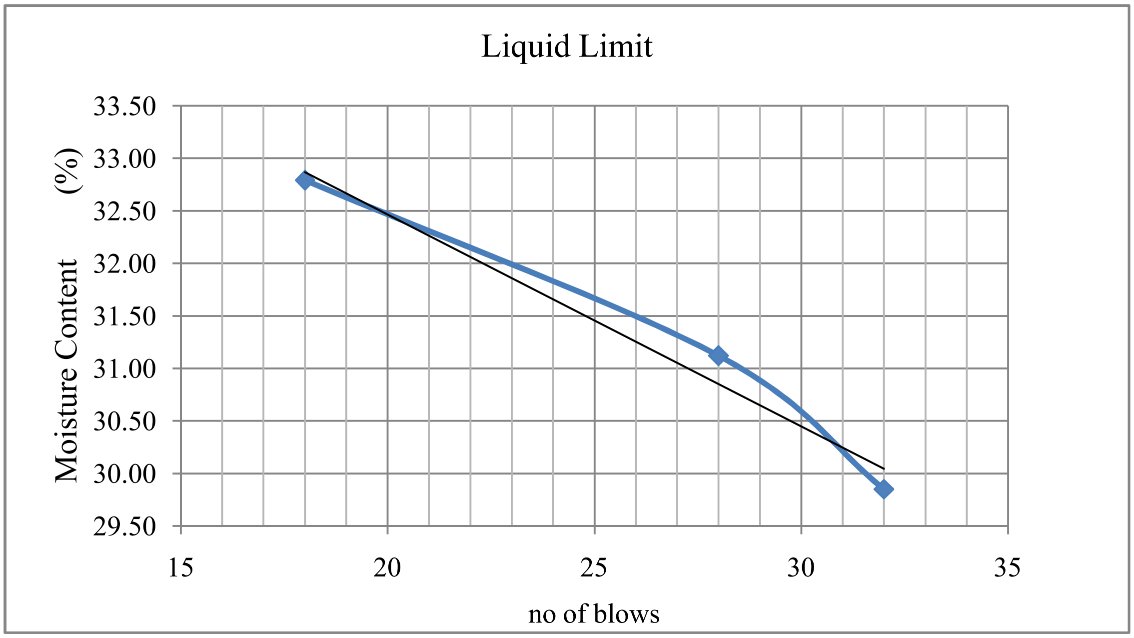



Figure 4.8 Liquid limit graph of weathered yellowish laterite

The liquid limit of the material is the moisture content at 25 no of blows. Results indicated 31.47%.

PI= LL-PL

Table 4.3 Plastic index of weathered yellowish material

| | |
|---|---|
| LIQUID LIMIT : (%) | 31.47 |
| PLASTIC LIMIT : (%) | 20.66 |
| PLASTICITY INDEX : (%) | 10.8 |

The material plastic index resulted to 10.8%. Therefore, the material is classified as medium plastic material. It is because the plasticity of a material index is found in between 10 – 20, according to AASHTO, 1993.

The weathered yellowish laterite has a good plasticity character. That it has a good bonding character. The plasticity of the weathered yellowish material is enough good to stick together. The cohesiveness of a material shows the ability to stick together. Soil, because of its fine-grained content will form a coherent mass at suitable moisture contents.

#### 4.1.2.4 Gradation of weathered yellowish laterite

Table 4.4 Weathered yellowish laterite gradation

| | Weathered yellowish | | | | | |
|---|---|---|---|---|---|---|
| | trial 1 | | | trial 2 | | |
| | Total weight before washing 8900.000 | | | total weight before washing 8852.0 | | |
| | total weight after washing 7251.00 | | | total weight after washing 6653 | | |
| sieve size | weight retained | % retained | % pass | weight retained | % retained | % pass |
| 63.000 | 0.000 | 0.000 | 100.000 | 0.000 | 0.000 | 100.00 |
| 37.500 | 0.000 | 0.000 | 100.000 | 0.000 | 0.000 | 100.00 |
| 20.000 | 266.000 | 2.989 | 97.011 | 116.000 | 1.310 | 98.690 |
| 4.750 | 2234.00 | 25.101 | 71.910 | 2000.00 | 22.594 | 76.096 |
| 2.000 | 1503.00 | 16.888 | 55.022 | 1365.00 | 15.420 | 60.676 |
| 0.425 | 1963.00 | 22.056 | 32.966 | 2103.00 | 23.757 | 36.918 |
| 0.075 | 1285.00 | 14.438 | 18.528 | 1069.00 | 12.076 | 24.842 |
| Total | 7251.00 | | | 6653.00 | | |

The weathered yellowish laterite is a very appreciable in its fines content character because the material is highly weathered. Gradation depends on the amount of weathering of the material. Highly weathered materials are composed of fine materials. From Table 4.4 the materials are composed more of sand fraction materials. Rather, the weathered yellowish material is composed of sand fraction materials. The sand fraction of a material composed of particles between the sizes of 4.75 mm and 0.075 mm. Sand fraction materials are essential if there is sufficient amount of coarse and medium gravel particles. The sand fractioned material uses by increasing the shear resistance of the grain material when transferring the load.

#### 4.1.2.5 Loss of weathered yellowish laterite

Figure.4.9 Preparation of weathered yellowish laterite for wet sieving

Table 4.5 Loss for weathered yellowish laterite

| | For trial 1 | For trial 2 |
|---|---|---|
| Total weight before washing (gm) | 8900 | 8850 |
| Total weight after washing (gm) | 7251 | 6653 |
| Total lost (gm) | 8900-7251 = 1649 | 8850- 6653 = 2197 |
| Total lost (%) | (1649/8900)*100 = 18.53% | (2197/8850) = 24.82% |

Table 4.15 the result showed that the weathered yellowish laterite has more percent of silt clayey fraction that passes the 0.075 mm sieve size. Increase in excess amount of friction is not very necessary, rather in a good composition is very necessary.

#### 4.1.2.6 Gradation of weathered yellowish against the specification

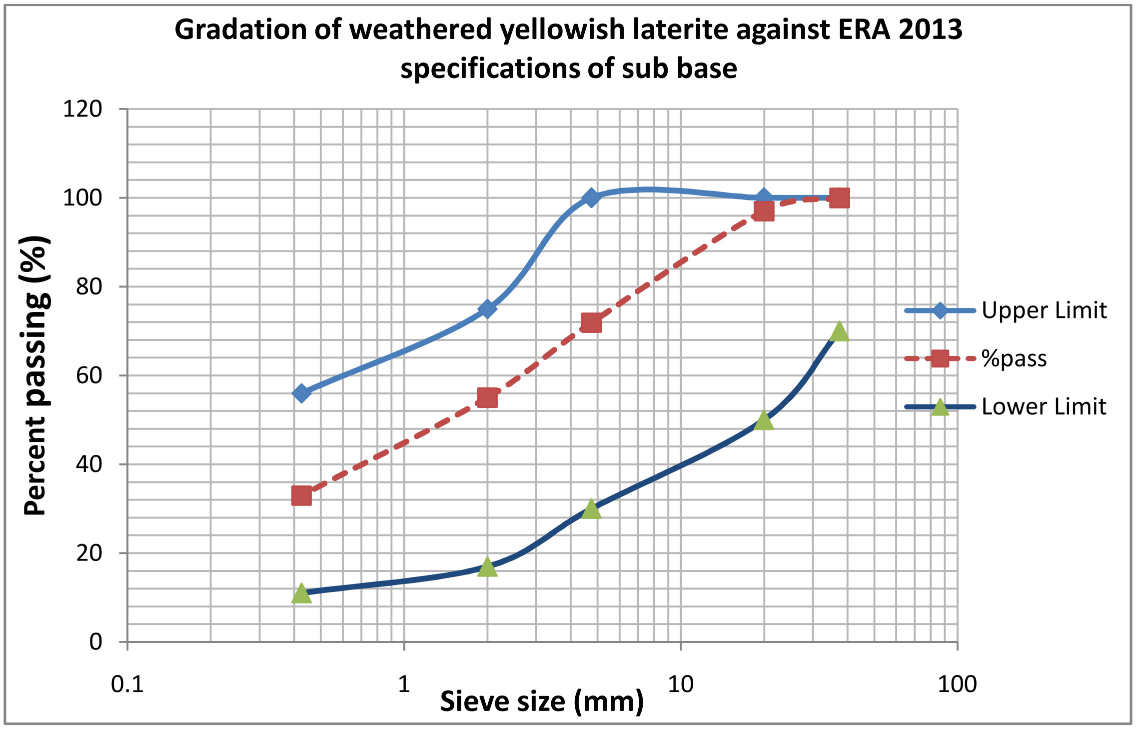



Figure 4.10 Gradation curve weathered yellowish material for trial 1

Based on the laboratory test results, the weathered yellowish laterite satisfied the gradation requirement for sub base limit showed by blue color in Figure 4.10. However, these materials meet the gradation but it has no or very minimum content of boulder (coarse gravel and medium gravel) particles to carry the load in wet condition and to transfer the load by grain to grain contact. That it cannot resist the load from the upper structure and cannot transfer load from one to another grain.

From the gradation curve, it indicated that the material has more fine that has effect on the performance of the structure. The sand fraction cannot transfer large load rather it displaced from the place and decrease the load carrying capacity of the structure. Increasing the % of gravel can improve the carrying capacity of the material.

Ethiopia road authority standard specifications 2013 did not recommend the using of weathered materials for the sub base construction in its natural condition, unless it is modified.

#### 4.1.2.7 Classification of weathered yellowish laterite

The material classification is based on AASHTO classification of soils and soil-aggregate mixtures. To classify material first evaluate the percentage passing of No. 200 sieve. That in both trials the ratio that pass no 200 sieve is

In trial 1 (1649/7398)*100 = 18.53% and in trial 2 (2197/8850) = 24.82%

Therefore, the material is generally classified as granular materials which 35% or less of total sample passing No. 200 sieve. And in group classification it has a plasticity index of greater than 10, in both case the percentage passing in No. 200 is less than 35 and the liquid limit of maximum 40 the material is classified as **A-2-6**. Thus the strength is low than cinder but by itself it has a good strength.

Therefore blending the cinder material with the weathered yellowish material is a good solution to improve the engineering behavior of the materials and to use them as the sub base course in road construction.

## 4.2 Stabilizing the cinder and weathered yellowish material

### 4.2.1 Proportioning the cinder and weathered yellowish material

As discussed in the literature there are 4 types of stabilization technique to change the character and increase the performance of the material. From these 4 techniques mechanical stabilization is used in this research because the problem of cinder material is gradation problem. Mechanical stabilization is best technique to change the particle size distribution of the material. Thus did by proportioning of the materials to be blended.

Proportioning the mixtures used to determine the satisfactory when tested in the laboratory. In proportioning the cinder and the weathered yellowish laterite the specification of gradation is used. The stabilization of the materials did by arithmetic or graphical method for mechanical stabilization. To give the chance for more investigation the graphical method is used. By considering the strength and classification characteristics of the cinder and weathered yellowish laterite the graphical proportioning gives different proportioning levels. Therefore, for further investigation it is the best suitable method.

**Proportioning by graphical method**

In graphical method the cinder is plotted at right and labeled as material B where as weathered yellowish laterite material is plotted at left labeled as material A. For both materials the percentage passing of the material is plotted at left and right for weathered yellowish laterite and cinder material, respectively.

For lower limit or the right limit, it was combined by considering the percentage pass of No. 40 or 0.425 mm sieve. Its specification is 11- 56, but evaluation is done with the lowest value that is 11.

Table 4.6 Trial combination for lowest value

| Trial combination | % pass of Cinder in 0.425mm sieve. | % pass of weathered yellowish laterite in 0.425mm sieve. | 72 %of cinder and 28 % of weathered yellowish laterite | 73%of cinder and 27 % of weathered yellowish laterite | 74%of cinder and 26 % of weathered yellowish laterite |
|---|---|---|---|---|---|
| 1x1 | 3.149 | 32.966 | 11.498 | 11.200 | 10.901 |
| 1x2 | 3.149 | 36.918 | 12.604 | 12.267 | 11.929 |
| 2x2 | 3.581 | 36.918 | 12.915 | 12.582 | 12.249 |
| 2x1 | 3.581 | 32.966 | 11.809 | 11.515 | 11.221 |

From the Table 4.6 the proportion that % of pass satisfies the given minimum specification range is 73% of cinder and 27% of weathered yellowish laterite. If the proportion of cinder is 72% that means it is going decrease and the percentage of weathered yellowish is 28% that means it is going increase, the stabilization going satisfies the specification that give the value greater than 11. If the proportion of cinder is 74% that means it is going increase and the percentage of weathered yellowish is 26% that means it is going decrease, the stabilization cannot satisfies the specification that give the value less than 11.

For upper limit or to the left side limit percent passing in No. 200 or 0.075 mm sieve is recommended and the limit in the specification is that 5-25 percentage of passing, so it was used a value of 25% because it the upper value.

Table 4.7 Maximum proportion

| Trial combination | Proportion of the cinder material and weathered yellowish that pass 0.075 mm sieve in trial combination | Highest value for No. 200 sieve or 0.075 mm. |
|---|---|---|
| 1x1 | 1.582*0 + 1*18.528 | 18.528 |
| 1x2 | 1.582*0 + 1*24.842 | 24.84 |
| 2x2 | 1.824*0 + 1*24.842 | 24.84 |
| 2x1 | 1.824*0 + 1*18.528 | 18.52 |

In this case the upper limit is 0 for cinder and 100 for the weathered yellowish material. Thus From combination of the four trials for the lowest minimum value the combination of trial 1x1 governs. Thus, the cinder material composed of 73% of and 27% of weathered yellowish laterite.

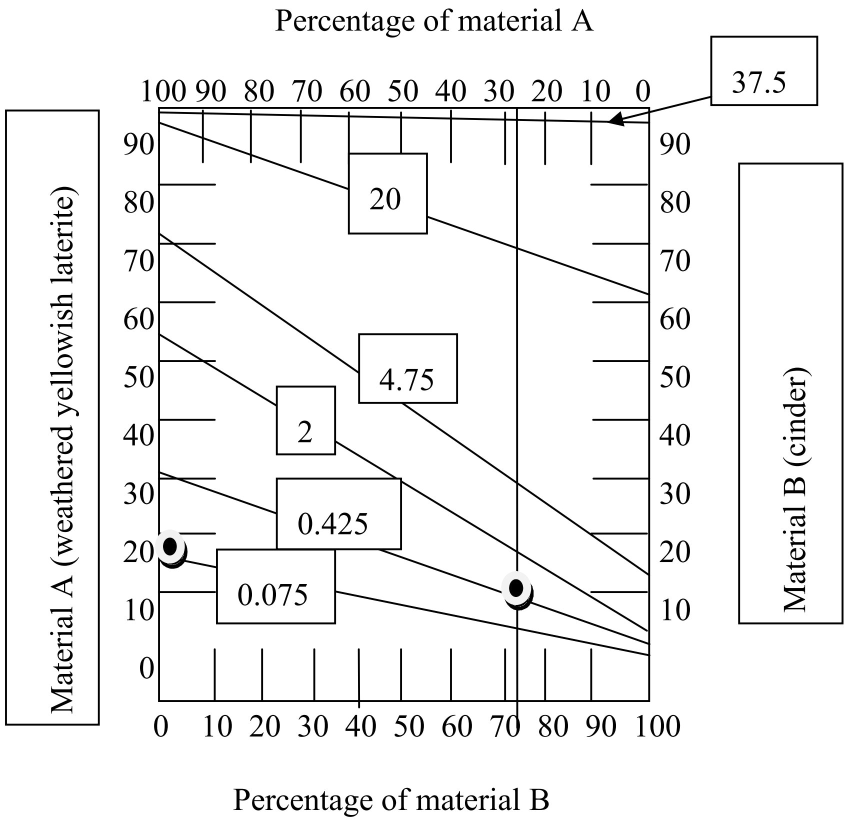



Figure 4.11 Graphical method proportioning

Therefore, to practical case 70% of cinder and 30% of weathered yellowish material are used as optimum in this research.

