GEOTECHNICAL INVESTIGATION ON CAUSES OF ROAD FAILURE OF DODOLA –GOBA ASPHALT ROAD PROJECT AND THEIR REMEDIAL MEASURES

MSc IN GEOTECHNICAL ENGINEERING

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HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA

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A THESIS SUBMITTED TO THE DEPARTMENT OF CIVIL ENGINEERING, FACULTY OF CIVIL ENGINEERING AND BUILT ENVIRONMENT, SCHOOL OF GRADUATE STUDIES HAWASSA UNIVERSITY.

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ADVISORS' APPROVAL SHEET

This is to certify that the thesis entitled "Geotechnical Investigation on Causes of Road Failure of Dodola –Goba Asphalt Road Project and Their Remedial Measures" submitted in partial fulfillment of the requirements for the degree of Master's with specialization in Geotechnical Engineering, the Graduate Program of the School of Civil Engineering, and has been carried out by Bereket Debebe Eshete, Id. No GPGeotR/007/11, under our supervision. Therefore, we recommend that the student has fulfilled the requirements, and hence here can submit the thesis to the school.

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We, the undersigned, members of the Board of Examiners of the final open defense by Bereket Debebe Eshete have read and evaluated his thesis "Geotechnical Investigation on Causes of Road Failure of Dodola –Goba Asphalt Road Project and Their Remedial Measures", and examined the candidate. This is, to certify that the thesis has been accepted in partial fulfillment of the requirement for the degree.

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DECLARATION

I certify that research work titled "Geotechnical Investigation on Causes of Road Failure of Dodola –Goba Asphalt Road Project and Their Remedial Measures" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

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LIST OF ABBREVATIONS

AASHTO	American Association of State Highway Transportation Officials
AC	Asphalt Concrete
ASTM	American State of Testing and materials
CBR	California Bearing Ratio
cm	Centimeter
ERA	Ethiopian Roads Authority
GB1	Graded crushed stone
GB2	Natural gravels
GB2A	Dry-bound and Water-bound Macadam
GB3	Weathered rocks
GS	Granular Sub-Bases
HMA	Hot mix asphalt
Km	Kilo meter
LL	Liquid Limit
MDD	Maximum dry density
mm	Millimeter
NMC	Natural Moisture Content
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plastic Limit
PM	Plasticity Modulus
PP	Plasticity Product
PSD	Particle Size Distribution
RSDP	Road Sector Development Program

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ABSTRACT

Dodola – Goba asphalt road, found in South East Ethiopia, was prematurely failed before serving its design life. From total stretch (130km) of the project, this study focus and detail investigations has been carried out on Adaba -Dinsho road section which was failed critically within 3 (three) years after it has been opened for traffic. A number of maintenance had been performed to make the road passable, but each maintained sections are failing with in short period. In this study, the real cause of the road failures has been identified and proper remedial measurements for failed section has been proposed. To asses this, field and laboratory investigations which includes visual inspection, field test, physical measurements and laboratory tests to determine geotechnical properties of sub grade, sub base and base course materials by comparison of the materials quality as per ERA 2014 and/or AASHTO design manuals were carried out. Finally based on the investigation, presence of too much moisture content on sub grade and pavement materials due to improper design and late maintenance of drainage, improper geometrical design, moisture sensitive geology/soil and seasonal variation of ground water are the major factors of failures. Also poor geotechnical properties of road base and sub base of materials due to substandard material quality and improper construction methods are the major causes of road failures. Correcting drainage system and geometric design of the road and reconstruction of pavement layers by standard material quality and methods are the proper remedial measure for failed section.

Key Words: Road Failures, Geotechnical Properties, Remedial Measurement

1. INTRODUCTION

1.1. Background

Road network is crucial infrastructure for the economic development of Ethiopia. Ethiopia's economic growth is highly dependent on the agricultural sector and the agricultural sector by itself need road for transportation of products to market centers. Therefore, development efforts to change the existing socio-economic condition of the nation would also be dependent on the efficiency of this road sector for the foreseeable future (ERA, 1997). The road network has been identified as a serious bottleneck for the economic development of the country. A better performance of the agricultural sector and the sustainable economic growth of the country at large would be achieved through an improvement of the basic road infrastructures (ERA, 2016).

Dodola –Goba road project was one of the main trunk road projects included in the Ethiopian government’s 10 years Road Sector Development Programmed (RSDP,1997- 2007) and financed by World Bank, as a major financer of RSDP, in a series of financing program, called Adaptable Program Lending (APL) (ERA, 2003).

The road is found in Oromia regional state, between Bale and Western Arsi zones which starts from the out strict of Dodola town passed through Adaba, Dinsho, and Robe town and terminates at the outskirt of Goba town (Togona River). The road was constructed by an international contractor called CGC Overseas International Construction limited while the consultancy services for the construction supervision of the project road was provided by a local firm called Transport Construction Design Share Company (TCSDCo) on 08 July 2006 commencement date and completed on July 2012. The scope of works was upgrading of the existing gravel road to double asphalt concrete road 130km length between Dodola junction –Goba town that part of the Nazereth –Asela –Dodola and Shashemene- Goba road project (TCDSCo, 2006- 2012).

Currently the road is not functional due to rapid failures on surface after opened to traffic, especially Adaba to Dinsho road section that has failed severely. This failure most significantly, causes road traffic accident that had resulted in to loss of lives and properties and a very serious problem that causes unnecessary delay in traffic flow, distorts pavement aesthetics, damages of vehicle.

If the real cause of failure is determined, it reduces the cost of maintenance. The usual way of maintenance, without investigation, failures may be under estimated and the maintenance recommendation may not solve the problem at all or may be over-estimated and lead to extravagancy (ERA, 2013c).

The present study is conducted in order to identify the causes of road failures and to propose proper remedial measures for failed road sections.

1.2. Statement of the Problem

Construction and maintenance of roads is not simple investment. It requires a huge amount of expenditure with foreign currency. In Ethiopia failures of roads are being observed before their design period and are greatly affecting the economic growth of the country (ERA, 2013c). Addis Ababa – Jimma (Awoke Sime, 2005) and Addis Ababa – Mojo (Asnake Abate, 2015) are, examples of roads which were failed after few years of construction.

Dodola - Goba road was one of recently constructed road and have reported to road failure rapidly after opened to traffic. The road was expected for 15 (fifty) years' service life time and to be make major contribution on sustainable development of the country by improving the level of transportation services to the rural areas and to support transporting of agricultural products, distribution of inputs, improved social services and reduce vehicle operating costs (TCDSCo, 2006- 2012).

From total stretch of Dodola –Goba road project, Adaba -Dinsho road section was failed within 3(three) years after it has been opened for traffic and a number of maintenance had been performed to make the road passable however; each maintained section fail again within three to four months Currently this road section has failed severely and results accidents that have caused considerable damages to human lives and properties. In addition to the assets and lives lost due to crash and damage of vehicle spares, the route is known for numerous cases of armed robbery attacks, owing to slow movement of the plying vehicles causes from road failures, as we hear from broadcasts. This has reduced the movement of people and goods, thus reducing the rate of socio-economic growth and development in the corridor.

Therefore, this study concern and detail investigations has been carried out on Adaba – Dinsho road sections to identify the causes of road failures and to propose proper remedial measure for this section.

1.3. Objectives

1.3.1. General Objectives

The general objective of this research work is undertaking detail geotechnical investigations to identify the causes of road failures on Dodola – Goba asphalt road project for severely failed Adaba –Dinsho road section and proposing proper remedial measures to solve the problem.

1.3.2. Specific objectives

The specific objectives of this research

- To identify the major failure types on road section and assess impact of the topography, geology and drainage conditions.
- To determine the geotechnical properties of sub grade, sub base, and base course materials and compare with standard specification of the materials.
- To identify the real causes of road failure based on the findings.
- To Evaluate the performance of maintenance methods previously done on failed road sections
- To propose the possible remedial measures for failed road sections.

1.4. Research Questions

This study will be conduct to answer the following research questions:

- What are major types of failures on this road section?
- What are the impacts of the topography, geology and drainage conditions on this road section?
- What are the geotechnical properties of sub grade, sub base, and base course materials this failed sections?
- What are the real causes of road failure?
- What are the performance of maintenance methods which done previously?
- What are the possible remedial measures for this failed road sections?

1.5. Significance of the study

Proper understanding of the causes of road failure of the study area lead to proper remedial measures and suggests the appropriate maintenance strategy to solve the problem by rational maintenance cost without over estimated cost or extravagancy. This in turn will be helpful for the people using this road in particular by increasing quality, safety and durability of the road with attainable design life by reducing the maintenance and vehicle running costs.

Additionally, identifying the real causes of road failures of this study area lead to serve experience by designing, construction and maintenance methods for new under construction roads linked with this study area i.e. Bale Robe- Ginnir and Ali – Arsi Robe, and have similar nature of topography, geology and climate condition with the study area. Even if the study will be very specific area in the country that is on Dodola- Goba road project, Adaba- Dinsho section, the results obtained from this research could be useful for studying the complex problems in general, or in the particular areas of different road networks. In addition, the study has used as a reference to those researchers who want to conduct studies for the future detailed on road failure and other related issues.

1.6. Scope of the Study

This research study has to be focused and detailed investigations has been carried out on the most critically failed road section of Adaba to Dinsho town, from total stretch of 130 km Dodola –Goba road project road. The study will start by field investigation to identify the major types of failures and with their respective cause. Detail field and laboratory investigations were carried out to identify the real cause of failures. On field investigation, visual inspection on actual condition of failed road with regards of geometry/design, geology, topographical and drainage characteristics of the road and physical measurements and sampling were carried out. Then laboratory investigations were conducted for geotechnical properties on sampled materials i.e. sub grade, embankment, sub base and base course of failed stations. Finally, it will suggest the most appropriate measure that should be taken based on the Ethiopian roads authority maintenance and rehabilitation guidelines. Total Six (6) sample pits of failed sections are selected to approximate the entire two failure sections of the road. It is important to use result of this research by taking these limitations into consideration.

The limitations may broadly include: -

- Although causes of failure include many issues, this paper has focused on material engineering properties.
- The investigation of material characteristic focuses on materials of subgrade/embankment, sub-base and base coarse.

1.7. Organization of the Thesis

This thesis has been organized and presented in five Chapters. In the first Chapter a general description on the pavement performance and affecting factors. Furthermore; the back ground and objective have also been discussed in this introductory part of the thesis. The second Chapter gives a brief literature review which discusses about general pavement classification, distress types, classification, causes of distress relative to Engineering properties material properties used for construction and impact of the topography, geology and drainage conditions. And also countermeasures to be taken to overcome. The third chapter presents a description of the study area as well as materials and methods of the study. Chapter four includes the field and laboratory investigation results, analysis and discussion. In this chapter, the different laboratory test results have been thoroughly conducted and analyzed. Finally, conclusions and recommendations drawn from the research are discussed in chapter five. Following chapter five, reference materials used in this thesis work are appropriately sited and listed. The thesis ends with appendices which contain detail experimental results of laboratory investigation and field investigation.

2. LITERATURE REVIEW

2.1. General

Flexible pavements embraced of several layers of carefully selected materials designed to gradually distribute loads from the pavement surface to the layers underneath material (Adlinge and Gupta, 2009). In flexible pavement, it is typically a multi-layered system comprising the sub-grade (support), sub-base, base and surfacing. Its principal function is to receive load from the traffic and transmit it is through the layer to the sub grade (Adetoro and Adams, 2014).

Road pavements are intended to limit the stress created at the subgrade level by the traffic traveling on the pavement surface, so that the subgrade is not subject to significant deformations. In effect, the concentrated loads of the vehicle wheels are spread over a sufficiently larger area at subgrade level. At the same time, the pavement materials themselves should not deteriorate to such an extent as to affect the riding quality and functionality of the pavement. Pavements do deteriorate, however, due to time, climate and traffic. Therefore, the goal of the pavement design is to limit, during the period considered, deteriorations which affect the riding quality, such as, in the case of flexible pavements, cracking, rutting, potholes and other such surface distresses to acceptable levels (ERA, 20013a).

All roads deteriorate with time as a result of traffic and environmental effects. The deterioration may be relatively easy to correct or may require major maintenance works, depending on the causes and extent of deterioration (ERA, 2013c). A pavement failure is defined in terms of decreasing serviceability caused by the development of surface distresses such as cracks, potholes and ruts (Adetoro and Adams, 2014). Road pavement failure may be considered as structural, functional, or materials failure, or a combination of these factors. They described structural failure is the loss of load carrying capability, where the pavement is no longer able to absorb and transmit the wheel loading through the structure of the road without causing further deterioration. Functional failure is a broader term, which may indicate the loss of any function of the pavement such as skid resistance, structural capacity, and serviceability or passenger comfort. Materials failure occurs due to the disintegration or loss of material characteristics of any of the component materials (Woods W. and Adcox A., 2004).

Deterioration of highway pavement is a very serious problem that causes unnecessary delay in traffic flow, distorts pavement aesthetics, damages of vehicle and most significantly, causes road traffic accident that had resulted into loss of lives and properties (Merwe and Ahronovitz, 2005). Pavement surface deformation affects the safety and riding quality on the pavement as it may lead to premature failures.

Subgrade conditions, environmental conditions, properties of construction materials, traffic loading, lack of proper drainage and poor workmanships are factors that credited to cause of road failures (Jegede,2004). The performance of pavement influenced by unsuitable subgrade material, lack consideration of traffic loading, uncontrolled moisture, lack of construction quality and lack of proper maintenance (Adlinge and Gupta, 2009).

This chapter discusses literatures by different authors and researchers on, factors affecting pavement failure, type of pavement failures and failure mitigation measures.

2.2. Factors Influencing the Performance of the Road

2.2.1. Heavy traffic loading

Traffic load is the total load that the pavement will carry within the design life and a major factor in pavement design. Traffic loading greatly affects the performance of an asphalt pavement (AASHTO, 2001).

The magnitude of the individual wheel loads and the number of times these loads are applied caused by traffic will result of road pavement deterioration. The traffic or load carrying ability of an asphalt pavement is a function of both the thickness of the pavement materials and its stiffness. Traffic is the most important load factor influencing road performance (ERA, 2013a). If the performance of pavement reduced by traffic, water may seep through underneath layers and make the layers unstable.

Traffic is the most important factor influencing pavement performance. Loading magnitude, configuration and the number of load repetitions by heavy vehicles are the most factors which influence the pavement performance. The damage caused per pass to a pavement by an axle is defined relative to the damage per pass of a standard axle load, which is defined as an 80 KN single axle load. Thus a pavement is designed to withstand a certain number of standard axle load repetitions that will result in a certain terminal condition of deterioration (Adlinge and Gupta, 2009).

P. Croney, and D. Croney (1998) reported that fatigue cracking and deformations of pavements are the major defects caused by heavy traffic due to the increased traffic overloading which is more than the design load. Deterioration of pavements arises from deformation generally associated with cracking under heavy commercial vehicles.

Omer et al. (2014) observed that pavement failures on west lane of the ring road that might have been caused by the movement of heavy loaded truck trailers, tippers, as well as loaded fuel tankers. Road surfaces often wear under the action of traffic, particularly during the very early life of the road. However, the action of traffic continues to wear the surface texture and thus gradually reduces the high speed skidding resistance (Oguara, 2010). He also reported that with the increase of traffic loads (volume and axle loads) the road network was experiencing a deterioration equivalent to a loss of billions dollars due to road deterioration and vehicle operating cost.

2.2.2. Environmental Variation

Environment has great impact on material selection and thickness design of a pavement. The two critical areas of the environment that cause pavement failure are temperature and water/rainfall. Temperature affects the selection of which grade asphalt binder that would be used in the asphalt pavement. Asphalt pavements are susceptible to damage by water. Water increase moisture in the pavement and reduce bearing capacity, it saturates subgrade or base of an asphalt pavement and causes structural damage to pavement in climates that have extensive rainfall. The best prevention to structural damage due to provision of water is proper drainage. Inspection and cleaning of drainage system insures that they are working properly and will eliminate some of major causes of pavement failure (AASHTO, 2001).

Wee et al. (2009) reported that, climatic changes in temperature and rainfall can interact together. Rainfall can alter moisture balances and influence pavement deterioration while the temperature changes can affect the aging of bitumen which causes the surface to crack, with a consequent loss of waterproofing of the surface seal.

Climate is one of the major factors that affect design and performance of pavements. Subgrade soils being beneath the pavement structure are most susceptible to moisture variation due to their exposure to precipitation, impaired drainage and fluctuation of the ground water table. Moreover, the strength and deformation of fine-grained soils is highly influenced by the water content in the soil. In cold climate regions, temperature also plays a critical role in the freezing and thawing of soils in roads. This phenomenon affects the performance of pavements in two ways. First, the volume to the soil generally increases during freezing. Eventually, this process may cause a non-uniform heave on the pavement surface. Second, the water generated during thawing period reduces the strength of subgrade soils, resulting in a reduction of bearing capacity of roads (Yesuf, G.Y, 2014).

Rainfall if allowed to penetrate the pavement structure or roadbed soil will influence the properties of those materials. During the heavy rainy seasons, it expected that some seasonal variation in bearing capacity would occur. Thus, the resilient modulus of road bed soil changes with seasonal variation, even when there is no freezing and thawing (AASHTO, 1993). In addition to the effect of freeze, thaw, and seasonal variation on sub grade and granular materials, temperature will also influence the characteristics of the asphalt concrete. Cracking, fatigue cracking and rutting are major failures type caused by environmental variations (AASHTO, 1993).

2.2.3. Poor Drainage and Excess Moisture

Poor design or maintenance of side and cut-off drains and cross drainage structures are often result localized pavement failures. When side drains and culverts silt up, water ponds against the road embankment, eventually weaken the lower pavement layers. Conversely, if the water velocity in the side drain is too high it erodes the road embankment and shoulders. More general failures occur when there is no drainage within the pavement layers themselves. Paved roads do not remain waterproof throughout their lives and, if water is not able to drain quickly, it weakens the lower pavement layers and results in rapid road failure. However, pavement deterioration as a result of poor drainage may not be obvious in the dry season hence discussions with local people may be necessary to establish the situation in the wet season (ERA, 2013c).

Drainage is the single most important factor governing the performance of road substructure Li (2004). He also concludes that properly functioning drainage system provides the following, 1) intersects the water seeping up from the subgrade. 2) Diverts the surface water flowing toward the track. 3) Removes water falling onto the track. 4) Carry off stone dust, sand, and other debris that otherwise could foul the road.

Patil Abhijit et al. (2011) investigated the effect of poor drainage on road pavement condition and found that the increase in moisture content decreases the strength of the pavement. Therefore, poor drainage causes the premature failure of the pavement. The highway drainage system includes the pavement and the water handling system which includes pavement surface, shoulders, drains and culverts. These elements of the drainage system must be properly designed, built, and maintained. When a road fails, inadequate drainage often is a major factor. Poor design can direct water back onto the road or keep it from draining away. Too much water remaining on the surface combine with traffic action may cause potholes, cracks and pavement failure. Inadequate drainage leads to major cause of pavement distress due to large amount of costly repairs or replacements long before reaching their design life. Drainage design for pavement is to keep the base, sub-base, subgrade, and other susceptible paving materials from becoming saturated or even being exposed to constant high moisture levels over time (Patil Abhijit et al, 2011).

Poor drainage also causes early pavement distresses leading to driving problems and structural failures of road as pointed out by researchers (TIZA, 2016). Water migrates into the pavement structure through combinations of surface infiltration (e.g., through cracks in the surface layer), edge inflows (e.g., from inadequately drained side ditches or inadequate shoulders), and from the underlying groundwater table (e.g., via capillary potential in fine grained foundation soils). It reduces the strength and stiffness of the unbound pavement materials, promotes contamination of coarse granular material due to fines migration, and can cause swelling (e.g., frost heave and/or soil expansion) and subsequent consolidation (FHWA, 2016). Moisture can also introduce substantial spatial variability in the pavement properties and performance, which can be manifested either as local distresses, like potholes, or more globally as excessive roughness (FHWA, 2016).

A research by, Awoke Sime (2005) found that water from rain or/and water from showering of pavement layers due to poor drainage condition is a major causes of Addis Ababa – Jimma road failure.

Moisture can significantly weaken the support strength of natural gravel materials, especially the subgrade. Moisture can enter the pavement structure through cracks and holes in the surface, laterally through the subgrade, and from the underlying water table through capillary action. The result of moisture ingress is the lubrication of particles, loss of particle interlock and subsequent particle displacement resulting in pavement failure (Adlinge and Gupta, 2009).

Moisture is one of the main factors which affects the strength and stiffness of a subgrade. It also controls the ease of compaction (density) and the compressibility and swelling/shrinkage characteristics of subgrade soils. There are numerous ways through which water can percolate and affect the moisture content of the subgrade. For example, seepage from higher ground, either along the pavement or within cuts can cause fluctuations in subgrade moisture conditions. In order to determine the possible consequences, the sensitivity of the subgrade strength and stiffness to changes in moisture content should be evaluated during or before design (ERA, 2013b).

Generally, in sandy and gravelly soils, small fluctuations in water content produce little change in their strength and stiffness. In silty soils, however, any fluctuation in water content can bring about a certain change in volume, and may also produce large changes in strength and stiffness. Typically, these soils attract and retain water through capillary action, and do not drain well. The change in water content in clays often results in large variations in volume, and there may be large changes in strength and stiffness too, particularly if the moisture content is near or above optimum. Typically, these soils attract and retain water through matrix suction (ERA, 2013b).

2.2.4. Poor Design and Improper Construction Methods

A road section for which a pavement design is to undertaken should be subdivided into sub grade areas, and soils investigations should then delineate sub grade design units on the basis of geology, drainage conditions and topography, and consider soil categories which have fairly consistent geotechnical characteristics (e.g. grading, plasticity, CBR) (ERA, 2013a). Thus, topography is another controlling criterion in road design. Horizontal and vertical alignments designed based on the actual topography of the route. Topography data obtained by field ground survey should be taken in consideration during design to ensure that alignment does not run parallel to major drainages, which may result in channel changes, extra culvert lengths due to skews, etc.

The design problems are the one cause of immediate road failure. Poor design can often be traced back to inappropriate cost-cutting during the initial design and installation, either because the property owner insisted on cheap options, the engineer didn’t know enough to put in the right materials, or the construction company cut corners where they shouldn't have (Greene, 2016). However, a deficit in structural integrity can usually overcome by adding thickness (Greene, 2016).

A research study by Hagos (2006) indicated that failure on Addis Ababa-Nekempt road is due to poor workmanship and design problem. It is shown that the main reason for the failures of the Addis Ababa-Nekempte road is not the red clay soil used as a subgrade, but failure is due to the improper use of the overlaying materials and poor construction. The base and sub base layer thicknesses used at all sections are too thin to support the traffic loading. From the design the required depth for base is 20cm, but 11.4cm thickness have been on the roads with equivalent traffic loading as Addis Ababa-Nekempte such as some part of Addis Ababa- Jimma and Addis Ababa-GohaTsion, no failure is observed as these roads are constructed on red clay soils with adequate layer thickness and proper construction.

The design of the geotechnical aspects of pavements must consequently focus on the selection of moisture-insensitive free-draining base and sub base materials, stabilization of moisture-sensitive sub grade soils, and adequate drainage of any water that does infiltrate into the pavement system (ERA, 2013b).

Failure to obtain improper compaction, improper moisture conditions during construction, quality of materials, and accurate layer thickness (after compaction) all directly affect the performance of road and its pavement. These conditions stress the need for skilled staff and the importance of good inspection and quality control procedures during construction (TRL, 1988).

A major objective in pavement design is to prevent the base, sub base, sub grade, and other susceptible paving materials from becoming saturated or even exposed to constant high moisture levels in order to minimize moisture-related problems (FHWA, 2016). The three approaches for controlling or reducing the problems caused by moisture are; prevent moisture from entering the pavement system, use materials and design features that are insensitive to the effects of moisture, quickly remove moisture that enters the pavement system.

During road design when water table is close to the sub-grade finished level, it is appropriate to consider whether moisture content will reach saturation and the design strength must then be based on this assumption by testing samples compacted to the target density at optimum moisture content but then tested after a 94 hours period of soaking (ERA,2013a).On the other hand, when the water table is deep, but rainfall can influence the sub-grade moisture content under the road a conditions will occur when rainfall exceeds evaporation and transpiration for at least two months of the year (ERA, 2013a), the moisture condition for design purposes can be taken as the optimum moisture content given by ASTM Test Method D 698 (ERA, 2013a).This represent the majority of Ethiopia which is having a rainfall greater than 250 mm per year and is seasonal (ERA, 2013a).

2.2.5. Low Quality Construction Material

The use of low quality materials for construction adversely affects the performance of the road. This sometimes occurs in the form of the improper grading of aggregates for base or sub-base and poor subgrade soil of low bearing strength. The use of marginal or substandard base materials for pavement construction will affect pavement performance (Rollings, 1998). He found that these materials may accelerate deterioration of the pavement and often result in rutting, cracking, shoving, raveling, aggregate abrasion, low skid resistance, low strength, shortened service life, or some combination of these problems.

Osuolale et al. (2012) investigated the possible causes of highway pavement failure along a road in south western Nigeria. He stated that the materials used as sub-base have the geotechnical properties below the specification and this is likely to be responsible for the road failure.

The choice of materials used for the construction of pavement layers may also cause road deterioration. This is due to inherent variability in the materials used for road construction in terms of soil properties such as strength or load bearing capacity, gradation, mix properties, elastic and resilience modulus. Poor choice of materials used for pavement layers can have a drastic effect on the strength of the layers and their subsequent performance (TRL, 1988).

2.2.6. Lack of Road Maintenance Standards

The rate of pavement deterioration is directly affected by the maintenance standards applied to repair road defects. When a maintenance standard is defined, it imposes a limit to the level of deterioration that a pavement is allowed to attain. Low maintenance standard therefore causes roads to deteriorate at a faster rate (TRL, 1988).

Road performance also depends on what, when, and how maintenance performed. No matter how well the road built, it will deteriorate over time based upon different factors. The timing of maintenance is very important, if road is permitted to deteriorate to a very poor condition, postponed maintenance because of budget constraints, will result in a significant financial penalty within a few years (ERA, 2013c).

2.3. Major Geotechnical Problems Encountered on Road Projects

One of the most common reasons for pavement failure in Ethiopia is caused by underlying geotechnical problems. Pavements primarily designed to protect the underlying and weaker layers from failure caused by traffic-induced stresses. Even if such structural design of pavements can cope with structurally weak sub grades, it is necessary to assume that the sub grade soils are inherently stable and not moving in mass through land-sliding processes or through large scale settlement. Problems of soil instability cannot be solved by the thickness design of pavement layers alone; solving such problems will inevitably require some form of geotechnical solution (ERA, 2013c). The geotechnical aspects are particularly important for longer pavement performance periods. The maintenance and rehabilitation activities used in the pavement management process to achieve the design performance period require a competent support from the underlying materials (ERA, 2013b).

Geotechnical components of a pavement system include surfacing aggregate, unbound granular base, unbound granular sub-base, the sub-grade or roadbed (either mechanically or chemically stabilized, or both), aggregate and geo-synthetics used in drainage systems, graded granular aggregate and geo-synthetic used as separation and filtration layers, and the roadway embankment foundation (John Poullain, 2012).

The geotechnical design of the pavement will involve additional special considerations in cut and fill areas and Attention must also be given to transition zones - e.g., between a cut and an at-grade section - because of the potential for non-uniform pavement support and subsurface water flow (FHWA, 2016). The main additional concern for cut sections is drainage, as the surrounding site will be sloping toward the pavement structure and the groundwater table will generally be closer to the bottom of the pavement section in cuts (FHWA, 2016). Stabilization of moisture sensitive natural foundation soils may also be required. The major geotechnical problems encountered on road projects are discussed below.

2.3.1. Problematic Subgrade Soil

The applied wheel loads supported by underlying subgrade material. If the subgrade is too weak to support the wheel loads, the pavement will loosen up excessively which ultimately causes the pavement to fail. If natural variations in the composition of the subgrade are not adequately addressed by the pavement design, significant differences in pavement performance will be experienced (Adlinge and Gupta, 2009).

The characteristics of the subgrade significantly affect the performance of a road pavement surface. Strength, stiffness, good drainage, compaction, and compressibility and swelling are necessary properties for subgrade performance (ERA, 2013b). Subgrade materials and conditions are unique for each project which be governed by variables such as soil type and clay mineralogy along a length of a roadway, the geology (soil genesis), climate and topography (ERA, 2013b).

Expansive, compressible, collapsible and dispersive are most common problematic subgrade soils in region of Ethiopia.

2.3.1.1. Expansive Soils

Expansive soils are soils which exhibit significant volume changes in the presence of water are termed as expansive soils. However, typical damage to roads on expansive soils includes longitudinal unevenness and bumpiness, differential movement near culverts, and longitudinal cracking (ERA, 2013b). Local expansion may also occur from poor drainage. In addition, cut sections may lead to local heaving due to removal of surcharge and subsequent increase of moisture content. The most widely used indicators for identification of expansive soils are the index properties that are routinely determined by most road agencies. Experience has shown that the volume change behavior correlates reasonably well with liquid limit, plasticity index, and shrinkage limit. In most cases, a combination of observed Atterberg limits and prior experience with materials within a given area could be the main identification methods used for expansive soils.

Using highly expansive soils as embankment fill material and disturbance created during the rehabilitation work together with the poor drainage condition are the major causes of Addis Ababa – Jimma road failure (Awoke Sime, 2005). Similarly, major cause of road failure constructed on expansive soils from Addis Ababa to Nekempte is mainly due to the expansive nature of the subgrade material (Hagos,2006).

2.3.1.2. Compressible Soils

Compressible soils are soil deposits which are susceptible to large settlements and deformations because of a relatively rapid decrease in void volume upon loading. If compressible soils are not treated, large surface depressions with random cracking can develop. The surface depressions can allow water to pond on the road surface and readily infiltrate the pavement structure, potentially creating further problems (ERA 2013b).

