STABILIZATION OF NATURAL SAND WITH CEMENT, AND BITUMEN AND SULFUR FOR BASE COURSE

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By

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SCHOOL OF GRADUATE STUDIES

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Approved by Board of Examiners

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DECLARATION

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor Eng. Efrem Gebre-egziabher and has not been presented as a thesis for a degree in any other university. All sources of materials used for this thesis have also been duly acknowledged.

Eskedil Abebaw	____________	July, 2014
Name	Signature	Date

Dedicated to

All those who had contributed and are striving

for the good of human beings

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Acronyms/Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
AC	Asphalt Concrete
CBR	California Bearing Ratio
DBM	Dense Bitumen Macadam
Dia.	Diameter
ERA	Ethiopian Roads Authority
FM	Fineness Modulus
Ht.	Height
HMA	Hot Mix Asphalt
LL	Liquid Limit
Max.	Maximum
MDD	Maximum Dry Density
Min.	Minimum
MS-2	Asphalt Institute's Manual Series – 2
OBC	Optimum Binder Content
OMC	Optimum Moisture Content
PCA	Portland Cement Association
PDM	Pavement Design Manual
PI	Plasticity Index
PMDM	Pavement and Materials Design Manual
PP	Plasticity Product
SATCC	South African Transport and Communications Commission
SB	Sand-Bitumen
SCB	Sand-Cement-Bitumen
SSB	Sand-Sulfur-Bitumen
SLB	Sand-Limestone-Bitumen
SSD	Saturated Surface Dry
STS	Standard Technical Specification
TMSG	Theoretical Maximum Specific Gravity
TRL	Transport Research Laboratory
UCS	Unconfined Compressive Strength
USCS	Unified Soil Classification System
VFB	Voids Filled with Bitumen
VMA	Voids in the Mineral Aggregate

List of Symbols

D_{10}	Sieve size through which 10% by weight of the material passes
D_{30}	Sieve size through which 30% by weight of the material passes
D_{60}	Sieve size through which 60% by weight of the material passes
C_c	Coefficient of Curvature, $D_{30}^2/D_{60}*D_{10}$
C_u	Coefficient of Uniformity, D_{60}/D_{10}

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ABSTRACT

Absence of sound natural road construction materials, within the vicinity of the project compound, makes road construction in that area very expensive. The best alternative in those areas, where there is scarcity of suitable construction materials, is upgrading the locally available materials so that they can be used for the proposed construction. Soil stabilization by adding additives is one of the oldest methods of upgrading substandard materials.

In the south western part of Ethiopia, in Gambella region, rock and natural gravel, which are the necessary ingredients for construction of pavement structural components, are very scarce. The vast flat land of the region is covered by black clay soil which is underlain by sand deposits within 0.5m – 3m depth.

The focus of this study is to stabilize the natural sand, found underneath the black clay soil, with bitumen and cement so that it can be used for road base construction. Additives like, sulfur and cement were added to the sand-bitumen mixture to increase the Marshall stability of the mix. The sand-bitumen mixtures were designed according to Marshall method of mix design and the respective properties were assessed based on the Marshall mix design criteria. The sand-cement mixtures were, on the other hand, designed according to the Joint US Army and Air Force methods and the mixture quality was evaluated based on the specifications, developed by the same agencies.

It was found out that the natural sand underneath the black clay soil can be used for roadbase construction by stabilizing it either with bitumen or ordinary portland cement. In bitumen stabilization 15-20 percent sulfur has to be added so that the mix gain sufficient stability. The sand stabilized with 10% ordinary portland cement fulfilled all of the requirements set for base course materials.

Economically, cement treated natural sand is the most feasible alternative as compared to the crushed stone base course and bituminous (sand-sulfur-bitumen) base course.

Key words: sand-bitumen, sand-sulfur-bitumen, stabilization, sand-bitumen roadbase, cement stabilization, cement treated sand.

1 INTRODUCTION

1.1 Background

Availability of natural construction materials within reasonable hauling distance is one of the major factors that have a direct impact on the investment cost of road projects. In areas where natural construction materials are readily available, roads can be constructed on sound economic basis. However in some regions, natural construction materials are either not available or do not fulfill the quality requirements of road building materials. This is the case in western part of Ethiopia, in Gambella region.