If by adding a certain percent of other materials, let's say < 20% is said to be modifying, but if > 20% it is said to be mixing of a material (ERA, 2013).

**4.2.2 Studying the engineering properties of the stabilized materials.**

**4.2.2.1 General**

Studying the engineering properties of the stabilized cinder and weathered yellowish laterite incorporates the potential use of the material in the road construction. The stabilized materials showed good engineering properties from their unmixed condition. The materials are mixed by weight that means weight cinder is 70 % and 30% of the weight is from weathered yellowish laterite. If 100 kg material is needed 70 kg of the material is cinder that of weathered yellowish laterite is 30 kg.

Figure 4.12 Mixing cinder material with weathered yellowish material

#### 4.2.2.2 Evaluating for sub base

**Gradation**

Characterization of the stabilized material based on particle size is the first step to check the material for the road construction. That it can be checked for other tests if the gradation of material meets.

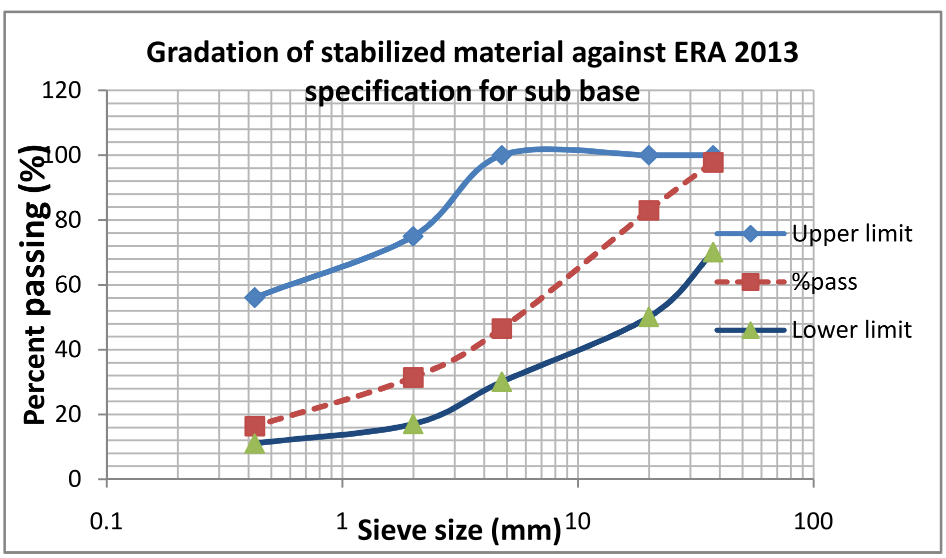



Figure 4.13 Gradation curve for the stabilized curve

The gradation requirement is full filed for sub base when the cinder mixed with weathered yellowish laterite. The gradation of the stabilized material is now best fit the gradation curve.

**Evaluating the curve is well graded or gap graded**

A curve with a hump represents the soil in which some of the intermediate size particles are missing. Such as is called gap – grading or skid – grading.

A flat S – curve represents a soil which contains the particles of different size in good proportion. Such a soil is called a well – graded (uniformly graded) soil (Arora, 2004).

According to Arora the uniformity of a soil is expressed qualitatively by a term known as uniformity coefficient. That it is the material uniformity percent.

$C_U$ given by

Uniformity coefficient $Cu = \frac{D60}{D10} \geq 6$……………………………………….Equation 2

Another parameter to describe the particle size distribution curve is Coefficient of curvature ($C_C$) and it is given by the formula

Coefficient curvature $Cc = \frac{(D30^2)}{D10*D30}$ .. $1\leq Cc\geq 3$ ……………………………Equation 3

Where

$D_{60}$ is the diameter of the soil particles for which 60% of the particles are finer,

$D_{10}$ is the diameter of the soil particles for which 10% of the particles are finer and

$D_{30}$ is the diameter of the soil particles for which 30% of the particles are finer

The 60% finer found in between 20 and 4.75 sieve size. 30% finer found in between 2 and 4.75 mm and 10% finer found in between 0.425mmand 0.075mm sieve sizes. By interpolation

For well graded gravel content > sand content $Cu \geq 4$; $1\leq Cc\geq 3$

Sand content > gravel content $Cu \geq 6$; $1\leq Cc\geq 3$

$D_{60}$ = 10.404,  $D_{30}$ 1.857 =  $D_{10}$ = 0.121,

Cu = $\frac{D60}{D10}$ = $\frac{10.404}{0.121}$ = 85.378  this shows uniformity of material is 85%

Cc = $\frac{(D30^2)}{D10*D30}$ = $\frac{(1.8574^2)}{0.121*1.8574}$ = 2.721 the gradation curve is very good.

Cu ≥ 6 and 1 ≤ Cc≥ 3 the material is well-graded and the sand content was greater than gravel content.

**Plasticity character**

The plasticity character of the stabilized cinder and weathered yellowish laterite shows deformation characteristics of the material. That it is the range of water content for which a soil will behave as a plastic material (deformation without cracking).

PI = LL – PL, but PL = cannot be determined the material is non plastic material.

**Moisture Density relationship**

Compaction may be defined as a process of increasing the soil unit weight by forcing the soil solids in to a denser state, reducing the air voids. It is accomplished by static or dynamic loads. Many types of earth construction such as dams, embankment, and highway and airport runways require soil fill which is placed in layers and compacted. A well compacted soil is mechanically more stable, has a high compressive strength and high resistance deformation than a loose soil (Molenaar, 2005).

Compaction is a process by means of which the soil can be densified. In soils there are some amount of air and water besides solid grains. Theoretically the density of soil can be increased by reducing the space occupied by the air and by elastic compression of soil grains. It means compaction takes place due to expulsion of air from the voids of the soil mass by applying any mechanical means

The purpose of the laboratory test is to determine the proper amount of moulding water to be added when compacting the soil in the field and the degree of compaction comparable to that obtained by the method used in the field.

It was done by standard and modified compaction test. In this research study, it was used modified proctor compaction test.

The optimum moisture content is the amount of moisture content at which specified amount of compaction will produce the maximum dry density. Dry density is the mass of the dry material after drying to content mass at 105 degrees Celsius.

Table 4.8 'Modified Proctors' compaction tests

| Type of test | Hammer mass (Kg) | Hammer drop (m) | Blows /layer | Number of layers | Compaction energy (Kg/cm$^3$) |
|---|---|---|---|---|---|
| Modified proctor | 4.5 | 0.45 | 56 | 5 | 2700 |

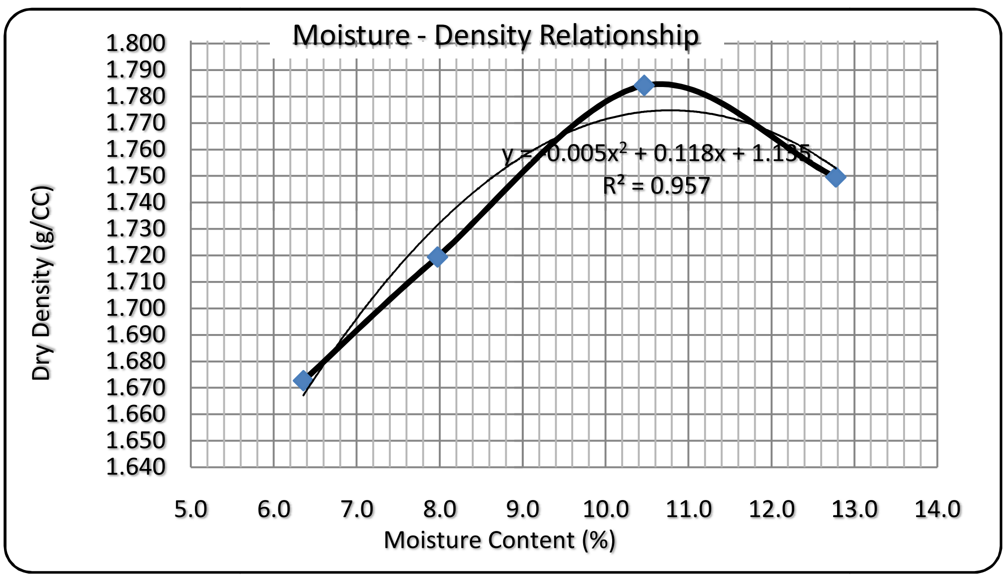



Figure 4.14 Maximum dry density and optimum moisture content graph

From Figure 4.1 equation the optimum moisture content is 10.7% and the maximum dry density is 1.786 %. It means the stabilized cinder material get its maximum dry density when the added amount of water is 10.7 % of the material weight. If it is used 10kg sample of stabilized cinder, the water content to be used is that 1070 mill-liters.

**California Bearing Ratio (CBR)**

California bearing ratio is the ratio of force per unit area required to penetrate into a soil mass with a circular plunger of 50mm diameter at the rate of 1.25mm/min.

The laboratory CBR test is generally carried out on remolded samples. The sample should be compacted to the expected field dry density at the appropriate water content.

Types of CBR are one point CBR and three point CBR

Their difference is in the number of molds and layers. If the process is within one point CBR the value is expected 100%and required one mold and 56 blows 5 layers, within three points CBR

Value is expected greater than 95% - 100% .and required three molds, 10 blows 30 blows and 65 blows

Performing CBR value has three steps -CBR compaction

-CBR socking to know percentage of swelling

-CBR penetration

The percent retain in number 20 sieve is 19.5 % that is less than 25 % the standard molds and standard plunger area used in the test.

The standard molds are within an internal diameter of 152.4± 0.66 mm and a height of 177.8 ± 0.4 mm with a colar height of 51 mm.

CBR value $= \frac{\text{load at ( 2.54 mm or 5.08 mm penetration)}}{standard\ load} * 100$ ……………………………equation 4

$= \frac{\text{Test load}}{standard\ load} * 100$

Table 4.9 Standard load /pressure

| Penetration (mm) | standard | |
|---|---|---|
| | Load (kg) | Pressure (MPa) |
| 2.54 | 1370 | 6.9 |
| 5.08 | 2055 | 10.3 |

The stress of the test is derived from the dial reading. The dial reading is a reading from the CBR instrument. The CBR instrument reading was changed to the load by a ring factor of 43.1152N/D that a force required to cause a circular piston that has an area or cross-sectional of 1935 $mm^2$ to penetrate the soil from the surface at a constant rate of 1.25 mm/min. the results are at appendix C.

Stress = 43.1152 /D * Dial reading ……………………………………………...equation 4

And D = 1935 $mm^2$

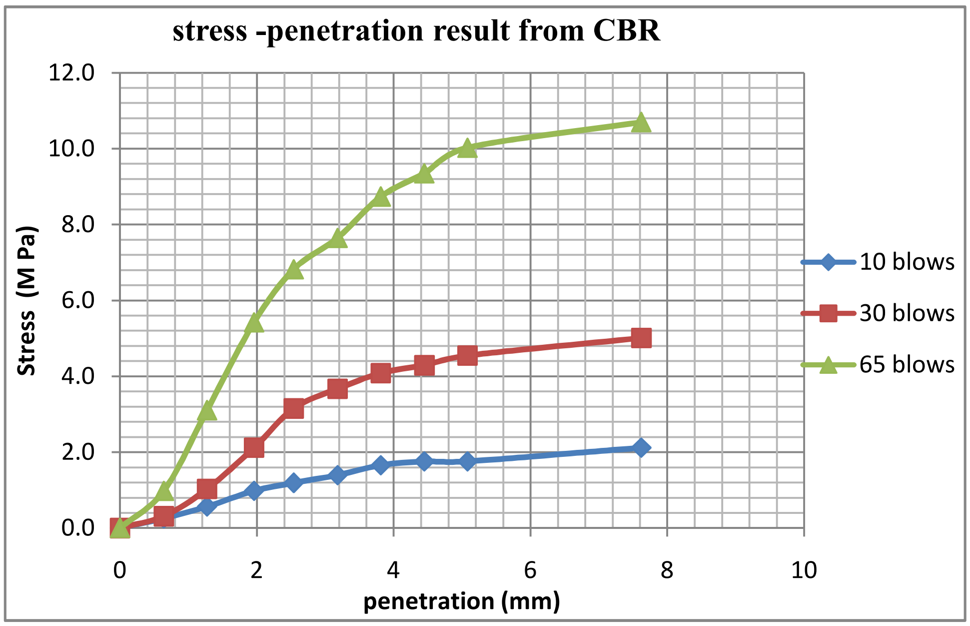



Figure 4.15 Stress - Penetration graph

Figure 4.15 shows highly compacted materials are penetrated by a large force. Thus the same level of penetration let us say 4 mm penetration is reached at different stress level for the three compaction level.  If the material is compacted well or that means if it is hard it cannot penetrated simply with a less amount of load. To get a maximum penetration from highly compacted material the stress level should be high. It means that the densely compacted material can resist a large amount of stress.

Table 4.10 Dry density and moisture content

| Blows | Before Soaking | | After Soaking | |
|---|---|---|---|---|
| | DD (g/cc) | Moisture (%) | DD (g/cc) | Moisture (%) |
| 10 | 1.497 | 10.4 | 1.64 | 11.1 |
| 30 | 1.661 | 10.8 | 1.69 | 12.5 |
| 65 | 1.821 | 10.6 | 1.85 | 13.2 |

Soaking the sample in water uses to estimate the worst case that happens in road structure at the field. If the material is not designed to the worst condition the material cannot resist the situations happened on the site. To consider the worst case the sample was soaked for 4 days.

According to ERA manual, for the sub-base material the minimum soaked California Bearing Ratio (CBR) shall be 30% when determined in accordance with the requirements of AASHTO T-193. The Californian Bearing Ratio (CBR) was determined at a density of 95% of the maximum dry density that AASHTO gives 95% to 100% when determined in accordance with the requirements of AASHTO T-180 method D.

So that, the 95% of maximum dry density of the sample founded by multiplying the maximum dry density that is 1.786. It means a compaction attained in filled is 95% of the relative maximum dry density in laboratory.

95% of MDD = 0.95 * 1.786

95% of MDD = 1.7

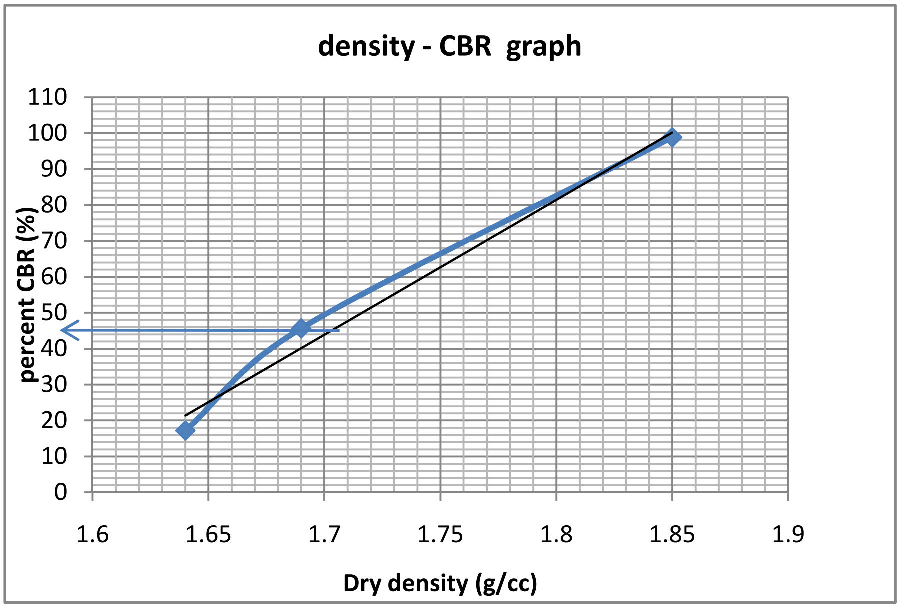



Figure 4.16 Density – CBR graph

From the Figure 4.16 the CBR value at 95% dry density or 1.7g/cc is the CBR value is 44%. Therefore the soaked CBR value is 44 % that used to design purpose. It is greater than 30 % that safe for sub base.