2.3.1.3. Collapsible soils

Collapsible soils are those that appear to be strong and stable in their natural (dry) state, but which rapidly consolidate under wetting, generating large and often unexpected settlements. On pavements, the result of collapse of the subgrade is mostly manifested by the development of a deeply rutted and often uneven road surface and significant deterioration of the riding quality of the road with or without cracking (ERA, 2013b). The potential for collapse at a specific location is initially evaluated based on the geological and environmental setting. The subsurface conditions are then evaluated by ground investigation, for example using power auger borings, trial pits and open excavations. These may be supplemented by boreholes that can extend to depths sufficient to define the thickness of the collapsible soil. If a pit is excavated, then filling the hole with the same amount of material can indicate whether the soils are collapsible, especially if the difference in levels is high after tamping and leveling.

2.3.1.4. Dispersive soils

Dispersive soils are those soils that, when placed in water, have repulsive forces between the clay particles that exceed the attractive forces. This results in the colloidal fraction going into suspension and, in still water, staying in suspension. In moving water, the dispersed particles are carried away or eroded (ERA, 2013b). Identification of dispersive soils should start with field investigations to determine if there are any surface indications such as unusual erosional patterns with sub-soil pipes and deep gullies, concurrent with excessive turbidity in any nearby bodies of water. Areas of poor crop production and stunted vegetation may indicate highly saline or carbonate-rich soils, many of which are dispersive. However, dispersive soils can also occur in neutral or acidic soils and can support lush grass growth (ERA, 2013b).

2.3.2. Slope Instability and Land Slide

Many roads in Ethiopia are constructed in mountainous or hilly areas. As a result, slopes that have been stable for many years may fail because of changes in stress conditions or the effects of road construction on drainage. Unstable natural slopes and road cuts often create a considerable problem to road users in Ethiopia during the rainy season. These unstable slopes or landslides typically occur where a natural slope is over-steep or where cut slopes in weathered rock and soil encounter groundwater (ERA, 2013b).

In Ethiopia, landslides often occur after periods of prolonged and heavy rainfall. Often, this is most likely to happen during the wet season between July and the end of September. In the southern part of the country, this period may extend into October and November as severe and localized rainfall can occur during this time (ERA, 2013b). (Kifle, 2014) was conducted the study on 54 road section which are affected by land slide in Ethiopia and he conclude that land slide in Ethiopia will continue to challenge on road and other infrastructure development.

The susceptibility of a slope to the occurrence of landslides is often related to an increase in shear stress (driving force) or a reduction in shear strength (resisting force). The processes that increase the shear stress or decrease the shear strength in a slope are normally termed as landslide preparatory or triggering factors. Preparatory (pre conditioning) factors make the slope increasingly susceptible to failure without actually initiating movement. Triggering factors are events that finally initiate movement. Thus geological and topographical parameters are usually regarded as landslide preparatory factors. Triggering factors include rainfall, earthquakes, toe erosion and man-made activities (ERA 2013 b).

2.4. Types of Pavement Failures

According Adlinge and Gupta (2009), Pavement deterioration is the process by which distress (defects) develop in the pavement under the combined effects of traffic loading and environmental conditions and asphalt pavement surface distresses categorized as either cracking, surface deformation, disintegration or surface defects. Cracking distress includes alligator, longitudinal, transverse edge and reflective cracking. Surface deformation is the result of weakness in one or more layers of the pavement that has experienced movement after construction and may be accompanied by cracking. Surface distortions can be a traffic hazard and basic types of surface deformation are including rutting, corrugations, shoving, depression and swell distresses. Disintegration is progressive breaking up of the pavement into small, loose pieces. If the integration is not repaired in its early stages, complete reconstruction of the pavement may be needed. The two most common types of disintegration are potholes and patches. Surface defects are related to problems in the surface layer and most common types of surface distress are raveling and bleeding.

Caltrans (2001), main types of pavement failures categorized as either deformation failures or surface texture failures. Deformation failures include corrugations, depressions, potholes, rutting and shoving. These failures may be due to either traffic (load associated) or environmental (no-load associated) influences. It may also reflect serious underlying structural or material problems that may lead to cracking. Surface texture failures include bleeding, cracking, polishing, stripping and raveling. These failures indicate that while the road pavement may still be structurally sound, the surface no longer performs the function it is designed to do, which is normally to provide skid resistance, a smooth running surface and water tightness. Other miscellaneous types of pavement failures include edge defects, patching and roughness.

Major types of type of road failure with respective possible causes are discussed as below

Fatigue cracking (Alligator cracking): This is a series of interconnected cracks creating small, irregular shaped pieces of pavement. It is caused by failure of the surface layer or base due to repeated traffic loading (fatigue). Eventually, the cracks lead to disintegration of the surface. The final result is potholes. Alligator cracking is usually associated with base or drainage problems. Small areas may be fixed with a patch or area repair. Larger areas require reclamation or reconstruction. Drainage must be carefully examined in all cases (Adlinge and Gupta, 2009).

The possible cause of fatigue or alligator cracking associated usually with base or drainage problems, decrease in pavement load support, stripping on the bottom of the HMA which decrease effective HMA thickness, increase in loading more than design load, poor construction e.g. inadequate compaction. Small, localized fatigue cracking is indicating a loss of sub-grade support and large fatigue cracked areas are signs of general pavement structural failure.

Crocodile cracking that is neither confined to the wheel paths nor associated with rutting is indicative of a fault in the construction of the surfacing. The more common production faults are poor particle size distribution, low binder contents, overheated bitumen and the use of absorptive aggregate. Construction faults include poor compaction, segregation of the mix and poor bonding, either between layers of bituminous material or the granular layer beneath. In these cases, the precise cause of failure can only be determined by destructive sampling and laboratory testing (ERA, 2013c).

Longitudinal cracking: Poor drainage, shoulder settlement, weak joints between adjoining spread of pavement layers or differential frost heave are major cause of longitudinal crack (Irshad Ahmad,2016). Longitudinal cracks are long cracks that run parallel to the center line of the roadway. These may be caused by frost heaving or joint failures or they may be load-induced. Multiple parallel cracks may eventually form from the initial crack. This phenomenon, known as deterioration, is usually a sign that crack repairs are not the proper solution (Adlinge and Gupta, 2009).

Transverse cracking: Transverse cracks form at approximately right angles to the centerline of the roadway. They are regularly spaced and have some of the same causes as longitudinal cracks. Transverse cracks will initially be widely spaced (over 20 feet apart). They usually begin as hairline or very narrow cracks and widen with age. If not properly sealed and maintained, secondary or multiple cracks develop, parallel to the initial crack. The reasons for transverse cracking, and the repairs, are similar to those for longitudinal cracking. In addition, thermal issues can lead to low-temperature cracking if the asphalt cement is too hard (Adlinge and Gupta, 2009). Shrinkage of the HMA surface due to low temperature or asphalt binder hardening or reflective crack caused by cracks beneath the surface layer major cause of traverse crack (K C Sethi, 2016).

Edge cracking: Edge cracks typically start as crescent shapes at the edge of the pavement. They will expand from the edge until they begin to resemble alligator cracking. This type of cracking results from lack of support of the shoulder due to weak material or excess moisture. They may occur in a curbed section when subsurface water causes a weakness in the pavement. At low severity the cracks may be filled. As the severity increases, patches and replacement of distressed areas may be needed. In all cases, excess moisture should be eliminated, and the shoulders rebuilt with good materials (Adlinge and Gupta, 2009).
K C Sethi (2016) described that the possible causes of edge crack are lack of support of shoulders due to weak materials or excess moisture, frost heave and inadequate pavement width also contributes towards formation of this type of distress.

Depressions: - Localized consolidation or movement of the supporting layers beneath surface course due to instability, inadequate compaction during construction or poor quality (soft) sub-grade (Irshad Ahmad,2016). Depressions are small, localized bowl-shaped areas that may include cracking. Depressions cause roughness, are a hazard to motorists, and allow water to collect. Depressions are typically caused by localized consolidation or movement of the supporting layers beneath the surface course due to instability. Repair by excavating and rebuilding the localized depressions. Reconstruction is required for extensive depressions (Adlinge and Gupta, 2009).

Localized depressions caused by settlement of the pavement layers, construction faults and differential movement at structures, particularly culverts, should be recorded. These are easy to see after periods of rain because they take longer to dry than the rest of the road. When the road is dry, they can sometimes be identified by the oil stains that occur where vehicles cross the depression. The depth should be measured using the 2-metre straightedge and calibrated wedge (ERA, 2012).

Potholes: Potholes are bowl-shaped holes. They are a progressive failure. First, small fragments of the top layer are dislodged. Over time, the distress will progress downward into the lower layers of the pavement. Potholes are often located in areas of poor drainage. Potholes are formed when the pavement disintegrates under traffic loading, due to inadequate strength in one or more layers of the pavement, usually accompanied by the presence of water. Most potholes would not occur if the root cause was repaired before development of the pothole. Repair by excavating and rebuilding. Area repairs or reconstruction may be required for extensive potholes (Adlinge and Gupta,2009).

According to Ahmed (2008), potholes are an indication of structural surface failure and they result from growth of a break in the surfacing, often as a result of severe alligator cracking. Once water enters pavement layers, the base and/or subgrade become wet and unstable, and the resultant degradation leads to rapid growth of pothole area and depth.

P. Sikdar et al (1999) reported that, if the potholes are numerous or frequent, it may indicate underlying problem such as inadequate pavement or aged surfacing requiring rehabilitation or replacement. Water entering pavement is often the cause, and could be caused by a cracked surface, high shoulders or pavement depressions ponding water on pavement, porous or open surface, or clogged side ditches.

Potholes are structural failures which include both the surfacing and road base layer. They are usually caused by water penetrating a cracked surfacing and weakening the road base. Further trafficking causes the surfacing to break up and a pothole develops. Because of the obvious hazard to the road user, potholes are usually patched as a matter of priority (ERA 2013c).

Rutting: The permanent downward deformation of the surface within wheel paths is called rutting. It may result from either deformation of the surfacing, the pavement materials or the underlying subgrade, or a combination of these. It is important to determine which layer is rutting since this will influence the optimal maintenance strategy. The worse level of rutting is the higher variation in the transverse profile of road surface. Because of this, ruts interfere with surface run-off patterns and increase the risk of wetting in the upper pavement layers. Rutting can also initiate aquaplaning, and hence have adverse impact on safety (Caltrans, 2001).

According to Adlinge and Gupta (2009), rutting is the displacement of pavement material that creates channels in the wheel path. Very severe rutting will actually hold water in the rut. Rutting is usually a failure in one or more layers in the pavement. The width of the rut is a sign of which layer has failed. A very narrow rut is usually a surface failure, while a wide one is indicative of a sub-grade failure. Inadequate compaction can lead to rutting. Deeper ruts may be shimmed with a truing and leveling course, with an overlay placed over the shim. If the surface asphalt is unstable, recycling of the surface may be the best option. If the problem is in the subgrade layer, reclamation or reconstruction may be needed.

K C Sethi, (2016) describe the cause of rutting is instability of the sub-grade, sub-base, base course or surface course or few of these pavement layers, inadequate compaction of the subgrade or any of the pavement layers, channelized movement of heavy wheel loads causing significant vertical stress on the sub-grade, improper design and specification of bitumen mix, inadequate thickness of the pavement or weak pavement structure. The width of the rut is a sign of which layer has failed.

Rutting is a load associated deformation or rutting appears as longitudinal depressions in the wheel paths. It is the result of an accumulation of non-recoverable vertical strains in the pavement layers and in the subgrade caused by traffic loads (ERA, 2013 c). In its early stages this type of rutting is not associated with shear failure (or shoving) in the upper layers of the pavement until it becomes very severe. Rutting can also be the result of shear failure in either the unbound or the bituminous pavement layers resulting in shoving at the edge of the road pavement. Where the shear failure is occurring in the unbound road base or sub-base the displaced material will appear at the edge of the surfacing. Where the failure is occurring in the bituminous material, the displaced material will be evident in the surfacing itself. Pavement failure due to inadequate support of upper layers, or to rutting, will usually necessitate complete pavement reconstruction, and not just the repair of the pavement surface where the problem is visible (ERA 2002).

2.5. Major Remedial Treatments

All roads deteriorate with time as a result of traffic and environmental effects. The deterioration may be relatively easy to correct or may require major works, depending on the causes and extent of deterioration (ERA, 2013c). Roads can fail in many different ways and the repairs that they require depend on the causes of deterioration. This is the guiding principle of pavement rehabilitation namely that ‘the repairs are determined by the cause or causes of the deterioration and the degree to which the deterioration has progressed’. Identifying these causes is therefore of paramount importance. Applying the wrong remedial treatment could be a waste of time and money. Thus the basic principle is to evaluate or assess the road to diagnose the cause (or causes) of deterioration and the severity of the deterioration so that the correct remedial treatment can be applied (ERA, 2013c).

An important element in the selection of rehabilitation treatments is the decision to either strengthen by overlaying or to choose some form of partial or full reconstruction. The evaluation procedure is designed to help with this decision, but there are several overarching principles that apply to these judgments (ERA 2013c). These are described briefly as below.

2.5.1. Treatment of deterioration and failure within the pavement structure

The types of deterioration that are occurring within the pavement must be addressed adequately. For example, one of the most common problems afflicting flexible pavements is cracking of the asphalt surface. The rapid reappearance (or reflection) of these cracks through an asphalt surface that is laid directly over them is a well-known phenomenon that must be solved as part of the rehabilitation process. If this is not done, the long-run consequences will be considerably more expensive. Another example concerns structural failures that are occurring (or have already occurred) deeper within the pavement. Ideally, rehabilitation should be carried out at the first signs of such failures in order to prevent their more widespread occurrence, but the point is that they must be treated; simply covering them up merely delays reconstruction that will inevitably become necessary. Deep patching is usually required and if too much deep patching is needed, then reconstruction is usually indicated (ERA 2013c).

Pavement failure due to inadequate support of upper layers, or to rutting, will usually necessitate complete pavement reconstruction, and not just the repair of the pavement surface where the problem is visible (ERA 2002).

2.5.2. Correcting drainage problems

If the drainage problems are serious they will normally have resulted in failures somewhere within the pavement. Such drainage problems must be addressed as part of rehabilitation. Sometimes this may require reconstruction to provide access to susceptible layers, even though this may not be strictly necessary to solve the particular problem resulting from the poor drainage itself (ERA 2013c).

2.6. Non-standard construction techniques, design and sub-standard materials

Quantifiable risks can only be based on options that have been tried before and for which there are precedents and an element of predictability. For example, if the properties of a road base or sub-base are found to lie outside any recognized specifications (or reliable research results that have not yet been incorporated into national standards and specifications) it is not possible to predict performance and to guarantee adequate pavement life. Under these circumstances, partial or full reconstruction to replace or modify the materials to meet specifications is the only option that can be considered. This category includes lack of adequate shoulders or other suitable edge support, which leads to shear failures along the outer wheel paths (ERA 2013c).

2.6.1. Thickness of overlays

Even if no serious failures have been experienced and overlaying is the logical method of strengthening, the required thickness can sometimes be excessive and it may be more cost effective to strengthen one or more of the existing layers of the pavement or to reconstruct completely (ERA 2013c).

2.6.2. High surface roughness, deformations and general unevenness

The high level of roughness observed on many roads is the result of many processes including settlement, poor maintenance work, poor re-instatement of utility trenches and poor initial construction where roads have literally simply 'evolved' over many years. Although some forms of deformation are indicative of soil instability and other forms of failure, there are also circumstances where the underlying structure is now stable. However, the nominal thickness of overlay required to correct these shape problems and the construction difficulties associated with so doing often make full or partial reconstruction a more economic option (ERA 2013c).

2.6.3. Utility trenches, particularly water pipes

Trenches for water pipes are often not backfilled and compacted properly leading to deformation and poor surface shape – in effect, very sub-standard bases and sub-bases have been introduced within a thin strip down the road. To correct this defect, the water pipes, need to be re-laid properly. This can be done without complete reconstruction, except if two trenches occur, one on either side of the road. In this case reconstruction may be the best option, especially if the water pipes have regularly leaked and created additional damage to the pavement structure (ERA 2013c).

2.7. Major Geotechnical Solutions

One of the most common reasons for pavement reconstruction in Ethiopia is pavement failure caused by underlying geotechnical problems. Pavements are primarily designed to protect the underlying and weaker layers from failure caused by traffic induced stresses. Although the structural design of pavements can cope with structurally weak subgrades, it is necessary to assume that the subgrade soils are inherently stable and not moving in masse through land-sliding processes or through large scale differential settlement. Problems of soil instability cannot be solved by the thickness design of pavement layers alone; solving such problems will inevitably require some form of geotechnical solution ERA (2013c).

Proper treatment of subgrade soils and the preparation of the foundation are important to ensure a long-lasting pavement that does not require excessive maintenance. The subgrade should be treated to form a construction pad or a long-term subsurface layer capable of carrying pavement applied loads (2013b).

ERA (2013), Geotechnical design manual recommends the techniques that can be used to improve the strength and stiffness of the subgrade and increase pavement performance described as below.

2.7.1. Moisture control

It is known that excess moisture has a damaging effect on pavement structures. This also applies to expansive and to a lesser extent, dispersive soils. Moisture, in combination with other factors, can have a profound negative effect on both material properties of the subgrade and the overall performance of the pavement. A major issue in geotechnical design of pavements is specially to prevent the subgrade from becoming saturated or even exposed to constant high moisture levels. The three main approaches for controlling or reducing the problems caused by moisture on pavements are:

- Prevent moisture from entering the pavement.
- Use materials and design features that are insensitive to the effects of moisture.
- Quickly remove moisture that enters the pavement.

Deep subsurface drains are usually installed to reduce groundwater levels. These drains intercept the lateral flow of subsurface water beneath the pavement structure, and remove the water that infiltrates the pavement surface. Various types of longitudinal roadside drains placed in trenches beneath shoulders at shallower depths can also be used to handle water infiltrating the pavement from above and the sides. In some cases, subsurface drainage may remove water, but may not significantly reduce the moisture content of fine grained soils in the subgrade. Drying should then be accomplished through evaporation of soil moisture at the time of construction. These methods are generally effective only in the top 200 to 300 mm of the subgrade, and are highly dependent on weather and environmental conditions.

2.7.2. Removal and Replacement

Removal of naturally occurring soil and replacing it with a suitable material is the most obvious method of eliminating many of the subgrade problems. In some cases, this approach may be economical if the thickness of the layer to be removed is less than about 1 to 2 m and suitable replacement material is available. Unfortunately, this is generally not the case, and the excavation and replacement solution is extended only to a depth which will reduce the subgrade problem to a tolerable minimum. Hence the required depth of excavation depends upon the suitability of the subgrade soil and the anticipated characteristics of fill that is available nearby.

Usually, the selection of appropriate material for replacing poor subgrade soils is a critical issue. Some road agencies use granular materials to replace unsuitable subgrade soils for structural and drainage reasons. Often a granular layer is used to provide uniformity and support as a construction platform. A thick mixed gravel/sand/silt layer may be used as an alternative to soil stabilization for subgrade improvement in areas with large quantities of readily accessible, good quality borrow material. Subgrade improvement is often the preferred way of dealing with weak and poorly drained soils compared to increasing the pavement layer thicknesses. The objectives and benefits of thick mixed gravel/sand/silt layers for subgrade improvement are to:

- Increase the supporting capacity of weak, fine-grained subgrades.
- Provide a minimum bearing capacity for the design and construction of pavements.
- Provide uniform subgrade support over sections with highly variable soil conditions.
- Reduce the seasonal effects of moisture and temperature on subgrade support.

2.7.3. Soil Stabilization

Soil stabilization is a general term that involves the use of mechanical or chemical modifiers to enhance the strength of soils and reduce the change in moisture. The process is often called soil modification when the purpose is to change the physical properties and thereby improve the quality of the subgrade soil. Soil stabilization is usually performed for the following reasons:

- As a construction platform to dry very wet soils and facilitate compaction of the upper layer. For this case, the stabilized soil is usually not considered as a structural layer in the pavement design process.
- To strengthen a weak soil and restrict the volume change potential of a highly plastic (expansive) or compressible soil. For this case, the modified soil is usually given some structural value in the pavement design process.

2.8. Pavement Materials

2.8.1. Subgrade Soils

Sub grade is the surface up on which the pavement structure and the shoulder are constructed. It is the portion of the natural soil, either undisturbed (but re compacted) local material in cut sections, or soil excavated in cut or borrows areas and placed as compacted embankment

(ERA, 2013a). The sub-grade is the underlying soil that supports the applied wheel loads. If the sub-grade is too weak to support the wheel loads, the pavement will flex excessively which ultimately causes the pavement to fail (AASHTO, 2001). In addition, if natural variations in composition of the sub-grade are not adequately addressed, significant differences in pavement performance will be experienced. Desirable properties that the subgrade should possess include high strength and stiffness, good drainage, eases of compaction, and low compressibility and swelling (ERA, 2013b).

The existing ground beneath embankments prior to their construction, existing level ground, or the ground at the bottom of cuttings prior to construction of pavement layers shall have a density of not less than 93% or 95% of the maximum dry density as required, determined in accordance with AASHTO T-180. In-situ material that is classified as suitable material but with a density of less than 93% or 95% of the maximum dry density, as required, determined in accordance with AASHTO T-180 shall be scarified to a minimum depth of 200mm that yields a minimum compacted layer thickness of 150mm, watered and re-compacted to the specified density (ERA, 2014).

Unsuitable material consists of peat and other organic materials, clay material having a liquid Limit (LL) exceeding 60 or a Plasticity Index (PI) exceeding 30 or CBR value less than 3% at 95% of modified AASHTO or a swell value of more than 3% ,other problematic soils such as expansive clays or collapsible sands or dispersive soils or saline soils or micaceous soils or low strength soils are removed shall be backfilled with approved imported material of CBR shall not be less than 5% at 95% of Modified AASHTO (T180) (ERA, 2014).

The AASHTO classification is given in AASHTO M145. It includes seven basic groups (A-1 to A-7) and twelve subgroups as table 2.1.

Table 2-1 AASHTO Soil Classification System (from AASHTO M 145 or ASTM D3282)

General Classif.n.	**Granular Materials (35% or less passing the 0.075 mm sieve)**							**Silt-Clay Materials (>35% passing the 0.075 mm sieve)**				
Group Classif.	A-1		A-3	A-2				A-4	A-5	A-6	A-7	
	A-1-a	A-1-b		A-2-4	A-4-5	A-2-6	A-2-7				A-7-5	A-7-6
2.00mm (No.10)	≤ 50											

0.425 (No.40)	≤ 30	≤ 50	≤ 51								
0.075 (No.200)	≤ 15	≤ 25	≤ 10	≤ 35	≤ 35	≤ 35	≤ 35	≥36	≥36	≥36	≥36
LL	…		….	≤ 40	≥41	≤ 40	≥41	≤ 40	≥41	<40	≥41
PI	6 Max		N.P	≤ 10	≤ 10	≥11	≥11	≤ 10	<10	≥11	≥11
Usual types of significant constituent materials	Stone Fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soil		Clayey soil	
General rating as subgrade	Excellent to good							Fair to poor			

* For PL <30, the classification is A-7-6; for PL ≥30, it is A-7-5.

Materials falling within group A-1, A-2-4, A-2-5 and A-3 are satisfactory as sub grade when properly drained and compacted under moderate thickness pavement (base and/or surface course) of a type suitable for the traffic to be carried, or can be made satisfactory by additions of small amount of natural or artificial binders (AASHTO, 2001). Materials falling within groups of A-2-6 and A-2-7 and silt-clay groups of A-4, A-5, A-6, and A-7 will range in quality from the approximate equivalent of the good A-2-4, A-2-5 to fair and poor sub grades (AASHTO, 2001). Poor groups require a layer of sub base material or an increased thickness or base coarse in order to furnish adequate support for traffic loads (AASHTO, 2001).

2.8.2. Sub-Bases (GS)

Sub base is an important load spreading layer in the completed pavement. It enables traffic stresses to be reduced to acceptable levels in the subgrade, it acts as a working platform for the construction of the upper pavement layers and it acts as a separation layer between subgrade and base course. Under special circumstances, it may also act as a filter or as a drainage layer. In wet climatic conditions, the most stringent requirements are dictated by the need to support construction traffic and paving equipment. In these circumstances, the sub-base material needs to be more tightly specified. In dry climatic conditions, in areas of good drainage, and where the road surface remains well sealed, unsaturated moisture conditions prevail and sub-base specifications may be relaxed. The selection of sub-base materials will therefore depend on the design function of the layer and the anticipated moisture regime, both in service and at construction (ERA, 2013a).

The materials used for the construction of sub-base layers shall be either natural gravel or crushed stone. Gravel material to be used for sub-base shall be obtained from approved sources in borrow areas, cuttings or existing pavement layers. The aggregate used for crushed stone sub-base shall be derived from a parent rock that is hard, sound, durable, and un-weathered and obtained from an approved quarry or clean sound boulders (ERA,2013a).

Sub-base quality governed by its ability to support construction traffic without excessive deformation. The minimum dry density to which the material shall be compacted shall be 95% of the MDD obtained in the AASHTO T-180 method D and at the highest expected moisture, a CBR > 30% is required at a minimum of 95% of Maximum Dry Density (MDD) AASHTO T-193 (ERA, 2013a).

In order to achieve required bearing capacity, and for uniform support to be provided to the upper pavement, limits on soil plasticity and particle size distribution is required (ERA, 2013a).3

Table 2-2 Grading Requirements for Sub-Base Material (ERA, 2014).

ISO Sieve Size(mm)	**Percentage Passing by Mass**		
	Natural Gravel	**Crushed Stone**	
	A	**B (37.5mm)**	**C (28mm)**
63	100		
50		100	
37.5	70-100	95-100	100
26.5			
20	50-100	60-80	70-85
9.5		40-60	50-65
4.75	30-100	25-40	35-55
2.36		15-30	25-40
1	17-75		
0.425	11-56	7-19	12-24
0.075	5-25	5-12	5-12

Natural gravel sub-base materials shall have a maximum Plasticity Index, when determined in accordance with AASHTO T-90, depending on the climate as shown in Table 2.3 and Crushed stone sub-base shall a Plasticity Index of less than 6% when determined in accordance with AASHTO T-90.

Table 2-3 Recommended plasticity of granular sub-bases (ERA, 2014).

Climate	Typical Annual Rainfall (mm)	Liquid limit	Plasticity Index	Linear Shrinkage
Moist tropical and wet tropical	>1000	<35	<6	<3
Seasonally wet tropical	>500	<45	<12	<6
Arid and semi- arid	<500	<55	<20	<10

2.8.3. Base Course

A wide range of materials can be used as unbound base course including crushed quarried rock, crushed and screened, mechanically stabilized, modified or naturally occurring „as dug‟ or „pit run‟ gravels. Their suitability for use depends primarily on the design traffic level of the pavement and climate. However, all base course materials must have a particle size distribution and particle shape which provide high mechanical stability and should contain sufficient fines (amount of material passing the 0.425 mm sieve) to produce a dense material when compacted (ERA, 2013).

2.8.3.1. Crushed Stone (GB1)

Graded crushed stone (GB1) material produced by crushing quarried rocks (ERA, 2013a). The propose of constructing crushed stone base is to achieve maximum impermeability compatible with good compaction and high stability under traffic (ERA, 2013a). To make the material durable, aggregate crashing value should preferably be less than 25 and always less than 29 and the fine fraction of a GB1 material should be non-plastic (ERA,2013a).

In situ dry density of placed Crashed Base Coarse (GB1) material shall be a minimum of 98% of the MDD in ASTM Test Method D 1557 (Heavy Compaction) and compaction thickness of each layers not exceed 200mm. Crushed stone base courses constructed with proper care in the materials described above have CBR values well in excess of 100 per cent (ERA, 2013a).

Table 2-4 Grading Limits for GB1 (ERA, 2014).

Test Sieve(mm)	Percentage by mass of total aggregate passing test sieve		
	Nominal maximum particle size		
	37.5mm	28mm	20mm
50	100	-	-
37.5	95-100	100	-

28	-	-	100
20	60-80	70-85	90-100
10	40-60	50-65	60-75
5	25-40	35-55	40-60
2.36	15-30	25-40	30-45
0.425	7-19	12-24	13-27
0.075	5-12	5-12	5-12

2.8.3.2. Naturally Occurring Granular Materials, Boulders, Weathered Rocks

Normal requirements for natural gravels and weathered rocks (GB2, GB3) include lateritic, calcareous and quartz gravels, river gravels, boulders and other transported gravels, or granular materials resulting from the weathering of rocks (ERA, 2014).

The fines of these materials should preferably be non-plastic or PI < 6. For Plasticity index (PI) close to 6, fines content should be restricted to lower end of the range and to ensure this, a maximum Plasticity Product (PP) of 60 is recommended or maximum Plasticity Modulus (PM) of 90 where PM is Plasticity Index times % passing 0.425 mm sieve (ERA, 2014).

Table 2-5 Recommended Particle Size Distribution for GB2 and GB3 (ERA, 2014).

Test Sieve(mm)	**Percentage by mass passing test**		
	` Natural Gravel		**Chemically Stabilized**
	37.5mm	**20mm**	**37.5mm**
50	100	-	-
37.5	80-100	100	-
19	60-80	80-100	100
9.75	45-65	55-80	80-100
4.75	30-50	40-60	50-70
2	20-40	30-50	35-50
0.425	10-25	12-27	12-30
0.075	5-15	5-15	5-15

When natural gravels and weathered rocks (GB2, GB3) is used as a base course, the material should be compacted to a density equal to or greater than 98 per cent of the maximum dry density achieved in the ASTM Test Method D 1557 (Heavy Compaction) and have a minimum CBR of 80% after four days soaking (ASTM D 1883) (ERA, 2014).

Table 2-6 Properties of unbound materials (ERA, 2013a)

Code	Description	Summary of Specification
GB1	Fresh, crushed rock	Dense graded, un-weathered crushed stone, non-plastic fines
GB2	Crushed weathered rock, gravel or boulders	Dense grading, PI<6, soil or parent fines ; PP<60
GB2A	Dry bound and water-bound Macadam	Aggregate properties as for GB2 PI<6; [1]PP<60
GB3	Natural Coarsely graded granular material including processed and modified gravels	Dense grading, PI<6 CBR after soaking >80%
GS	Natural Gravel	CBR after Soaking > 30%
GC	Gravel or gravel -soil	Dense graded CBR after soaking >15%

[1] PP = Plasticity Product = PI x (per cent passing 0.075mm sieve)

2.8.3.3. Bitumen-Bound Materials / Asphalt Concrete (AC)

Hot mix asphalt is the most critical layer in a pavement structure and must be of high quality and have predictable performance (ERA, 2013a). It possesses high resistance to fatigue and the ability to withstand high strains, i.e. need to be flexible, sufficient stiffness to reduce stresses in the underlying layers, high resistance to environmental degradation i.e. good durability, low permeability to prevent the ingress of water, good workability to allow adequate compaction to be obtained during construction, sufficient surface texture to provide good skid resistance in wet weather, and predictable performance (ERA, 2013a).