Gambella is located in south western part of Ethiopia and has a total area of 25,802 square kilometer. The regional landscape is characterized as flat or very gently undulating plains with altitude varying from 405m to 435m above mean seal level[14]. The regional geology is predominantly composed of alluvial and lacustrine deposits[16]. According to the soil map in the National Atlas of Ethiopia, 1984 edition[11], the dominant types of soil in the region are chromic and pellic vertisols. Unpublished site investigation reports of the on-going road projects also confirmed that black cotton soil is the dominant superficial material that covers vast area. This soil is underlain by sandy soil after depths varying from 0.5m to 3m below the surface. It was also reported that natural gravel and rock are very scarce materials in the region[8].

In areas with scarce resources of suitable construction materials, it is common to upgrade the available materials using appropriate stabilization technique, and utilize them for the intended construction. In many cases materials that are not satisfactory in their natural state can be modified using admixtures and made suitable for the proposed construction[31]. Generally, stabilization can be applied to soil to improve strength, durability, workability, and/or waterproofing quality[21].

Economical stabilization of the natural sand underneath the black cotton soil in Gambella region can be a means to tackle the shortage of materials that are required for road base construction.

The strength of natural sand can be improved by using cement and/or bitumen stabilization. Cement stabilization consists of adding Portland cement to the sand and allow the mixture to gain strength by hydration of cement. In bituminous stabilization, the bitumen provides cohesion to the sand mass and waterproofing to any clay constituents in the sand[31]. The addition of sulfur to the sand-bitumen mixes substantially improves their quality. Sand-sulfur-bitumen mixes can be designed to possess sufficient strength so that they can be used as road base and surfacings[7].

Various researches had been carried out to stabilize natural sand with cement, bitumen, and sulfur-bitumen combination. A research by TRL on Nigerian fine sand proved that it is possible to stabilize natural sand with bitumen for road base construction[18]. Similarly a research on Saudi Arabian dune sand using asphalt cement and industrial wastes, like kiln dust, gypsum, and iron oxide, confirmed the possibility of improving the strength of natural sand to meet the road base requirement[15]. A study on cement stabilized wind-blown sand indicated that the strength of the sand can be improved due to the cohesional resistance imparted by the cementing agent[2].

The purpose of this research is to stabilize the sand, found underneath the black clay soil, with cement and bitumen so that it can be used for road base construction. The Marshall method of mix design was employed for the bituminous stabilization of the sand and the conventional mix design procedure was followed to stabilize the natural sand with Ordinary Portland Cement. In the bituminous mix, sulfur and ordinary Portland cement were used as admixtures and their effect on the Marshall stability of the sand-bitumen mix was examined.

1.2 Problem Statement

Rock is a very scarce resource in Gambella region where the geology is predominantly composed of alluvial and lacustrine deposits. Finding sound rock sources for road base construction, within a reasonable hauling distance, is a rare success in the region. Thus utlization of unbound granular materials, which fulfill ERA Standard Technical Specification, unreasonably escalates the investment cost of road projects due to the long hauling distance. To cope up with this problem, there must be a means to use the locally available materials for construction of pavement structural components. This is the problem that initiated this research.

1.3 Research Questions

The research is aimed to answer the following research questions:

- Is it possible to use the locally available abundant natural sand for base course construction?
- Is it possible to stabilize the locally available natural sand with bitumen without additives?
- Is it possible to further increase the stability of the natural sand – bitumen mixture with the help of additives?
- What composition of bituminous mix optimally fufill the base course requirements?
- What composition of ordinary Portland cement and natural sand produce materials that meet the road base requirement?

1.4 Research Objective

1. To assess the possibility of using the locally available material for road base construction.
2. To determine appropriate mix composition so that the natural sand-bitumen mixture provides the required stability without compromising other mix criteria;
3. To find methods that help to increase the stability of natural sand-bitumen mixture without compromising other mix criteria.
4. To determine the optimum cement content at which the natural sand-cement mixture meets base course requirements.

2 LITERATURE REVIEW

2.1 Introduction

In many parts of tropical areas, materials that fulfill the requirements of untreated road pavement materials are not always readily available. Scarcity in the natural construction materials increases the production cost of the materials, which would be hauled from elsewhere. The feasible alternative to counteract this problem is either to improve the locally available materials with stabilizing agent or to use the locally available materials in their natural state[27]. Inferior quality materials can be made to resist excessive deformation and deflection by applying soil stabilization or modification techniques[9].