**SWELLING**

Swelling is the process opposites to consolidation. That means it is expansion of a soil on reduction of pressure due to water being drawn into the voids between particles.

$$\% \text{ swell} = \frac{\text{Reading after soaking} - \text{Reading before soaking}}{\text{Height of specimen}} * 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{equation 5}$$

Height of specimen is considered as 4.584 in given AASHTO T- 193

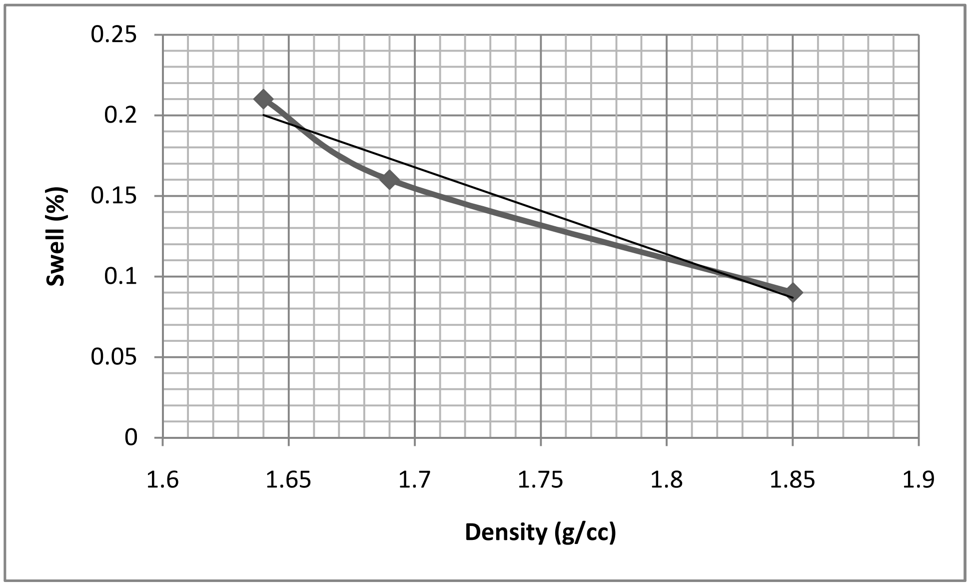



Figure 4.17 Density – swell curve

From the graph the swelling at 95% maximum dry density that is 1.7 g/cc is 0.16%. That means the stabilized material swells at 0.16% when harsh case encountered. The volume of the stabilized material increases by 0.16%. That is less than 0.2% that is safe based ERA standard.

**Flakiness index**

The flaky particles are undesirable because they cause inherent weakness with possibilities of breaking down under heavy loads. The flakiness index of an aggregate sample is found by separating the flaky particles and expressing their mass as a percentage of the mass of the sample, the test is applicable to material passing a 63 mm sieve and retained on a 6.3 mm sieve**.**

Table 4.11 Flakiness index calculation

| Sieve analysis | | Gauging | |
|---|---|---|---|
| Sieve size (mm) | Wt. ret | Gauge range | Wt. of sample passing the gauge |
| 50 | | 63-50 | |
| 37.5 | | 50-37.5 | |
| 28 | 433.6 | 37.5-28 | 82.3 |
| 20 | 952 | 28-20 | 303.2 |
| 14 | 1483.9 | 20-14 | 472.2 |
| 10 | 622 | 14-10 | 10.3 |
| 6.3 | 261 | 10-6.3 | 120.5 |
| Weight after discarding 5% or less ,M2 | 3752.5 | Combined wt, M3 | 988.5 |

$FI = (M_3/M_2) \times 100$……………………………………………………………..Equation 6

Where: FI = Flakiness Index

$M_2$= Total mass of all fractions retained on each sieve

$M_3$= Total mass of particles passing the slots

F.I = (988.5/3752.5)*100 = 26.34% < 35% given by ERA standard specification.

The flakiness index of the material is 26.34% that means the material is not flaky for the sub base. It means that the breaking down of the material when the load is applied does not significantly damage the road structure.

**Linear shrinkage**

The linear shrinkage test offers a convenient method to confirm that the test results for the plasticity index are reasonable. Most types of soil exhibit a relationship between the plasticity index and the linear shrinkage of the material. The linear shrinkage is considered a more reliable indicator than the plasticity index for materials with very low plasticity (Eshete, 2011).

$$LS = (1 - \frac{LD}{Lo}) * 100 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{equation 7}$$

Where, $L_D$ is the length of the oven-dry specimen (mm)

Lo is the original length of the specimen (mm)

$L_D$ = 138.7 mm, Lo = 140 mm, Ls = (1-138.7/140) * 100 = 1.94 % < 6 it is suitable because it is in the recommended range by ERA 2013 specification.

The stabilized cinder material with weathered yellowish laterite satisfied the density and strength standard specifications of ERA, 2013. Therefore, the stabilized cinder can be used as sub base material for the road structure.

## 4.3 Effect of compaction on resilient deformation and permanent deformation characteristic

### 4.3.1 General

The mechanical behavior of the materials shows the response of the materials when subjected to the traffic loading. The mechanical value is the best used to select the material in its performance.

As the load is applied the stress increases also the strain increase. When the stress is reduced, the strain also reduces but all the strain is not recovered after the stress is removed. The total strain consists the recoverable/elastic/ or resilient and non-recoverable or permanent. Repeated load CBR is used to determine the resilient deformation and permanent deformation of the material.

The resilient modulus is a fundamental property that is similar in a concept to the modulus of elasticity. Resilient modulus $= \frac{stress}{strain}$, stress = loads/area of the specimen and strain = recoverable deformation/original height. However, this result is known from the tri–axial test. As it discussed in literature part using the repeated load CBR test we can assess the deformation formed in the pavement structure.

In this research, the laboratory test is carried out by the result of stabilized material of 70% cinder with 30% of weathered yellowish laterite content the repeated load CBR.

The repeated load CBR was done by three compaction, those are 10 blows, 30 blows and 65 blows. For all cases the un-loaded case used stress is 0.1 Mpa that is to simulate the over laying thus it shows the condition of layer in field. The test is unloaded for about 9 seconds. The test is loaded for about 1 second (Nagula et al., 2017).

To simulate the loading condition in the field, it was used to that the stresses in 2.54 mm. The stress of the 10 blows, test is 1.19 Mpa, 3.15 Mpa for 30 blows and 6.82 Mpa for 65 blows.

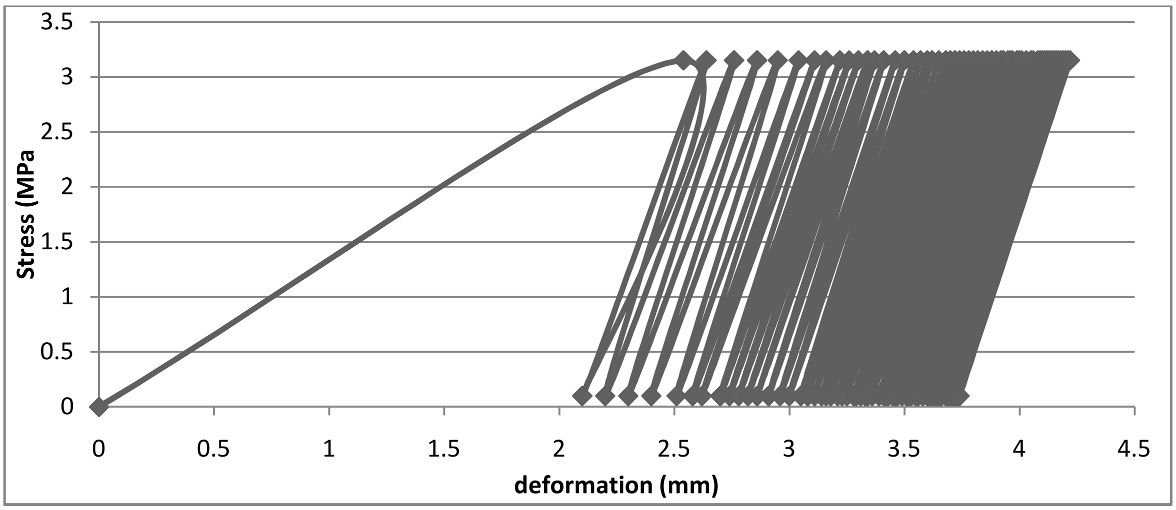



Figure 4.18 Laboratory test result of no 30 blows

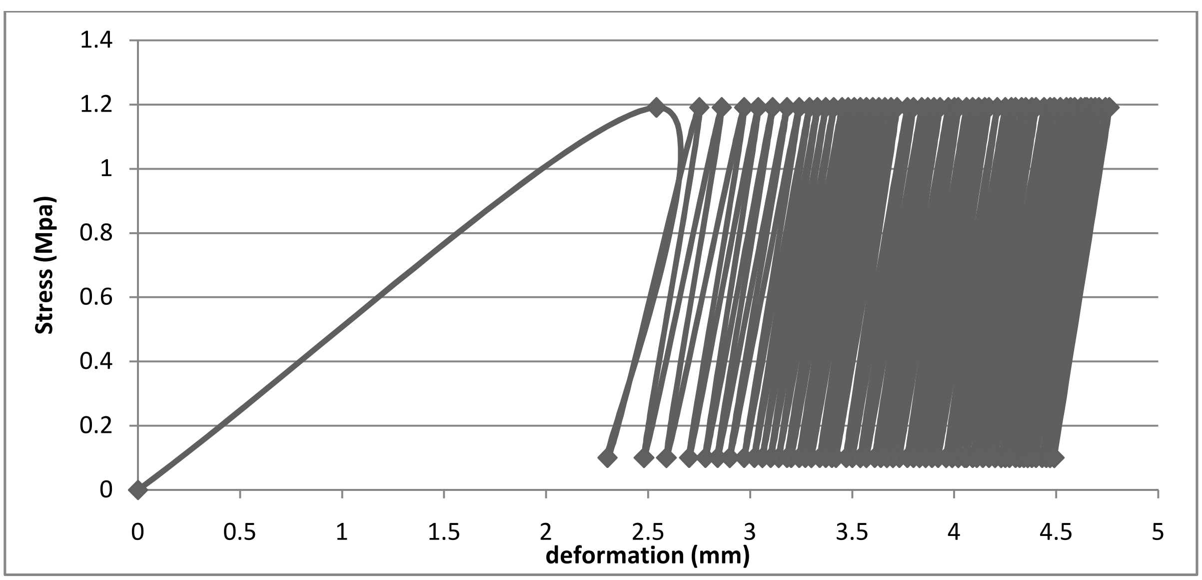



Figure 4.19 Repeated load test result of 10 blows.

Ms- Excel results Figure 4.18 and 4.19 which illustrates the load – deformation relation for both the initial loading, i.e. standard CBR loading, and the repeated load cycles. It also shows the hysteresis loop between the loading and unloading curves and how the permanent deformation per cycle decreases as the number of loading cycles progresses. This figure demonstrates the fundamental stress – strain behavior of unbound granular materials under repeated loading (Araya et al., 2011). Thus, the repeated load CBR graph shows loading and unloading. That used to predict the resilient deformation and permanent deformation. That the deformation or the penetration going over one on another repeatedly.

### 4.3.2 Permanent deformation

The permanent deformation is used to characterize the pavement structure in mechanistic approach. The permanent deformation is a deformation created as permanent and non-recoverable to the original place. The total permanent deformation is calculated for each cycle is the difference between deformation on unloading and the original deformation or 0. Thus, the permanent deformation is non-recoverable.

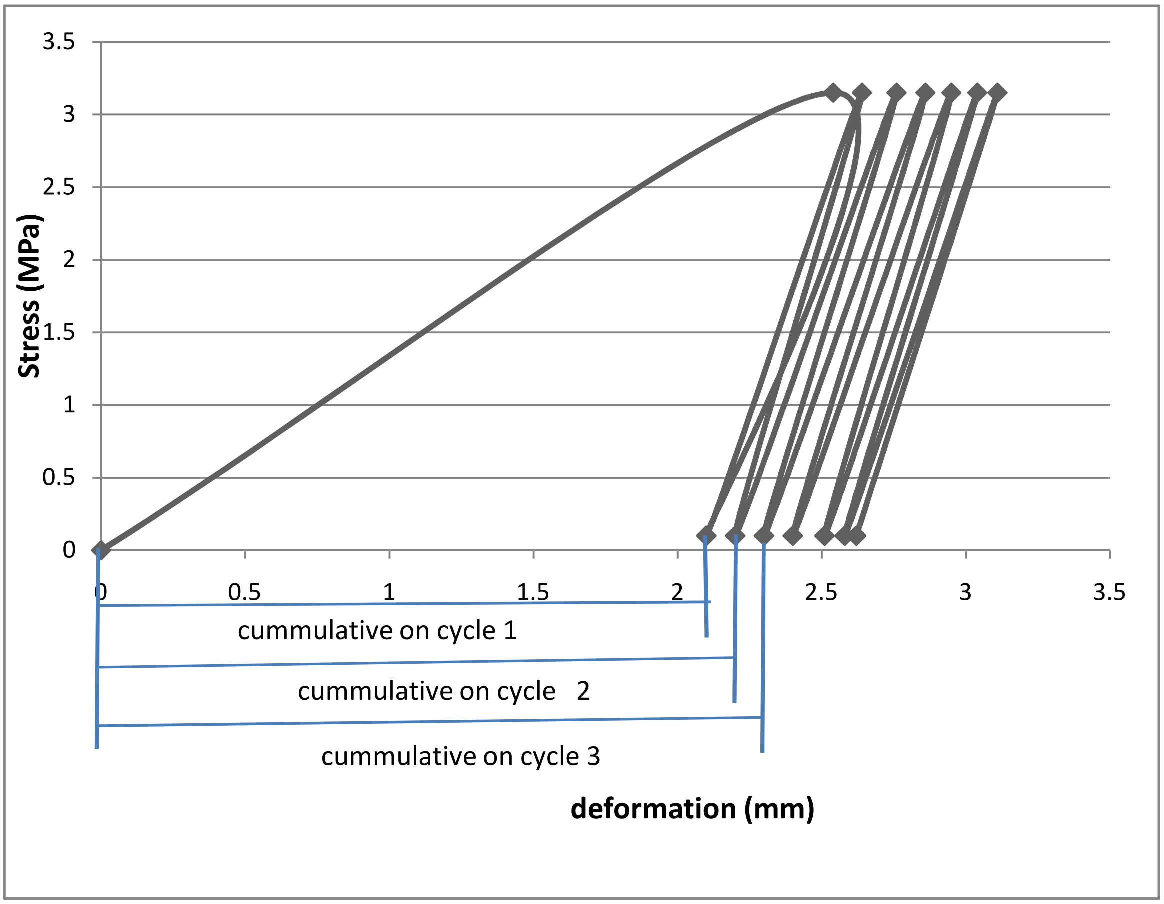



Figure 4.20 Part from the 30 blows for permanent deformation on different cycles

Figure 4.20 show that the permanent deformation is a cumulative deformation. That comes from when the application of load increases the material non recoverable deformation also going to increase. It means that the loads pass through the road causes a portion of non reversible deformation which can added in each load repetition.