It is the most common types of HMA, has a continuous distribution of aggregate particle sizes that often designed to give the maximum particle density after compaction but adjusted slightly to make room for sufficient bitumen.

3. MATERIALS AND METHODS

3.1. Description of the study area.

3.1.1. Project Location

The road project of this runs from Dodola to Goba towns, where they located to the South East part of Ethiopia direction from the capital city, Addis Ababa at distance of 314+000 and 442+000 kilometers respectively. The total length of the road is about 130 km asphalt pavement which connects Bale zone and West Arsi zone of Oromia Regional State, starting at out strict of Dodola town and terminates at out strict of Goba town (Togona river), passes through Adaba, Dinsho and Robe towns (TCDSCo, 2006- 2012). From the total stretch 130km of the total Dodola –Goba road project road, this study has focused and detailed investigations has been carried out on the most critically failed road section of Adaba to Dinsho town of 59 kilometers which located between 339+000 and 398+000 kilometer from Addis Ababa

The road is the only link route of Bale zone to central Ethiopia and has national importance, where the area is highly fertile agricultural land and have two terms production per year which crucial for the economy of the country. The road is also access for numerous tourist attractions like **Sof-Omar Cave** which the longest natural cave in Ethiopia at 15.1 kilometers long and **Bale Mountains National Park** which founded in 1962 that comprises about 64 types of mammals' species among this 11 is found only in this park, 225 species of birds in the parks from this birds 16 are found only in this park and above 1000 species of plant found in the park (Bale Zone socio economic data, 2005).

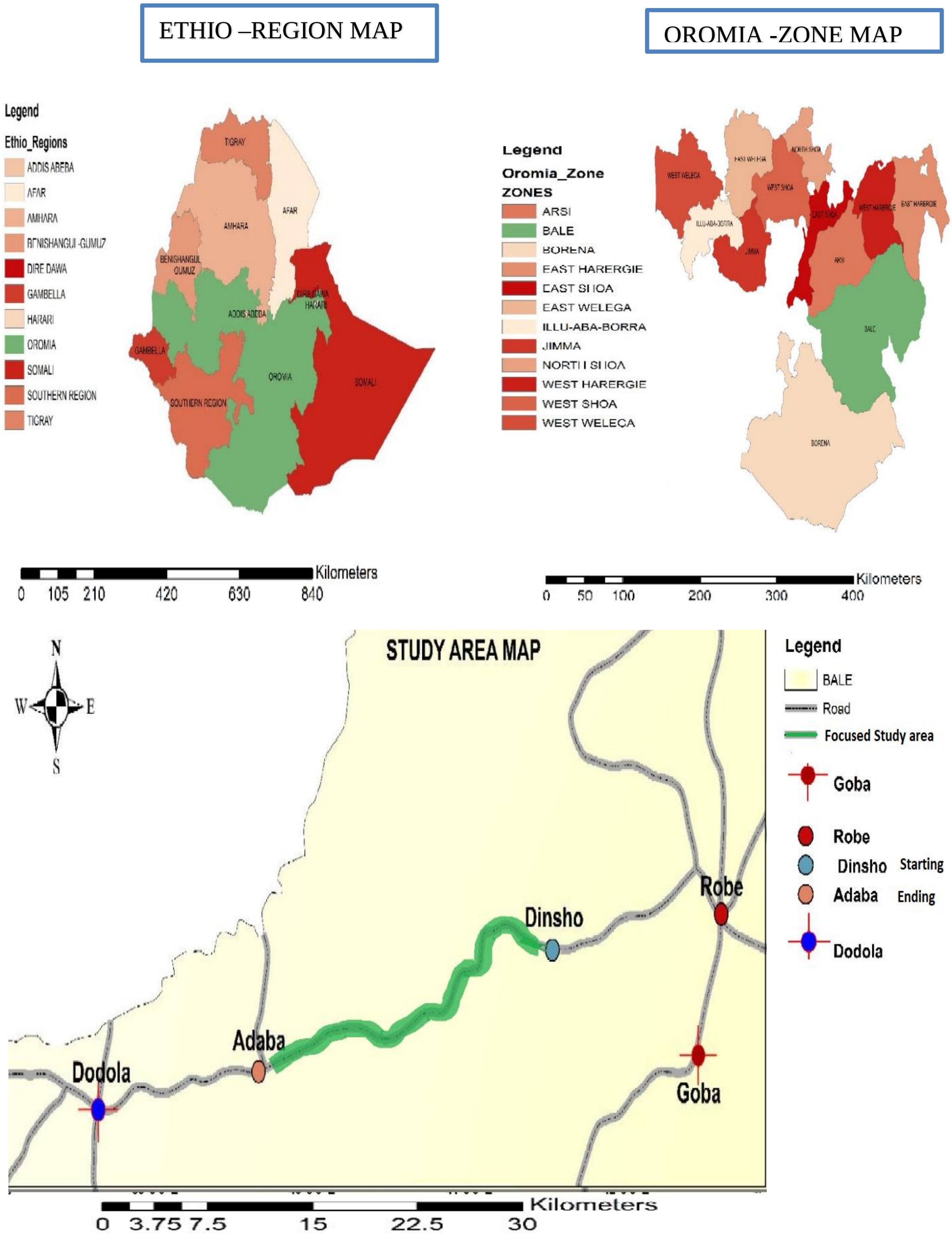

Figure 3-1 Map of the study area (source: - GIS Map)

3.1.2. Physiographic and Topography

Dodola –Goba road project route corridor is broadly classified into three physiographic regions mainly based on terrain (TCDSCo, 2006- 2012). These are flat, mountain and gentle curve and flat physiographic regions.

Table 3-1 Physiographic Regions (TCDSCo, 2006- 2012).

Item No	Section	Physiography
1	314+000 - 347+000	Flat
2	347+000 - 390+000	Mountain
3	390+000 - 447+000	Gentle Curve and Flat

As we have seen from table 3.1 the focused area of Adaba –Dinsho road mainly passes through mountain physiography regions and also minor sections of this road passes on flat and gentle curve regions. The road on focused study area lies through generally high elevation and side of many parts of the road is steep to very steep valley and ranges from 2300-3600m above sea level (TSDCo, 2006-2012).

3.1.3. Climate

The project area has long rainy period from the beginning of May to the end of October with mean annual rainfall 1100mm.The mean annual temperature in the area is max.23^{0}c and min.13^{0}c. The climate of the project route is generally pleasant with extreme cool temperatures (TSDCo, 2006-20012). The zone of the study area is located in dega zone. As a result, the dominant prevailing weather conditions of the project area is mostly rainy and very cold on most the construction period hence the construction activities were seriously affected by such weather.

3.2. Materials Used

In the initial stage of the study, on literature review, online and printed books, journals, government standard documents, articles of conferences and edited books were studied. Then a field inspection and sampling proceed. On field inspection car was used to visually asses the entire road length and digital camera were used for take photos and documentations. Disturbed samples taken manually through hand tools of shovels, digging bar. Measurements were taken by using hand tape. All samples were taken from the center of respective pavements structures, embankment and sub grade. The depth, which sampling made, predicted from the thickness recommended by ERA design recommendation.

On field density testing, sand and excavated pavement structural materials of embankment, sub grade and bas course, and equipment's of sand pouring cylinder, scarper with handles, metal containers, and balance used. Furthermore, warning flags placed on opposite sides of sampling locations for traffic management.

In laboratory testing, disturbed sample materials (embankment, sub base, base course, sub grade) and distillated water for compaction, California bearing ratio (CBR), atterburg limit tests were used.

In conducting laboratory tests, equipment's of Casagrande in liquid limit testing, CBR machine in CBR testing, glass plate in plastic limit testing, mold with removable collar and base in CBR and compaction tests, graduated jar 400mm and 100mm, for distilled water measuring and free swell testing, Stock of Sieve from 50 mm up to 0.075mm and for gradation analyses or Particle Size Distribution (PSD) were used.

3.3. Methodology

The procedure utilized throughout the conduct of this research study is as follows: Related literatures review, field inspections, field tests and measurements were carried out and laboratory tests were under taken from representative samples. After necessary data collection, organization, comparison and analysis were obtained and then subsequently compared the results with preexisting literature and standard specifications. Based on findings from result the real causes of road failure were identified and previous maintenance methods also evaluated. Possible remedial were proposed for failed road sections. Finally, conclusion and recommendation were done.

3.3.1. Review Related Literatures

Related literatures with the study title were reviewed which includes articles, reference books, research papers, class lecture notes, project specifications, standards specifications like ERA, AASHTO and ASTM. Technical standard, design requirements and other relevant information such as topography, soil or geological records, and temperature, weather or rainfall data of the study area were reviewed from construction progress report and other related data.

3.3.2. Field Investigations

3.3.2.1. Visual Inspection

Visual inspection was done on whole stretch of Adaba –Dinsho road of study area. The major failure types of the road were identified and impact of the topography, geology and drainage conditions were assessed. Additionally, major geotechnical problems like problematic soils, land slide and slope instability along the road section were assessed. This helped to relate the pavement failures noticed on the surface with possible causes. It was very clear that before testing any pavement structure, the impacts of the topography, geology and drainage conditions were checked during site survey. An effective visual inspection of pavement failures was essential, to ensure that the cause of the failure can be diagnosed efficiently and it is a guide to what testing should be carried out and where (ERA, 2013c).

3.3.2.2. Field Tests and Samplings

Two critically failed road sections were selected that were uniform failure types, topography, geology and drainage conditions based on field inspection. Three representative test pits location were selected for each critically failed two sections observed during field inspections and totally six test pits location were selected as shown table below with their respective stations and coordinate.

Table 3-2 Location of Selected Test Pits

Road Sections	Selected Test Pits Locations		
		Coordinate	
	Station	Northing	Easting
Section 1 (from 370+000 to 380+000)	377+600	570939	779309
	376+500	570809	779186
	375+500	570604	779163
Section 2 (from station 380+000 to 388+000)	386+800	575946	785871
	387+800	579660	786050
	388+300	576251	786333

Selected test pit locations were striped by using asphalt cutter and retrieving samples from each layer for the determination of field density test was made at each selected test pit stations for each layer of the road using Sand Replacement Method by AASHTO T-191 and sample taken for in situ moisture content covered by plastic and placed at the internal part of each disturbed samples within the sample bugs as shown on fig 3.2.

Figure 3-2 Field density determination and thickness measurements.

Disturbed base course, sub base, subgrade (in situ suitable or selected fill) and natural sub ground soil material of each approximately 70kg were taken from each selected failed sections and various depths below the pavement surface to get base samples, subbase samples and subgrade or embankment samples at varying depths of 0 - 0.20m, 0.20m – 0.4m and 0.4m – 0.6m respectively as shown on fig.3.2. Additional samples were taken on natural ground soil at road side of 0.40m depth. Samples of base course and sub base materials were taken at 5 cm depth from top layer after field tests and samples of subgrade (in situ suitable or selected fill) and natural ground soil below embankment fill were taken at 25cm depth from top layer. Each sample bugs were identified by name and transported to Arsi Robe – Agarfa –Ali Road Project laboratory. After the field tests and sampling is done, proper maintenance was made using left over excavation materials and/or side borrows materials. Up on sampling, the width of the existing road surface and the thickness of each layer were taken measured using a meter tape.

3.3.3. Laboratory Tests

Enough representative samples were collected from each test pit and each layer of pavement. Samples were collected, labeled and transported to the laboratory for tests. Before starting Laboratory test, these samples were first air-dried under the sun to allow moisture to evaporate before starting the required test. The tests were performed according to the AASHTO and ASTM Specification and Procedure. The following tests were undertaken such as Atterberg Limits, Grain size Analysis, Compaction tests, and California Bearing Ratio (CBR) tests were made to understand the physical properties of the road materials on the selected road sections.

3.3.3.1. Compaction Test (AASHTO T-180)

Compaction increases shear strength, stability and bearing capacity, and reducing the compressibility and permeability of the soil. To conduct this test, first samples were air dried thoroughly. Then representative sample pass 19mm sieves was weighted and mixed with water thoroughly with a measured quantity of water. Mixed specimen was compacted in mold with 56 blows in five layers of approximately equal height. This process was repeated on other samples until weight of sample compacted in a mold was decreased. The maximum volume of mix compacted per mold volume was recorded as wet density of soil. Then dry density determined by wet density divided by one plus moisture content, of the mix.

3.3.3.2. In Situ Moisture content (AASHTO T 265-93)

The test conducted in AASHTO T265-93 as recommended in ERA practical construction tastings. A test specimen was oven at a temperature of 110+/- 5°C. The loss of mass due to drying considered water. Then water content taken as the percentage loser of mass.

3.3.3.3. Liquid Limit Test (AASHTO T-89)

The liquid limit defined as the water content at which soil changes from the liquid state to the plastic state. AASHTO T-89 is the most practical standard method of testing used. In this testing, 300gm of soil sample passing No.40 sieve was weighted and mixed with distilled water, such that the number of blows required to close the groove is about 15-40, to form a uniform paste and soaked for 24 hr. After 24 hours, the past smoothed and grooved on Casagrande cup to a maximum depth of 12.7mm. By turning the handle of apparatus at a rate of two revolutions per second, the number of blows necessary to close gap for a distance of about 13mm recorded and sample near the closed portion taken for moisture content determination. This were repeated for at least two additional trials producing successively lower number of blows to close the grove which should yield a blow count of 15-20, 20-25 and 25-35 blows respectively

3.3.3.4. Plastic Limit (AASHTO T-90)

Plastic limit is the water content below which the soil stops behaving as plastic material. At this state, sample begins crumbled when rolled by palm into a tread of 3mm diameter over a glass plate. The water content of crumbled pest of the thread takes, for moisture content determination. The water content at which the soil cracks when it rolled to 3mm diameter was taken as Plastic Limit.

3.3.3.5. Sieve Analyses (AASHTO T88/311/ T27-99)

This test is made in AASHTO T88/311 for soil samples and T27-99 standard methods of testing for aggregate (coarse grained) samples. The test conducted by first taking a representative sample of material from disturbed sample, in methods of reducing laboratory samples BS812-102, taken from test pits. Then the sample was shacked, in a stalk of sieve, from 37.5 up to 0.075mm sieve sizes, for a minimum of 10minuites. The percentage retained of sample on each sieve was weighted and recorded. The percentage pass was then determined by deducting the retained amount over each sieve from 100.

3.4. Data Collection Process

In order to achieve the objectives of the research stated in chapter one, data were required. The methodologies applied to collect the data are discussed below. Before starting any data collection formal letter was obtained from IoT-HU and an official permission was obtained from ERA Regional Shashemene office district.

Field visits in the study area were undertaken and type of failures occurred on the road surface of study area were identified and recorded. Field measurements and tests also were collected. Representative samples at each test pits and photographs at representative location were collected. Finally, the collected representative samples were tested in laboratory

3.5. Data Analysis Procedure

In this study, first, a detail review of similar research works, previously conducted in study area assessed, with the main interests of supporting real cause of the problem. Similarly, climate, topography and drainage characteristics of the road reviewed and correlate how these are interrelated with the road design recommendations stipulated in ERA 2013 design manual, and the actual road construction.

Then geotechnical properties of embankment, sub grade, sub base, and base course materials utilized is determined. Based on the results, classification of materials associated with the design and construction requirements expected, as per ERA design manual, studied. Consequently, results evaluated as per ERA (2013) design recommendations of the respective areal climate, drainage and topography, criteria, if any.

At the end, according to the evaluation of materials, i.e. best or poor, in correlation with climate, drainage and topographic conditions, their quality recommendations in ERA (2013), and the relation with previous findings, methods of improvement suggest on conclusion.

4. RESULT AND DISCUSSION

4.1. General

In this chapter, results of filed investigations and laboratory tests to be interpreted and analyzed. The discussion has to be made on the interpretation of all result from the study sections that had been obtained and comparisons to be had with ERA and/or AASHTO standard specifications.

4.2. Field Investigation

4.2.1. Visual Inspection

The visual inspection was done on whole stretch of Adaba –Dinsho road from ending July to August 2020. This period was pick rainfall season of the study area and good period for visual inspection to assesses the major failure types, major geotechnical problems encountered (problematic soils, land slide and slope instability) and impacts of the topography, geology and drainage conditions along the road sections.

According to visual inspection, the road was critically failed/ damaged on two major road sections. These sections were categorized in to section 1, where from station 370+000 to 380+000 km and section 2, where from station 384+000 to 388+000 based on uniformity of failure types, topography, geology and drainage conditions.

Major failure types, impact of the topography, geology and drainage conditions with major geotechnical problems of road sections and performance of previous maintenance methods are described as below.

4.2.1.1. Section 1

On this section, the road stretch runs from station 370+000 to 380+000 km i.e. from Haricho kebele to Geremba Dima kebele. The road was constructed on mountainous area, along valley side. Cut slopes were done from hill to widen the road way partially and used fill adjacent valley for purpose of supporting the road way. Narrow horizontal and steep vertical alignments were constructed based on the actual topography of the route. The subgrade of this section were constructed from the in-situ suitable material below cut section by re-compaction and breaking up undesirable formations of hard or rocky materials treatments.

The road was in bad condition and high amount of critically failed sections are revealed on the road surface as per observation. Repetitive maintenances were under taken by ERA regional Shashemene district office to solve problems on road performance. But the road still cannot serve properly as intended purpose and causes road traffic accident that had resulted into loss of lives and properties and a very serious problem that causes unnecessary delay in traffic flow, distorts pavement aesthetics, damages of vehicle. Potholes, alligator cracks, edge cracks and rutting were majorly observed types of failures along this section during observation.

Series of interconnected cracks creating small, irregular shape pieces of pavement results large alligator or fatigue cracks were observed during field inspection as shown in fig 4.1. Alligator cracks indicate a loss of sub grade support and sign of a general pavement structure failure, associated with base or drainage problems as described by KC Sethi (2016). During field inspection, the surface water from rain fall or mountain penetrate with in cracks in to pavement materials and sub grade, causes for other type of failure like potholes. Additionally, wider rutting's failure associated with fatigue cracking are observed on the surface of this sections as shown in fig 4.1 which indicates of mostly subgrade material failure and also some pavement material failure is occur due to heavy traffic load, in adequate compaction and excess moisture as discussed in chapter two.

Figure 4-1 Alligator cracking and rutting failures observed at location around 375+000km

Similarly, potholes are widely observed failure type at this section which results from several alligator cracks due to inadequate strength in one or more layers of the pavement and subgrade, accompanied by the presence of water and drainage problems. During observation water ponds on potholes are common during rainy seasons of this section as shown on fig 4.2a and the water infiltrate in to pavement layers, base, sub base and subgrade material become wet and unstable, this results a growth of number of pot holes with area and depth. Interconnected potholes are become large hole on the road surface that cause danger for motorist and road traffic accident results loss of life and properties were seen during observation as fig 4.2b. Edge crack are also one of failure type of this section and they occur at edge of pavement due to lack support or wet shoulder material and excess moisture as shown fig 4.2b.

The geology of this section show that highly weathered Basalt rock type on bottom of cut

Figure 4-2 a) Water ponds on potholes failures observed at location around 377+600km.
b) Interconnected potholes and edge failures on the road surface at location around 375+000km.

slope profile and overlaid by residual soil of block cotton, reddish or orange, molted appearance and gray or bluish gray clay soils with gravel placed from top to bottom respectively from hill cut section as fig 4.3.

The observation shows that black to grey color soils of this section may typically generated from highly basic basalt rocks with factors of climates and floods, or to impede drainage. This indicates that, clay materials are more sensitive to moisture. The ground water rises during rainy season intercepts the cut slope causes flood flows through cut face from top hill to toe slope as shown on fig 4.3.

Figure 4-3 Geological profile the road and falling flow of ground water from upper hill at location around 377+000 and 376+000

The drainages of this road section were poor and susceptible to saturate sub grade and paving materials. During observation side ditches of this section silts up by eroded or slide soil material from hill cut slope and cannot cache water from cut slope and road surface at toe of each cut section. During sampling of this section, side drains and culverts was silt up by soil and rock material slides from failed slope, that results water fall on the surface of asphalt and shoulder from cut slope and surface drainage. This water infiltrate through the surface, or could flow laterally from the pavement edges and shoulder ditches.

Figure 4-4 Side ditches silts up by eroded or slide soil material from hill cut slope and water falls on the surface of asphalt and shoulder location around 377+000 and 376+000

The moistures entry from the side caused by poorly constructed surface drainage, and lateral movement of water into pavement. Eventually reduces strength and stiffness of pavement materials, being worse for the subgrade material. Improper design and late maintenance of side and cross drainage structures are back onto the road or not keep it from draining away flows on road surface were observed as shown on fig.4.4. Too much water remaining on the surface combine with traffic action are major causes potholes, cracks and other pavement failure of this section.

The ground water rises during rainy season intercepts the cut slope causes flood flows through cut face from top hill to toe slope and initiate land slide and slope failures during observation.

Landslides or slope failures are common on this road sections and triggering from week geology, topography, climate variation, and effect of ground water as shown on fig 4.5. Week geology or weak soils overlying harder rocks provide an interface or potential shear surface along which landslides can initiate. Topographical or steep hillsides promote gravity induced slope failures and concave slope facets which can hold moisture for long periods of time, this leads to lower effective stress and reduced strength. Groundwater increases the weight on a slope there by increasing the total stress or driving force and also clays are hydrate and expand and make them susceptible to failure.

Figure 4-5 Land slide occurs during sampling location around 376+800km

Currently the most stations of this road section are failed beyond critical state and repetitive maintenances were done on these stations by ERA regional shashemene district office. Patches and rigid concrete pavement overlaying rehabilitations types were done for failed stations. Patches repair were done on potholes, but after few months later the surface became fail again. This indicate that patches cannot proper maintenance methods potholes failures and the cause failure are structural problems induced from subgrade and pavement materials (base and sub base) rather than surface layers. Applying the wrong remedial treatment could be a waste of time and money. Concrete pavement over laying rehabilitations up to 20meter length with 7meter width were done on rutting, cracking and pothole failures of this road sections. Rehabilitations made by concrete pavement overlaying methods have good performance, but suddenly alligator cracks and potholes failures were initiated between flexible pavement and rigid concrete slab due to non-uniform transition of materials and piping were seen during observation. Concrete overlaying slabs cannot proper maintenance methods due to uneconomical solution using on all failed sections.

Figure 4-6 Sudden progresses of alligator cracking and potholes at transition of flexible pavement and rigid concrete slab 375+800km

Improper design and late maintenance of drainage, seasonal variation of ground water, substandard construction material, improper construction methods and workmanships and heavy traffic loads are the possible major factors for road failure of this section.

4.2.1.2. Section 2

On this section of the road stretch runs from station 380+000 to 388+000 km i.e. from Geremba Dima kebele to Hor Soba kebele outlet. The road section was constructed on further flat areas and selected subgrade fills / embankments were done on cut original natural soil by removes and replaces methods to stabilize the natural sub grade. The road was highly deteriorated with high amount of deformation on the road surface and embankments. Alligator cracking, rutting and potholes were major observed types of failures of this section.

Alligator cracking that associated with rutting were observed during field inspection as shown in fig 4.7. Alligator cracks with large fatigue cracked areas are signs of general pavement structural failure (KC Sethi, 2016) and rutting is usually a failure in one or more layers in the pavement. Rutting result of an accumulation of non-recoverable vertical strains in the pavement layers and in the subgrade caused by traffic loads, inadequate compaction of the subgrade or any of the pavement layers and inadequate thickness of the pavement or weak pavement structure. Run-off or surface water on the ruts and alligator cracking enters pavement layers, the base and/or subgrade become wet and unstable that results the development of pothole failures of this section as shown during observation.

The area of this section has flat and swampy and may not drain fast when high amount of precipitation conceded. The geology of this section was majorly covered by residual block cotton clay soil which generated from highly weathered basic basalt rocks.

Figure 4-7 Alligator cracking, potholes and rutting type road failure at station 386+800 and station 388+300.

Proper drainages were not provided on this road section due to poor design and the water direct back onto the road or cannot keep it from draining away. During sampling, the pavement materials (base and sub base) were highly saturated due to the water migrates into the pavement structure through combinations of surface infiltration through cracks in the surface layer and edge inflows from inadequately drained side ditches. Additionally, inadequate shoulders were provided on this section due to poor design which does not prevent the base, sub base, sub grade, and other susceptible paving materials from becoming saturated were shown during observation.

The geometric design of this section was poor and adequate drainage were not provided on this section which cannot prevent the edge flows to the surface of the pavement as observed during field inspection. The top of the pavement surface was averagely 60cm higher than the side natural ground soil as it measures during trial pit excavation. which the side run-off from catchments simply migrates into the pavement structure. This shows that no enough cover is provided to overcome the swell effect of the expansive soil. The ground water of this section was rises and more closed to natural soil during the rainy season. This results the water migrate in to embankment and pavement structure from the underlying groundwater table by capillary potential in fine grained foundation soils. Too much water remaining on the surface combine with traffic action may cause potholes, cracks and rutting failure of this section.

Patches maintenance and concrete pavement overlaying rehabilitation were done for failed stations of this section which done previously by ERA regional shashemene district office. As we discussed in section 1, patches cannot proper maintenance methods for potholes failures of this sections because, the surface fail again after few months later. On the other hand, concrete pavement overlaying rehabilitation has good performance on the maintained section, but other similar failures were initiated nearby to maintained sections as observed from field inspection.

Improper geometric design, poor workmanship, using sub-standard quality material during construction and heavy traffic loads are the major possible cause of failure of this section as per visual field inspection.

4.2.2. Existing Width and Thickness of the Pavement Material

The width of the existing road surface on test pit locations were measured using a meter tape during test pitting and sampling. The design width of the road in in small towns and rural areas has 7-meter carriageway width and 1.5-meter shoulders on both sides as per TCDSCo annual report of this road (TCDSCo, 2006- 2012). Hence the width of the road on test pit locations are established mostly by judgment and measured. The mean width of the carriage way and shoulder is around 7.0m and 1.5m respectively. This indicate that the width of the road on test pit locations are fulfill the design width, despite with difficulties on establishment pavement at pavement edge failures caused by repetitive rutting by traffic load and edge crack.

The thicknesses of the road materials were measured using a meter tape of each test pit at failed section are given in Table 4.1.

Table 4-1 Existing thickness of pavement layers taken during soil sampling

Sections	Sampled Locations	Material Thickness(cm)	
		Base	Sub Base
1	377+600	19	19
	376+500	19.5	20
	375+500	18.5	19
	Average	19	19.3
2	386+800	19.5	21
	387+800	20	19.5
	388+300	19.5	20
	Average	19.7	20.2

As the document obtained from TCDSCo annual construction report this road indicates that, the design thickness of upgrading pavement layer would consist of 20cm crushed aggregate base course and 200mm sub base course respectively (TCDSCo, 2006- 2012). The thickness of the existing pavement from the field investigation results were given in table 4.2 and the result shows that, average thickness of the base course aggregates at section 1 is 19 cm, at section 2 is 19.7 cm, and the average thickness of natural sub base course at section 19.3, at section 2 is 20.2 cm. This indicates that the base course at section 1 and section 2 are less than the minimum thickness of 20 cm and the sub base course of the section 1 are also less than the minimum thickness of 20cm. on the other hand, sub base course at section 2 showed that it would be able to carry the traffic loads throughout its service life because it is equal to the minimum required thickness as per approved plan of the project.

The lowering of thickness of pavement material may be due to improper construction methods, voids due to fines material migration by water and more fine material due to heavy traffic loading. This will not be able to carry the traffic loading at its service life time that results rutting and potholes road failures.

4.2.3. In-situ density survey

An in-situ density measurement was carried out on selected test pits where samples have been recovered for laboratory tests. It was conducted by the sand replacement method in accordance with AASHTO-191(1993). Such in situ tests were performed on subgrade/fill, sub base and base layers, where the material thickness is 10cm or more and without oversized stone or gravel.

Table 4-2 Field densities and field water content results.

Sections	Sampled Station	Type of Sampled Material	Field Bulk Density g/cm^3	Field Dry Density g/cm^3	Field Water Content (%)
Section 1	377+600	Base Coarse	2.45	2.29	7.30
		Sub base	1.93	1.64	17.11
		Sub grade	1.76	1.51	16.96
	376+500	Base Coarse	2.49	2.32	7.31
		Sub base	1.92	1.65	15.97
		Sub grade	1.80	1.49	20.73
	375+500	Base Coarse	2.45	2.31	6.27
		Sub base	1.88	1.59	8.58
		Sub grade	1.86	1.61	15.20
Section 2	386+800	Base Coarse	2.39	2.21	8.21
		Sub base	1.79	1.47	22.04
		Sub grade	1.90	1.64	16.16
	387+800	Base Coarse	2.28	2.11	8.28
		Sub base	1.66	1.42	17.35
		Sub grade	1.84	1.61	14.60
	388+300	Base Coarse	2.38	2.23	6.33
		Sub base	1.73	1.45	18.88
		Sub grade	1.82	1.57	15.81

The bulk density was computed upon completion of each test. The field dry densities were later computed based on results of the natural moisture content determined in the laboratory.

The field dry densities and the field moisture content conducted on the subgrade/fill, sub base and base layers are tabulated below.

As per result table 4.2, at section 1, the average field dry density and field water content of in-situ suitable sub grade material is 1.81g/cm^3 and 17.63%. The average field dry density and field water content is 1.91g/cm^3 and 17.22% respectively for sub base material and 2.47g/cm^3 and 6.96% respectively for course base material.

Correspondingly at section 2, the average field dry density and field water content of selected subgrade fill material is 1.85g/cm^3 and 15.52%. The average field dry density and field water content is 1.73g/cm^3 and 19.42% respectively for sub base material and 2.35g/cm^3 and 7.61% respectively for course base material.