Soil stabilization and soil modification are both related to improvement of the soil properties so that they suit a particular purpose. Soil modification often refers to soil improvements that happen during or shortly after mixing. Modified soils are those whose consistency, gradation, and/or swelling properties are improved to the desired extent and strength is increased to a certain extent[9,10,29]. Soil stabilization on the other hand refers to significant improvement in the strength of the soil due to the long term reaction that takes place between the soil and stabilizing agent. The reaction can be due to hydration reaction in portland cement or class C fly ash, or pozzolanic reactivity between free lime and soil pozzolans[9]. Soil Stabilization in general is defined as improvement of soil so that it can be used for subbases, bases, and, in some rare instances, surface courses[31]. It was stated in the Department of Army report[10] that 'Stabilization is the process of blending and mixing materials with a soil to improve certain properties of the soil'. This can be done by mixing of soils to meet the target gradation or by combining additives that are commercially available so as to change the gradation, texture, plasticity, or act as a binder for cementation of the soil. However, treatment of soils with mechanical means is also refered as stabilization[9]. There are different approaches that are used to distinguish modified and stabilized soils. Most of the approaches are tied with the strength of the treated materials. To mention some:

1. If a soil gains seven days UCS of 0.8MPa and above after treatment, then it can be considered as stabilized soil, else it is a modified soil[29];

2. If the strength of the soil after treatment exceed its strength before treatment by 350kPa and above, under the same condition of compaction and cure, then it can referred as stabilized soil[9];

3. If a treated soil fulfills the durability requirements of the specifications for a particular material and fails to meet the strength requirements, then it can be considered as modified soil. However, if the treated soil meets both the strength and durability requirements, the soil can be said it is stabilized[10].

Soil stabilization is broadly classified into four types, namely: thermal, electrical, mechanical, and chemical. In thermal stabilization the soil properties are modified by either heating or freezing. Heating the soil to 600^{o}C irreversibly dehydrate or fuse the soil particles whereas freezing strengthen the soil by solidifying the water in the soil. Electrical stabilization is carried out by applying a direct electrical current to the soil. As a result the water in the soil migrate out to an electrode and the soil get strong. Thermal and electrical stabilization are rarely used options for soil. Mechanical stabilization refers to mixing (of two or more soils), draining, and/or compacting of the soils. Application of fibrous or other nonbiodegradable reinforcing materials is also grouped under mechanical stabilization. In chemical stabilization the soil is mixed with chemicals like cement, lime, fly ash, bitumen or combination of these materials. Chemical stabilizers are most widely used in road construction industry. They are broadly classified into three groups, namely: Traditional stabilizers, non-traditional stabilizers, and by-product stabilizers[9,19].

Traditional Stabilizers

Traditional stabilizers are hydrated lime, portland cement, and fly ash. Traditional stabilizers modify or improve soil properties through hydration reaction, pozzolanic activity, cation exchage or a combination of these.

Lime is one of the most frequently used traditional stabilizer. It stabilizes/modifies the soil mainly in two steps, namely: cation exchange and pozzolanic reactions. Cation exchange is the first step that takes place rapidly as compared to the pozzolanic reaction. In this process the calcium in the lime replaces the potassium and sodium ions in the soil. As a result the thickness of the water bound around the clay particles decrease significantly and cause flocculent structures to develop. Consequently, the plasticity, shrinkage, and workability of the soil are improved. The second process, pozzolanic reaction, is the chemical reaction of the pozzolans in the soil with lime (calcium hydroxide) and water. This reaction leads to the formation of hydrated calcium silicate, C-S-H, which is cementatious material. Pozzolanic reactions resulted in strength improvement in the soil. The reaction is a long-term process that progresses as far as the PH of the soil is kept above 12.4 to dissolve the silicates and aluminates in the soil matrix.

Portland cement consists of calcium-silicates and calcium-aluminates that hydrate to form cementatious products. The hydration of portland cement is fast and causes rapid strength development in the stabilized soil. During hydration reaction, free lime [Ca(OH)] is produced. In high PH soil, this lime can react pozzolanically and add to the strength of the soil.

Fly-ash is a by-product of burning coal. As a result its properties vary based on the types of coal and burning processes. According to AASHTO M 295 (ASTM C 618), fly ash is divided into two types: Class C (self-cementing) and Class F (non self-cementing). Class C fly ash is produced from lignite or sub-bituminous coal and possesses both pozzolanic and cementitious properties. Class F fly ash, on the other hand, is produced from the burning of anthracite or bituminous coal. It has only pozzolanic properties. To use Class F fly ash as soil stabilizer, certain amounts of portland cement or lime shall be added as activating admixtures so that there is sufficient lime to begin the pozzolanic reaction[9,21,29].