The permanent deformation on cycle 1 is 2.1-0 = 2.1

Table 4.12 Permanent deformation from the repeated load CBR

| Penetration (mm) | Load (Mpa) | no of load repetition | Permanent deformation (mm) |
|---|---|---|---|
| 0 | 0 | 0 | 0 |

| | | | |
|---|---|---|---|
| 2.54 | 3.15 | 1 | 2.1 - 0 = 2.1 |
| 2.1 | 0.1 | 2 | 2.2 - 0 = 2.2 |
| 2.64 | 3.15 | 3 | 2.3 – 0 = 2.3 |
| 2.2 | 0.1 | 4 | 2.4 – 0 = 2.4 |
| 2.76 | 3.15 | 5 | 2.51 – 0 = 2.51 |
| 2.3 | 0.1 | 6 | 2.58 – 0 = 2.58 |
| 2.86 | 3.15 | | |
| 2.4 | 0.1 | | |
| 2.95 | 3.15 | | |
| 2.51 | 0.1 | | |
| 3.04 | 3.15 | | |
| 2.58 | 0.1 | | |

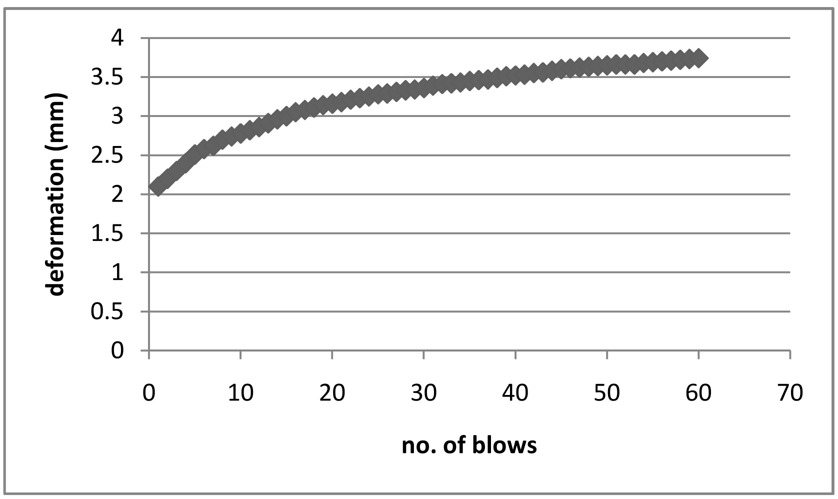



Figure 4.21 Permanent deformation of number 10 blows

Figure 4.21 shows that the load increases when the pavement structure opened for service, and traffic loading. So that, the loading increase within the repetition of loading and the permanent deformation going far from the original place or level. That means the pavement is deformed to unrecoverable depth when the load repetition is going to increase. The traffic passed on the pavement structure deforms the structure for the permanent and the deformation on the pavement structure increases rapidly at initial steps, whereas going increase slowly after a number of load repetition.

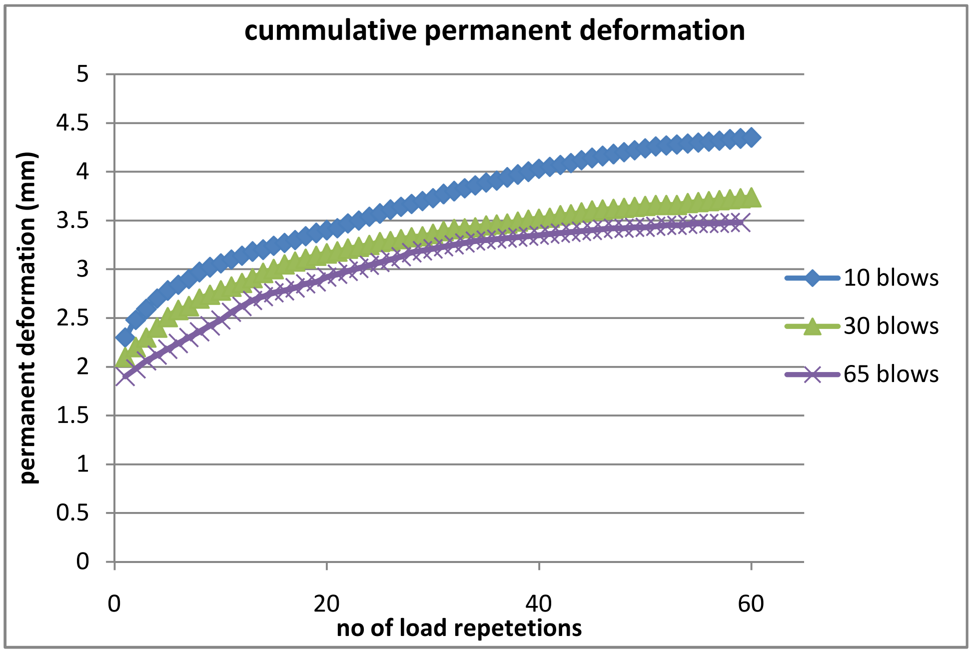



Figure 4.22 Permanent deformations for 10 blows, 30 blows and 65 blows

The loads are applied from the values read from the standard CBR. Each blow in the repeated load CBR is subjected to the corresponding load from the standard CBR. That means the 10 blows is subjected to 0.1 MPa in unloading case and repeated load of 1.19 MPa loading case, the 30 blows is subjected to 0.1 MPa in unloading case and repeated load of 3.15 MPa loading case and the 65 blows is subjected to 0.1 MPa in unloading case and repeated load of 6.82 MPa loading case. The loading is for 1 seconds, and unloading for 9 seconds.

However, the specimen of each blow is subjected to their respective loading from the standard CBR, the permanent deformation is different. The Figure 4.22 shows that the compaction of material affects the response of the material to the applied cyclic load. However, the same instruments are used to measure, it shows different permanent deformation.

Araya (2011) discusses the factors that affect the deformation. The material with low compaction has different dry density, moisture content, and number of blows that affect the deformation.

The graph showed that the permanent deformation is affected by the number of blow or the compaction level. Low compacted materials have high permanent deformation, which means their non-recoverable deformation is high.

### 4.3.3 Resilient deformation

The traffic loading is not a static load that it is movable load, so that it deforms the pavement structure and the deformation may be recoverable or non-recoverable. Elastic deformation is recoverable deformation that the deformation is again going to the point. The material can get its original shape after the load removed.

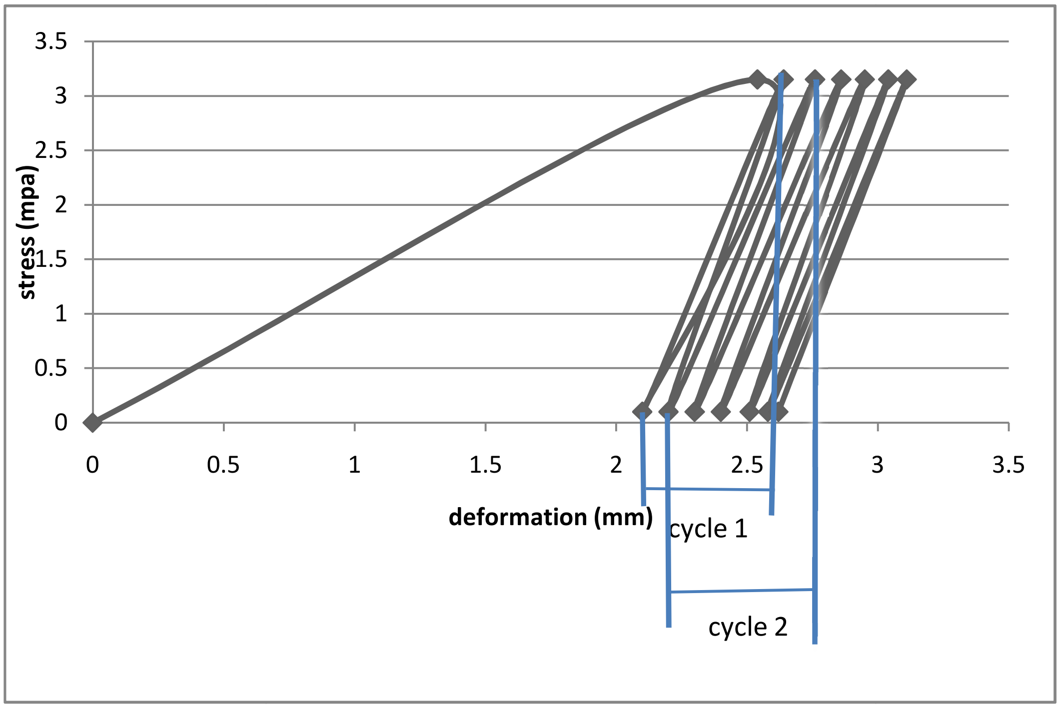



Figure 4.23 Part from the 30 blows for resilient strain

Moving traffic passes by creating a deformation on the pavement structure. The created deformation may be recoverable or non-recoverable. Recoverable deformation is elastic deformation. The elastic deformation is a deformation that can be recoverable after the structure is unloaded. The elastic deformation is the difference of the deformation at loaded case and the deformation at unloaded.

Elastic deformation for cycle 1 is 2.64 – 2.1 = 0.54 mm.

Table 4.13 Resilient deformation from repeated load CBR

| Penetration (mm) | Load (Mpa) | no of load | elastic deformation (mm) |
|---|---|---|---|
| 0 | 0 | 1 | 2.64 – 2.1 = 0.54 |
| 2.54 | 3.15 | 2 | 2.76 – 2.2 = 0.56 |
| 2.1 | 0.1 | 3 | 2.86 – 2.3 = 0.56 |
| 2.64 | 3.15 | 4 | 2.95 – 2.4 = 0.55 |
| 2.2 | 0.1 | 5 | 3.04 – 2.51 = 0.53 |
| 2.76 | 3.15 | 6 | 3.11 – 2.58 = 0.53 |
| 2.3 | 0.1 | | |
| 2.86 | 3.15 | | |
| 2.4 | 0.1 | | |
| 2.95 | 3.15 | | |
| 2.51 | 0.1 | | |
| 3.04 | 3.15 | | |
| 2.58 | 0.1 | | |
| 3.11 | 3.15 | | |

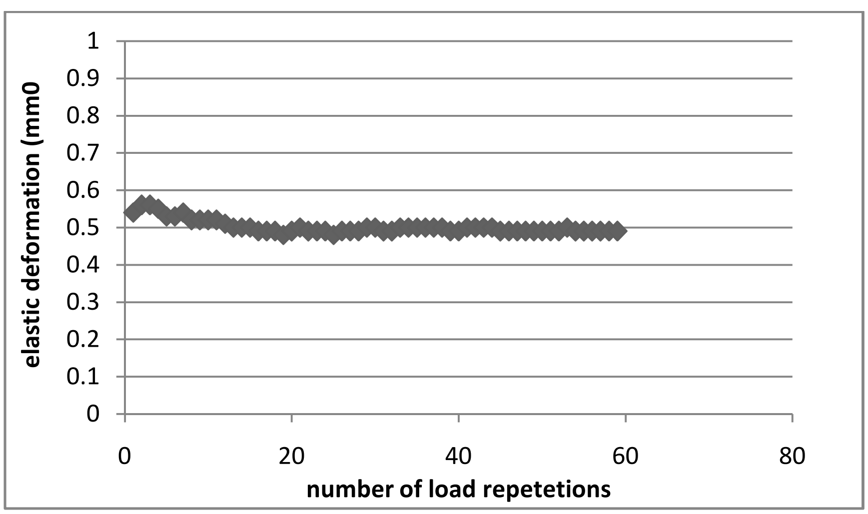



Figure 4.24 Elastic deformation for 30 blows

Elastic deformation going to decrease at the initial loading and it is constant after a number of load repetitions. The elastic deformation going to constant means that, the material recovers with the constant recoverable power. The material deforms to transfer the load and then recover with a constant deformation. That doesn't mean that the total deformation is constant, because the deformation is cumulative effect of recoverable and non-recoverable deformation.

The elastic deformation for the three blows is given in the Figure 4.25.

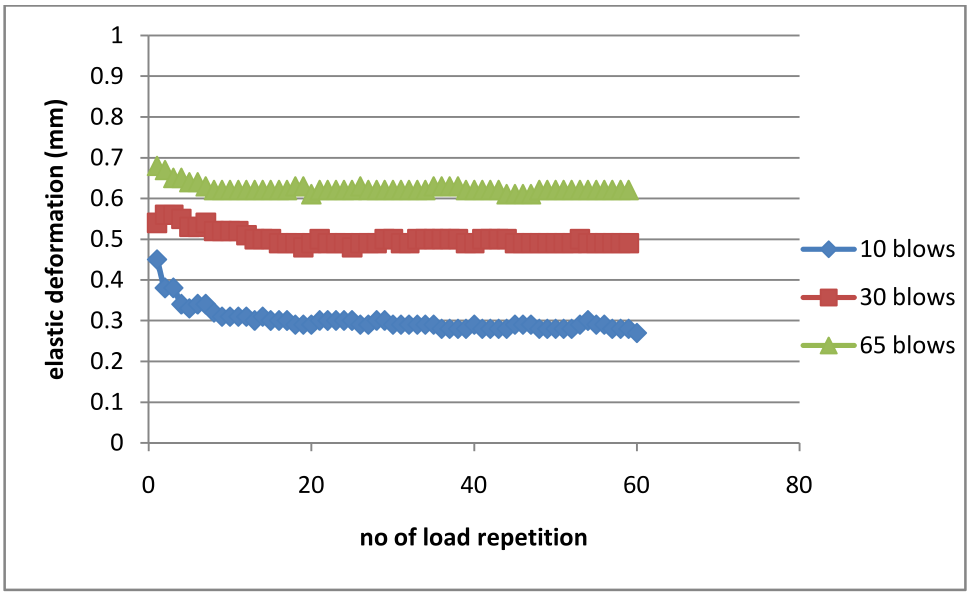



Figure 4.25 Resilient deformation of10 blows, 30 blows and 65 blows

From the graph 4.25 the elastic deformation of 10 blows, 30 blows and 65 blows showed the elastic deformation decreases at initial loading situations and going to constant when the repetition of load increases. But the compaction of a material affects the elastic modulus that the 10 blow has low elastic modulus and the 30 blows has a greater than 10 blows and 65 blows has greater value than 30 blows.

The Figure 4.25 shows that the material can undergo higher recoverable deformation if the material is compacted well. The material with high compaction has high recoverable deformation. Highly compacted material can recover highly when they are unloaded. That is because of the material cannot accumulate stress inside it.