4.3. Laboratory Tests

4.3.1. Compaction Test and Percentage of Compaction

Soil compaction is one of the most important geotechnical concerns that improve quality of highway pavements and related fills and/or embankments. Compaction improves soil properties of elastic stiffness, strength and/or bearing capacity and reduces short-term deformations during cyclic loading, compressibility and potential for excessive long-term settlement, instability, hydraulic conductivity, thus, helps maintain desired strength and stiffness properties, decreased erosion resistance.

The laboratory compaction tests were done for determination the maximum dry density with the optimum moisture content. The Modified Proctor test methods were used in accordance with (AASHTO T-180) (1993). Degree of compaction were computed from field dry densities determined from in situ density test at section 4.3 and maximum dry density determined from laboratory compaction tests for each samples.

Table 4-3 Compaction tests results and percentage of compaction summary.

Road Sections	Sampled Station	Type of Sampled Material	Field Dry Density (g/cm^3)	Maximum Dry Density (g/cm3)	Compaction Degree (%)	Mini. Specification Requirement (%)
Section 1	377+600	Base Coarse	2.29	2.34	97.88%	98%
		Sub base	1.64	1.78	92.16%	95%
		Sub grade	1.51	1.81	83.20%	95%
	376+500	Base Coarse	2.32	2.37	98.05%	98%
		Sub base	1.65	1.80	91.76%	95%
		Sub grade	1.49	1.71	87.11%	93%
	375+500	Base Coarse	2.31	2.39	96.52%	98%
		Sub base	1.59	1.77	89.59%	95%
		Sub grade	1.61	1.83	88.05%	95%
Section 2	386+800	Base Coarse	2.21	2.33	94.77%	98%
		Sub base	1.47	1.67	87.86%	95%
		Sub grade	1.64	1.81	90.43%	95%
	387+800	Base Coarse	2.11	2.28	92.53%	98%
		Sub base	1.42	1.63	86.99%	95%
		Sub grade	1.61	1.76	91.41%	95%
	388+300	Base Coarse	2.23	2.38	93.89%	98%
		Sub base	1.45	1.62	89.57%	95%
		Sub grade	1.62	1.77	91.31%	95%

According to ERA, (2014) standard technical and specification manual, minimum practical maximum dry density of 95 % for in-situ suitable subgrade material and embankment fill that constitutes the subgrade layer (including capping and selected layers), 95% for sub base and 98% for base course, in modified method of compaction (ERA, 2014). In this study, Modified compaction conducted as in AASHTO T-180 standard method of testing and the results drawn in table 4.5 including their respective field density results found.

From the result above, the average percentage of compaction of in-situ suitable sub grade material at section 1 and selected subgrade fill material at section 2 averagely are 86.1% and 91.0% respectively, which are less than the required of 95% based on specification.

The percentage of compaction of the natural sub-base material at section 1 averagely is 91.2% and at section 2 averagely is 88.1%, which was less than the required of 95% based on specification. On the same manner the average percentage of compaction of base course material at section 1 is 97.5% which is closed to the required 98% based on specification, but at section 2, the average compaction percentage is 93.7% which was less than the required of 98% based on specification and do not satisfy density criteria.

As we have discussed in chapter two of the study, lowering of density may causes from inadequate compaction during constructions, loss of fine materials or mean much amount of void filled with either in water or air due to improper drainage and heavy traffic load. During lower density materials may flex high under the application of traffic loading and the movement exert high amount of strain on the asphalt surface. This results potholes, fatigue cracking, and rutting failures for these failures sections of the road. But lowering density may not only be the cause and responsible for failure of sections, because of tests conducted on failed sections of the road, behind loosen parts.

4.3.2. Moisture Variation

The increase in moisture content decreases the strength of the pavement. The increasing of moisture content results due to poor drainage on road pavement condition, exposure of surface to rain during construction, or porous or open graded asphalt.

Prediction of pavement structural materials in proper drained condition or not are evaluated by comparing the in situ and optimum moisture contents of each sample. The optimum moisture content can found in compaction test analysis. This is a sufficient amount of water required for better stability and load bearing capacity of each material. The in situ moisture content of samples determined by AASHTO T265-93 as recommended in ERA practical construction tastings. The variation of in situ moisture in percentage, with the optimum is tabulated as shown below.

Table 4-4 Variation of in situ moisture from optimum

Road Sections	Type of Sampled Material	Sample Locations	Moisture Contents		
			Field / In situ (%)	Lab. Optimum (%)	Variations (%)
Section 1	Base	377+600	7.30	7.00	4.25
		376+500	7.31	6.94	5.30
		375+500	6.27	6.00	4.58
Section 2		386+800	8.21	7.40	10.98
		387+800	8.28	7.50	10.40
		388+300	6.33	5.70	11.04
Section 1	Sub Base	377+600	17.11	15.30	11.80
		376+500	15.97	14.30	11.65
		375+500	18.58	16.36	13.55
Section 2		386+800	22.04	19.00	16.02
		387+800	17.35	14.70	18.04
		388+300	18.88	16.56	13.99
Section 1	Sub grade	377+600	16.96	13.52	25.42
		376+500	20.73	17.00	21.95
		375+500	15.20	12.80	18.75
Section 2		386+800	16.16	14.00	15.41
		387+800	14.60	13.10	11.46
		388+300	15.81	14.00	12.94

Result in table 4.4 indicates that, field or in situ moisture content of treated in-situ suitable subgrade material of station 1 and selected subgrade fill material of section 2 increased from laboratory optimum moisture content averagely by 22.04 % and 13.27% respectively. The field or in situ moisture content of sub base increased at section1 averagely by 12.33% and at section 2 increased by 16.02% from optimum moisture content. Also, field or in situ moisture content of base course increased averagely at section 1 by 4.71% and at section-2 by 10.81% from optimum moisture content.

From the result and analyses above the increasing field or in situ moisture content at section 1 increased due to mainly rises of underlying ground water during rainy season, edge flow though shoulder from flood flow and in adequate side ditch drainage. Also at sections 2, the field or in situ moisture content increased due to infiltration through cracks and potholes on the road surface, surface water towards to in to pavement due to poor drainage design and the ground water rises during rainy season moisturize embankment/ fill material due to capillary action from natural black cotton clay sub grade soil.

As we have seen the above discussion, too much moisture increases on pavement materials and sub grade is responsible for potholes, cracks and rutting failures of the road of failed sections and results density of a certain layer is significantly lower than optimum because of the in situ moisture content deviation from optimum moisture mean much amount of void filled with either in water or air causes flex higher and cause road failure.

4.3.3. Atterberg Limits Test

Plasticity index and liquid limit are the important factors that help an engineer to understand the consistency or plasticity of clay (Gowans, 2011).

4.3.3.1. Liquid limit test

According to ERA (2014) Standard technical and specification manual, liquid limit of natural gravel sub base construction has depends on climate of the area. Since the climate of our study area has mean annual rainfall 1100mm, which fall under category of moist tropical and wet tropical area on amount of 35% the maximum of liquid limit. The recommended maximum amount of liquid limit of subgrade or fill material is 60% and also the liquid limit of natural subgrade below cut section or fills section and embankment foundation must be less than 60% (ERA, 2014).

Table 4-5 Liquid limit results summery

Type of Sample Material	Liquid Limits (%) of Sampled Stations					
	Section 1			Section 2		
	377+600	376+500	375+500	386+ 800	387+800	388+300
Base	23.86	23.73	25.49	27.14	28.78	23.98
Sub Base	42.38	39.05	44.12	45.69	41.35	39.72
Subgrade	46.71	48.87	43.15	48.43	42.54	48.87
Natural ground soil	-	-	-	100.24	92.73	97.93

As result from Table 4.5, the liquid limit of subgrade (in situ or selected fill) materials of section1 and section 2 are averagely 46.24% and 46.61% respectively which are below the required of 60% specification requirement and safe a head. Additionally, the liquid limit of natural subgrade material below embankment of section 2 is 96.97% which is highly increased from maximum specification requirement of 60%.

The liquid limit of natural gravel sub base material of all tested samples station of section 1&2 are above the specification requirement on the moist and wet tropical climate of the study area and does not satisfy the requirement. The result averagely 41.85% at section1 and 42.25% at section 2 which is greater than the required of 35% based on specification. In the same manner the liquid limit of base coarse material also increased on both section 1and 2 for non-plastic base course as per manual requirement.

As we have seen from above discussion, high amount of liquid limits on base, sub base materials and natural sub grade soil below embankment indicates, having more clay content and having high instability in moisture variation which do not advised in construction and responsible for failures on both section 1 and 2.

4.3.3.2. Plasticity Index (PI %)

High plasticity of material decreases its permeability and hydraulic conductivity which may be a factor of water logging and flooding both of which results in road failure (Onuoha, 2014). Usually highly plastic soil has the ability to retain appreciable amount of total moisture in the diffuse double layer, especially by means of absorption and are usually susceptible to high compressibility and also this clay material is seen as troublesome and difficult in construction (Onuoha, 2014).

According to ERA (2014) recommendation, the recommended maximum amount of plastic index of subgrade /fill and embankment materials are 30% and natural gravel sub-base materials shall have a 6%, under category of moist tropical and wet tropical climate of this study area. In similar manner graded crushed stone (GB1) base materials of should be non-plastic which considered in this study and maximum 6% of naturally occurring granular base materials (GB2 & GB3) (ERA, 2014). Similarly, ERA (2013) flexible pavement design manual recommends, fill material over the expansive soils shall be impermeable soils with a plasticity index of greater than 15%.

Table 4-6 Plastic limit results summary

Type of Sampled Material	Plastic Index (%) of Sampled Stations					
	Section 1			Section 2		
	377+600	376+500	375+500	386+800	387+800	388+300
Base	5.74	5.04	7.38	8.38	8.01	5.44
Sub Base	13.71	14.34	15.40	19.42	15.29	14.51
Subgrade	21.85	20.74	19.17	19.82	17.27	20.97
Natural ground soil				61.99	45.81	52.11

As per result from table 4.6, the average plastic index of base course material of is 6.05% at section 1 and 7.28 at section 2 which are above the required of non- plastic based on specification. Similarly, the average plastic indexes of tested samples of sub base materials of section 1and section 2 are 14.48% and 16.41% respectively, which are above the required of 6% the moist and wet tropical climate of the study area.

On other hand plastic index of embankment /fill subgrade of all tested samples of section1 and section 2 are satisfied the specification requirement and safe a head. Additionally, the plastic index of natural subgrade material below fill subgrade of section 2 tasted stations is above the specification requirement of 30%.

As we have seen the above discussion, the plastic index of base and sub base materials increase either using substandard quality materials during construction or by contamination with expansive clay by water through infiltration through the cracks and pothole or poor drainage problems as we discussed on visual inspection. This increases of PI reduces the aggregate strength (load carrying capacity) by reducing grain to grain contact friction hence it is responsible for the cause of ruts in base layer and accelerates deterioration of the pavement and results rutting, cracking and potholes failures on this study area.

4.3.4. Grain Size Analysis /Particle Size Distribution (PSD)/

Well-graded particle size distributions provide good material interlock and high mechanical stability and dense material when compacted, at the required optimum moisture. In this study, AASHTO T88/311 standard methods for soil samples and T27-99 standard methods of testing for aggregate (coarse grained) samples is used on a representative sample taken in methods of reducing laboratory samples BS812-102. The sieve analysis test results for base and sub material from laboratory test result are shown below.

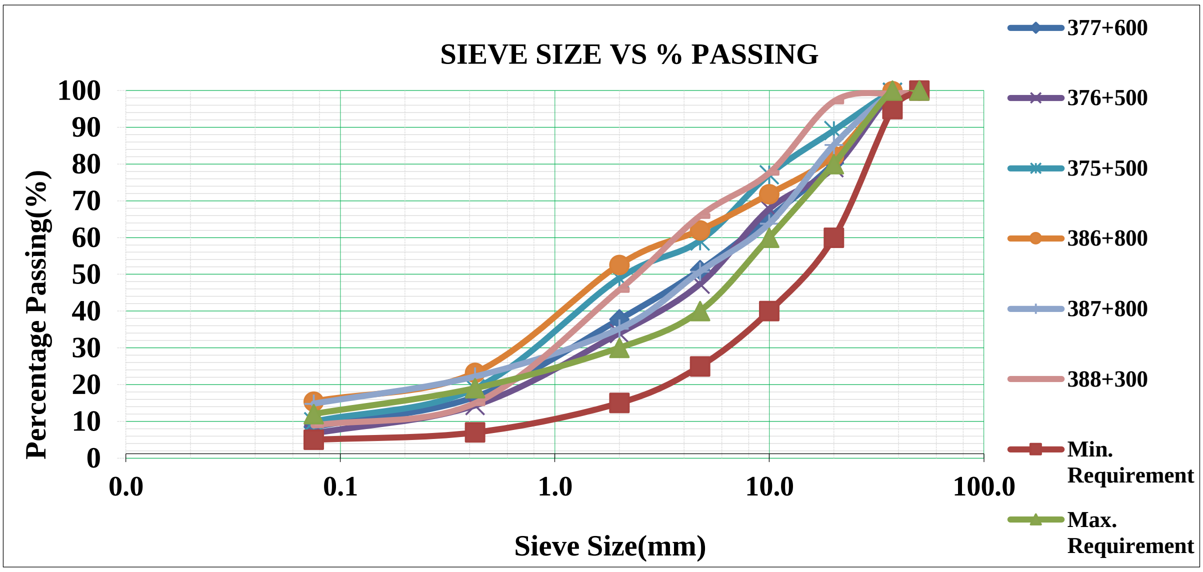

Figure 4-9 Sieve analyses results of base course material

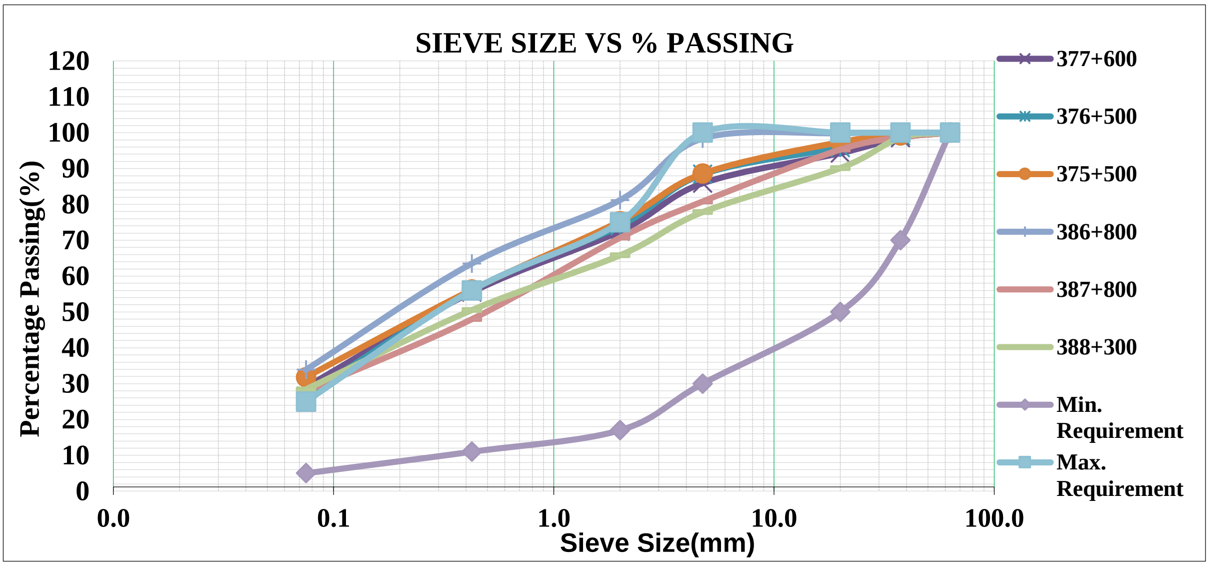

Figure 4-8 Sieve analyses results of sub base material

Mining the plots 4.8 and 4.9, comparing the laboratory test results for gradation with that of the specification, all base course and sub base materials of section 1 and section 2 are out of requirement specification envelop recommended by ERA and they are finer as shown on plots above. These results indicate that the samples contain more fine particles such high clay content could be responsible for instability of road pavement in the area and they have more affinity for water and high compressibility. This may cause due to using poorly graded or substandard quality materials during construction and break down of material by heavy traffic loading. Bearing in mind that, the specification proposed by carful investigation, the deviation in particle distribution would provide an inadequate material stability. This means the materials flex high, under the application of traffic load.

Improper gradation of material causes instability. That is the materials will not interlock and bonded together to produce a stable layer. This will cause settlement of the road structure over it and finally wearing course failure. This will further result deformation and pothole formation. Poor particle size distribution is the major causes for crocodile/ alligator cracking and rutting failures of failed sections of the study area. When this distress occur surface drainage will pond over it and seepage towards the layers underneath. The seepage increase moisture of pavement structures and embankment, and finally, the road will totally fail.

4.3.5. California Bearing Ratio (CBR)

The strength of the road subgrade for flexible pavements is commonly assessed in terms of the California Bearing Ratio (CBR) and this is dependent on the type of soil, its density, and its moisture content (ERA, 2013a). The higher the CBR value of a subgrade, the more strength it has to support the pavement (ERA, 2013b).

Due to unsealed shoulders and conditions of poor surface drainage, and crashed aggregate base course allows water to drain into the lower layers; the saturation of the sub-base is likely. Therefore, it is recommended that CBR is determined on samples soaked for a period of 96 hours or four days (ERA, 2013a).

According to ERA (2014) recommendation, the CBR value shall be greater than 30%, for sub base at 95 percent of maximum dry density and 100% percent of CBR, at 98 percent maximum dry density for base course materials. At the same manner, the CBR shall not be less than 5% at 95% of Modified AASHTO (T180) for embankment or subgrade fill and minimum 3% at 95% of modified AASHTO compaction (AASHTO method T-180) for natural subgrade material.

The California Bearing Ratio/CBR/ test at the maximum dry density results of all samples which soaked for four- days are shown below.

Table 4-7 CBR test results summery

Type of Sample Material	CBR (%) of Sampled Stations						Min. Specif. Requir. (%)
	Section 1			Section 2			
	377+600	376+500	375+500	386+800	387+800	388+300	
Base	98.19	101.44	97.02	93.25	90.55	91.17	100
Sub Base	15.88	18.92	19.91	13.17	16.58	15.09	30
Subgrade	14.24	13.45	14.93	14.88	17.20	16.99	5
Natural subgrade soil	-	-	-	2.54	1.57	2.84	3

Based on result from table 4.7, the average CBR value of in situ suitable subgrade at section 1 is 14.2% and selected fill sub grade material at section 2 is 16.36% which both section meets the requirement of minimum 5% CBR value on specification and the material is suitable to use as subgrade material when compacted at its optimum moisture content and compacted to its maximum dry density. At section 2, average CBR value of natural soil below selected fill result is 2.32% which is below 3% based on the requirement and that is why, the removal and replacement of this unsuitable soil were done during construction.

The average CBR value of sub base material of section 1 and section 2 is 18.24% and 14.95% respectively which both sections cannot meet the requirement of minimum 30% CBR value on specification and do not satisfy density criteria.

On the same manner, the CBR of the crushed base course material at section 1 is 98.88% averagely which is closed to the required minimum of 100% based on specification and also at section 2, the average compaction percentage is 91.66% which was less than the required of 100% based on specification and do not satisfy density criteria.

The lowering of CBR value of base and sub base material caused from substandard material quality, fault on construction, progressive material deterioration under heavy traffic loading and loss of support due to presence of water, inadequate compaction and poor or soft underlying materials. This accelerates deterioration of the pavement and responsible for rutting, cracking and potholes failures on this study area.

4.3.6. AASHTO Soil Classification

AASHTO classification system works based on particle-size distribution, liquid limit and plasticity index properties. The AASHTO classification system is used mainly for classification of highway sub-grades (Gowans, 2011). To classify a soil, apply the test data from left to right by process of elimination on Table 2.1 and the first group from the left into which the test data satisfy is the correct classification (AASHTO, 2001).

Table 4-8 AASHTO Classification of sub base material

Table 4-9 AASHTO Classification of natural ground soil below embankment fills.

Sections	Sampled Locations	Sieve Size			LL	PI	Group	Usual types of significant constituent materials
		2	0.425	0.075				
Section 1	377+600	72.50	55.5	29.00	42.38	13.71	A-2-7	Clayey gravel and sand
	376+500	74.00	56.00	26.50	39.05	14.34	A-2-6	Clayey gravel and sand
	375+500	75.30	56.20	31.80	44.12	15.40	A-2-7	Clayey gravel and sand
Section 2	386+800	81.16	63.50	33.90	45.69	19.42	A-2-7	Clayey gravel and sand
	387+800	70.68	47.98	27.88	41.35	15.29	A-2-7	Clayey gravel and sand
	388+300	65.78	50.50	28.40	39.72	14.51	A-2-6	Clayey gravel and sand

Road sections	Sampled Locations	Sieve Size			LL	PI	Group	Usual types of significant constituent materials
		2	0.425	0.075				
Section 2	386+800	99.40	93.60	80.00	100.24	61.99	A-7-5	Clayey soil
	387+800	98.60	88.80	72.60	92.73	45.81	A-7-5	Clayey soil
	388+300	98.80	90.20	75.40	97.93	52.11	A-7-5	Clayey soil

Table 4-10 AASHTO Classification of subgrade (suitable in-situ/selected fill)

		Sieve Size	LL	PI	Group	

Road Sections	Sampled Location	2	0.425	0.075				Usual types of significant constituent materials
Section 1	377+600	75.50	53.60	32.80	46.71	21.85	A-2-7	Clayey gravel and sand
	376+500	74.70	55.40	34.00	48.87	20.74	A-2-7	Clayey gravel and sand
	375+500	66.00	50.98	29.80	43.15	19.17	A-2-7	Clayey gravel and sand
Section 2	386+800	72.60	50.20	30.10	48.43	19.82	A-2-7	Clayey gravel and sand
	387+800	72.90	53.50	26.40	42.54	17.27	A-2-7	Clayey gravel and sand
	388+300	76.52	53.10	33.00	48.87	20.97	A-2-7	Clayey gravel and sand

As per AASHTO (2001) classification tables, materials falling within group A-1, A-2-4, A-2-5, A-2-6, A-2-7 and A-3 are excellent to good in that order when used as sub grade. Materials falling within group A-4, A-5, A-6 and A-7 are fair to poor in that order when used as sub grade. Soils classified in A-1, A-2-4, A-2-5, or A-3 group as in AASHTO M 145 shall be used as an embankment, when available, and shall compacted to the depth specified to not less than 95% of MDD as per AASHTO T99 (ERA, 2013a).

Based on above results, the sub base materials of tested samples at both sections are classified in A-2- 6 and A-2-7 group of clayey gravel and sand material type. Even if the AASHTO (2001) classification cannot describe sub base material rating, the materials contained finer materials and poor for sub base, even below the requirement as use an embankment.

All samples of suitable in situ subgrade of section1 and selected fill subgrade of section 2 were found to have comprised A-2-7 group of clayey gravel and sand material type which indicate the materials are good for subgrade. The natural sub grade materials under embankments of all tested section 2 were classified in A-7-5 group as most clayey soil which is greater than PL-30 and poor in order to use as subgrade material.

4.4. Proposed Remedial Measures

A number of repetitive maintenances were done on both section 1and section 2 by ERA regional shashemene district office as observed during field investigations. Patches and rigid concrete pavement overlaying rehabilitations types were done for failed stations. Patches repair were done on potholes, but after few months later the surface became fail again. This indicate that patches cannot proper maintenance methods potholes failures and the cause failure are structural problems induced from subgrade and pavement materials (base and sub base) rather than surface layers. Applying the wrong remedial treatment could be a waste of time and money. Concrete pavement over laying rehabilitations up to 20meter length with 7meter width were done on rutting, cracking and pothole failures of this road sections. Rehabilitations made by concrete pavement overlaying methods have good performance on maintained sections, but suddenly alligator cracks and potholes failures were initiated between flexible pavement and rigid concrete slab due to non-uniform transition of materials and piping were seen during observation. Concrete overlaying slabs cannot proper maintenance methods due to uneconomical solution using on all failed sections and other failures initiated suddenly beside of the maintained section due to non- uniform material transition and piping.

The results from field and laboratory investigations of the study indicates that the major causes of road failure at section 1 are improper design and late maintenance of side and cross drainages, seasonal ground water variations and poor geotechnical properties of road base and sub base of materials, especially on sub base materials. The moisture increase on sub grade and pavement materials increased due to flood flows generated from hill mountain which saturate the materials by edge flow through shoulder and infiltration through surface cracks results from improper design and late maintenance of side /cross drainage and the rise of ground water. The properties of road base and sub base lowered due to using substandard material quality, fault on construction, progressive material deterioration under heavy traffic loading and loss of support due to presence of water and inadequate compaction. Based on findings from above results the proper remedial measure suggested is reconstruction of drainage and pavement layers. The reconstruction includes, correcting drainage problems by providing adequate side and cross drainage structure (to prevent water migration in to pavement from mountains and providing sub surface drainage to remove the moisture inters on pavement and to control seasonal ground water variation) and replace the road base and sub base material by standard material as per standard specification and standard construction techniques and design which recommended by ERA.

Similarly, the major causes of road failure at section 2 are improper geometry design which results excess moisture on sub grade and pavement materials and poor geotechnical properties of road base and sub base of materials, especially on sub base materials. Excess moisture that saturate the subgrade and pavement materials results due to improper geometry design of the road which the top of the pavement surface was averagely 60cm higher than the side natural ground soil that no enough cover to prevent the side run-off from catchment that infiltrate through cracks and shoulder. Additionally, at this section the natural soil below embankment /sub grade fill is highly expansive and have low CBR value which result swell effect on embankment and piping due to capillary action. On the same manner the properties of road base and sub base under standard due to using substandard material quality, fault on construction, progressive material deterioration under heavy traffic loading and loss of support due to presence of water and inadequate compaction.

Reconstruction of pavement layers and correcting the geometry design is the suggested proper remedial measure suggested based on findings from above results. During reconstruction, increasing the height of the top of the pavement by constructing embankment fill and provide cross drainage to prevent side run off on pavement surface. Filing the embankment foundation by rock fill to increase the stability and to reduce swelling effect of natural black cotton soil on embankment fill. Replace the base and sub base course material by standard material as per standard specification and standard construction techniques and design which recommended by ERA.

5. SUMMARY AND CONCLUSIONS

5.1. General

In this chapter, summary has to be made on aggregate of filed investigations and laboratory tests results of the study. A common conclusion has to be made from findings obtained from result summaries.

5.2. Summary

Most of the roads constructed in Ethiopia fail prematurely before serving the design life due to various causes arising from many factors. One of these roads failed before reaching design life time is the Dodola - Goba road project. From total stretch of the project, this study focus on Adaba -Dinsho road section which was failed critically within 3(three) years after it has been opened for traffic and a number of maintenance had been performed to make the road passable. however the maintenance did not work. This study, investigate the real cause of the road failure along Adaba-Dinsho road section and propose proper remedial measurement. Detail field and laboratory investigations were carried out to the real cause of failures. On field investigation, visual inspection on actual condition of failed road with regards of geometry/design, geology, topographical and drainage characteristics of the road and physical measurements and sampling were carried out. Then laboratory investigations were conducted for quality tests on sampled materials used in this road construction i.e. sub grade, embankment, sub base and base course of failed stations. Field investigation and laboratory tests results on selected critically failed road sections (Section 1 and Section 2) as summarized below

The road sections (Section 1 and Section 2) were failed critically with high amounts which are not function as expected design life and causes road traffic accidents. Potholes, alligator cracks, edge cracks and rutting were majorly and repetitively observed types of failures as per visual field inspection. This indicates that cause of failure may due to subgrade material failure associated with presence of water/drainage problems and also pavement material failure are occur due to heavy traffic load, in adequate compaction and excess moisture as discussed in chapter two.

The geology, at section 1 covered by gray or bluish gray clay with highly weathered rock from Basalt rock type and at section 2 covered by residual block cotton clay soil which generated from highly weathered basic basalt rocks. This indicates that the area is so sensitive to moisture. Proper drainages were not provided on both failed road section due to poor design and the water direct back onto the road or cannot keep it from draining away and lack of maintenance of side drain and culverts were major problem on the road of sections. During sampling, the pavement materials (base and sub base) were highly saturated due to the water migrates into the pavement structure through combinations of surface infiltration through cracks in the surface layer and edge inflows from poor or inadequately drained side ditches. Additionally, at section1, the rise of ground water during rainy season intercepts cut slope and initiate land slide and slope failures which promoted from gravity induced by topographical or steep hillsides and increases the weight on a slope by ground water. Similarly at section 2, the ground water more closed to natural soil during the rainy season results the water migrate in to embankment and pavement structure from the underlying groundwater table by capillary potential in fine grained foundation soils. At section-2 the top of the pavement surface was averagely 60cm higher than the side natural ground soil due to improper geometry design which shows that there is no enough cover is provided to overcome the swell effect of the expansive soil and to prevent the side run-off from catchments which simply migrates into the pavement structure.

The average thickness of base course at section 1 and section 2 are less than the minimum thickness of 20 cm and the sub base course of the section 1 are also less than the minimum required thickness of 20cm as per design of the project which would not be able to carry the traffic loads throughout its service life. On the other hand, sub base course at section 2 is equal to the minimum required thickness as per approved design of the project and safe a head. The lowering of thickness may major contribute for rutting and potholes road failures.

The laboratory investigations results indicate, the percentage of compaction of sub grade material on both section 1 and section 2 are less than the requirement. On the same time, the field or in situ moisture content increased from laboratory optimum moisture content. These indicate, the decreasing of dry density may cause from inadequate compaction during constructions, loss of fine materials and heavy traffic load. The increasing of field moisture contents is due to poor drainage during the rainy season and due to infiltration through the cracks and potholes on the road surface. Decreasing of dry densities and increasing of field moisture content are major responsible causes of potholes, cracks and rutting failures of failed sections. The liquid limit and plasticity index of both section 1 and section 2 are below the maximum requirement values and safe a head. The CBR values of both section 1 and section 2 are above the minimum requirement and the material is suitable to use as subgrade material when compacted at its optimum moisture content and compacted to its maximum dry density and also the soil classified in A-2-7 group of clayey gravel and sand material type and good material for subgrade material, as per AASHTO (2001) classification.