Non-traditional Stabilizers

There are about seven types of non-traditional stabilizers namely chlorides, clay additives, electrolyte emulsions, lignosulfonates, synthetic-polymer emulsions, and tree-resin emulsions. Most of this additves are used to prevent dust and stabilize the soil based on the rate of application.

Chlorides are one of non-traditional stabilizers. They are used in unbound roads for dust suppression. Chlorides contain chloride salts and exist in liquid or solid states. They absorb the moisture in the air and keep the road surface moist and hence reduce dust nuisance. Chlorides are also used to reduce the evaporation of road surface moistures.

Clay additives are soils made up of montmorillonite minerals, which is a highly plastic clay mineral with high affinity for water. The clay additives are used to stabilize the non-plastic crushed aggregates so as to prevent raveling and washboarding.

Electrolyte emulsion are composed of chemicals that can affect the electro-chemical bonding of the soil and replace water molecules in the soil. They are used for dust suppression and soil stabilization purposes. When applied in low quantity to the unbound road surface, electorlyte emulsions suppress dust nuisance. If applied at higher rates, the electrolyte emulsions stabilize the soil and create hard bound layer that can even be used as road surfacing.

Enzymatic emulsions contain enzymes that can react with soil molecules to form a cementing bond. Enzymatic emulsions reduce the soil's water affinity. They have similar purpose as the electrolyte emulsions in that when applied at lower amount, they are used to suppress dust, and at higher amount, they stabilize the soil.

Lignosulfonates are made up of lignin. They can draw moisture from the air and also have cementitious properties. At lower applications, lignosulfonates suppress dust and at higher applications they can be used to stabilize subgrade or base materials containing fines. Generally lignosulfonates improve the compressive strength and load bearing capacity, bind materials to reduce particle loss, and provide a hard and dust-free surface.

Synthetic polymer emulsions are composed of acrylic or acetate polymers. They are used to make dense and water-resistant road surface by improving the chemical bond between the soil particles. Alike electrolytes, enzymatic emulsions and lignosulfonates, polymers are used to suppress dust and stabilize the soil when applied in low and high amounts.

Tree resin emulsions are produced from tree resins. Similar to most of the non-traditonal stabilizers they are used to control dust nuisance and stabilize soils[19].

Even if the non-traditional stabilizers serve similar functions of dust suppression and soil stabilization, they are not equally effective in all types of soils. Their effectiveness depends on the type of soil, weather conditions, terrain, and other factors. Therefore, due emphasis shall be given while selecting a particular type of additives so that the target requirements are met.

By-product Stabilizers

By-product stabilizers are industrial by-products like cement kiln dust and lime kiln dust. Lime kiln dust and cement kiln dust are the by-products of lime and cement production plants respectively. These materials contain considerable amounts of both lime and pozzolanic materials. Therefore, they help to stabilize or modify soils both through cation exchange and pozzolanic reaction[19].

In general presently there are a number of stabilizing additives that play various roles in the civil construction industry. The stabilizing additives include cementing agents, modifiers, waterproofing agents, water-retaining agents, water-retarding agents, and other various chemicals as summarized in table 2.1[31]. In road construction industry, many natural materials can be stabilized to make them suitable for road pavements provided that the cost of overcoming a deficiency in the locally available material does not exceed the cost of importing another material which is good without stabilization[13].

Table 2.1 Types and Effects of Stabilizers [ref 31]

	Type	Admix	Primary Mechanics of Stabilization	Use	Situations Best Suited	Approximate Quantity by Weight
Increase Strength by Cementing Action	Cementing agents[b]	Portland cement	Principally hydration Some modification of clay minerals	Base and subbase	Sandy soils or lean clays	A-7 9–15% to A-2 5–9%
		Lime	Change water film, flocculation, chemical	Some base and subbase, shoulders	Granular materials or lean clays	2–5%
		Lime flyash	Pozzolanic action of lime and silica, some modification of clay minerals	Some base and subbase, shoulders	Granular materials or lean clays	2–5% lime 10–20% flyash
		Bitumen		Base and subbase	Granular	2–5%
Improve Plasticity May or May Not Increase Strength	Modifiers	Cement	Modification of clay Change water film	Improve poorly graded base and subbase	Improve existing road metal, clays	½–4%
		Lime	Change water film, modification of clay minerals	Do	Do	½–4%
		Bitumen	Retards moisture absorption	Do	Improve existing road metal	1–3%
Little or No Increased Strength	Water-proofing agents	Bitumen	Retards moisture sorption by coating soil grains	Primarily subbase	Sandy soils or poor quality base materials, some clays	4–6%
		Membranes	Prevents movement of free water and water vapor	Primarily subbase and subgrades	Soils that may be improved by compaction	
	Water-retaining agents	Calcium chloride	Deliquescent properties, lowers freezing point, base exchange	Construction expedient, traffic binding	Graded aggregate	½–1½%
		Sodium chloride	Deliquescent properties, lowers freezing point	Do	Do	½–1½%
Little or No Increased Strength	Water retarding	Organic cationic compounds	Alters clay minerals to act as a hydrophobic agent	Subbases		Trace

[a] In some cases a slight increase is shown for granular soils.