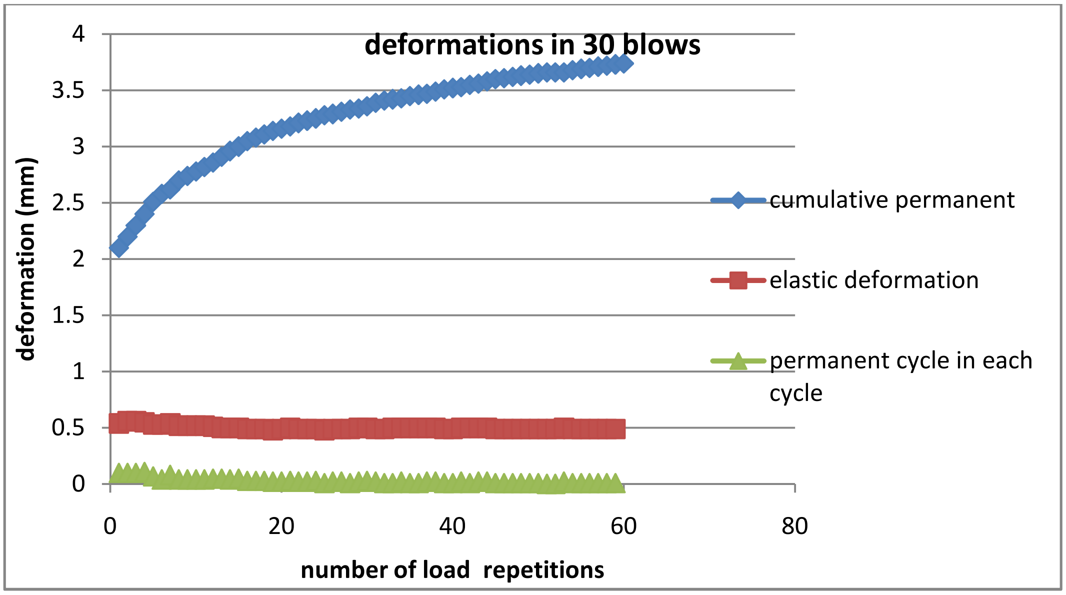



Figure 4.26 Deformations in 30 blows

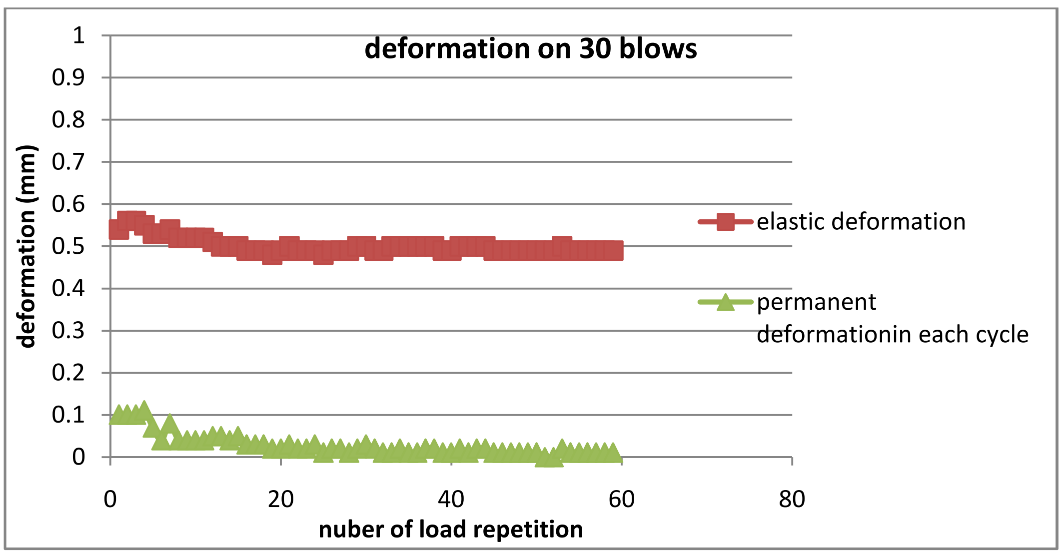



Figure 4.27 Permanent and elastic deformation in 30 blow sample

Figure 4.26 and 4.27 Shows the permanent deformation formed in a material at each cycle is less than that the elastic deformation formed at each cycle, except the first cycle. It means the material elastic responsibility is greater than that of permanently deformed in each cycle. But cumulatively, the cumulative permanent deformation of the material is larger than that of elastic deformation. It is because of the permanent deformation is accumulated in each cycle whereas; the elastic deformation cannot be cumulated.

### 4.3.4 Summary and discussion of the repeated load CBR results

The result of repeated load CBR shows that compaction of material affects the deformation characteristics of the material. That means the compaction has different effect on permanent and elastic deformation. The repeated load CBR results shows that the permanent deformation of the material increases highly in the first few repeated load, and after a number of loading is applied it increases gradually. The elastic deformation of the material decreases suddenly at the first few loading and going constant for the next repeated loading.

From the Figure 4.25 it can be understood that highly compacted materials have low permanent deformation and low elastic deformation characteristic than lightly compacted materials. The lightly compacted materials have high permanent deformation and low elastic deformation than highly compacted materials.

The high value of permanent deformation of lightly compacted material shows that a material undergoes compaction at the time of loading of repeated load CBR.

According to Jan Englund, the permanent deformation and elastically deformation are formed because of the contact between the grains in the structure. When the material is subjected to external stress it will be deformed both elastically and permanently. The elastic deformation is caused mainly by deformation in the contact between the particles, the increase in the contact area of the grain and which can back to its place, as described in Figure 4.28.


Figure 4.28 Elastic increase in the contact area between the particles due to compaction *source : (Jan, 2011).*

The contact area of each contact is deformed and the area between the contacts is increased. During elastic compression more contacts are created, and grains back to the original place after the load is removed, as described in Figure 4.29.

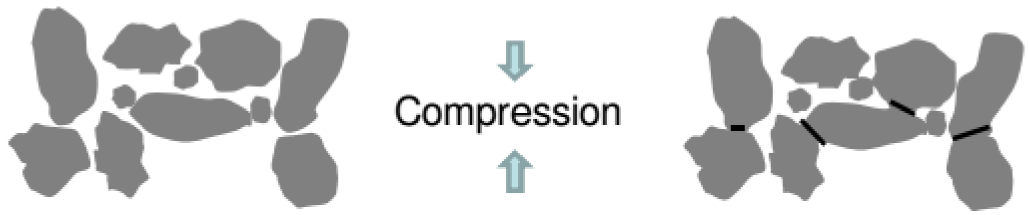



Figure 4.29 More contacts between particles are created elastically due to compression of the unbound granular material represented by the black lines *source: (Jan, 2011)*.

The increased contact area and the creation of new contacts between the particles make it more difficult to deform the unbound material elastically the higher the stress level. The response is, thus, stress hardening and that creates permanent deformation. The deformed material cannot retain to its original shape.

Repeated loading cause permanent deformation, in turn caused by crushing and sliding in the contacts between the grains, and the grains start to move and rotate as shown in Figure 4.30.

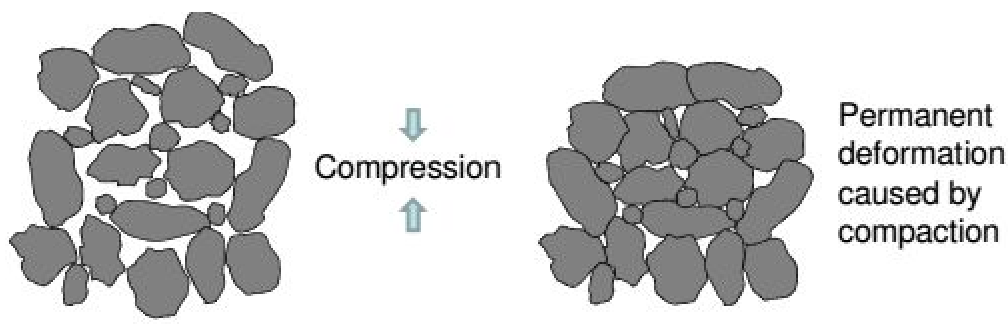



Figure 4.30 Permanent deformation cause crushing and sliding in the contacts between the particles and the grains start to move and rotate *source: (Jan, 2011).*

The pore volume is reduced and the volume of the unbound material decreases. When the pore volume is decreased, the number of contacts between the particles increases. This increases the deformation modulus but also the stress hardening due to the higher number of contacts, the area of which can be larger during the elastic deformation. The denser structure also makes it possible to increase the number of contacts during the elastic deformation, which will cause even greater stress hardening.

In permanent deformation the effect becomes even greater due to that fact new contacts between the particles appear when the particles are subjected to increase compression, and the number of contacts per particle will be increased (Jan, 2011).

Generally, the grain to grain contact has effect on the deformation characteristics of the material. Thus, the grain to grain contact effect rises because of many reasons. The gradation, compaction, loading, moisture continent has their effect on the deformation. From these parameters the compaction effect is studied in this research that the permanent deformation low when the compaction is high. And the resilient modulus is high for grain to grain contact is good.

# 5 CONCLUSIONS AND RECOMMENDATIONS

## 5.1 Conclusion

The cinder material classified as excellent in its strength, and it has very minimum particles loss in the wet sieve, It is naturally occurring material. However, the cinder materials are lack of sandy fraction that less than 4.75 mm. And also the cinder material is non plastic material. Thus, the cinder material cannot meet the specification of sub base of ERA, 2013. Therefore, the cinder in the road construction is used when it blended with other material which has good fine composition.

The weathered yellowish laterite is good in its strength characteristic, it has large proportion of fine/sand sized/ materials and it is a medium plastic material. It is naturally occurring material. But the weathered yellowish material has not gravel size particles in proportion and the material has a significant proportion of silt clay fraction that used to give cohesion in structure of the sub base. Therefore, it is not selected for sub base material alone. The weathered yellowish laterite used when it is blended with a material of coarser proportion.

By stabilizing the cinder material with a weathered yellowish laterite the density and strength are improved, and the suggested optimum suitable proportion to use the material for sub base is 30 % of cinder material and 70 % of weathered yellowish laterite.

The permanent deformation indicates an increase when the load repetition increases. Thus, non-recoverable deformation shows steep increase at the beginning, and increases slightly when the load repetition increases. The elastic deformation of the material decreases suddenly at the first few loading and going constant for the next repeated loading. In repeated load at specific loading point the elastic deformation is greater than the cycle permanent deformation.

Well compacted materials have low permanent deformation, and high elastic deformation characteristic than lightly compacted materials. The lightly compacted materials have high permanent deformation and low elastic deformation than highly compacted materials. Therefore, well compacted material recovers with high elastic deformation level because the material initially compacted to high level so that it cannot accumulate the applied stress in it.

The repeated load CBR uses to study the mechanical properties. Therefore, by using CBR the recoverable deformation and non-recoverable deformation can be determined.

## 5.2 Recommendation

### 5.2.1 Based on the findings of the study, the following recommendations are drawn:

The cinder material is a lack of fine materials and to use the material in road construction, it is recommended to blend the material to change the gradation of cinder, with fine materials and it is possible to use.

However, the result in stabilization for sub bases gives any value less than 70 % of cinder and greater than 30 % of weathered yellowish material it is not recommended to use 0 % of cinder. Because of the coarse gravel material is needed to transfer the load by grain to grain contact in wet season.

To get a minimum cumulative permanent deformation and maximum elastic or recoverable deformation generally, to get a good performance; it is recommended the materials should be compacted well, as their dry density.

Contractors and consultants must use the mechanical concept because it can show relevant information on the performance character of the pavement material.

### 5.2.2 Recommendations for future study

To characterize a material broadly for the mechanistic – empirical additional mechanical characteristics like resilient modulus, shear strength should be studied. And to improved and modified tests and values tri–axial tests should be carried and characterizing of the material should be developed.

To use the mechanical approach widely as a design criteria correlation should be developed for the resilient deformation and resilient modulus or equivalent modulus, for materials tested in standard repeated load CBR. That means the fine grained materials that pass 19 mm used in the standard repeated load CBR.

Further study should be done on weathered yellowish laterite to know the chemical composition of the material, and to increase the potential usage of it. Because, it is naturally occurring and its engineering property is good, the study on the material should increase.

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# APPENDIX A

**Engineering properties of cinder and weathered yellowish laterite**

| DETERMINING LIQUID LIMIT, PLASTIC LIMIT, PLASTIC INDEX & CLASSIFICATION OF WEATHERED YELLOWISH MATERIAL |
|---|
| TEST AND CLASSIFICATION METHODS : AASHTO T:89,T90 & M145 |

Table A.1 Liquid Limit determination

| | LIQUID LIMIT | | |
|---|---|---|---|
| Test No. | 1 | 2 | 3 |
| Number of blows | 32 | 28 | 18 |
| Container no. | b1 | 6 | E |
| Wet Soil+Cont (g) | 43.70 | 43.5 | 41.8 |
| Dry Soil+Cont (g) | 37.7 | 37.40 | 35.80 |
| Mass Container (g) | 17.6 | 17.8 | 17.5 |
| Mass Moisture (g) | 6 | 6.1 | 6 |
| Mass Dry Soil (g) | 20.1 | 19.6 | 18.3 |
| Moisture Content (%) | 29.85 | 31.12 | 32.79 |

Table A.2 Plastic limit determination for weathered yellowish material

| | PLASTIC LIMIT | |
|---|---|---|
| Test No. | 1 | 2 |
| Container no. | B3 | A |

| | | |
|---|---|---|
| Wet Soil+Cont (g) | 24.4 | 23.2 |
| Dry Soil+Cont (g) | 23.00 | 22.40 |
| Mass Container (g) | 17.60 | 17.2 |
| Mass Moisture (g) | 1.40 | 0.80 |
| Mass Dry Soil (g) | 5.40 | 5.20 |
| Moisture Content (%) | 25.93 | 15.38 |
| | 20.66 | |

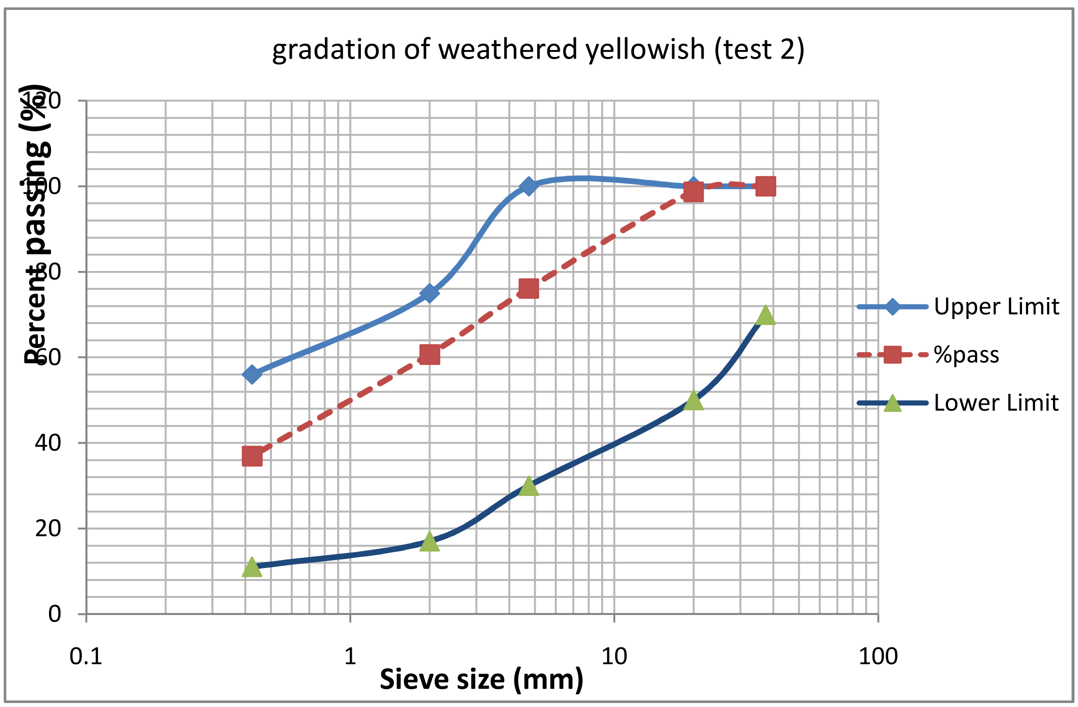


Figure A.1 Gradation percent pass of weathered yellowish laterite

# APPENDIX B

Stabilization of the cinder material and weathered yellowish laterite

Table B.1 Combination of 70% cinder and 30% weathered yellowish material trial 1X1

| | Cinder | weathered yellowish | | | |
|---|---|---|---|---|---|
| sieve size | % pass | % pass | 0.7*Cinder (4) | 0.3*weathered yellowish (5) | 4+5 |
| 63 | 100 | 100 | 70 | 30 | 100 |
| 37.5 | 98.067 | 100 | 68.6469 | 30 | 98.6469 |
| 20 | 68.478 | 97.011 | 47.9346 | 29.1033 | 77.0379 |
| 4.75 | 17.464 | 71.91 | 12.2248 | 21.573 | 33.7978 |
| 2 | 8.218 | 55.022 | 5.7526 | 16.5066 | 22.2592 |
| 0.425 | 3.149 | 32.966 | 2.2043 | 9.8898 | 12.0941 |
| 0.075 | 1.582 | 18.528 | 1.1074 | 5.5584 | 6.6658 |