The sub base material results the percentage of compaction on both section 1 and section 2 are less than the requirement and also the field or in situ moisture content are increased from laboratory optimum moisture content. The liquid limit and plastic index of natural gravel sub base material of section 1& 2 are above the specification requirement on the moist and wet tropical climate of the our study area and does not satisfy the requirement. The CBR value is below minimum requirement on both sections 1 & 2 and the materials are not suitable to use as sub-base material when compacted at its optimum moisture content and compacted to its maximum dry density. The gradation of this materials are out of requirement specification envelop recommended by ERA and the material have contain more fine particles such high clay content and they have more affinity for water and high compressibility. The soil classified in A-2-6 & A-2-7 group of clayey gravel and sand material type, as per AASHTO (2001) classifications which indicate the material contain finer and even below requirement to use as embankment material. Therefore, the sub base material shows, lower compaction degree, much moisture content, high plasticity index, low CBR and contains finer particles and that could be responsible for instability of road pavement in the area.

The percentage of compaction of base course material at section 1 is relatively good which closed to the requirement, whereas at section 2 percentage of compaction lower than minimum requirement. On the same manner the field or in situ moisture content has increased from laboratory optimum moisture content with some amount relatively at section1 and increased highly at section 2. The plastic index on both section 1and section 2 are above the required of non- plastic based on specification. The CBR value is relatively good at section 1 which is closed to the minimum requirement, whereas at section 2 CBR value is lower than minimum requirement. The gradation of this material on both section 1and section 2 are out of requirement specification envelop recommended by ERA and the material contain more fine particles such have high clay content and they have more affinity for water and high compressibility. Therefore, the quality test of base course material at section 1 is relatively meets the specification requirement except high plastic index and more contains finer particles and that could be responsible for instability of road. On other hand at section 2 the base course material is lower compaction degree, increased moisture content, high plasticity index, low CBR and contains finer particles and that could be responsible for instability.

Patches on potholes and concrete overlaying pavement rehabilitations on critical failures i.e. rutting, cracking and edge failures were done previously, were not the long term remedial measures on this failed section as shown from observation. This indicate that patches cannot proper maintenance methods potholes failures and the cause failure are structural problems induced from subgrade and pavement materials (base and sub base) rather than surface layers and also the concrete overlaying rehabilitation cannot the proper maintenance method due to uneconomical solution using on all failed sections and other failures initiated suddenly beside of the maintained section due to non- uniform material transition and piping.

At section 1 improper design and late maintenance of side and cross drainages, seasonal ground water variations are major factors for excess moisture condition. Lowered thickness, high plastic index and more contain finer particles of base coarse material and lower thickness, lower compaction degree, high plastic index, low CBR value and contain more fine particle on gradation sub base of materials due to substandard material quality, fault on construction, progressive material deterioration under heavy traffic load and loss of support due to excess water are the major factors for poor geotechnical properties. On the same manner at section 2, improper geometry design which not enough cover to prevent the side run-off from catchment that infiltrate through cracks and shoulder and seasonal ground water variations are major factors for excess moisture condition. And also at this section the natural soil below embankment /sub grade fill is highly expansive and have low CBR value which result swell effect on embankment and piping due to capillary action. Lower thickness, lower compaction degree, high plastic index, low CBR value and contain more fine particle on gradation of base and sub base of materials due to substandard material quality, fault on construction, progressive material deterioration under heavy traffic load and loss of support due to excess water are the major factors for poor geotechnical properties.

Based on findings from above results reconstruction of drainage and pavement layers for section 1 failures and reconstruction of pavement layers and correcting the geometry design for section 2 are the suggested proper remedial measure. The reconstruction at section 1 includes, correcting drainage problems by providing adequate side and cross drainage structure. Filing the subgrade by rock fill to free movement of water and to reduce effect water pressure and also replace the road base and sub base material by standard material as per standard specification and standard construction techniques which recommended by ERA. Additionally, at section1, reducing steepness of hill side slope by excavation to reduce the weight of slopes, diversion of run offs, constructing filters and horizontal drains at different elevation along the slopes to prevent wash out of fines and save passage of water and constructing retaining structures are the proper remedial measure for slope failure and land slide.

At section 2 the reconstruction includes correcting the geometry design is by increasing the height of the top of the pavement by constructing embankment fill and provide cross drainage to prevent side run off on pavement surface. Filing the embankment foundation by rock fill to increase the stability and to reduce swelling effect of natural black cotton soil on embankment fill. Replace the base and sub base course material by standard material as per standard specification and standard construction techniques and design which recommended by ERA.

5.3. Conclusions

Based on the study findings from field and laboratory investigations the following conclusions are drawn.

1. The investigation revealed that two critically failed road sections (section 1 & section 2) which failed majorly and repetitively by potholes, alligator cracks, edge cracks and rutting failure types.
2. The field and laboratory investigations indicates, presence of too much moisture content on sub grade and pavement materials due to improper design and late maintenance of drainage, improper geometrical design, moisture sensitive geology/soil and seasonal variation of ground water, poor geotechnical properties of road base and sub base of materials due to substandard material quality, improper construction methods, progressive material deterioration under heavy traffic load and loss of support due to excess water are the major causes of road failures on critically failed sections of this road.
3. The results of section 1 indicates

a) Improper design and late maintenance of side and cross drainages, seasonal ground water variation results increase field moisture content and reducing field dry densities on sub grade and sub base materials which cause of failure.
b) Insufficient thickness of base and sub base layer than the minimum required design thickness and not be able to carry the traffic loads and basic cause for failed section.
c) Poor geotechnical properties on sub-base materials which have high plasticity index, low CBR and improper gradation that contains more fine particles and the material also classified in A-2-6 & A-2-7 group of clayey gravel and sand material type which is poor for sub base, even below the requirement as use an embankment. We can conclude that sub base qualities are the main causes of failures. And also on base coarse material which have high plastic index and improper gradation that contains more fine particles.
d) Land slide and slope failures initiated due to the rise of ground water during rainy season intercepts cut slope increases the weight on a slope and steep hillside slope or topographical.

4. The results of section 2 indicates
   a) Improper geometry design which is not enough cover to prevent the side run-off from catchment results and rise of ground water which results increase field moisture content and reducing field dry densities on sub grade, sub base and base coarse materials.
   b) The natural soil below embankment /sub grade fill is highly expansive and have low CBR value which result swell effect and piping due to capillary action on embankment.
   c) Insufficient thickness of sub base layer than the minimum required design thickness and not be able to carry the traffic loads.
   d) Poor geotechnical properties on base and sub-base materials which have high plasticity index, low CBR and improper gradation that contains more fine particles. Additionally, the sub base material is classified in A-2-6 & A-2-7 group of clayey gravel and sand material type which is poor for sub base, even below the requirement as use an embankment.
5. Previous maintenance methods of patches and concrete overlaying are not long term remedial measures for both failed sections due to the cause of failures are structural

problems induced from excess moisture on or in subgrade and pavement materials (base and sub base) and rather than surface layers to patches on potholes and also the concrete overlaying rehabilitation cannot the proper maintenance method due to uneconomical solution using on all failed sections and other failures initiated suddenly beside of the maintained section due to non- uniform material transition and piping.

6. Reconstruction of drainage system, correcting the geometry design and of pavement layers are the suggested proper remedial measure for failed section. The reconstruction take account of
   a) At section 1 correcting drainage problems by providing adequate side and cross drainage structure. Filing the subgrade by rock fill to free movement of water and to reduce effect water pressure and also removing substandard materials of road base and sub base replace by standard material. Additionally, reducing steepness of hill side slope by excavation to reduce the weight of slopes, diversion of run offs, constructing filters and horizontal drains at different elevation along the slopes to prevent wash out of fines and save passage of water and constructing retaining structures are the proper remedial measure for slope failure and land slide.
   b) At section 2 correcting the geometry design is by increasing the height of the top of the pavement by constructing embankment fill and provide cross drainage to prevent side run off on pavement surface. Filing the embankment foundation by rock fill to increase the stability and to reduce swelling effect of natural black cotton soil on embankment fill. Replace the base and sub base course material by standard material as per standard specification and standard construction techniques and design which recommended by ERA.

5.4. Recommendations

To the end, the following recommendations are proposed.

a) Further investigation on traffic load analysis and properties of AC material shall be done by pavement design concept to identify the influence on pavement performance hence this study focus on geotechnical investigation.

b) Adequate routine and emergency maintenance of drainage and culverts shall be done on road constructed on cut section to prevent silt up by off stone dust, sand and other debris.
c) During repair or rehabilitation by reconstruction for failed section of this study, the quality pavement material shall be bearing in mind the climate condition of the area. Because climate is the major factor for selection of material quality, particularly for sub base materials.
d) The real cause of failures shall be determined before undertaking maintenance or repair on other failed section of this road. Applying the wrong remedial treatment could be a waste of time and money
e) Detail investigation should be carried out in project areas; also the properties of material and method of construction should be according to the design specification of the project in order to serve the design period of a project in order to avoid the failure.
f) It is further recommended that in-depth investigation should be undertaken on the other parts of Ethiopia, where asphalt pavement is to be constructed with similar soil types and topography to avoid similar problems that have been encountered within the study area.

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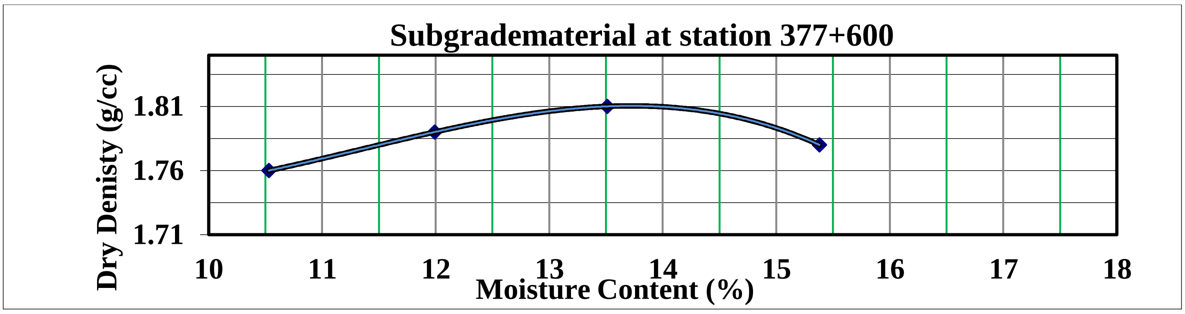
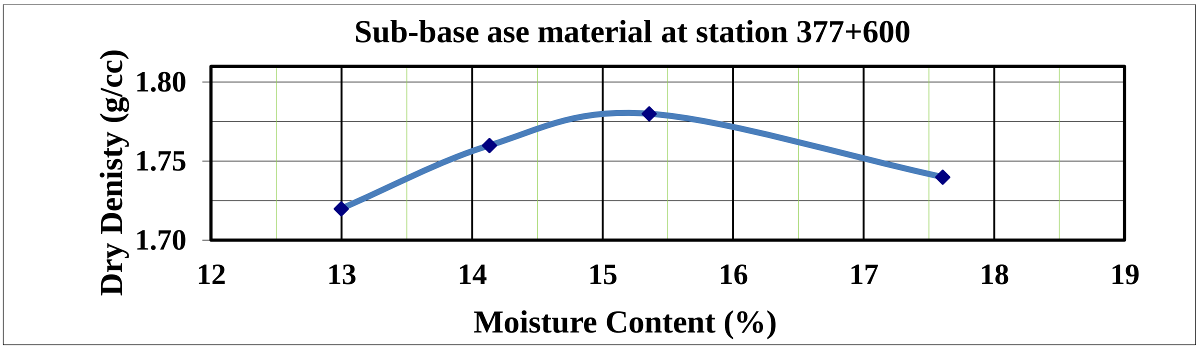
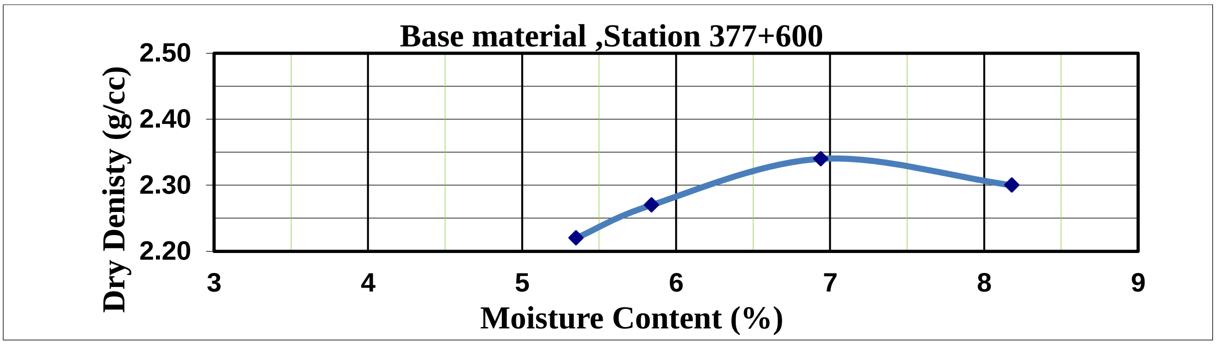
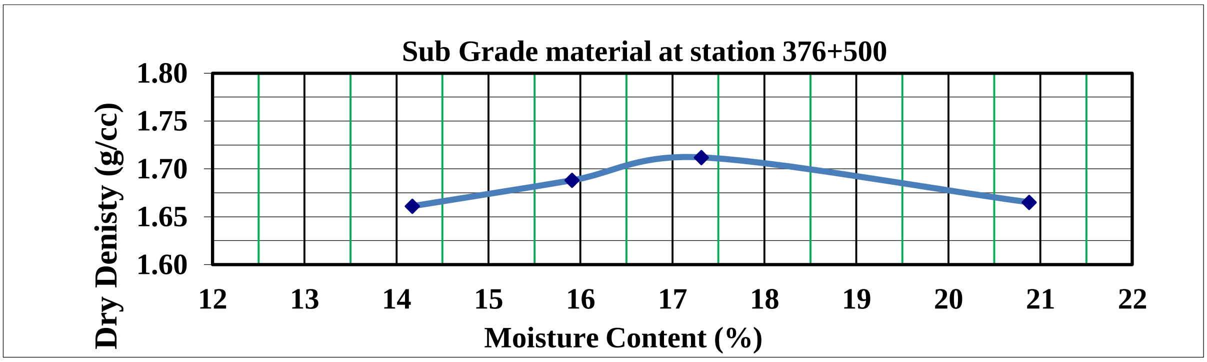
APPENDIXES

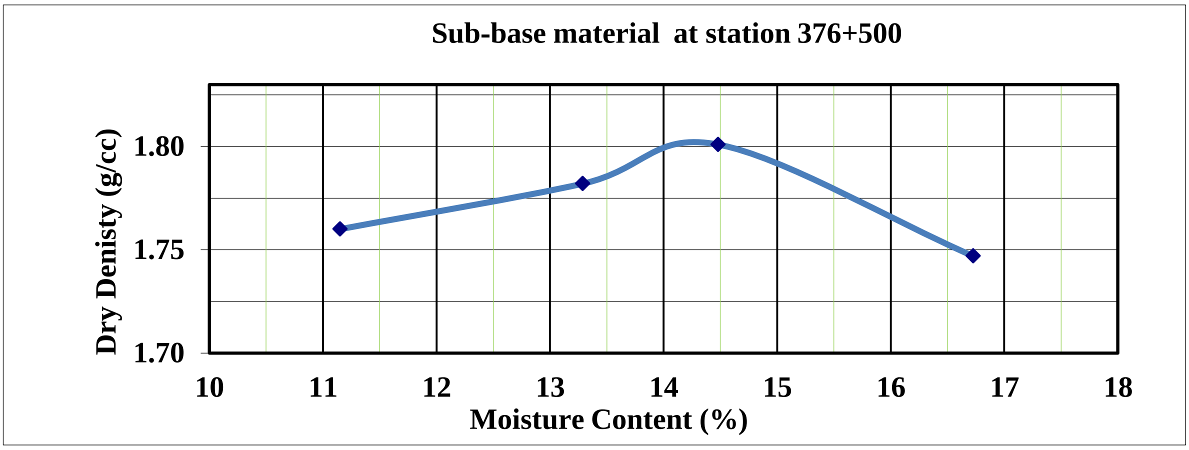
Appendix A: Compaction Test (Moisture Density Relation)

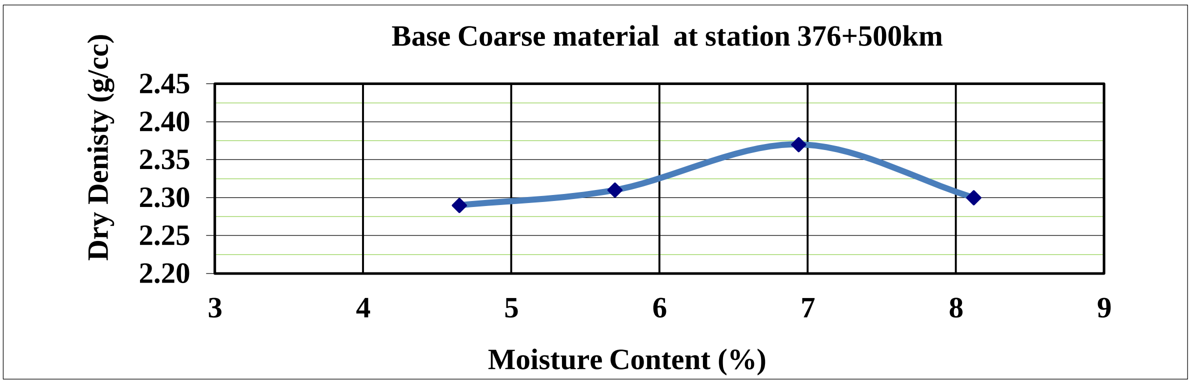


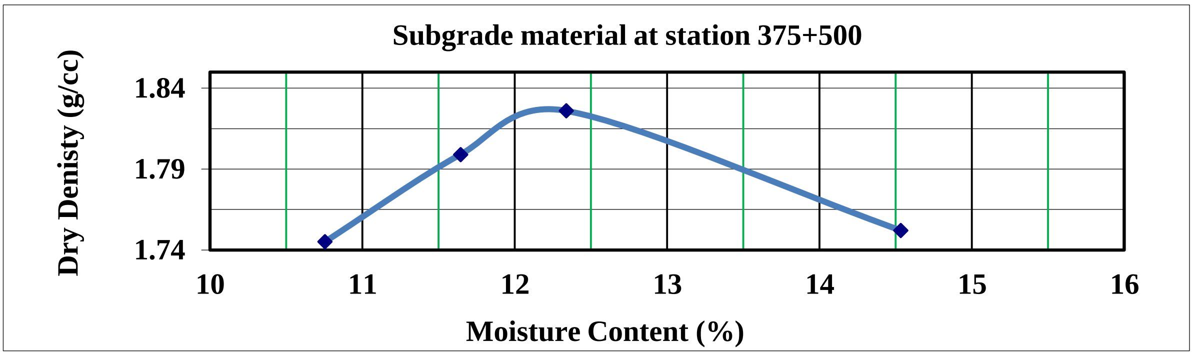


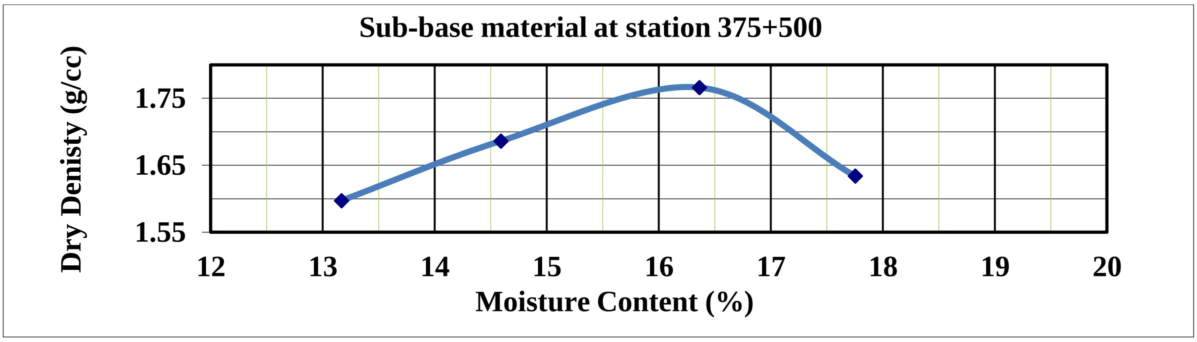


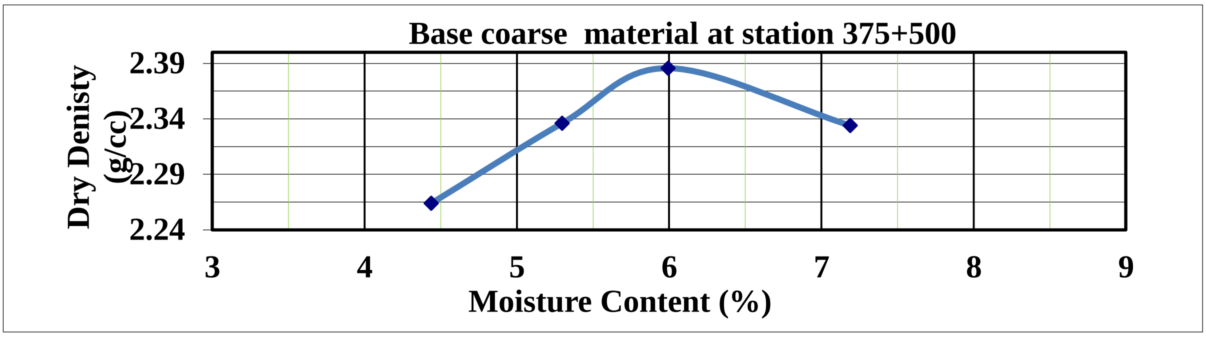


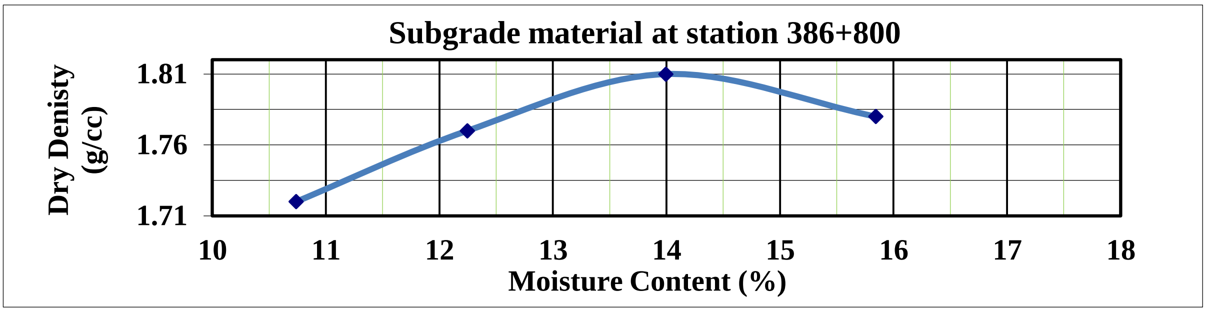

Sub-base material at station 376+500
Dry Denisty (g/cc)
1.80
1.75
1.70
10
11
12
13
14
15
16
17
18
Moisture Content (%)

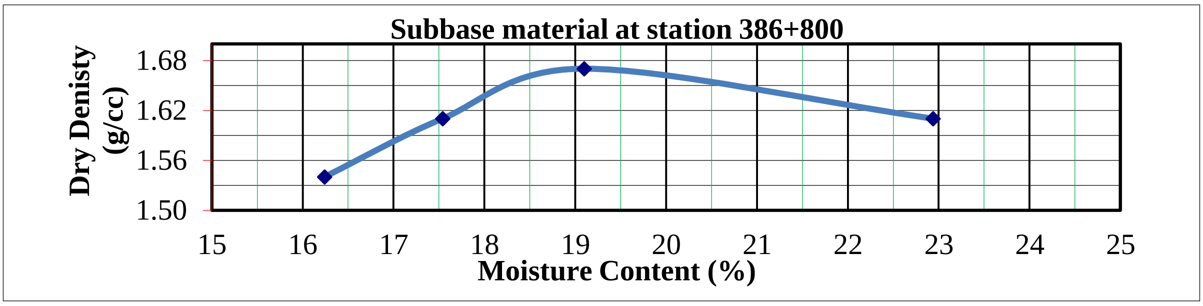

Base Coarse material at station 376+500km
Dry Denisty (g/cc)
2.45
2.40
2.35
2.30
2.25
2.20
3
4
5
6
7
8
9
Moisture Content (%)

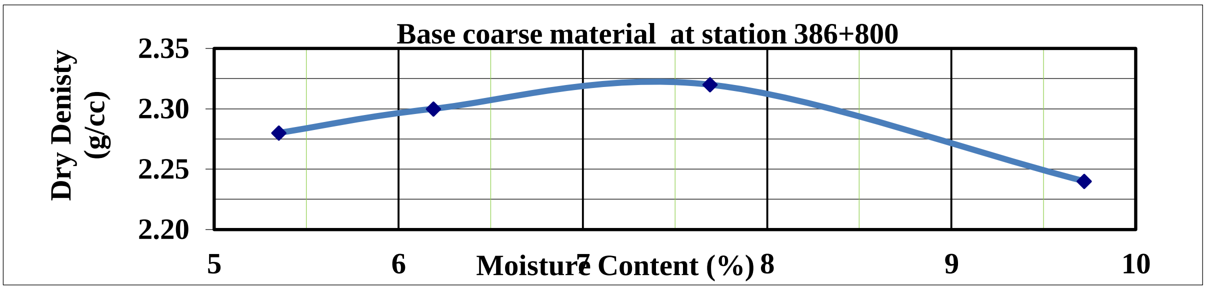

Subgrade material at station 375+500
Dry Denisty (g/cc)
1.84
1.79
1.74
10
11
12
13
14
15
16
Moisture Content (%)

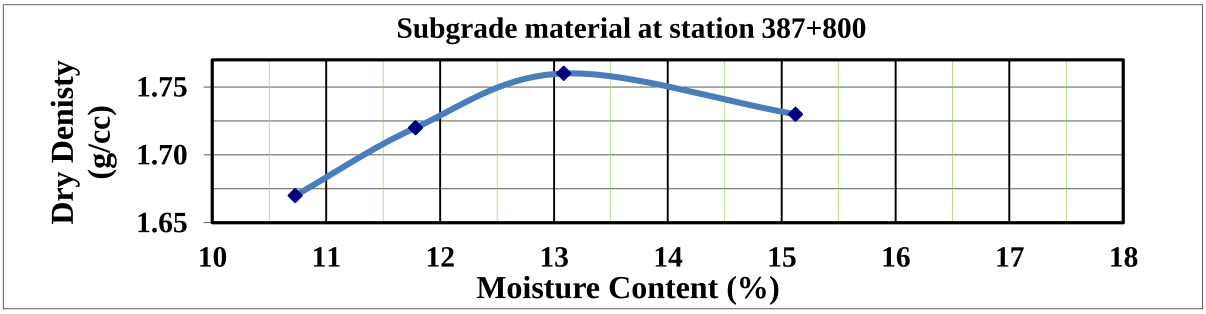

Sub-base material at station 375+500
Dry Denisty (g/cc)
1.75
1.65
1.55
12
13
14
15
16
17
18
19
20
Moisture Content (%)

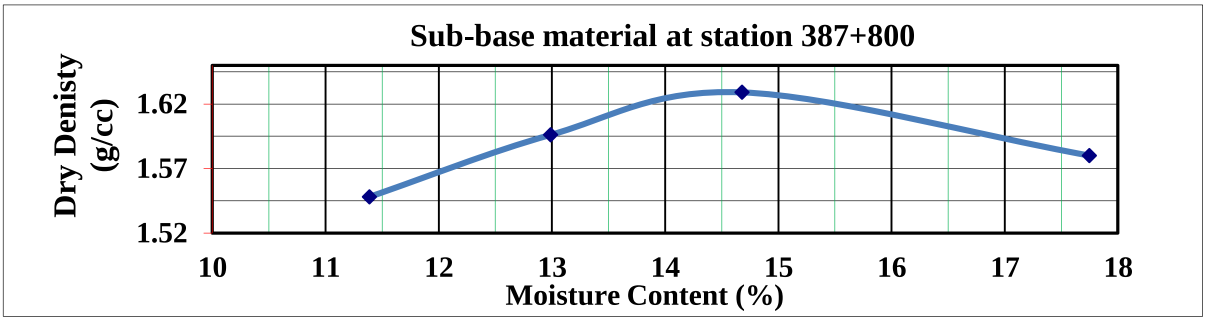

Base coarse material at station 375+500
Dry Denisty (g/cc)
2.39
2.34
2.29
2.24
3
4
5
6
7
8
9
Moisture Content (%)

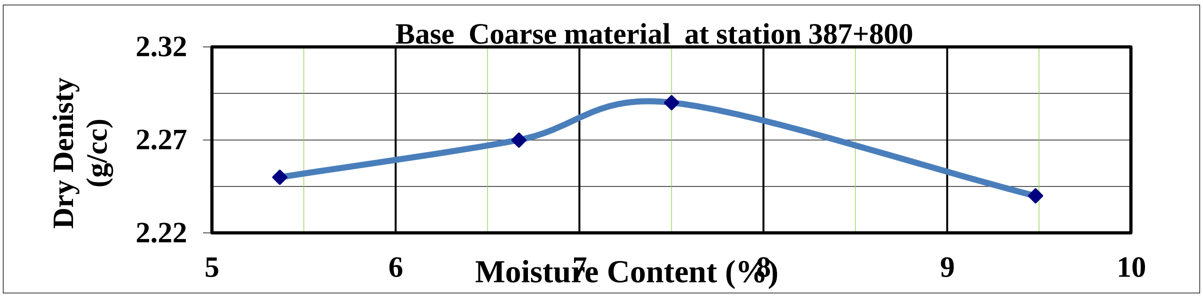

Subgrade material at station 386+800
Dry Denisty (g/cc)
1.81
1.76
1.71
10
11
12
13
14
15
16
17
18
Moisture Content (%)