Table B.2 Combination of 70% cinder and 30% weathered yellowish material trial 2X2

| | Cinder | weathered yellowish | | | |
|---|---|---|---|---|---|
| sieve size | % pass | % pass | 0.7*Cinder (4) | 0.3*weathered yellowish (5) | 4+5 |
| 63 | 100 | 100 | 70 | 30 | 100 |

| | | | | | |
|---|---|---|---|---|---|
| 37.5 | 92.486 | 100 | 64.7402 | 30 | 94.7402 |
| 20 | 71.135 | 98.69 | 49.7945 | 29.607 | 79.4015 |
| 4.75 | 22.324 | 76.096 | 15.6268 | 22.8288 | 38.4556 |
| 2 | 9.959 | 60.676 | 6.9713 | 18.2028 | 25.1741 |
| 0.425 | 3.581 | 36.918 | 2.5067 | 11.0754 | 13.5821 |
| 0.075 | 1.824 | 24.842 | 1.2768 | 7.4526 | 8.7294 |

Table B.3 Combination of 70% cinder and 30% weathered yellowish material trial 1X2

| | Cinder | weathered yellowish | | | |
|---|---|---|---|---|---|
| sieve size | % pass | % pass | 0.7*Cinder (4) | 0.3*weathered yellowish (5) | 4+5 |
| 63 | 100 | 100 | 70 | 30 | 100 |
| 37.5 | 98.067 | 100 | 68.6469 | 30 | 98.6469 |
| 20 | 68.478 | 98.69 | 47.9346 | 29.607 | 77.5416 |
| 4.75 | 17.464 | 76.096 | 12.2248 | 22.8288 | 35.0536 |
| 2 | 8.218 | 60.676 | 5.7526 | 18.2028 | 23.9554 |
| 0.425 | 3.149 | 36.918 | 2.2043 | 11.0754 | 13.2797 |
| 0.075 | 1.582 | 24.842 | 1.1074 | 7.4526 | 8.56 |

Table B.4 Combination of 70% cinder and 30% weathered yellowish material trial 2X1

| | cinder | weathered yellowish | | | |
|---|---|---|---|---|---|
| sieve size | % pass | % pass | 0.7*Cinder (4) | 0.3*weathered yellowish (5) | 4+5 |
| 63 | 100 | 100 | 70 | 30 | 100 |
| 37.5 | 92.486 | 100 | 64.7402 | 30 | 94.7402 |
| 20 | 71.135 | 97.011 | 49.7945 | 29.1033 | 78.8978 |
| 4.75 | 22.324 | 71.91 | 15.6268 | 21.573 | 37.1998 |
| 2 | 9.959 | 55.022 | 6.9713 | 16.5066 | 23.4779 |
| 0.425 | 3.581 | 32.966 | 2.5067 | 9.8898 | 12.3965 |
| 0.075 | 1.824 | 18.528 | 1.2768 | 5.5584 | 6.8352 |

# APPENDIX C

## Stabilized material test for sub base course result

Table C.1 Gradation result of stabilized material

| Test method :AASHTO T 27 | | | | | |
|---|---|---|---|---|---|
| Trial-1 | | | | | |
| Total weight before washing (gm) = 5635 | | | | | |
| Total weight after washing (gm) = | | | | | |
| Sieve Sizes (mm) | Weight Retained (gm) | % Retain | % Pass | SPEC- LIMIT | |
| | | | | upper | limit |
| 63 | 0 | | 100 | 100 | |
| 37.5 | 125 | 2.22 | 97.78 | 70 | 100 |
| 20 | 834 | 14.80 | 82.98 | 50 | 100 |
| 4.75 | 2058 | 36.52 | 46.46 | 30 | 100 |
| 2 | 851 | 15.10 | 31.36 | 17 | 75 |
| 0.425 | 846 | 15.01 | 18.34 | 11 | 56 |
| 0.075 | 412 | 7.31 | 9.02 | 5 | 25 |

Table C.2 Gradation of stabilized material

| Trial-2 | | | | | |
|---|---|---|---|---|---|
| Total weight before washing (gm) =6359 | | | | | |
| Total weight after washing (gm) = | | | | SPEC LIMIT | |
| Sieve sizes (mm) | Weight retained (gm) | %retain | %pass | upper | limit |
| 63 | 0 | | 100 | 100 | |
| 37.5 | 0 | 0.00 | 100.00 | 70 | 100 |
| 20 | 703 | 11.06 | 88.94 | 50 | 100 |
| 4.75 | 2377 | 37.38 | 51.56 | 30 | 100 |
| 2 | 1215 | 19.11 | 32.46 | 17 | 75 |
| 0.425 | 1115 | 17.53 | 14.92 | 11 | 56 |
| 0.075 | 322 | 5.06 | 9.85 | 5 | 25 |
| Pan | 15 | | | | |

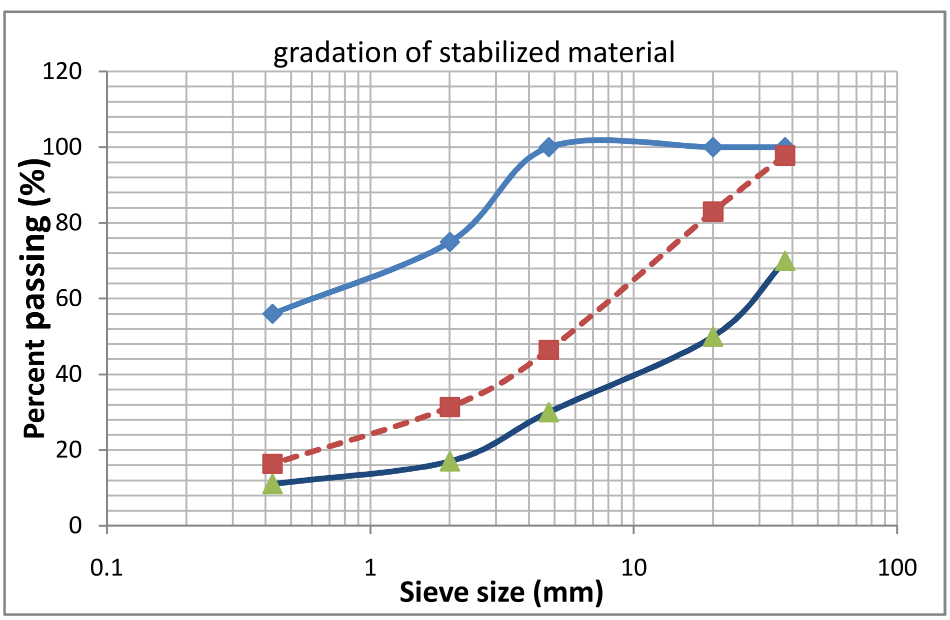


Figure C 1 Stabilized material gradation

Table C.3 Modified compaction data

| Test methods : AASHTO T – 180, method D | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| WT OF MAT'L TO EACH TRIAL (g) | | | 6000 | | | | | |
| AMOUNT OF MIXING WATER TO EACH TRIAL (lt) | | | | | | | | |
| DENSITY | | TRIAL NUMBER | 1 | 2 | 3 | 4 | 5 | 6 |
| | 1 | WEIGHT OF SOIL + MOLD(g) | 9297 | 9467 | 9718 | 9723 | | |
| | 2 | WEIGHT OF MOLD(g) | 5394 | 5394 | 5394 | 5394 | | |
| | 3 | WEIGHT OF SOIL, (g) (1-2) | 3903 | 4073 | 4324 | 4329 | | |
| | 4 | VOLUME OF MOLD (cc) | 2194 | 2194 | 2194 | 2194 | | |
| | 5 | WET DENSITY OF SOIL, (3/4),( g/cc) | 1.779 | 1.856 | 1.971 | 1.973 | | |
| MOISTURE | 6 | CONTAINER NUMBER | B1 | A1 | A2 | P | NMC | |
| | 7 | WET SOIL + CONTAINER(g) | 215.0 | 210.0 | 220.0 | 230.0 | | |
| | 8 | DRY SOIL + CONTAINER(g) | 204.0 | 197.0 | 202.0 | 207.0 | | |
| | 9 | WEIGHT OF WATER, (g) (7-8) | 11.0 | 13.0 | 18.0 | 23.0 | | |
| | 10 | WEIGHT OF CONTAINER (g) | 31.0 | 34.0 | 30.0 | 27.0 | | |
| | 11 | WEIGHT OF DRY SOIL, (g) (8-10) | 173.0 | 163.0 | 172.0 | 180.0 | | |
| | 12 | MOISTURE CONTENT,(9/11)*100, % | 6.4 | 8.0 | 10.5 | 12.8 | | |
| | DRY DENSITY OF SOIL, (5/(100+12))*100, g/cc | | 1.673 | 1.719 | 1.784 | 1.750 | | |

Table C.4 CBR Test description

| | | | |
|---|---|---|---|
| MATERIAL DESCRIPITION | cinder material and highly weatherd yellowish laterite | DATE AND TIME SOAKED | |
| PURPOSE: | sub base | DATE AND TIME TESTED: | 21/9/2019 |
| Ring Factor:<br>( 43.1152N/D) | 0.0431152 | SAMPLE REF NO : | 25/9/2019 |
| CALIFORNIA BEARING RFATIO<br>TEST METHOD : AASHTO T 193 | | | |

Table C.5 Density Distribution

| Soaking condition | | 10 blows | | 30 blows | | 65 blows | |
|---|---|---|---|---|---|---|---|
| | | Before | After | Before | After | Before | After |
| MOLD NUMBER | | M2 | M2 | C | C | P3 | P3 |
| WEIGHT OF SOIL + MOLD g | | 9864 | 10200 | 10300 | 10415 | 10440 | 10600 |
| WEIGHT OF MOLD g | | 6500 | 6500 | 6552 | 6552 | 6341 | 6341 |
| WEIGHT OF SOIL g | | 3364 | 3700 | 3748 | 3863 | 4099 | 4259 |
| VOLUME OF MOLD g | | 2036 | 2036 | 2036 | 2036 | 2036 | 2036 |
| WET DENSITY OF SOIL g/cc | | 1.652 | 1.817 | 1.841 | 1.897 | 2.013 | 2.092 |
| DRY DENSITY OF SOIL g/cc | | 1.497 | 1.636 | 1.661 | 1.687 | 1.821 | 1.848 |

Table C.6 Moisture content

| Soaking condition | | 10 blows | | 30 blows | | 65 blows | |
|---|---|---|---|---|---|---|---|
| | | Before | After | Before | After | Before | After |
| CONTAINER NUMBER | | M | B3 | T | B | Q | A2 |
| WEIGHT OF SOIL + CONTAINER G | | 200.0 | 200.0 | 205.0 | 210.0 | 210.0 | 220.0 |
| DRY SOIL + CONTAINER G | | 184.0 | 183.0 | 188.0 | 190.0 | 193.0 | 198.0 |
| WEIGHT OF WATER G | | 16.0 | 17.0 | 17.0 | 20.0 | 17.0 | 22.0 |
| WEIGHT OF CONTAINER G | | 30.0 | 30.0 | 31.0 | 30.0 | 32.0 | 31.0 |
| WEIGHT OF DRYSOIL % | | 154.0 | 153.0 | 157.0 | 160.0 | 161.0 | 167.0 |
| MOISTURE CONTENT | % | 10.4 | 11.1 | 10.8 | 12.5 | 10.6 | 13.2 |

Table C.7 Penetration data for 10 and 30 blows

| PENETRATION | 10 Blows | | | | 30 Blows | | | |
|---|---|---|---|---|---|---|---|---|
| | DIAL RDG | LOAD (kN) | COR.LOAD (kN) | CBR % | DIAL RDG | LOAD (kN) | COR.LOAD (kN) | CBR % |
| 0 | 0 | 0.00 | | | 0 | 0.00 | | |
| 0.64 | 12 | 0.52 | | | 15 | 0.65 | | |
| 1.27 | 26 | 1.12 | | | 47 | 2.03 | | |
| 1.96 | 44 | 1.90 | | | 94 | 4.05 | | |
| 2.54 | 54 | 2.33 | 2.3 | 17.2 | 142 | 6.12 | 6.1 | 45.7 |
| 3.18 | 62 | 2.67 | | | 165 | 7.11 | | |
| 3.81 | 79 | 3.41 | | | 184 | 7.93 | | |
| 4.45 | 75 | 3.23 | | | 193 | 8.32 | | |
| 5.08 | 80 | 3.45 | 3.4 | 17.1 | 205 | 8.84 | 8.8 | 44.2 |
| 7.62 | 94 | 4.05 | | | 226 | 9.74 | | |

Table C.8 Penetration data for 65 blows

| PENETRATION | 65 Blows | | | |
|---|---|---|---|---|
| | DIAL RDG | LOAD (kN) | COR.LOAD(kN) | CBR % |
| **0** | **0** | **0.00** | | |
| **0.64** | **45** | **1.94** | | |
| **1.27** | **140** | **6.04** | | |
| **1.96** | **244** | **10.52** | | |
| **2.54** | **306** | **13.19** | **13.2** | **98.9** |
| **3.18** | **343** | **14.79** | | |
| **3.81** | **393** | **16.94** | | |
| **4.45** | **419** | **18.07** | | |
| **5.08** | **450** | **19.40** | **19.4** | **97.3** |
| **7.62** | **480** | **20.70** | | |
| **10.16** | | | | |
| **12.7** | | | | |

Table C.9 Swelling results

| SWELL % [Height of Specimen (mm) = 116.43] | | | |
|---|---|---|---|
| No. OF BLOWS | 10 | 30 | 65 |
| Reading Before soaking | 0 | 0 | 0 |
| Reading after soaking | 0.23 | 0.15 | 0.1 |
| PERCENT SWELL | 0.2 | 0.13 | 0.09 |

Table C.10 CBR value and swelling

| BLOW | LOAD (KN) | | CBR (%) | | Swell (%) |
|---|---|---|---|---|---|
| | 2.54mm | 5.08mm | 2.54mm | 5.08mm | |
| 10 | 2.30 | 3.40 | 17.2 | 17.10 | 0.2 |
| 30 | 6.10 | 8.80 | 45.7 | 44.20 | 0.13 |
| 65 | 13.20 | 19.40 | 98.9 | 97.30 | 0.09 |

Table C.11 Moisture Density relation before and after soaking

| Blows | Before Soaking | | After Soaking | |
|---|---|---|---|---|
| | DD (g/cc) | Moisture (%) | DD (g/cc) | Moisture (%) |
| 10 | 1.497 | 10.4 | 1.64 | 11.1 |
| 30 | 1.661 | 10.8 | 1.69 | 12.5 |
| 65 | 1.821 | 10.6 | 1.85 | 13.2 |

Table C.12 Swell CBR relation

| BLOW | LOAD (KN) | | CBR (%) | | Swell (%) |
|---|---|---|---|---|---|
| | 2.54mm | 5.08mm | 2.54mm | 5.08mm | |
| 10 | 2.30 | 3.40 | 17.2 | 17.10 | 0.21 |
| 30 | 6.10 | 8.80 | 45.7 | 44.20 | 0.16 |
| 65 | 13.20 | 19.40 | 98.9 | 97.30 | 0.09 |

Table C.13 Dry density - Swell

| Blow | Dry Density (g/CC) | CBR (%) | Swell (%) |
|---|---|---|---|
| 10 | 1.640 | 17.20 | 0.21 |
| 30 | 1.690 | 45.70 | 0.16 |
| 65 | 1.850 | 98.90 | 0.09 |
| CBR at 95 % of MDD(%): | | | 46% |
| Percent Swell (%): | | | 0.15% |

# APPENDIX D

**Repeated load CBR result**

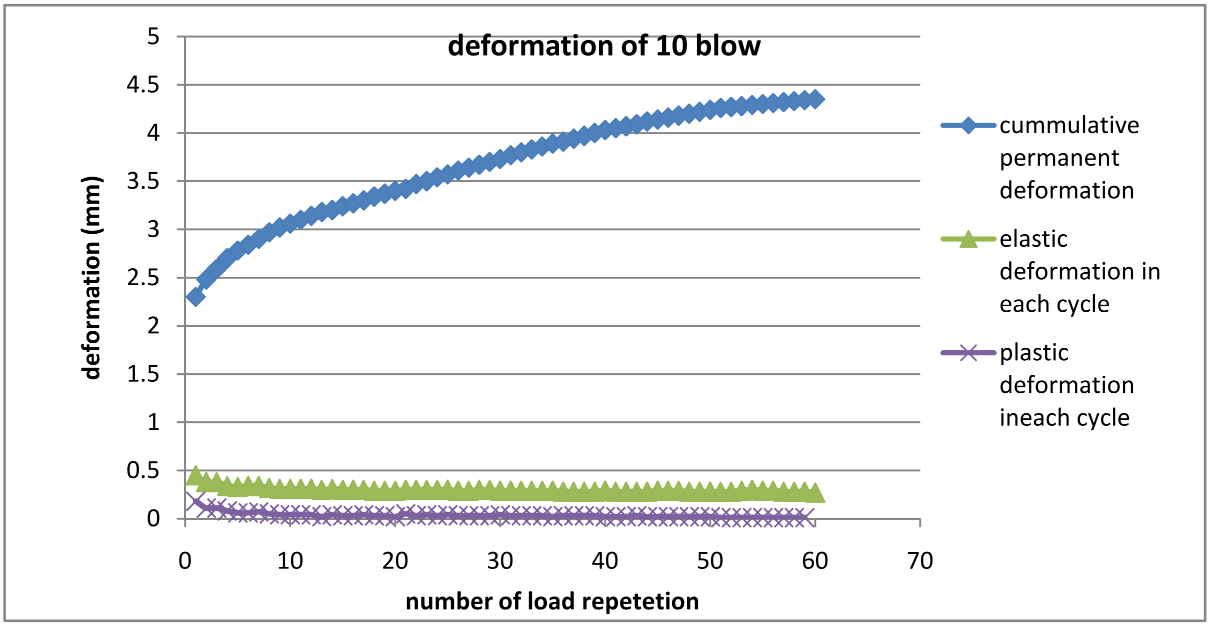


Figure D.1 Deformation on 10 blows

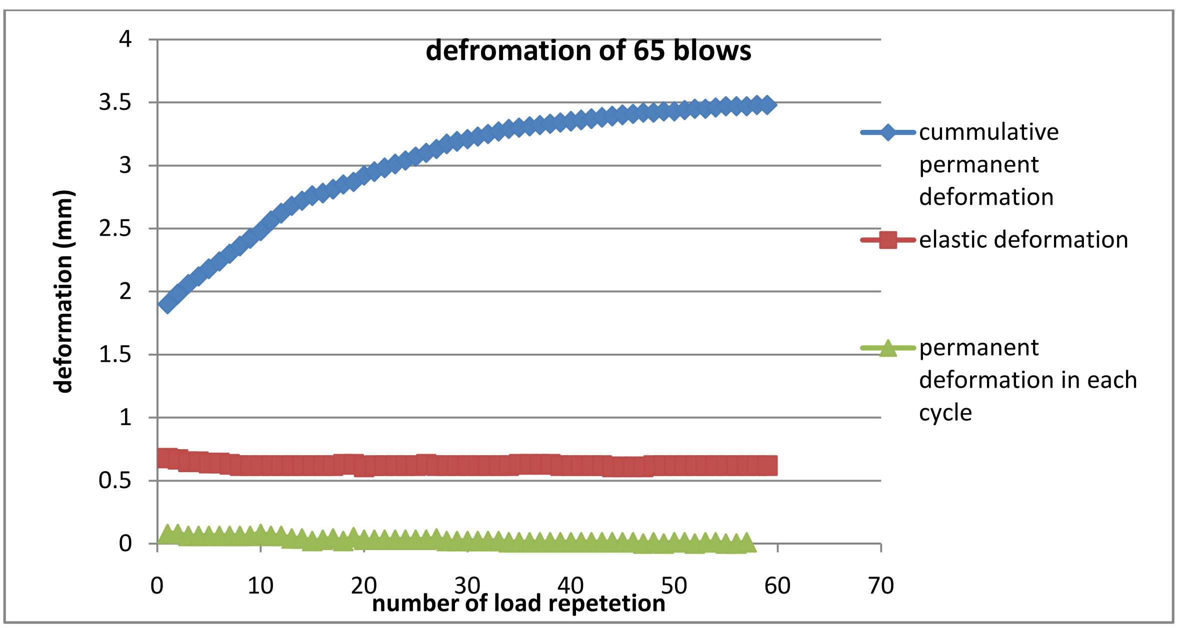


Figure D.2 Deformation 65 blows

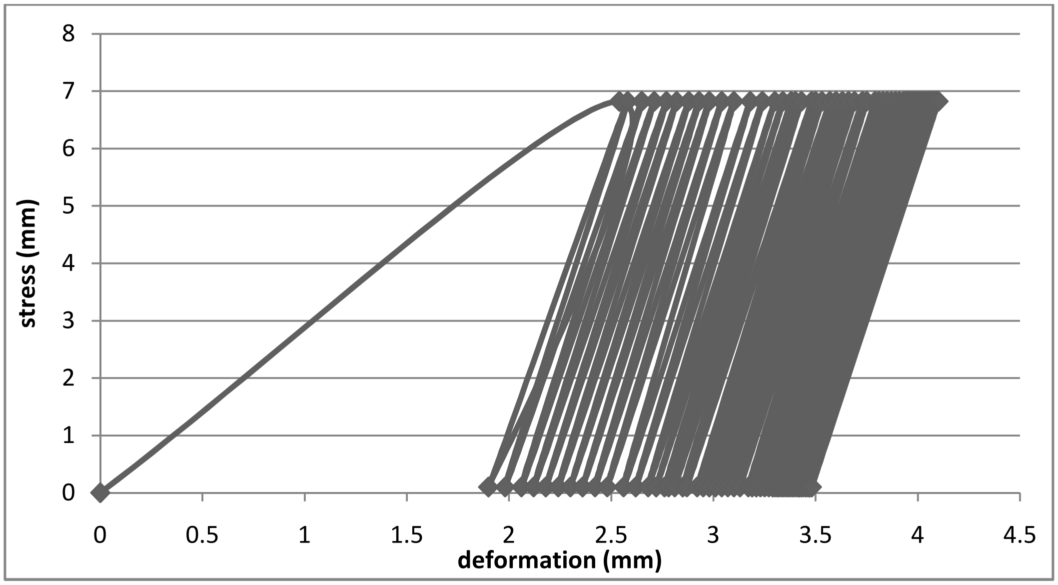


Figure D.3 Repeated load test result for 65 blows.

Table D.1 Repeated load penetration load relation for 10 blows

| penetration | Load | | Permanent deformation | | Elastic deformation (resilient deformation) | |
|---|---|---|---|---|---|---|
| 0 | 0 | | no of load | Deformation (mm) | no of load | Deformation (mm) |
| 2.54 | 1.19 | | 1 | 2.3 | 1 | 0.45 |
| 2.3 | 0.1 | | 2 | 2.48 | 2 | 0.38 |
| 2.75 | 1.19 | | 3 | 2.59 | 3 | 0.38 |
| 2.48 | 0.1 | | 4 | 2.7 | 4 | 0.34 |
| 2.86 | 1.19 | | 5 | 2.78 | 5 | 0.33 |
| 2.59 | 0.1 | | 6 | 2.84 | 6 | 0.34 |
| 2.97 | 1.19 | | 7 | 2.9 | 7 | 0.34 |
| 2.7 | 0.1 | | 8 | 2.97 | 8 | 0.32 |
| 3.04 | 1.19 | | 9 | 3.02 | 9 | 0.31 |
| 2.78 | 0.1 | | 10 | 3.06 | 10 | 0.31 |
| 3.11 | 1.19 | | 11 | 3.1 | 11 | 0.31 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 2.84 | 0.1 | | 12 | 3.14 | 12 | 0.31 |
| 3.18 | 1.19 | | 13 | 3.18 | 13 | 0.3 |
| 2.9 | 0.1 | | 14 | 3.2 | 14 | 0.31 |
| 3.24 | 1.19 | | 15 | 3.24 | 15 | 0.3 |
| 2.97 | 0.1 | | 16 | 3.27 | 16 | 0.3 |
| 3.29 | 1.19 | | 17 | 3.3 | 17 | 0.3 |
| 3.02 | 0.1 | | 18 | 3.34 | 18 | 0.29 |
| 3.33 | 1.19 | | 19 | 3.37 | 19 | 0.29 |
| 3.06 | 0.1 | | 20 | 3.4 | 20 | 0.29 |
| 3.37 | 1.19 | | 21 | 3.42 | 21 | 0.3 |
| 3.1 | 0.1 | | 22 | 3.47 | 22 | 0.3 |
| 3.41 | 1.19 | | 23 | 3.5 | 23 | 0.3 |
| 3.14 | 0.1 | | 24 | 3.54 | 24 | 0.3 |
| 3.45 | 1.19 | | 25 | 3.57 | 25 | 0.3 |
| 3.18 | 0.1 | | 26 | 3.61 | 26 | 0.29 |
| 3.48 | 1.19 | | 27 | 3.64 | 27 | 0.29 |
| 3.2 | 0.1 | | 28 | 3.67 | 28 | 0.3 |
| 3.51 | 1.19 | | 29 | 3.7 | 29 | 0.3 |
| 3.24 | 0.1 | | 30 | 3.73 | 30 | 0.29 |
| 3.54 | 1.19 | | 31 | 3.77 | 31 | 0.29 |
| 3.27 | 0.1 | | 32 | 3.8 | 32 | 0.29 |
| 3.57 | 1.19 | | 33 | 3.83 | 33 | 0.29 |
| 3.3 | 0.1 | | 34 | 3.86 | 34 | 0.29 |
| 3.6 | 1.19 | | 35 | 3.89 | 35 | 0.29 |
| 3.34 | 0.1 | | 36 | 3.91 | 36 | 0.28 |
| 3.63 | 1.19 | | 37 | 3.94 | 37 | 0.28 |
| 3.37 | 0.1 | | 38 | 3.97 | 38 | 0.28 |
| 3.66 | 1.19 | | 39 | 4 | 39 | 0.28 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 3.4 | 0.1 | | 40 | 4.03 | 40 | 0.29 |
| 3.69 | 1.19 | | 41 | 4.05 | 41 | 0.28 |
| 3.42 | 0.1 | | 42 | 4.07 | 42 | 0.28 |
| 3.72 | 1.19 | | 43 | 4.09 | 43 | 0.28 |
| 3.47 | 0.1 | | 44 | 4.12 | 44 | 0.28 |
| 3.77 | 1.19 | | 45 | 4.14 | 45 | 0.29 |
| 3.5 | 0.1 | | 46 | 4.16 | 46 | 0.29 |
| 3.8 | 1.19 | | 47 | 4.18 | 47 | 0.29 |
| 3.54 | 0.1 | | 48 | 4.2 | 48 | 0.28 |
| 3.84 | 1.19 | | 49 | 4.22 | 49 | 0.28 |
| 3.57 | 0.1 | | 50 | 4.24 | 50 | 0.28 |
| 3.87 | 1.19 | | 51 | 4.26 | 51 | 0.28 |
| 3.61 | 0.1 | | 52 | 4.27 | 52 | 0.28 |
| 3.9 | 1.19 | | 53 | 4.28 | 53 | 0.29 |
| 3.64 | 0.1 | | 54 | 4.29 | 54 | 0.3 |
| 3.93 | 1.19 | | 55 | 4.3 | 55 | 0.29 |
| 3.67 | 0.1 | | 56 | 4.31 | 56 | 0.29 |
| 3.97 | 1.19 | | 57 | 4.32 | 57 | 0.28 |
| 3.7 | 0.1 | | 58 | 4.33 | 58 | 0.28 |
| 4 | 1.19 | | 59 | 4.34 | 59 | 0.28 |
| 3.73 | 0.1 | | 60 | 4.35 | 60 | 0.27 |
| 4.02 | 1.19 | | | | | |
| 3.77 | 0.1 | | | | | |
| 4.06 | 1.19 | | | | | |
| 3.8 | 0.1 | | | | | |
| 4.09 | 1.19 | | | | | |
| 3.83 | 0.1 | | | | | |
| 4.12 | 1.19 | | | | | |

| | | | | | | |
|---|---|---|---|---|---|---|
| 3.86 | 0.1 | | | | | |
| 4.15 | 1.19 | | | | | |
| 3.89 | 0.1 | | | | | |
| 4.17 | 1.19 | | | | | |
| 3.91 | 0.1 | | | | | |
| 4.19 | 1.19 | | | | | |
| 3.94 | 0.1 | | | | | |
| 4.22 | 1.19 | | | | | |
| 3.97 | 0.1 | | | | | |
| 4.25 | 1.19 | | | | | |
| 4 | 0.1 | | | | | |
| 4.29 | 1.19 | | | | | |
| 4.03 | 0.1 | | | | | |
| 4.31 | 1.19 | | | | | |
| 4.05 | 0.1 | | | | | |
| 4.33 | 1.19 | | | | | |
| 4.07 | 0.1 | | | | | |
| 4.35 | 1.19 | | | | | |
| 4.09 | 0.1 | | | | | |
| 4.37 | 1.19 | | | | | |
| 4.12 | 0.1 | | | | | |
| 4.41 | 1.19 | | | | | |
| 4.14 | 0.1 | | | | | |
| 4.43 | 1.19 | | | | | |

| | | | | | | |
|---|---|---|---|---|---|---|
| 4.16 | 0.1 | | | | | |
| 4.45 | 1.19 | | | | | |
| 4.18 | 0.1 | | | | | |
| 4.46 | 1.19 | | | | | |
| 4.2 | 0.1 | | | | | |
| 4.48 | 1.19 | | | | | |
| 4.22 | 0.1 | | | | | |
| 4.5 | 1.19 | | | | | |
| 4.24 | 0.1 | | | | | |
| 4.52 | 1.19 | | | | | |
| 4.26 | 0.1 | | | | | |
| 4.54 | 1.19 | | | | | |
| 4.27 | 0.1 | | | | | |
| 4.56 | 1.19 | | | | | |
| 4.28 | 0.1 | | | | | |
| 4.58 | 1.19 | | | | | |
| 4.29 | 0.1 | | | | | |
| 4.58 | 1.19 | | | | | |
| 4.3 | 0.1 | | | | | |
| 4.59 | 1.19 | | | | | |
| 4.31 | 0.1 | | | | | |
| 4.59 | 1.19 | | | | | |

| | | | | | | |
|---|---|---|---|---|---|---|
| 4.32 | 0.1 | | | | | |
| 4.6 | 1.19 | | | | | |
| 4.33 | 0.1 | | | | | |
| 4.61 | 1.19 | | | | | |
| 4.34 | 0.1 | | | | | |
| 4.61 | 1.19 | | | | | |