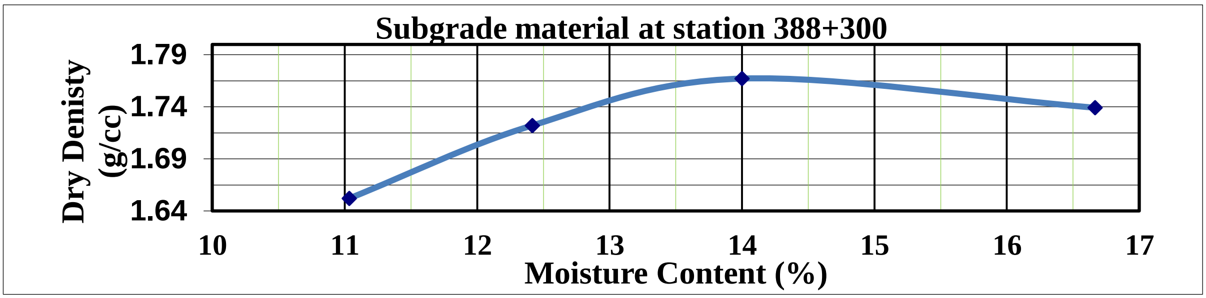

Subbase material at station 386+800
Dry Denisty (g/cc)
1.68
1.62
1.56
1.50
15
16
17
18
19
20
21
22
23
24
25
Moisture Content (%)

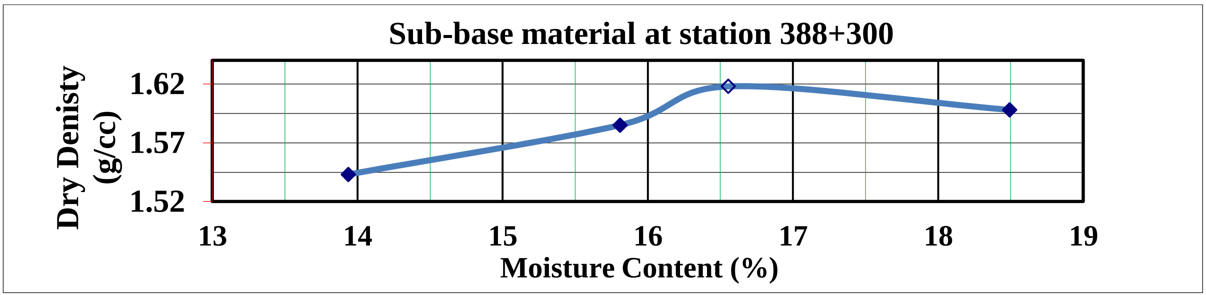

Base coarse material at station 386+800
Dry Denisty (g/cc)
2.35
2.30
2.25
2.20
5
6
7
8
9
10
Moisture Content (%)

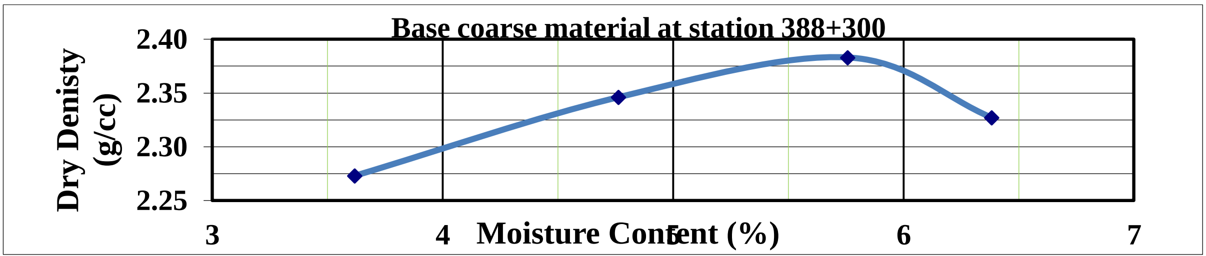

Subgrade material at station 387+800
Dry Denisty (g/cc)
1.75
1.70
1.65
10
11
12
13
14
15
16
17
18
Moisture Content (%)


Sub-base material at station 387+800
Dry Denisty
(g/cc)
1.62
1.57
1.52
10
11
12
13
14
15
16
17
18
Moisture Content (%)


Base Coarse material at station 387+800
Dry Denisty
(g/cc)
2.32
2.27
2.22
5
6
7
8
9
10
Moisture Content (%)


Subgrade material at station 388+300
Dry Denisty
(g/cc)
1.79
1.74
1.69
1.64
10
11
12
13
14
15
16
17
Moisture Content (%)


Sub-base material at station 388+300
Dry Denisty
(g/cc)
1.62
1.57
1.52
13
14
15
16
17
18
19
Moisture Content (%)


Base coarse material at station 388+300
Dry Denisty
(g/cc)
2.40
2.35
2.30
2.25
3
4
5
6
7
Moisture Content (%)

Appendix B: Field Density and In-situ Moisture Test

No.	Description	Unit	377+600			376+500			375+500		
			Sample Type			Sample Type			Sample Type		
			Base Coarse	Sub-Base	Sub Grade	Base Coarse	Sub-Base	Sub Grade	Base Coarse	Sub-Base	Sub Grade
Bulk Density	Wt of sand in hole	gm	2,322.0	2,495.0	2,459.5	2,444.0	2,600.0	2,663.5	2,466.0	2,476.0	3,246.0
	Density of sand used	gm/cc	1.26	1.26	1.26	1.26	1.26	1.26	1.26	1.26	1.26
	Volume of sand in hole	gm	1,842.9	1,980.2	1,952.0	1,939.7	2,063.5	2,113.9	1,957.1	1,965.1	2,576.2
	Bulk Density of Wet Soil	gm/cc	2.45	1.93	1.76	2.49	1.92	1.80	2.45	1.88	1.86
Moisture Content Determinations	Container No.										
	Wt of wet soil+Container	gm	387.00	382.00	384.00	394.00	391.00	412.00	386.00	415.00	403.00
	Wt of dry soil+Container	gm	368.50	343.00	345.00	375.00	353.00	361.00	370.00	368.00	365.00
	Wt of water	gm	18.50	39.00	39.00	19.00	38.00	51.00	16.00	47.00	38.00
	Wt of Container	gm	115.00	115.00	115.00	115.00	115.00	115.00	115.00	115.00	115.00
	Wt of dry soil	gm	253.50	228.00	230.00	260.00	238.00	246.00	255.00	253.00	250.00
	Moisture Content	%	7.30	17.11	16.96	7.31	15.97	20.73	6.27	18.58	15.20
Field dry density		gm/cc	2.29	1.64	1.51	2.32	1.65	1.49	2.31	1.59	1.61

No.	Description	Unit	386+800			387+800			388+300		
			Sample Type			Sample Type			Sample Type		
			Base Coarse	Sub-Base	Sub Grade	Base Coarse	Sub-Base	Sub Grade	Base Coarse	Sub-Base	Sub Grade
Bulk Density Determinations	Wt of sand in hole	gm	2,916.	2,569.0	2,975.0	3,124.0	3,216.0	3,208.0	3,098.0	2,669.0	2,723.
	Density of sand used	gm/cc	1.26	1.26	1.26	1.26	1.26	1.26	1.26	1.26	1.26
	Volume of sand in hole	gm	2,314.3	2,038.9	2,361.1	2,479.4	2,552.4	2,546.0	2,458.7	2,118.2	2,161. 1
	Bulk Density of Wet Soil	gm/cc	2.39	1.79	1.90	2.28	1.66	1.84	2.38	1.73	1.82
Moisture Content Determinations	Container No.										
	Wt of wet soil+Container	gm	451.00	342.00	381.00	455.00	372.00	374.00	451.00	348.00	386.00
	Wt of dry soil+Container	gm	425.50	301.00	344.00	429.00	334.00	341.00	431.00	311.00	349.00
	Wt of water	gm	25.50	41.00	37.00	26.00	38.00	33.00	20.00	37.00	37.00
	Wt of Container	gm	115.00	115.00	115.00	115.00	115.00	115.00	115.00	115.00	115.00
	Wt of dry soil	gm	310.50	186.00	229.00	314.00	219.00	226.00	316.00	196.00	234.00
	Moisture Content	%	8.21	22.04	16.16	8.28	17.35	14.60	6.33	18.88	15.81
Field dry density		gm/cc	2.21	1.47	1.64	2.11	1.42	1.61	2.23	1.45	1.57

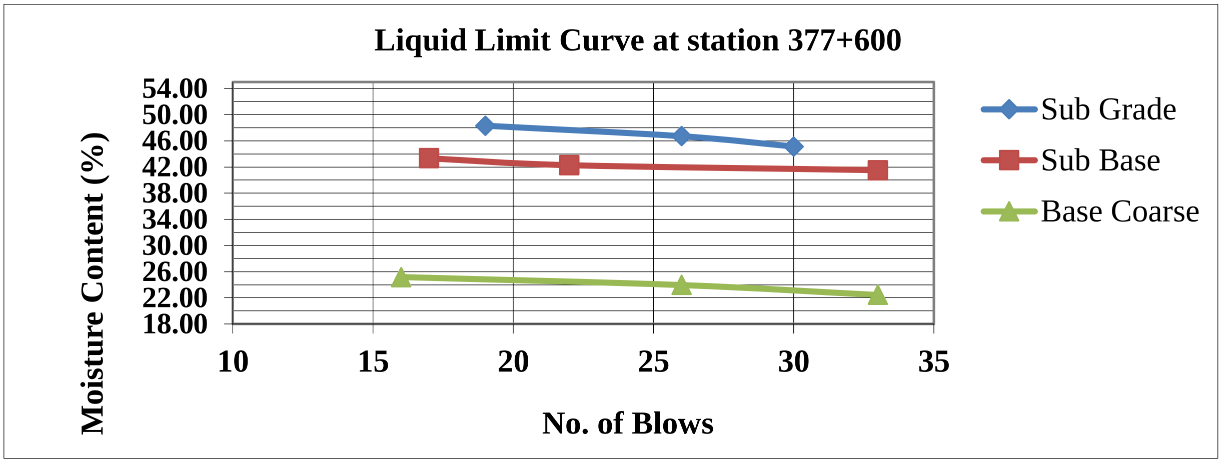
Appendix C: Atterberg Limits Test Results

Sub Grade Material	**Sampled Station:-377+600**				
	Liquid Limit (%)			**Plastic Limit (%)**	
Container No.	B1	B2	B3	Q4	J4
No. of Blows	30	26	19		
Wt. of cont. + wet soil (g) = (w_1)	41.48	41.86	41.98	26.92	26.96
Wt. of cont. + dry soil (g.) = (w_2)	35.60	35.70	35.68	26.08	26.14
Wt. of container (g.) = (w_3)	22.57	22.51	22.64	22.64	22.90
Mass of moisture (g.) (w_1-w_2) = x	5.88	6.16	6.30	0.84	0.82
Wt. of dry soil (g.) (w_2-w_3) = y	13.03	13.19	13.04	3.44	3.24
Moisture Content (%) = (100x/y)	45.13	46.70	48.31	24.42	25.31
Average Moisture Content (%)	----			**24.86**	
Moisture Content (%) at 25 no of blows From Liquid Limit Curve	**46.71**			**---**	
Plasticity Index = Liquid Limit - Plastic Limit				**21.85**	

Sub Base Material	**Sampled Station:-377+600**				
	Liquid Limit (%)			**Plastic Limit (%)**	
Container No.	A1	A2	A3	A5	M1
No. of Blows	33	22	17		
Wt. of cont. + wet soil (g) = (w_1)	42.01	42.20	42.77	26.55	26.56
Wt. of cont. + dry soil (g.) = (w_2)	36.24	36.42	36.72	25.66	25.75
Wt. of container (g.) = (w_3)	22.34	22.75	22.76	22.56	22.92
Mass of moisture (g.) (w_1-w_2) = x	5.77	5.78	6.05	0.89	0.81
Wt. of dry soil (g.) (w_2-w_3) = y	13.90	13.67	13.96	3.10	2.83
Moisture Content (%) = (100x/y)	41.51	42.28	43.34	28.71	28.62
Average Moisture Content (%)	----			**28.67**	
Moisture Content (%) at 25 no of blows From Liquid Limit Curve	**42.38**			**-----**	
Plasticity Index = Liquid Limit - Plastic Limit				**13.71**	

Base Material	**Sampled Station:-377+600**				
	Liquid Limit			**Plastic Limit**	
Container No.	C1	C2	C3	C4	C5
No. of Blows	33	26	16		
Wt. of cont. + wet soil (g) = (w_1)	43.08	43.19	43.35	27.53	27.77
Wt. of cont. + dry soil (g.) = (w_2)	39.31	39.18	39.16	26.78	27.02
Wt. of container (g.) (w_3)	22.52	22.44	22.51	22.63	22.89
Mass of moisture (g.) (w_1-w_2) = x	3.77	4.01	4.19	0.75	0.75

Wt. of dry soil (g.) (w_2-w_3) = y	16.79	16.74	16.65	4.15	4.13
Moisture Content (%) = (100x/y)	22.45	23.95	25.17	18.07	18.16
Average Moisture Content (%)	----			18.12	
Moisture Content (%) at 25 no of blows from Liquid Limit Curve	**23.86**			-----	
Plasticity Index = Liquid Limit - Plastic Limit				**5.74**	

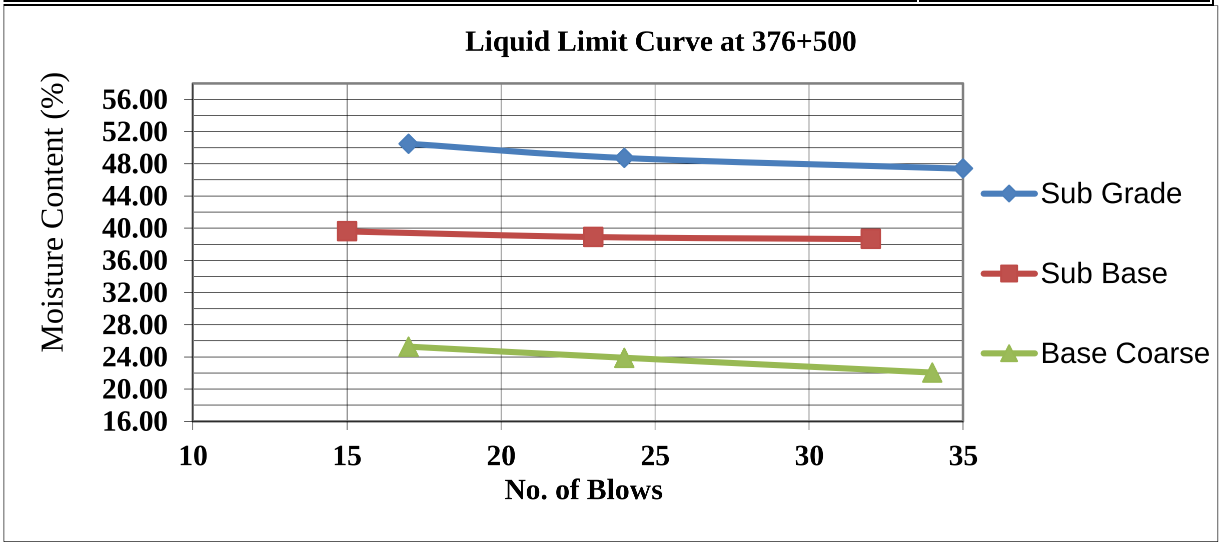


Sub Grade Material	**Sampled Station:-376+500**				
	Liquid Limit			**Plastic Limit**	
Container No.	B1	B2	B3	B4	B5
No. of Blows	35	24	17		
Wt. of cont. + wet soil (g) = (w_1)	41.51	42.28	42.88	26.11	26.14
Wt. of cont. + dry soil (g.) = (w_2)	35.42	35.80	36.09	25.38	25.40
Wt. of container (g.) = (w_3)	22.57	22.50	22.64	22.64	22.90
Mass of moisture (g.) (w_1-w_2) = x	6.09	6.48	6.79	0.73	0.74
Wt. of dry soil (g.) (w_2-w_3) = y	12.85	13.30	13.45	2.74	2.50
Moisture Content (%) = (100x/y)	47.39	48.72	50.48	26.64	29.60
Average Moisture Content (%)	---			**28.12**	
Moisture Content (%) at 25 blows from Liquid Limit Curve	**48.87**			-----	
Plasticity Index = Liquid Limit-Plastic Limit				**20.74**	

Sub Base Material	**Sampled Station:-376+500**				
	Liquid Limit			**Plastic Limit**	
Container No.	G1	G2	G3	K6	k4
No. of Blows	32	23	15		
Wt. of cont. + wet soil (g) = (w_1)	41.68	41.84	41.97	26.36	26.38
Wt. of cont. + dry soil (g.) = (w_2)	36.32	36.44	36.52	25.61	25.64

Wt. of container (g.) = (w_3)	22.45	22.56	22.76	22.56	22.66
Mass of moisture (g.) (w_1-w_2) = x	5.36	5.40	5.45	0.75	0.74
Wt. of dry soil (g.) (w_2-w_3) = y	13.87	13.88	13.76	3.05	2.98
Moisture Content (%) = (100x/y)	38.64	38.90	39.61	24.59	24.83
Average Moisture Content (%)				**24.71**	
Moisture Content (%) at 25 blows from Liquid Limit Curve	**39.05**				
Plasticity Index = Liquid Limit-Plastic Limit				**14.34**	

Base Material	**Sampled Station:-376+500**				
	Liquid Limit			**Plastic Limit**	
Container No.	C1	C2	C3	C4	C5
No. of Blows	34	24	17		
Wt. of cont. + wet soil (g) = (w_1)	43.04	43.26	43.34	27.56	27.74
Wt. of cont. + dry soil (g.) = (w_2)	39.34	39.26	39.14	26.78	26.98
Wt. of container (g.) = (w_3)	22.56	22.51	22.51	22.63	22.89
Mass of moisture (g.) (w_1-w_2) = x	3.70	4.00	4.20	0.78	0.76
Wt. of dry soil (g.) (w_2-w_3) = y	16.78	16.75	16.63	4.15	4.09
Moisture Content (%) = (100x/y)	22.05	23.88	25.26	18.80	18.58
Average Moisture Content (%)				**18.69**	
Moisture Content (%) at 25 blows from Liquid Limit Curve	**23.73**				
Plasticity Index = Liquid Limit-Plastic Limit				**5.04**	

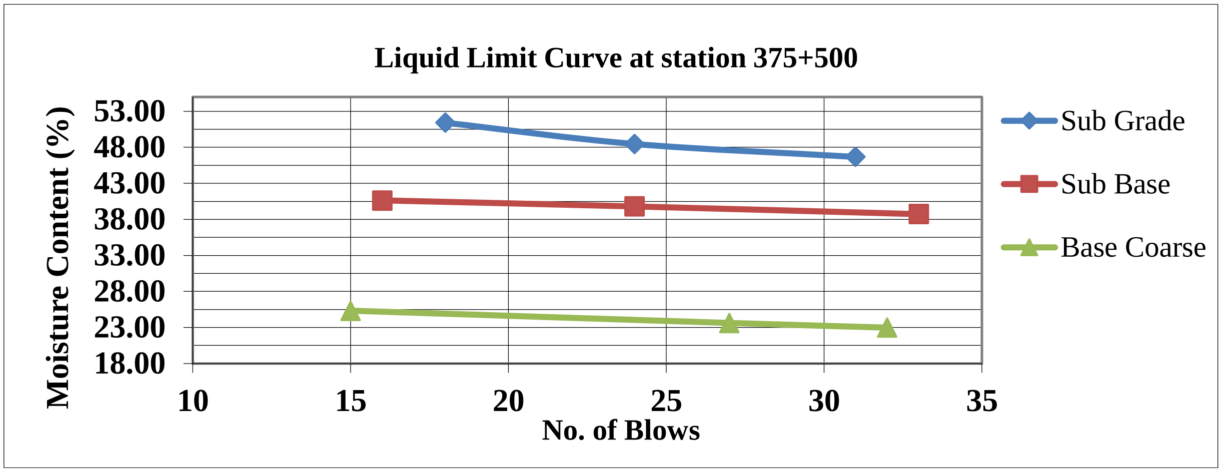


Sub Grade Material	Sampled Station:-375+500				
	Liquid Limit			Plastic Limit	
Container No.	Q1	Q2	Q3	Q4	Q5
No. of Blows	31	24	18		
Wt. of cont. + wet soil (g) = (w_1)	41.39	41.56	41.69	27.01	27.02
Wt. of cont. + dry soil (g.) = (w_2)	35.40	35.34	35.22	26.08	26.10
Wt. of container (g.) = (w_3)	22.57	22.51	22.64	22.64	22.90
Mass of moisture (g.) (w_1-w_2) = x	5.99	6.22	6.47	0.93	0.92
Wt. of dry soil (g.) (w_2-w_3) = y	12.83	12.83	12.58	3.44	3.20
Moisture Content (%) = (100x/y)	46.69	48.48	51.43	27.03	28.75
Average Moisture Content (%)	------			27.89	
Moisture Content (%) at 25 blows from Liquid Limit Curve	48.87			-----	
Plasticity Index (%) = Liquid Limit (%)-Plastic Limit(%)				**20.97**	

Sub Base Material	Sampled Station:-375+500				
	Liquid Limit			Plastic Limit	
Container No.	K7	K8	K9	VD1	VD2
No. of Blows	33	24	16		
Wt. of cont. + wet soil (g) = (w_1)	38.62	38.94	39.21	28.85	28.86
Wt. of cont. + dry soil (g.) = (w_2)	34.11	34.24	34.38	27.64	27.66
Wt. of container (g.) = (w_3)	22.47	22.43	22.49	22.86	22.88
Mass of moisture (g.) (w_1-w_2) = x	4.51	4.70	4.83	1.21	1.20
Wt. of dry soil (g.) (w_2-w_3) = y	11.64	11.81	11.89	4.78	4.78
Moisture Content (%) = (100x/y)	38.75	39.80	40.62	25.31	25.10
Average Moisture Content (%)	---------			25.21	
Moisture Content (%) at 25 blows from Liquid Limit Curve	39.72			-----	
Plasticity Index(%) = Liquid Limit- Plastic Limit				**14.51**	

Base Coarse Material	Sampled Station:-375+500				
	Liquid Limit			Plastic Limit	
Container No.	K1	K2	F13	F11	K14
No. of Blows	32	27	15		
Wt. of cont. + wet soil (g) = (w_1)	42.98	43.03	43.22	27.48	27.50
Wt. of cont. + dry soil (g.) = (w_2)	39.14	39.09	39.04	26.69	26.80
Wt. of container (g.) = (w_3)	22.44	22.42	22.53	22.58	22.88
Mass of moisture (g.) (w_1-w_2) = x	3.84	3.94	4.18	0.79	0.70
Wt. of dry soil (g.) (w_2-w_3) = y	16.70	16.67	16.51	4.11	3.92
Moisture Content (%) = (100x/y)	22.99	23.64	25.32	19.22	17.86

Average Moisture Content (%)	**---------**	**18.54**
Moisture Content (%) at 25 blows from Liquid Limit Curve	**23.98**	**-----**
Plasticity Index = Liquid Limit-Plastic Limit		**5.44**



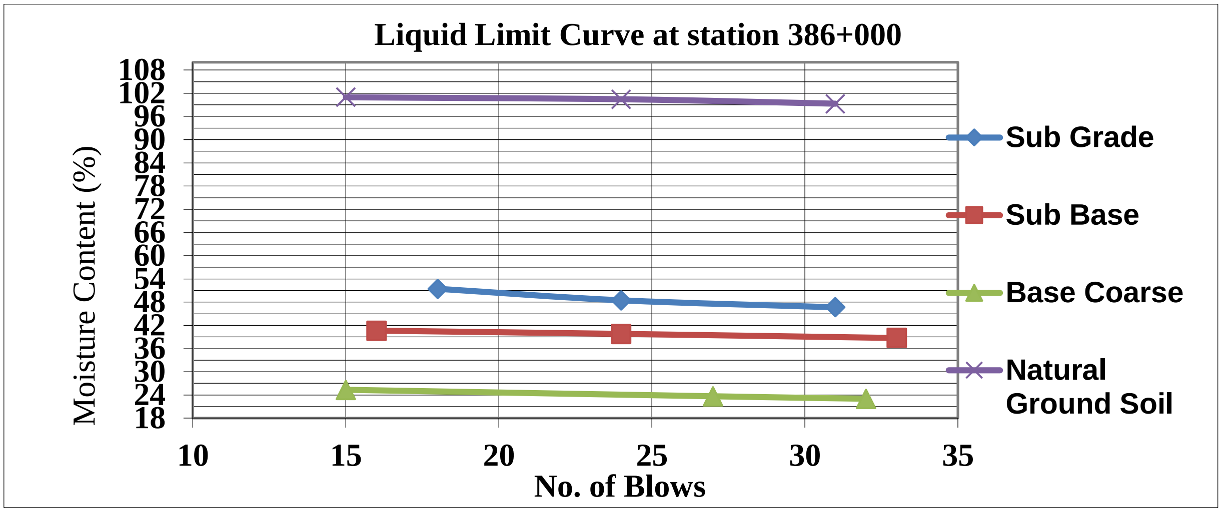
Natural Ground Soil	**Sampled Station:-386+800**				
	Liquid Limit			**Plastic Limit**	
Container No.	CK1	A1	CK2	J4	J7
No. of Blows	31	24	15		
Wt. of cont. + wet soil (g) = (w_1)	45.03	39.60	41.25	26.37	28.03
Wt. of cont. + dry soil (g.) = (w_2)	33.64	30.94	31.90	25.16	26.35
Wt. of container (g.) = (w_3)	22.17	22.32	22.64	22.01	21.94
Mass of moisture (g.) (w_1-w_2) = x	11.39	8.66	9.35	1.21	1.68
Wt. of dry soil (g.) (w_2-w_3) = y	11.47	8.62	9.26	3.15	4.41
Moisture Content (%) = (100x/y)	99.30	100.46	100.97	38.41	38.10
Average Moisture Content (%)	**---------**			**38.25**	
Moisture Content (%) at 25 blows from liquid limit curve	**100.24**			**-----**	
Plasticity Index = Liquid Limit-Plastic Limit				**61.99**	

Sub Grade Soil	**Sampled Station:-386+800**				
	Liquid Limit			**Plastic Limit**	
Container No.	A1	A2	A3	A4	A5
No. of Blows	33	24	18		
Wt. of cont. + wet soil (g) = (w_1)	42.32	42.90	42.96	27.20	27.30
Wt. of cont. + dry soil (g.) = (w_2)	35.92	36.30	36.22	26.16	26.32
Wt. of container (g.) = (w_3)	22.32	22.71	22.65	22.53	22.89
Mass of moisture (g.) (w_1-w_2) = x	6.40	6.60	6.74	1.04	0.98
Wt. of dry soil (g.) (w_2-w_3) = y	13.60	13.59	13.57	3.63	3.43
Moisture Content (%) = (100x/y)	47.06	48.57	49.67	28.65	28.57

Average Moisture Content (%)	---------	**28.61**
Moisture Content (%) at 25 blows from liquid limit curve	**48.43**	-----
Plasticity Index = Liquid Limit-Plastic Limit		**19.82**

Sub Base Material	**Sampled Station:-386+800**				
	Liquid Limit			**Plastic Limit**	
Container No.	Q7	Q8	Q3	B4	B5
No. of Blows	34	23	16		
Wt. of cont. + wet soil (g) = (w_1)	41.16	41.46	41.64	27.85	27.86
Wt. of cont. + dry soil (g.) = (w_2)	35.61	35.38	35.44	26.76	26.78
Wt. of container (g.) = (w_3)	22.47	22.43	22.49	22.62	22.66
Mass of moisture (g.) (w_1-w_2) = x	5.55	6.08	6.20	1.09	1.08
Wt. of dry soil (g.) (w_2-w_3) = y	13.14	12.95	12.95	4.14	4.12
Moisture Content (%) = (100x/y)	42.24	46.95	47.88	26.33	26.21
Average Moisture Content (%)	---------			**26.27**	
Moisture Content (%) at 25 blows from liquid limit curve	**45.69**			-----	
Plasticity Index = Liquid Limit-Plastic Limit				**19.42**	

Base coarse Material	**Sampled Station:-386+800**				
	Liquid Limit			**Plastic Limit**	
Container No.	E4	E3	E1	D4	D5
No. of Blows	32	23	16		
Wt. of cont. + wet soil (g) = (w_1)	40.36	40.46	40.65	27.83	27.90
Wt. of cont. + dry soil (g.) = (w_2)	36.61	36.63	36.71	27.00	27.12
Wt. of container (g.) = (w_3)	22.52	22.46	22.53	22.64	22.90
Mass of moisture (g.) (w_1-w_2) = x	3.75	3.83	3.94	0.83	0.78
Wt. of dry soil (g.) (w_2-w_3) = y	14.09	14.17	14.18	4.36	4.22
Moisture Content (%) = (100x/y)	26.61	27.03	27.79	19.04	18.48
Average Moisture Content (%)	---------			**18.76**	
Moisture Content (%) at 25 blows from liquid limit curve	**27.14**			-----	
Plasticity Index = Liquid Limit-Plastic Limit				**8.38**	



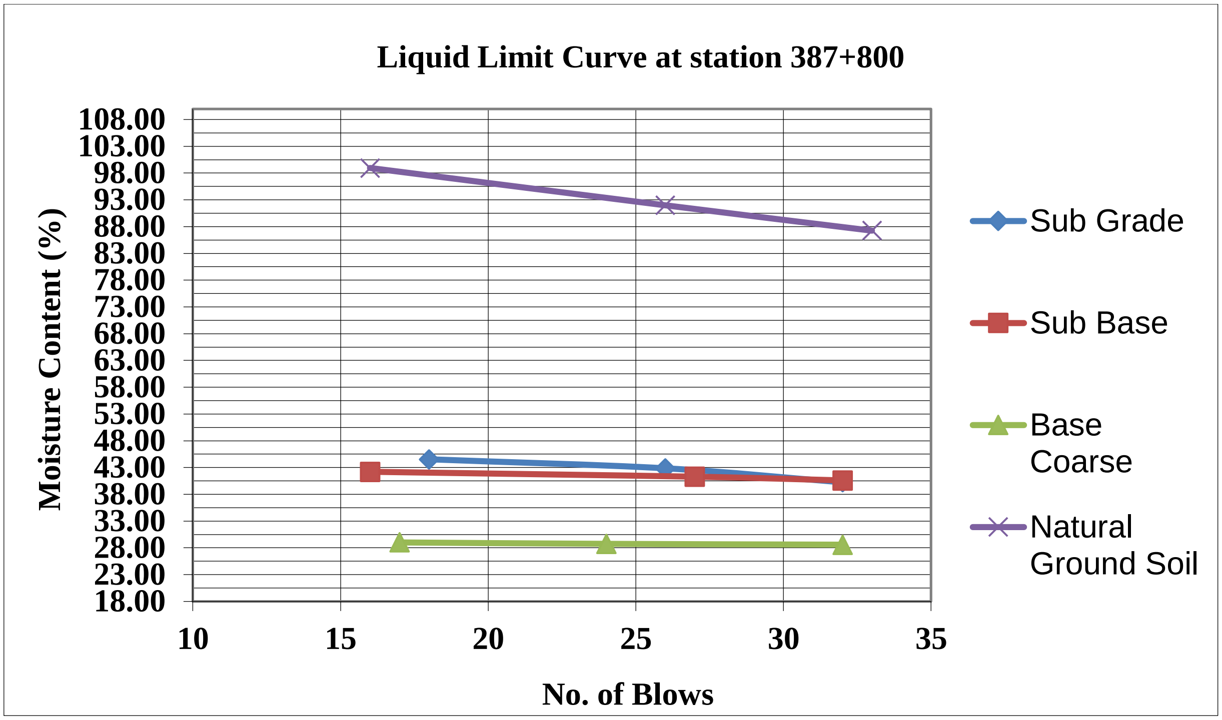
Natural Ground Soil	Sampled Station:-387+800				
	Liquid Limit			Plastic Limit	
Container No.	A1	A2	A3		
No. of Blows	33	26	16		
Wt. of cont. + wet soil (g) = (w_1)	42.90	43.40	43.55	27.40	27.58
Wt. of cont. + dry soil (g.) = (w_2)	33.24	33.30	33.15	25.64	25.82
Wt. of container (g.) = (w_3)	22.17	22.32	22.64	22.01	21.94
Mass of moisture (g.) (w_1-w_2) = x	9.66	10.10	10.40	1.76	1.76
Wt. of dry soil (g.) (w_2-w_3) = y	11.07	10.98	10.51	3.63	3.88
Moisture Content (%) = (100x/y)	87.26	91.99	98.95	48.48	45.36
Average Moisture Content (%)	-----			46.92	
Moisture Content (%) at 25 blows from liquid limit curve	92.73			-----	
Plasticity Index = Liquid Limit-Plastic Limit				45.81	

Sub Grade Material	Sampled Station:-387+800				
	Liquid Limit			Plastic Limit	
Container No.	A2	A1	A7	D4	D5
No. of Blows	32	26	18		
Wt. of cont. + wet soil (g) = (w_1)	42.08	42.18	42.29	27.43	27.84
Wt. of cont. + dry soil (g.) = (w_2)	36.41	36.34	36.24	26.64	26.66
Wt. of container (g.) = (w_3)	22.32	22.71	22.65	22.53	22.89
Mass of moisture (g.) (w_1-w_2) = x	5.67	5.84	6.05	0.79	1.18
Wt. of dry soil (g.) (w_2-w_3) = y	14.09	13.63	13.59	4.11	3.77
Moisture Content (%) = (100x/y)	40.24	42.85	44.52	19.22	31.30

Average Moisture Content (%)	-----	**25.26**
Moisture Content (%) at 25 blows from liquid limit curve	**42.54**	-----
Plasticity Index = Liquid Limit - Plastic Limit		**17.27**

Sub Base Material	**Sampled Station:-387+800**				
	Liquid Limit			**Plastic Limit**	
Container No.	45	24	B3	k-4	G-5
No. of Blows	32	27	16		
Wt. of cont. + wet soil (g) = (w_1)	42.08	42.42	42.64	27.63	27.65
Wt. of cont. + dry soil (g.) = (w_2)	36.42	36.58	36.66	26.62	26.64
Wt. of container (g.) = (w_3)	22.47	22.43	22.49	22.62	22.88
Mass of moisture (g.) (w_1-w_2) = x	5.66	5.84	5.98	1.01	1.01
Wt. of dry soil (g.) (w_2-w_3) = y	13.95	14.15	14.17	4.00	3.76
Moisture Content (%) = (100x/y)	40.57	41.27	42.20	25.25	26.86
Average Moisture Content (%)	-----			**26.06**	
Moisture Content (%) at 25 blows from liquid limit curve	**41.35**			-----	
Plasticity Index = Liquid Limit-Plastic Limit				**15.29**	

Base Coarse Material	**Sampled Station:-387+800**				
	Liquid Limit			**Plastic Limit**	
Container No.	JG1	JH2	JH3	JH4	JH5
No. of Blows	32	24	17		
Wt. of cont. + wet soil (g) = (w_1)	41.21	41.36	41.58	28.10	28.39
Wt. of cont. + dry soil (g.) = (w_2)	37.02	37.14	37.29	27.24	27.31
Wt. of container (g.) = (w_3)	22.36	22.46	22.51	22.69	22.54
Mass of moisture (g.) (w_1-w_2) = x	4.19	4.22	4.29	0.86	1.08
Wt. of dry soil (g.) (w_2-w_3) = y	14.66	14.68	14.78	4.55	4.77
Moisture Content (%) = (100x/y)	28.58	28.75	29.03	18.90	22.64
Average Moisture Content (%)	-----			**20.77**	
Moisture Content (%) at 25 blows from liquid limit curve	**28.78**			-----	
Plasticity Index = Liquid Limit - Plastic Limit				**8.01**	



Natural Ground Soil	Sampled Station:-388+300				
	Liquid Limit			Plastic Limit	
Container No.	m1	m2	C3	m3	C5
No. of Blows	32	26	16		
Wt. of cont. + wet soil (g) = (w_1)	44.94	39.60	41.25	25.37	26.03
Wt. of cont. + dry soil (g.) = (w_2)	33.84	31.06	31.90	24.31	24.75
Wt. of container (g.) = (w_3)	22.17	22.32	22.64	22.01	21.94
Mass of moisture (g.) (w_1-w_2) = x	11.10	8.54	9.35	1.06	1.28
Wt. of dry soil (g.) (w_2-w_3) = y	11.67	8.74	9.26	2.30	2.81
Moisture Content (%) = (100x/y)	95.12	97.71	100.97	46.09	45.55
Average Moisture Content (%)	----			**45.82**	
Moisture Content (%) at 25 blows from liquid limit curve	**97.93**			---	
Plasticity Index = Liquid Limit-Plastic Limit				**52.11**	

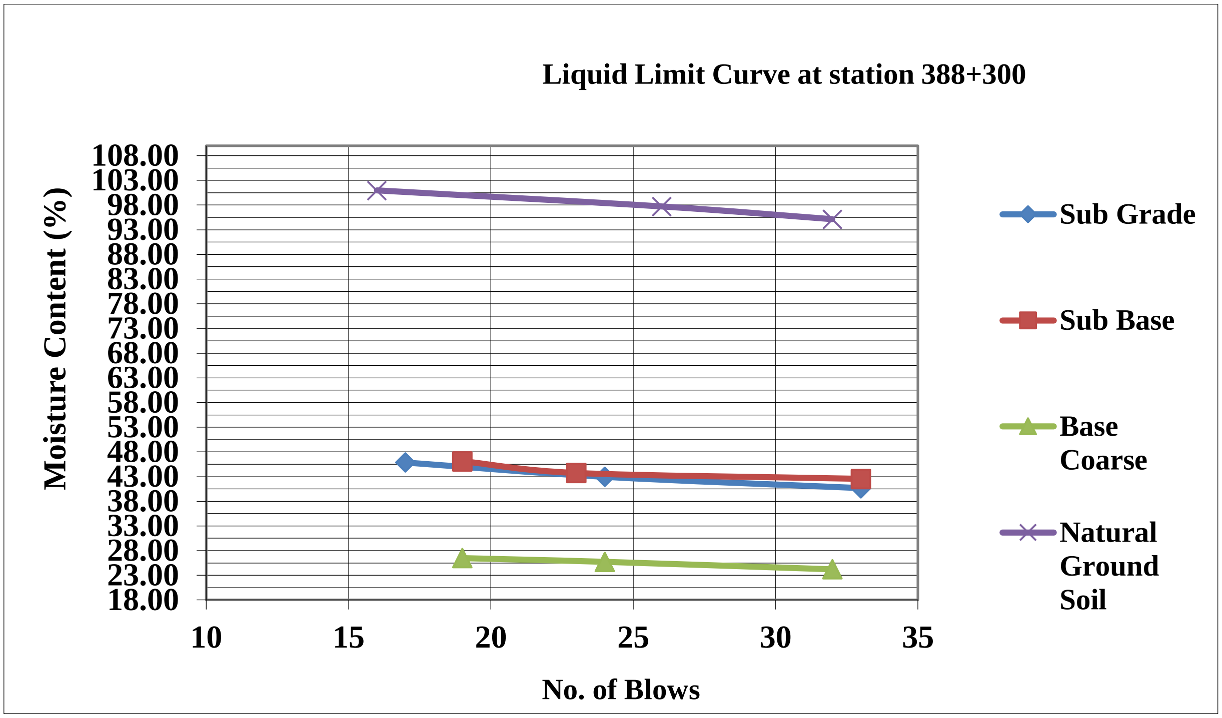
Sub Grade Material	Sampled Station:-388+300				
	Liquid Limit			Plastic Limit	
Container No.	X11	X12	X13	X14	X15
No. of Blows	33	24	17		
Wt. of cont. + wet soil (g) = (w_1)	42.64	42.62	42.74	26.66	26.68
Wt. of cont. + dry soil (g.) = (w_2)	36.88	36.62	36.42	25.89	25.92
Wt. of container (g.) = (w_3)	22.71	22.65	22.64	22.67	22.76

Mass of moisture (g.) $(w_1-w_2) = x$	5.76	6.00	6.32	0.77	0.76
Wt. of dry soil (g.) $(w_2-w_3) = y$	14.17	13.97	13.78	3.22	3.16
Moisture Content (%) = (100x/y)	40.65	42.95	45.86	23.91	24.05
Average Moisture Content (%)	-----			**23.98**	
Moisture Content (%) at 25 blows from liquid limit curve	**43.15**			-----	
Plasticity Index = Liquid Limit-Plastic Limit				**19.17**	

Sub Base Material	**Sampled Station:-388+300**				
	Liquid Limit			**Plastic Limit**	
Container No.	B1	B2	B3	B4	B5
No. of Blows	33	23	19		
Wt. of cont. + wet soil (g) = (w_1)	40.20	40.84	41.44	26.95	27.04
Wt. of cont. + dry soil (g.) = (w_2)	34.91	35.24	35.46	26.04	26.06
Wt. of container (g.) = (w_3)	22.47	22.43	22.49	22.62	22.88
Mass of moisture (g.) $(w_1-w_2) = x$	5.29	5.60	5.98	0.91	0.98
Wt. of dry soil (g.) $(w_2-w_3) = y$	12.44	12.81	12.97	3.42	3.18
Moisture Content (%) = (100x/y)	42.52	43.72	46.11	26.61	30.82
Average Moisture Content (%)	-----			**28.71**	
Moisture Content (%) at 25 blows from liquid limit curve	**44.12**			-----	
Plasticity Index = Liquid Limit - Plastic Limit				**15.40**	

Base Coarse Material	**Sampled Station:-388+300**				
	Liquid Limit			**Plastic Limit**	
Container No.	k4	k2	k3	F4	F5
No. of Blows	32	24	19		
Wt. of cont. + wet soil (g) = (w_1)	43.38	43.49	43.52	27.53	27.77
Wt. of cont. + dry soil (g.) = (w_2)	39.31	39.18	39.12	26.78	27.02
Wt. of container (g.) = (w_3)	22.52	22.44	22.51	22.63	22.89
Mass of moisture (g.) $(w_1-w_2) = x$	4.07	4.31	4.40	0.75	0.75
Wt. of dry soil (g.) $(w_2-w_3) = y$	16.79	16.74	16.61	4.15	4.13
Moisture Content (%) = (100x/y)	24.24	25.75	26.49	18.07	18.16
Average Moisture Content (%)	-----			**18.12**	
Moisture Content (%) at 25 blows from liquid limit curve	**25.49**			-----	

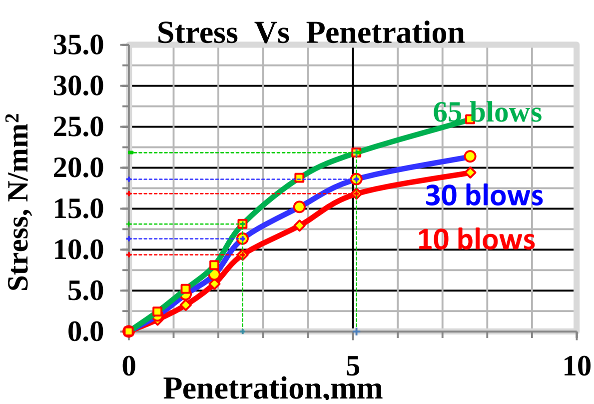
Plasticity Index = Liquid Limit-Plastic Limit	**7.38**

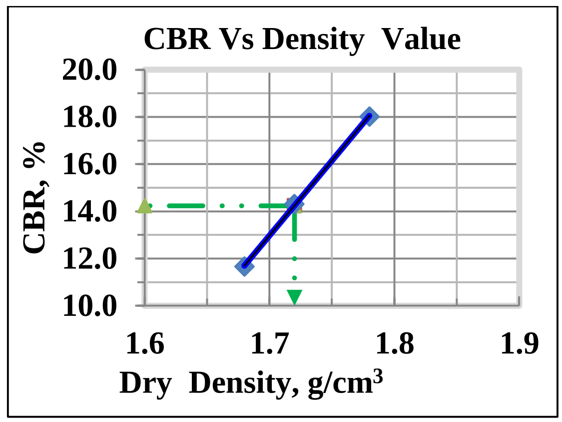


Appendix D: California Bearing Ratio Test Results

Sub Grade Material					Sampled Station:-377+600	
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	1.38	2.33	10.32	11.66	11.66	1.68
30	1.80	2.86	13.50	14.31	14.31	1.72
65	2.12	3.60	15.88	18.02	18.02	1.78

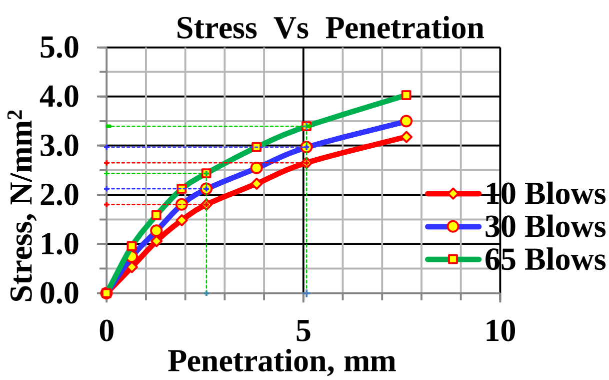
MDD, g/cm^3	**1.81**	**Target density, g/cm^3**	**1.72**
Density required, %	**95**	**CBR, %**	**14.24**

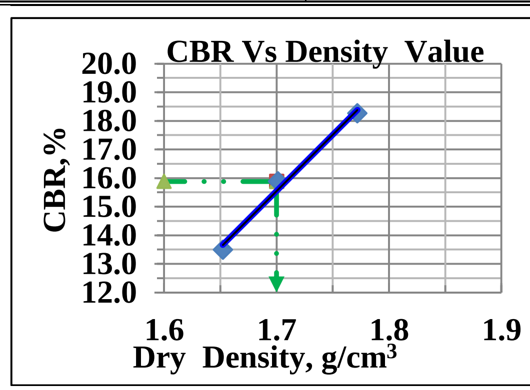




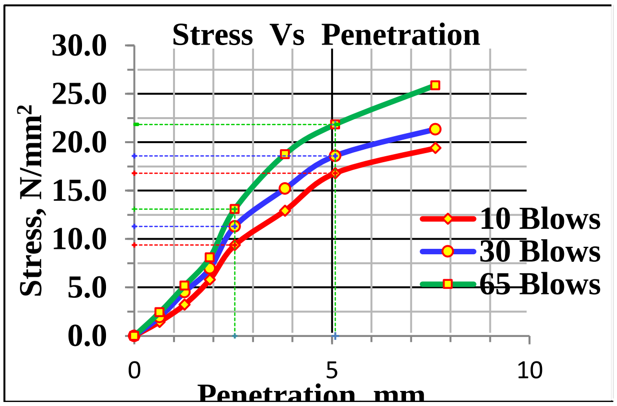
Sub Base Material					Sampled Station:-377+600	
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	1.80	2.65	13.50	13.25	13.50	1.65
30	2.12	2.97	15.88	14.84	15.88	1.70
65	2.44	3.39	18.26	16.96	18.26	1.77

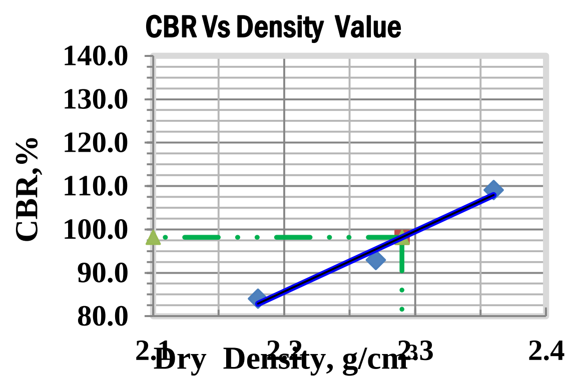
MDD, g/cm^3	**1.78**	**Target density, g/cm^3**	**1.70**
Density required, %	**95**	**CBR, %**	**15.88**



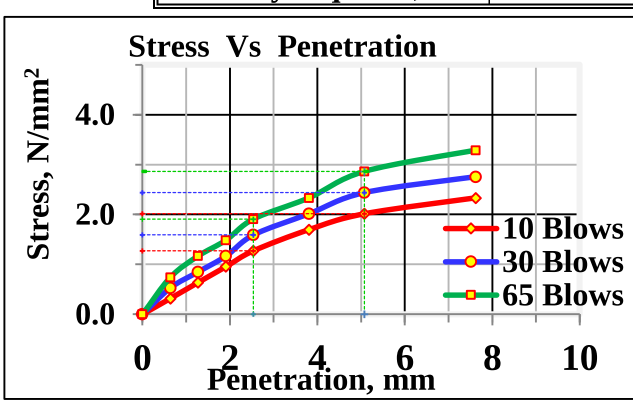


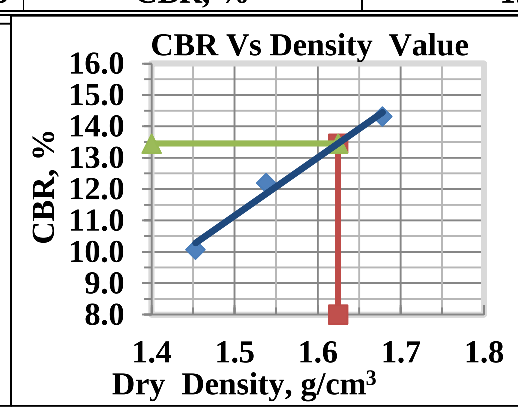
Base Coarse Material						Sampled Station:-377+600
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	9.38	16.82	70.25	84.08	84.08	2.18
30	11.32	18.60	84.79	92.98	92.98	2.27
65	13.10	21.83	98.11	109.15	109.15	2.36
	MDD, g/cm³		**2.34**	**Target density ,g/cm³**		**2.29**
	Density required, %		**98**	**CBR, %**		**98.19**



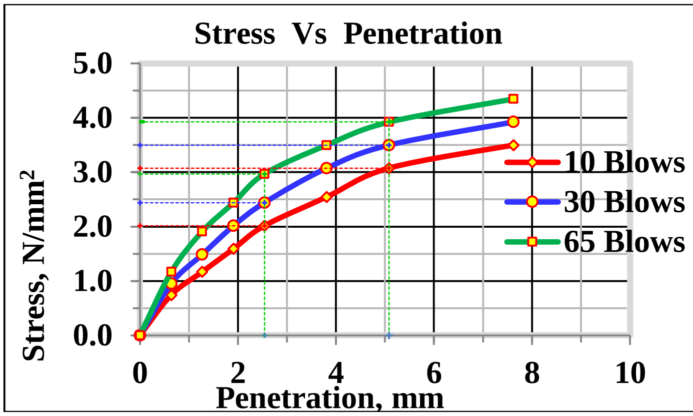


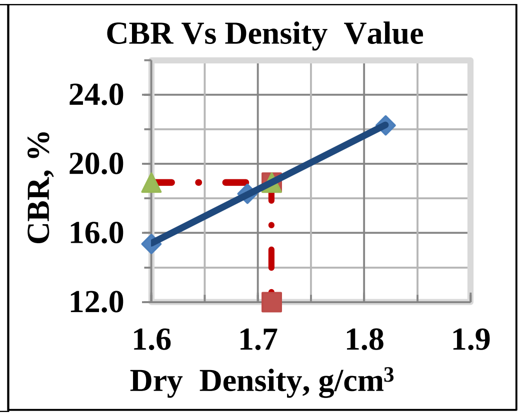
Sub Grade Material						Sampled Station:-376+500
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	1.27	2.01	9.53	10.07	10.07	1.45
30	1.59	2.44	11.91	12.19	12.19	1.54
65	1.91	2.86	14.29	14.31	14.31	1.68
	MDD, g/cm³		**1.71**	**Target density, g/cm³**		**1.62**
	Density required, %		**95**	**CBR, %**		**13.45**



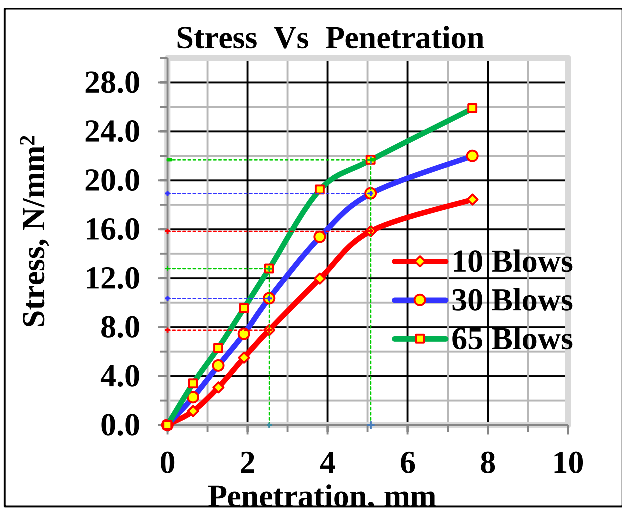


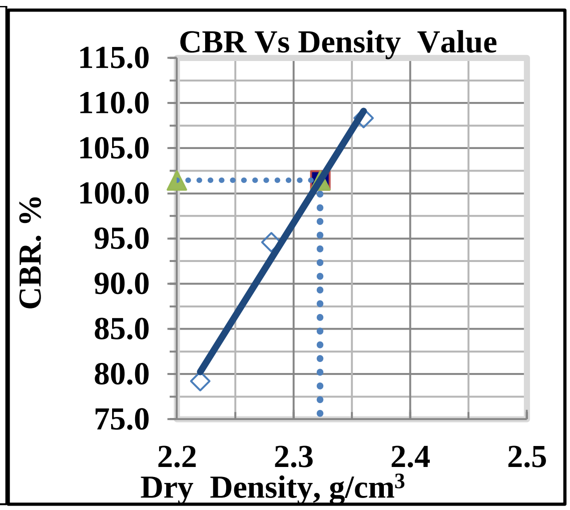
Sub Base Material					Sampled Station:-376+500	
	Load (KN)		CBR (%)		Max	Dry Density,
Blows	2.54mm	5.08mm	2.54mm	5.08mm	CBR(%)	g/cc
10	2.01	3.07	15.09	15.37	15.37	1.60
30	2.44	3.50	18.26	17.49	18.26	1.69
65	2.97	3.92	22.23	19.61	22.23	1.82
	MDD, g/cm³		**1.80**	**Target density ,g/cm³**		**1.71**
	Density required, %		**95**	**CBR, %**		**18.92**



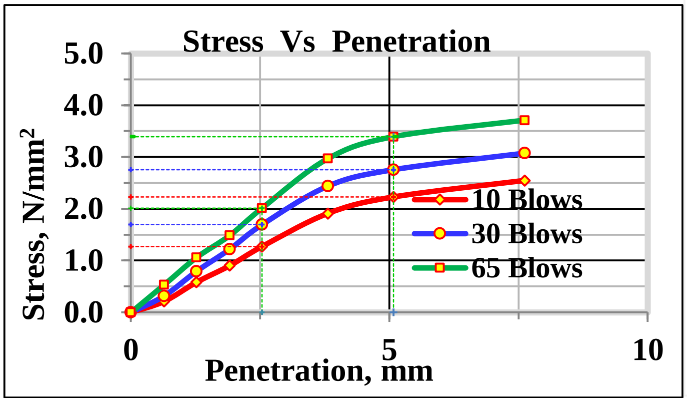


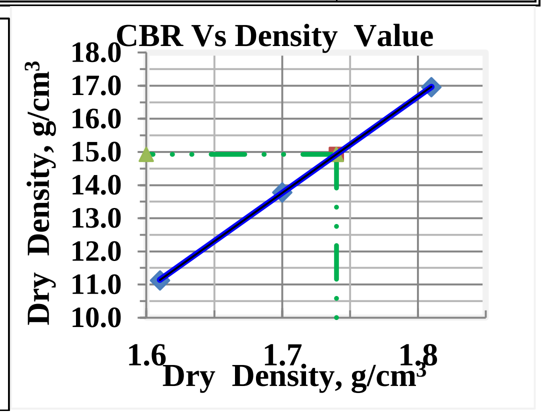
Base Coarse Material					Sampled Station:-376+500	
	Load (KN)		CBR (%)		Max	Dry Density,
Blows	2.54mm	5.08mm	2.54mm	5.08mm	CBR(%)	g/cc
10	7.76	15.85	58.14	79.23	79.23	2.22
30	10.35	18.92	77.52	94.59	94.59	2.28
65	12.77	21.67	95.69	108.34	108.34	2.36
	MDD, g/cm³		**2.37**	**Target density, g/cm³**		**2.32**
	Density required, %		**98**	**CBR, %**		**101.44**



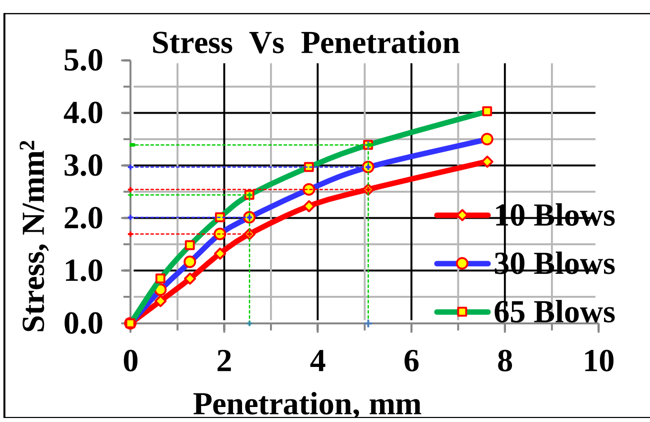


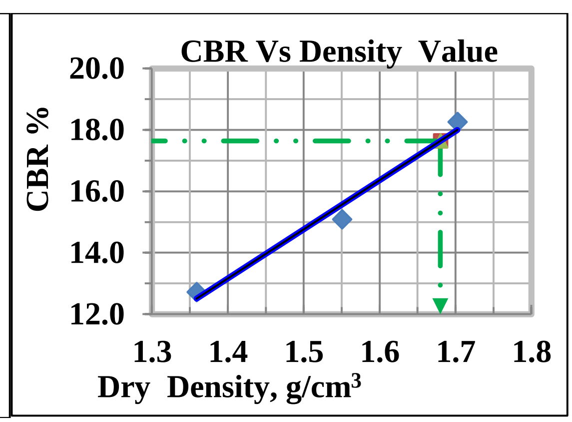
Sub Grade Material					Sampled Station:-375+500	
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density ,g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	1.27	2.23	9.53	11.13	11.13	1.61
30	1.70	2.76	12.70	13.78	13.78	1.70
65	2.01	3.39	15.09	16.96	16.96	1.81
	MDD, g/cm^3		**1.83**	**Target density ,g/cm^3**		**1.74**
	Density required, %		**95**	**CBR, %**		**14.93**



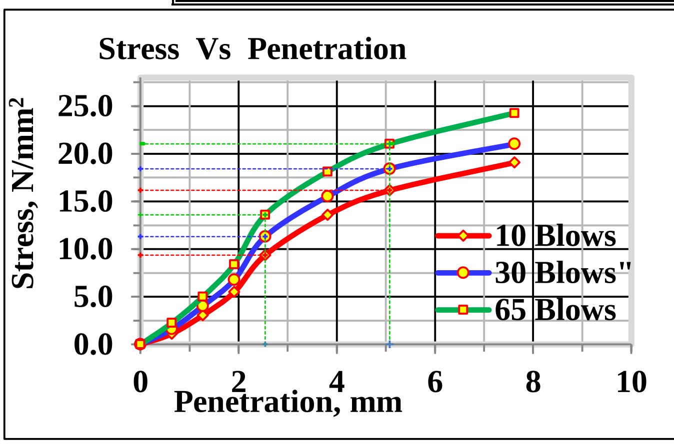


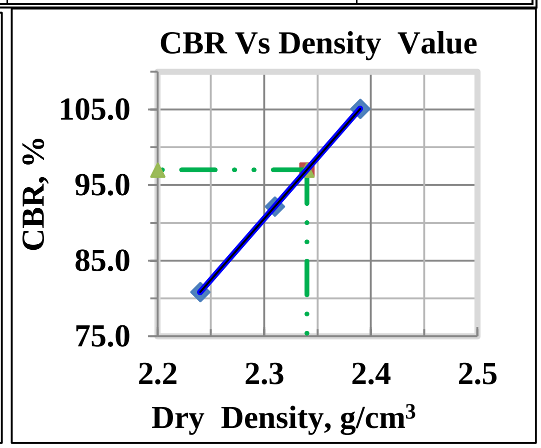
Sub Base Material					Sampled Station:-375+500	
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	1.70	2.54	12.70	12.72	12.72	1.36
30	2.01	2.97	15.09	14.84	15.09	1.55
65	2.44	3.39	18.26	16.96	18.26	1.70
	MDD, g/cm^3		**1.77**	**Target density, g/cm^3**		**1.68**
	Density required, %		**95**	**CBR, %**		**17.64**