Table D.2 Repeated load penetration load relation for 30 blows

| penetration | Load | | Permanent deformation | | Elastic deformation (resilient deformation) | |
|---|---|---|---|---|---|---|
| 0 | 0 | | no of load | Deformation (mm) | no of load | Deformation |
| 2.54 | 3.15 | | 1 | 2.1 | 1 | 0.54 |
| 2.1 | 0.1 | | 2 | 2.2 | 2 | 0.56 |
| 2.64 | 3.15 | | 3 | 2.3 | 3 | 0.56 |
| 2.2 | 0.1 | | 4 | 2.4 | 4 | 0.55 |
| 2.76 | 3.15 | | 5 | 2.51 | 5 | 0.53 |
| 2.3 | 0.1 | | 6 | 2.58 | 6 | 0.53 |
| 2.86 | 3.15 | | 7 | 2.62 | 7 | 0.54 |
| 2.4 | 0.1 | | 8 | 2.7 | 8 | 0.52 |
| 2.95 | 3.15 | | 9 | 2.74 | 9 | 0.52 |
| 2.51 | 0.1 | | 10 | 2.78 | 10 | 0.52 |
| 3.04 | 3.15 | | 11 | 2.82 | 11 | 0.52 |
| 2.58 | 0.1 | | 12 | 2.86 | 12 | 0.51 |
| 3.11 | 3.15 | | 13 | 2.91 | 13 | 0.5 |
| 2.62 | 0.1 | | 14 | 2.96 | 14 | 0.5 |
| 3.16 | 3.15 | | 15 | 3 | 15 | 0.5 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 2.7 | 0.1 | | 16 | 3.05 | 16 | 0.49 |
| 3.22 | 3.15 | | 17 | 3.08 | 17 | 0.49 |
| 2.74 | 0.1 | | 18 | 3.11 | 18 | 0.49 |
| 3.26 | 3.15 | | 19 | 3.14 | 19 | 0.48 |
| 2.78 | 0.1 | | 20 | 3.16 | 20 | 0.49 |
| 3.3 | 3.15 | | 21 | 3.18 | 21 | 0.5 |
| 2.82 | 0.1 | | 22 | 3.21 | 22 | 0.49 |
| 3.34 | 3.15 | | 23 | 3.23 | 23 | 0.49 |
| 2.86 | 0.1 | | 24 | 3.25 | 24 | 0.49 |
| 3.37 | 3.15 | | 25 | 3.28 | 25 | 0.48 |
| 2.91 | 0.1 | | 26 | 3.29 | 26 | 0.49 |
| 3.41 | 3.15 | | 27 | 3.31 | 27 | 0.49 |
| 2.96 | 0.1 | | 28 | 3.33 | 28 | 0.49 |
| 3.46 | 3.15 | | 29 | 3.34 | 29 | 0.5 |
| 3 | 0.1 | | 30 | 3.36 | 30 | 0.5 |
| 3.5 | 3.15 | | 31 | 3.39 | 31 | 0.49 |
| 3.05 | 0.1 | | 32 | 3.41 | 32 | 0.49 |
| 3.54 | 3.15 | | 33 | 3.42 | 33 | 0.5 |
| 3.08 | 0.1 | | 34 | 3.43 | 34 | 0.5 |
| 3.57 | 3.15 | | 35 | 3.45 | 35 | 0.5 |
| 3.11 | 0.1 | | 36 | 3.46 | 36 | 0.5 |
| 3.6 | 3.15 | | 37 | 3.47 | 37 | 0.5 |
| 3.14 | 0.1 | | 38 | 3.49 | 38 | 0.5 |
| 3.62 | 3.15 | | 39 | 3.51 | 39 | 0.49 |
| 3.16 | 0.1 | | 40 | 3.52 | 40 | 0.49 |
| 3.65 | 3.15 | | 41 | 3.53 | 41 | 0.5 |
| 3.18 | 0.1 | | 42 | 3.55 | 42 | 0.5 |
| 3.68 | 3.15 | | 43 | 3.56 | 43 | 0.5 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 3.21 | 0.1 | | 44 | 3.58 | 44 | 0.5 |
| 3.7 | 3.15 | | 45 | 3.6 | 45 | 0.49 |
| 3.23 | 0.1 | | 46 | 3.61 | 46 | 0.49 |
| 3.72 | 3.15 | | 47 | 3.62 | 47 | 0.49 |
| 3.25 | 0.1 | | 48 | 3.63 | 48 | 0.49 |
| 3.74 | 3.15 | | 49 | 3.64 | 49 | 0.49 |
| 3.28 | 0.1 | | 50 | 3.65 | 50 | 0.49 |
| 3.76 | 3.15 | | 51 | 3.66 | 51 | 0.49 |
| 3.29 | 0.1 | | 52 | 3.66 | 52 | 0.49 |
| 3.78 | 3.15 | | 53 | 3.66 | 53 | 0.5 |
| 3.31 | 0.1 | | 54 | 3.68 | 54 | 0.49 |
| 3.8 | 3.15 | | 55 | 3.69 | 55 | 0.49 |
| 3.33 | 0.1 | | 56 | 3.7 | 56 | 0.49 |
| 3.82 | 3.15 | | 57 | 3.71 | 57 | 0.49 |
| 3.34 | 0.1 | | 58 | 3.72 | 58 | 0.49 |
| 3.84 | 3.15 | | 59 | 3.73 | 59 | 0.49 |
| 3.36 | 0.1 | | 60 | 3.74 | 60 | |
| 3.86 | 3.15 | | | | | |
| 3.39 | 0.1 | | | | | |
| 3.88 | 3.15 | | | | | |
| 3.41 | 0.1 | | | | | |
| 3.9 | 3.15 | | | | | |
| 3.42 | 0.1 | | | | | |
| 3.92 | 3.15 | | | | | |
| 3.43 | 0.1 | | | | | |
| 3.93 | 3.15 | | | | | |
| 3.45 | 0.1 | | | | | |
| 3.95 | 3.15 | | | | | |

| | | | | | | |
|---|---|---|---|---|---|---|
| 3.46 | 0.1 | | | | | |
| 3.96 | 3.15 | | | | | |
| 3.47 | 0.1 | | | | | |
| 3.97 | 3.15 | | | | | |
| 3.49 | 0.1 | | | | | |
| 3.99 | 3.15 | | | | | |
| 3.51 | 0.1 | | | | | |
| 4 | 3.15 | | | | | |
| 3.52 | 0.1 | | | | | |
| 4.01 | 3.15 | | | | | |
| 3.53 | 0.1 | | | | | |
| 4.03 | 3.15 | | | | | |
| 3.55 | 0.1 | | | | | |
| 4.05 | 3.15 | | | | | |
| 3.56 | 0.1 | | | | | |
| 4.06 | 3.15 | | | | | |
| 3.58 | 0.1 | | | | | |
| 4.08 | 3.15 | | | | | |
| 3.6 | 0.1 | | | | | |
| 4.09 | 3.15 | | | | | |
| 3.61 | 0.1 | | | | | |
| 4.1 | 3.15 | | | | | |
| 3.62 | 0.1 | | | | | |
| 4.11 | 3.15 | | | | | |
| 3.63 | 0.1 | | | | | |
| 4.12 | 3.15 | | | | | |
| 3.64 | 0.1 | | | | | |
| 4.13 | 3.15 | | | | | |

| | | | | | | |
|---|---|---|---|---|---|---|
| 3.65 | 0.1 | | | | | |
| 4.14 | 3.15 | | | | | |
| 3.66 | 0.1 | | | | | |
| 4.15 | 3.15 | | | | | |
| 3.66 | 0.1 | | | | | |
| 4.15 | 3.15 | | | | | |
| 3.66 | 0.1 | | | | | |
| 4.16 | 3.15 | | | | | |
| 3.68 | 0.1 | | | | | |
| 4.17 | 3.15 | | | | | |
| 3.69 | 0.1 | | | | | |
| 4.18 | 3.15 | | | | | |
| 3.7 | 0.1 | | | | | |
| 4.19 | 3.15 | | | | | |
| 3.71 | 0.1 | | | | | |
| 4.2 | 3.15 | | | | | |
| 3.72 | 0.1 | | | | | |
| 4.21 | 3.15 | | | | | |
| 3.73 | 0.1 | | | | | |
| 4.22 | 3.15 | | | | | |

Table D.3 Repeated load penetration load relation for 65 blows

| penetration | Load | | Permanent deformation | | Elastic deformation (resilient deformation) | |
|---|---|---|---|---|---|---|
| 0 | 0 | | no of load | Deformation (mm) | no of load | Deformation (mm) |
| 2.54 | 6.82 | | 1 | 1.9 | 1 | 0.68 |
| 1.9 | 0.1 | | 2 | 1.98 | 2 | 0.67 |
| 2.58 | 6.82 | | 3 | 2.06 | 3 | 0.65 |
| 1.98 | 0.1 | | 4 | 2.12 | 4 | 0.65 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 2.65 | 6.82 | | 5 | 2.18 | 5 | 0.64 |
| 2.06 | 0.1 | | 6 | 2.24 | 6 | 0.64 |
| 2.71 | 6.82 | | 7 | 2.3 | 7 | 0.63 |
| 2.12 | 0.1 | | 8 | 2.36 | 8 | 0.62 |
| 2.77 | 6.82 | | 9 | 2.42 | 9 | 0.62 |
| 2.18 | 0.1 | | 10 | 2.48 | 10 | 0.62 |
| 2.82 | 6.82 | | 11 | 2.56 | 11 | 0.62 |
| 2.24 | 0.1 | | 12 | 2.62 | 12 | 0.62 |
| 2.88 | 6.82 | | 13 | 2.68 | 13 | 0.62 |
| 2.3 | 0.1 | | 14 | 2.72 | 14 | 0.62 |
| 2.93 | 6.82 | | 15 | 2.76 | 15 | 0.62 |
| 2.36 | 0.1 | | 16 | 2.78 | 16 | 0.62 |
| 2.98 | 6.82 | | 17 | 2.81 | 17 | 0.62 |
| 2.42 | 0.1 | | 18 | 2.85 | 18 | 0.63 |
| 3.04 | 6.82 | | 19 | 2.87 | 19 | 0.63 |
| 2.48 | 0.1 | | 20 | 2.92 | 20 | 0.61 |
| 3.1 | 6.82 | | 21 | 2.95 | 21 | 0.62 |
| 2.56 | 0.1 | | 22 | 2.98 | 22 | 0.62 |
| 3.18 | 6.82 | | 23 | 3.01 | 23 | 0.62 |
| 2.62 | 0.1 | | 24 | 3.04 | 24 | 0.62 |
| 3.24 | 6.82 | | 25 | 3.07 | 25 | 0.62 |
| 2.68 | 0.1 | | 26 | 3.1 | 26 | 0.63 |
| 3.3 | 6.82 | | 27 | 3.13 | 27 | 0.62 |
| 2.72 | 0.1 | | 28 | 3.17 | 28 | 0.62 |
| 3.34 | 6.82 | | 29 | 3.19 | 29 | 0.62 |
| 2.76 | 0.1 | | 30 | 3.21 | 30 | 0.62 |
| 3.38 | 6.82 | | 31 | 3.23 | 31 | 0.62 |
| 2.78 | 0.1 | | 32 | 3.25 | 32 | 0.62 |
| 3.4 | 6.82 | | 33 | 3.27 | 33 | 0.62 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 2.81 | 0.1 | | 34 | 3.29 | 34 | 0.62 |
| 3.43 | 6.82 | | 35 | 3.3 | 35 | 0.63 |
| 2.85 | 0.1 | | 36 | 3.31 | 36 | 0.63 |
| 3.48 | 6.82 | | 37 | 3.32 | 37 | 0.63 |
| 2.87 | 0.1 | | 38 | 3.33 | 38 | 0.63 |
| 3.5 | 6.82 | | 39 | 3.34 | 39 | 0.62 |
| 2.92 | 0.1 | | 40 | 3.35 | 40 | 0.62 |
| 3.53 | 6.82 | | 41 | 3.36 | 41 | 0.62 |
| 2.95 | 0.1 | | 42 | 3.37 | 42 | 0.62 |
| 3.57 | 6.82 | | 43 | 3.38 | 43 | 0.62 |
| 2.98 | 0.1 | | 44 | 3.39 | 44 | 0.61 |
| 3.6 | 6.82 | | 45 | 3.4 | 45 | 0.61 |
| 3.01 | 0.1 | | 46 | 3.41 | 46 | 0.61 |
| 3.63 | 6.82 | | 47 | 3.42 | 47 | 0.61 |
| 3.04 | 0.1 | | 48 | 3.42 | 48 | 0.62 |
| 3.66 | 6.82 | | 49 | 3.43 | 49 | 0.62 |
| 3.07 | 0.1 | | 50 | 3.43 | 50 | 0.62 |
| 3.69 | 6.82 | | 51 | 3.44 | 51 | 0.62 |
| 3.1 | 0.1 | | 52 | 3.45 | 52 | 0.62 |
| 3.73 | 6.82 | | 53 | 3.45 | 53 | 0.62 |
| 3.13 | 0.1 | | 54 | 3.46 | 54 | 0.62 |
| 3.75 | 6.82 | | 55 | 3.47 | 55 | 0.62 |
| 3.17 | 0.1 | | 56 | 3.47 | 56 | 0.62 |
| 3.79 | 6.82 | | 57 | 3.47 | 57 | 0.62 |
| 3.19 | 0.1 | | 58 | 3.48 | 58 | 0.62 |
| 3.81 | 6.82 | | 59 | 3.48 | 59 | 0.62 |
| 3.21 | 0.1 | | 60 | | 60 | |
| 3.83 | 6.82 | | | | | |
| 3.23 | 0.1 | | | | | |

| | | | | | | |
|---|---|---|---|---|---|---|
| 3.85 | 6.82 | | | | | |
| 3.25 | 0.1 | | | | | |
| 3.87 | 6.82 | | | | | |
| 3.27 | 0.1 | | | | | |
| 3.89 | 6.82 | | | | | |
| 3.29 | 0.1 | | | | | |
| 3.91 | 6.82 | | | | | |
| 3.3 | 0.1 | | | | | |
| 3.93 | 6.82 | | | | | |
| 3.31 | 0.1 | | | | | |
| 3.94 | 6.82 | | | | | |
| 3.32 | 0.1 | | | | | |
| 3.95 | 6.82 | | | | | |
| 3.33 | 0.1 | | | | | |
| 3.96 | 6.82 | | | | | |
| 3.34 | 0.1 | | | | | |
| 3.96 | 6.82 | | | | | |
| 3.35 | 0.1 | | | | | |
| 3.97 | 6.82 | | | | | |
| 3.36 | 0.1 | | | | | |
| 3.98 | 6.82 | | | | | |
| 3.37 | 0.1 | | | | | |
| 3.99 | 6.82 | | | | | |
| 3.38 | 0.1 | | | | | |
| 4 | 6.82 | | | | | |
| 3.39 | 0.1 | | | | | |
| 4 | 6.82 | | | | | |
| 3.4 | 0.1 | | | | | |
| 4.01 | 6.82 | | | | | |

| | | | | | | |
|---|---|---|---|---|---|---|
| 3.41 | 0.1 | | | | | |
| 4.02 | 6.82 | | | | | |
| 3.42 | 0.1 | | | | | |
| 4.03 | 6.82 | | | | | |
| 3.42 | 0.1 | | | | | |
| 4.04 | 6.82 | | | | | |
| 3.43 | 0.1 | | | | | |
| 4.05 | 6.82 | | | | | |
| 3.43 | 0.1 | | | | | |
| 4.05 | 6.82 | | | | | |
| 3.44 | 0.1 | | | | | |
| 4.06 | 6.82 | | | | | |
| 3.45 | 0.1 | | | | | |
| 4.07 | 6.82 | | | | | |
| 3.45 | 0.1 | | | | | |
| 4.07 | 6.82 | | | | | |
| 3.46 | 0.1 | | | | | |
| 4.08 | 6.82 | | | | | |
| 3.47 | 0.1 | | | | | |
| 4.09 | 6.82 | | | | | |
| 3.47 | 0.1 | | | | | |
| 4.09 | 6.82 | | | | | |
| 3.47 | 0.1 | | | | | |
| 4.09 | 6.82 | | | | | |
| 3.48 | 0.1 | | | | | |
| 4.1 | 6.82 | | | | | |
| 3.48 | 0.1 | | | | | |
| 4.1 | 6.82 | | | | | |