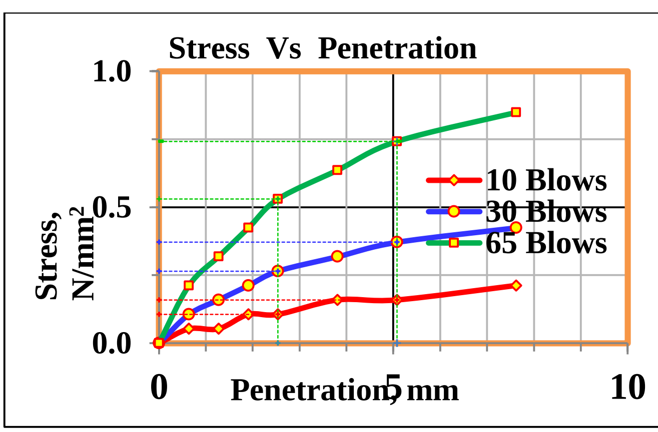


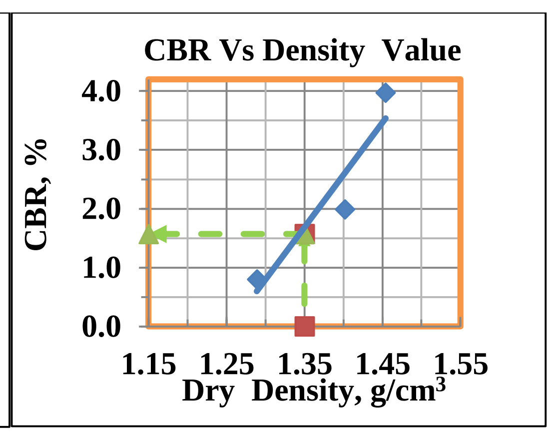
Base Coarse Material					Sampled Station:- 375+500	
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	9.38	16.17	70.25	80.85	80.85	2.24
30	11.32	18.43	84.79	92.17	92.17	2.31
65	13.58	21.02	101.74	105.11	105.11	2.39
	MDD, g/cm³		**2.39**	**Target density, g/cm³**		**2.34**
	Density required, %		**98**	**CBR, %**		**97.02**



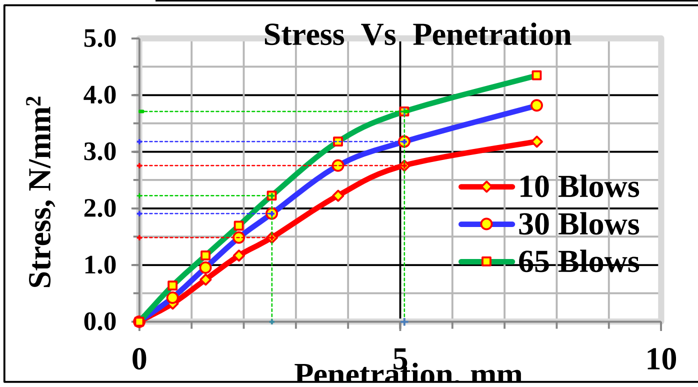


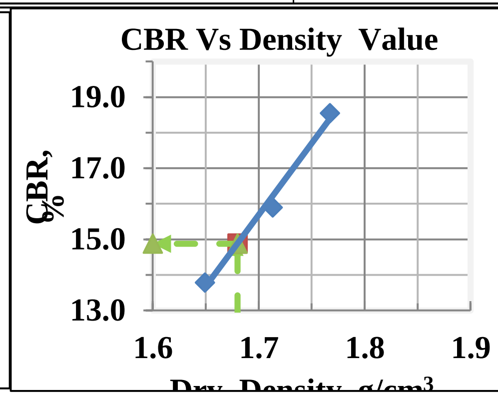
Natural Ground Soil					Sampled Station:-386+800	
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	0.11	0.16	0.79	0.80	0.80	1.29
30	0.27	0.37	1.99	1.86	1.99	1.40
65	0.53	0.74	3.97	3.71	3.97	1.45
	MDD, g/cm³		**1.42**	**Target density, g/cm³**		**1.35**
	Density required, %		**95**	**CBR, %**		**1.57**



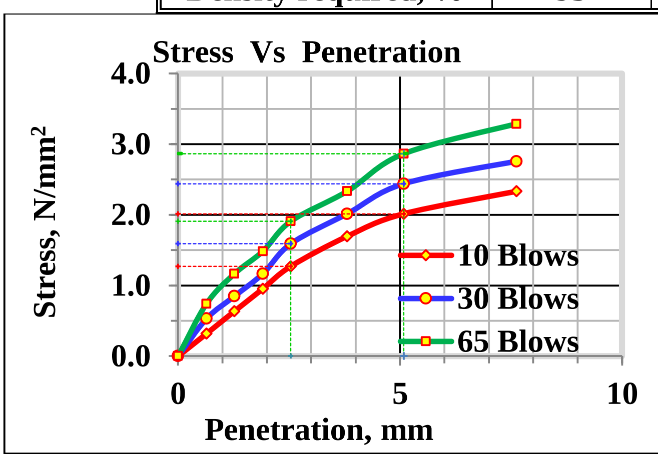


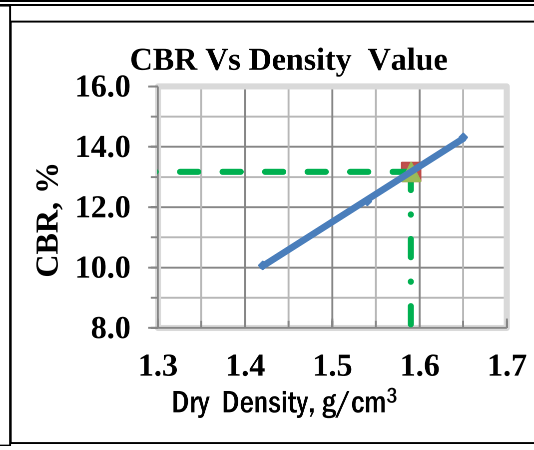
Sub Grade Material				Sampled Station:-386+800		
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	1.48	2.76	11.12	13.78	13.78	1.65
30	1.91	3.18	14.29	15.90	15.90	1.71
65	2.23	3.71	16.67	18.55	18.55	1.77
	MDD, g/cm³		**1.81**	**Target density, g/cm³**		**1.68**
	Density required, %		**93**	**CBR, %**		**14.88**



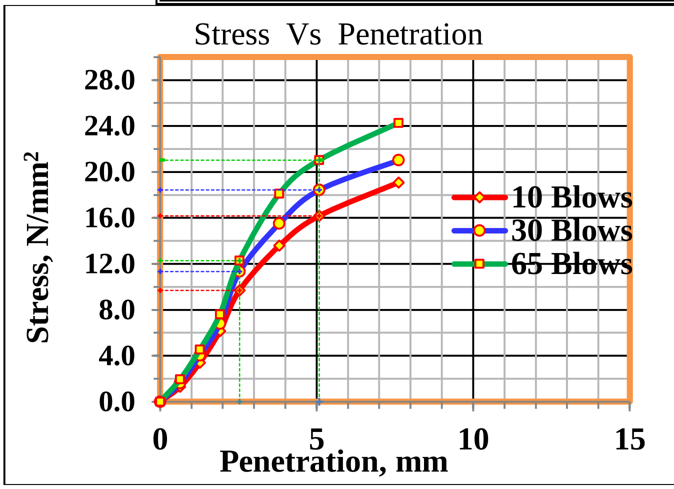


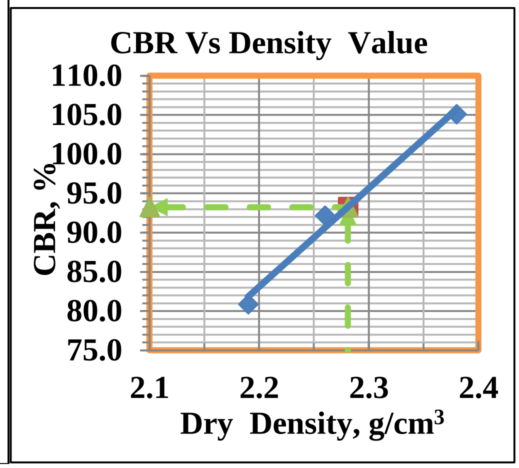
Sub Base Material				Sampled Station:-386+800		
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	1.27	2.01	9.53	10.07	10.07	1.42
30	1.59	2.44	11.91	12.19	12.19	1.54
65	1.91	2.86	14.29	14.31	14.31	1.65
	MDD, g/cm³		**1.67**	**Target density, g/cm³**		**1.59**
	Density required, %		**95**	**CBR, %**		**13.17**



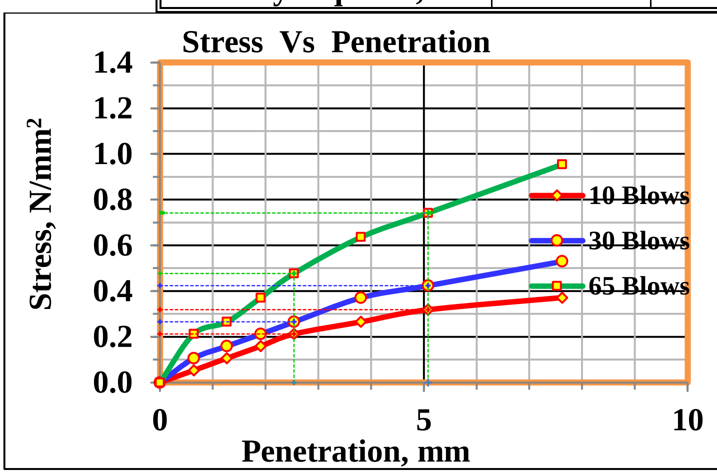


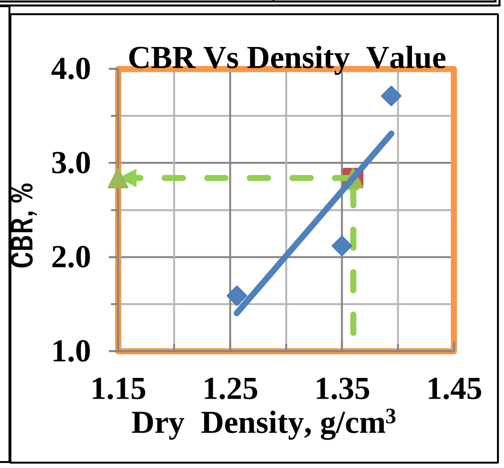
Base Coarse Material					Sampled Station:-386+800	
Blows	Load (KN) 2.54mm	Load (KN) 5.08mm	CBR (%) 2.54mm	CBR (%) 5.08mm	Max CBR(%)	Dry Density, g/cc
10	9.70	16.17	72.67	80.85	80.85	2.19
30	11.32	18.43	84.79	92.17	92.17	2.26
65	12.29	21.02	92.05	105.11	105.11	2.38
	MDD, g/cm³		**2.33**	**Target density, g/cm³**		**2.28**
	Density required, %		**98**	**CBR, %**		**93.25**



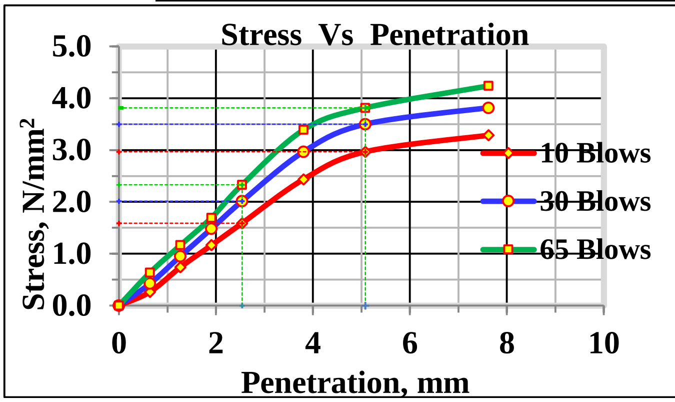


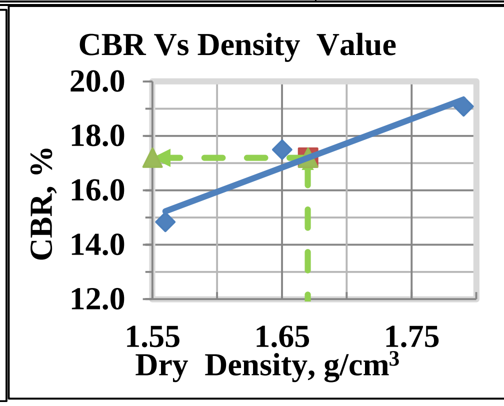
Natural Ground Material					Sampled Station:-3867+800	
Blows	Load (KN) 2.54mm	Load (KN) 5.08mm	CBR (%) 2.54mm	CBR (%) 5.08mm	Max CBR(%)	Dry Density, g/cc
10	0.21	0.32	1.59	1.59	1.59	1.26
30	0.27	0.42	1.99	2.12	2.12	1.35
65	0.48	0.74	3.57	3.71	3.71	1.39
	MDD, g/cm³		**1.43**	**Target density, g/cm³**		**1.36**
	Density required, %		**95**	**CBR, %**		**2.84**



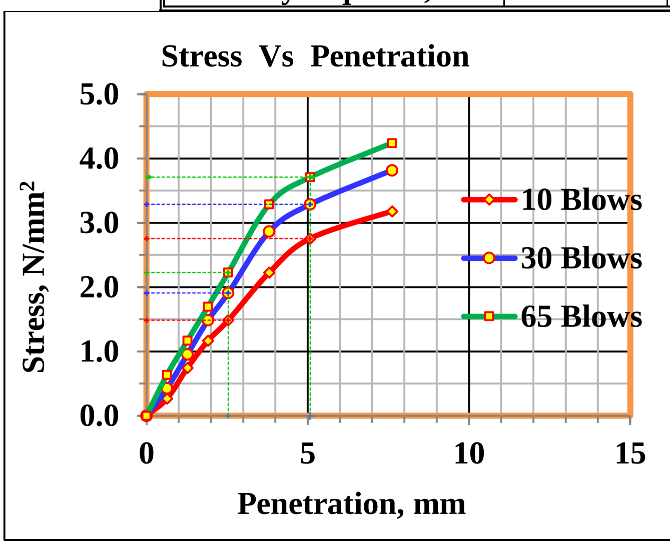


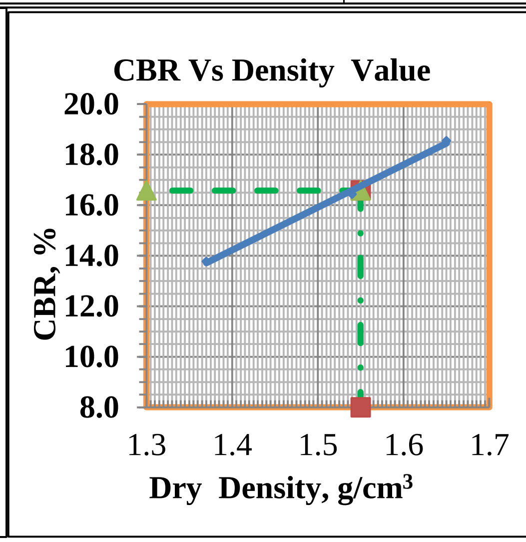
Sub Grade Material				Sampled Station:-387+800		
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	1.59	2.97	11.91	14.84	14.84	1.56
30	2.01	3.50	15.09	17.49	17.49	1.65
65	2.33	3.82	17.47	19.08	19.08	1.79
	MDD, g/cm³		**1.76**	**Target density, g/cm³**		**1.67**
	Density required, %		**95**	**CBR, %**		**17.20**



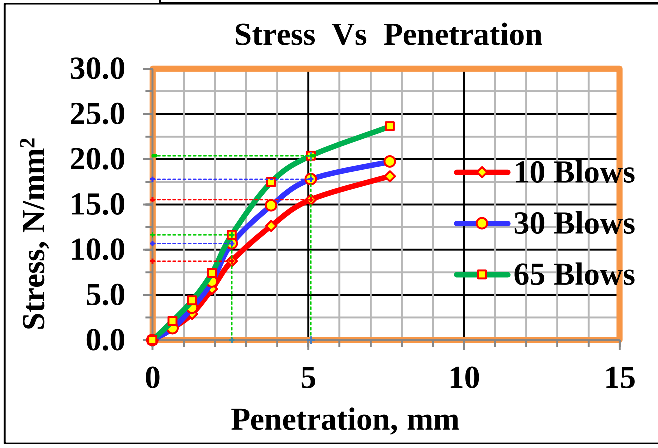


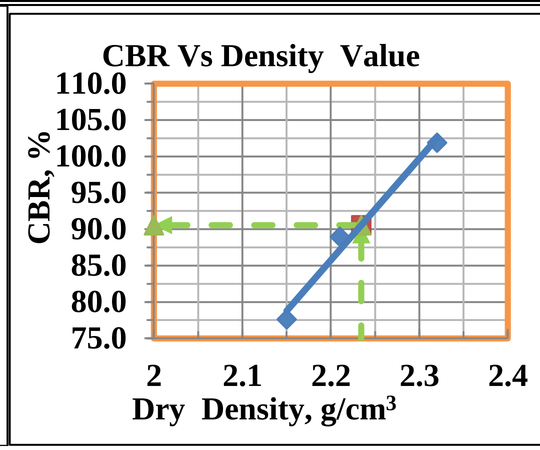
Sub Base Material				Sampled Station:-387+800		
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	1.48	2.76	11.12	13.78	13.78	1.37
30	1.91	3.29	14.29	16.43	16.43	1.54
65	2.23	3.71	16.67	18.55	18.55	1.65
	MDD, g/cm³		**1.63**	**Target density, g/cm³**		**1.55**
	Density required, %		**95**	**CBR, %**		**16.58**



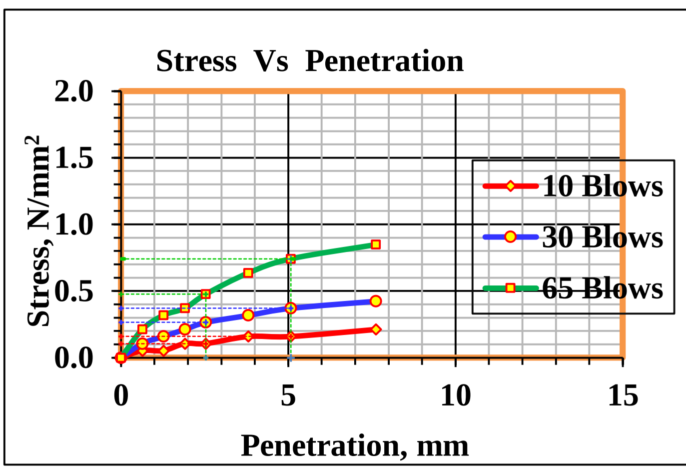


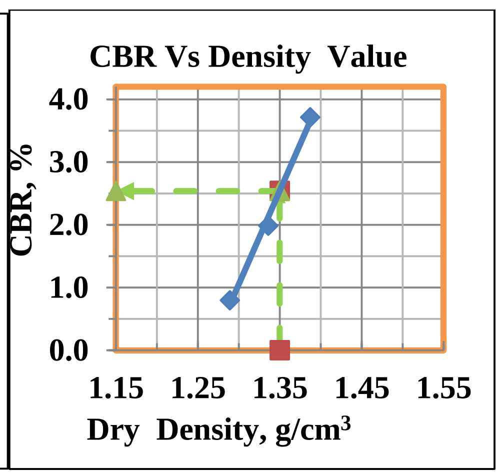
Base Coarse Material						
Sampled Station:-387+800						
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	1.59	2.97	11.91	14.84	14.84	1.56
30	2.01	3.50	15.09	17.49	17.49	1.65
65	2.33	3.82	17.47	19.08	19.08	1.79
	MDD, g/cm³		**1.76**	**Target density, g/cm³**		**1.67**
	Density required, %		**95**	**CBR, %**		**17.20**



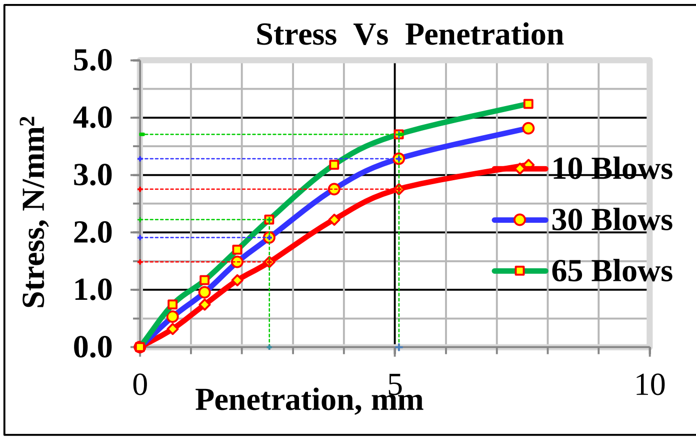


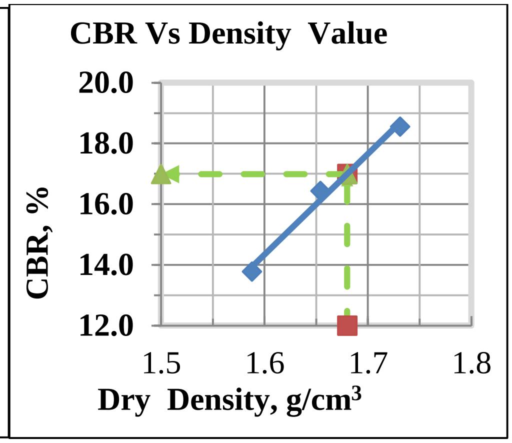
Natural Ground Soil Material						
Sampled Station:-388+300						
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	0.11	0.16	0.79	0.80	0.80	1.29
30	0.27	0.37	1.99	1.86	1.99	1.34
65	0.48	0.74	3.57	3.71	3.71	1.39
	MDD, g/cm³		**1.42**	**Target density, g/cm³**		**1.35**
	Density required, %		**95**	**CBR, %**		**2.54**



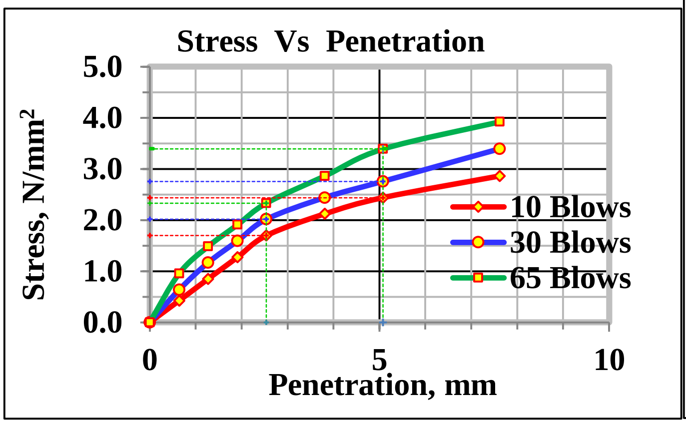


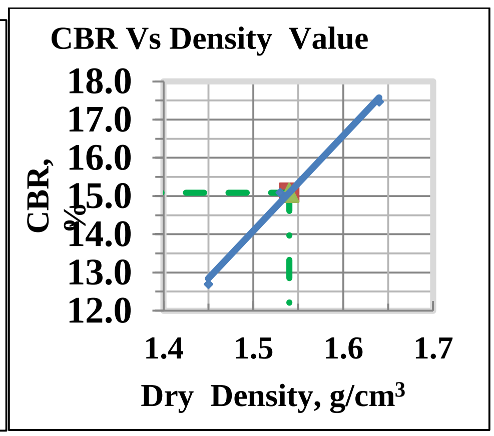
Sub Grade Material					Sampled Station:-388+300	
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	1.48	2.76	11.12	13.78	13.78	1.59
30	1.91	3.29	14.29	16.43	16.43	1.65
65	2.23	3.71	16.67	18.55	18.55	1.73
	MDD, g/cm³		**1.77**	**Target density, g/cm³**		**1.68**
	Density required, %		**95**	**CBR, %**		**16.99**



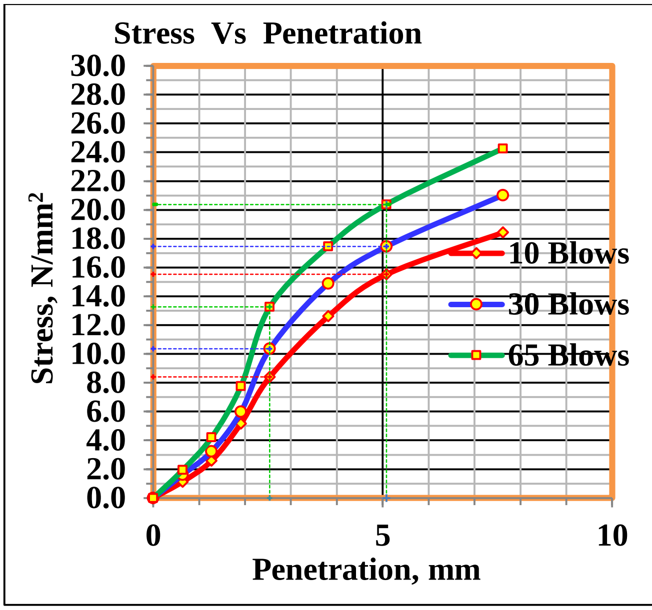


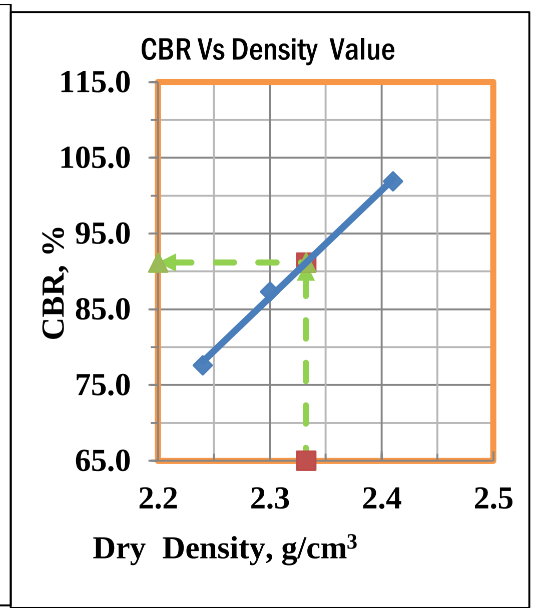
Sub Base Material					Sampled Station:-388+300	
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density, g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	1.70	2.44	12.70	12.19	12.70	1.45
30	2.01	2.76	15.09	13.78	15.09	1.53
65	2.33	3.39	17.47	16.96	17.47	1.64
	MDD, g/cm³		**1.62**	**Target density, g/cm³**		**1.54**
	Density required, %		**95**	**CBR, %**		**15.09**





Base Course Material					Sampled Station:-388+300	
Blows	Load (KN)		CBR (%)		Max CBR(%)	Dry Density ,g/cc
	2.54mm	5.08mm	2.54mm	5.08mm		
10	8.41	15.52	62.98	77.62	77.62	2.24
30	10.35	17.46	77.52	87.32	87.32	2.30
65	13.26	20.37	99.32	101.87	101.87	2.41
	MDD, g/cm^3		**2.38**	**Target density, g/cm^3**		**2.33**
	Density required, %		**98**	**CBR, %**		**91.17**





Appendix E: Particle Size Distribution Result

Sieve analyses results of base course material

Sieve Size (mm)	Percent of Passing (%) on Tasted Sampled Station						Min. Requir.	Max. Requi..
	377+600	376+500	377+500	386+800	387+800	388+300		
50	100.00	100.00	100	100.00	100	100.00	100	100
37.5	99.20	99.50	99.26	99.80	99.40	99.60	95	100
20	80.30	79.00	97.06	82.20	85.20	89.10	60	80
10	65.50	67.76	77.60	71.80	63.80	77.10	40	60
4.75	51.20	47.36	65.90	62.00	50.70	59.10	25	40
2	37.80	33.96	45.78	52.60	35.22	48.90	15	30
0.425	16.70	14.31	14.98	23.20	22.20	18.90	7	19
0.075	8.50	6.71	8.90	15.40	14.80	9.90	5	12

Sieve analyses results of sub base course material

Sieve Size (mm)	Percent of Passing (%) on Tasted Sampled Station						Min. Requir.	Max. Requir.
	377+600	376+500	375+500	386+800	387+800	388+300		
63	100.00	100.00	100.00	100.00	100.00	100.00	100	100
37.5	98.60	99.10	98.60	100.00	98.70	99.20	70	100
20	94.30	95.80	90.10	99.82	95.30	97.40	50	100
4.75	85.90	88.40	77.86	98.36	80.83	88.60	30	100
2	72.50	74.00	65.78	81.16	70.68	75.30	17	75
0.425	55.50	56.00	50.50	63.50	47.98	56.20	11	56
0.075	29.00	26.50	28.40	33.90	27.88	31.80	5	25

Sieve analyses results of sub grade course material

Sieve Size (mm)	Percent of Passing (%) on Tasted Sampled Station					
	377+600	**376+500**	**375+500**	**386+800**	**387+800**	**388+300**
63	100.00	100.00	100.00	100.00	100.00	100.00
37.5	98.70	99.70	98.70	98.70	99.10	99.00
20	96.70	98.50	95.80	96.30	97.10	96.60
4.75	88.60	90.10	88.72	82.70	83.10	79.20
2	75.50	74.70	76.52	72.60	72.90	66.00
0.425	53.60	55.40	53.10	50.20	53.50	50.98
0.075	32.80	34.00	33.00	30.10	26.40	29.80