2.2 Selection of Stabilizer

There are a number factors which need to be considered to select appropriate type of stabilizer. These are the type of soil to be stabilized, purpose of the stabilized layer, desired improvement, required strength and durability, cost of stabilization, and environmental conditions[10]. The TRL Overseas Road Note 31, from which ERA PDM had been developed, set the guideline in table 2.2 for selection of stabilizer based on particle size distribution and plasticity of soil.

Table 2.2 Road Note 31 Guideline for Selection for Stabilizer

Type of Stabilization	Soil Properties					
	More than 25% passing the 0.075mm sieve			Less than 25% passing the 0.075mm sieve		
	PI≤10	10<PI≤20	PI>20	PI≤6 PP≤60	PI≤10	PI>10
Cement	Yes	Yes	Marginally Effective	Yes	Yes	Yes
Lime	Marginally Effective	Yes	Yes	No	Marginally Effective	Yes
Lime-Pozzolan	Yes	Marginally Effective	No	Yes	Yes	Marginally Effective

The U.S Air Force on the other hand had set guidelines called Soil Stabilization Index System [SSIS] to select candidate stabilizers for a particular soil or base materials. These are shown in the form of decision trees in fig 2.1 and fig 2.2[9].

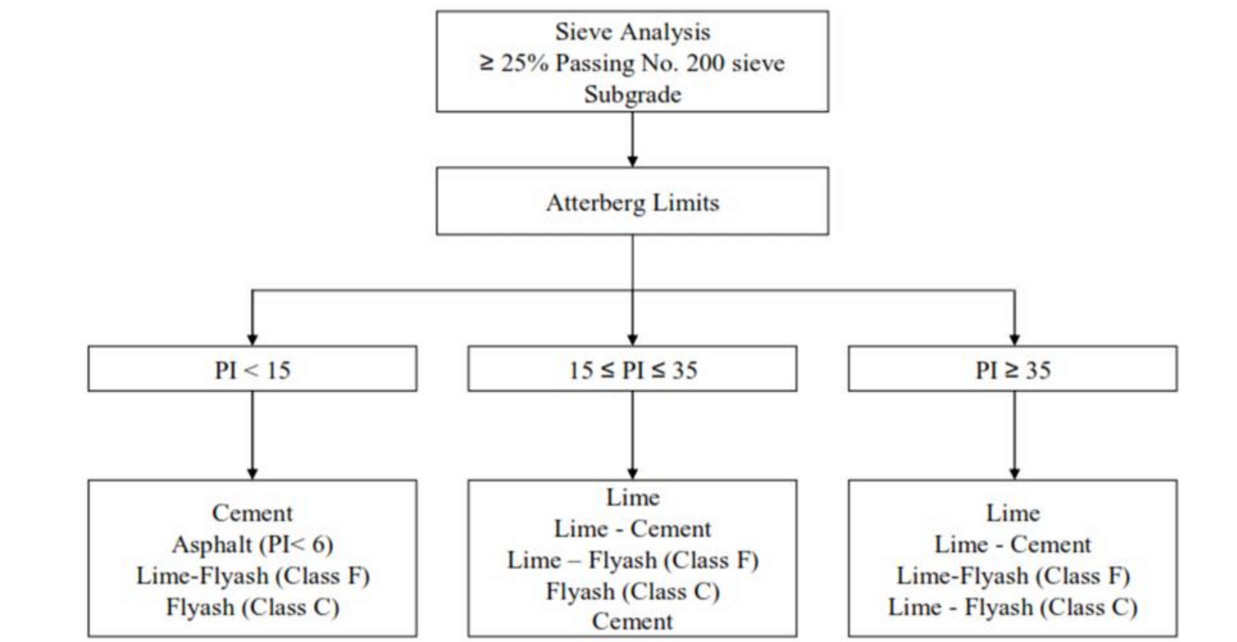


Figure 2.1 Decision tree for selecting stabilzers for use in subgrade soils

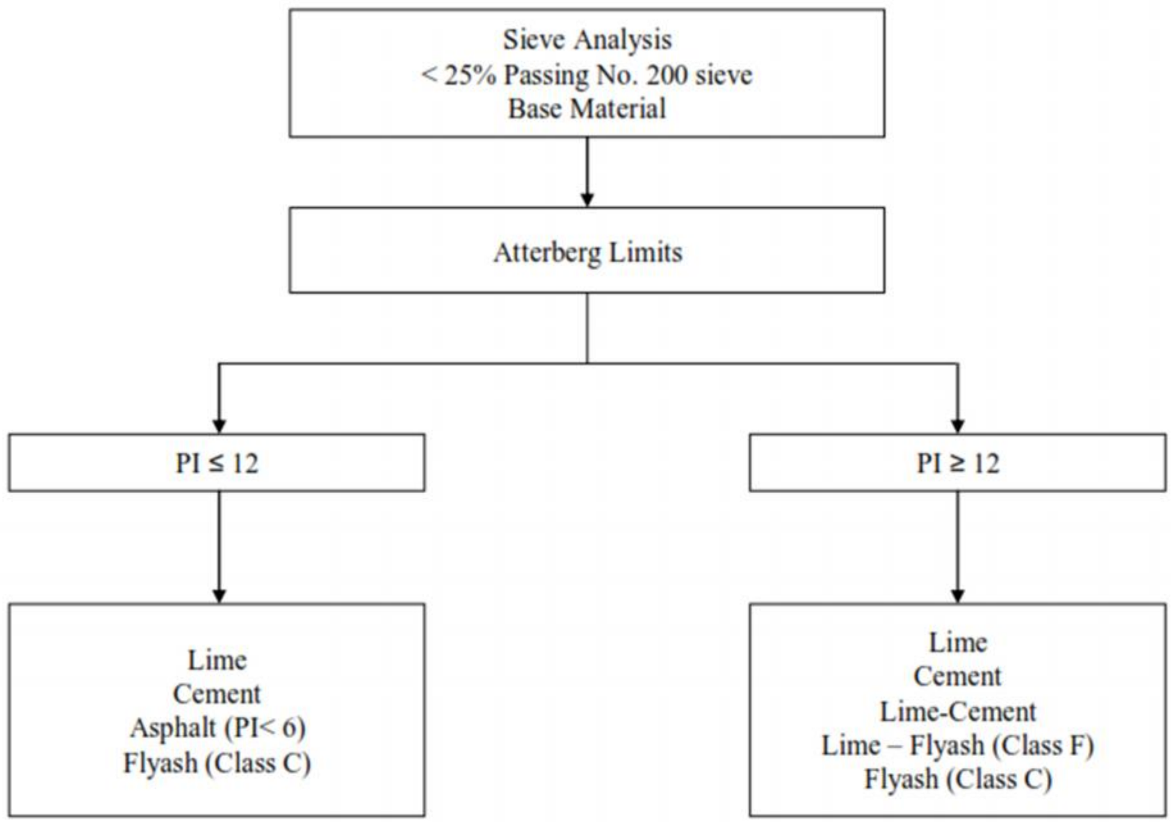


Figure 2.2 Decision tree for selecting stabilizers for use in base materials

It is possible that more than one type of stabilizer may be found suitable to treat a particular soil type. Under such circumstances, distinction can be made based on the granularity, plasticity, and texture of the soil to be treated. Well graded granular soils with sufficient amount of fines can be well treated with portland cement. However, portland cement is not recommended to stabilize highly plastic soils as it is difficult to mix the cement with the plastic clays[10]. In order to utilize portland cement to stabilize plastic clays, the clays have to first be modified with about 2% lime to make it more workable[28].

High plasticity of soils indicates higher concentration of reactive clay minerals which can readily be stabilized with lime. Lime is suitable stabilizer for medium and highly plastic soil. Lime can be used to stabilize weak subgrade soils and transform them into sub-base layer. Marginal quality granular materials can also be treated with lime so that they can serve as high quality base course materials[10].

The guideline in fig. 2.3 and table 2.3 was prepared by US Army and Air Force to help in selecting the suitable types of stabilizer for a particular soil type. The particle size distribution, the Atterberg limits, and the soil class (USCS) are the only required parameters to use the guideline. While using this guideline, if more than one stabilizer types are found appropriate for the type of soil to be stabilized, distinction shall be made based on other criteria like availability of the stabilizer, cost, ease of construction, etc.

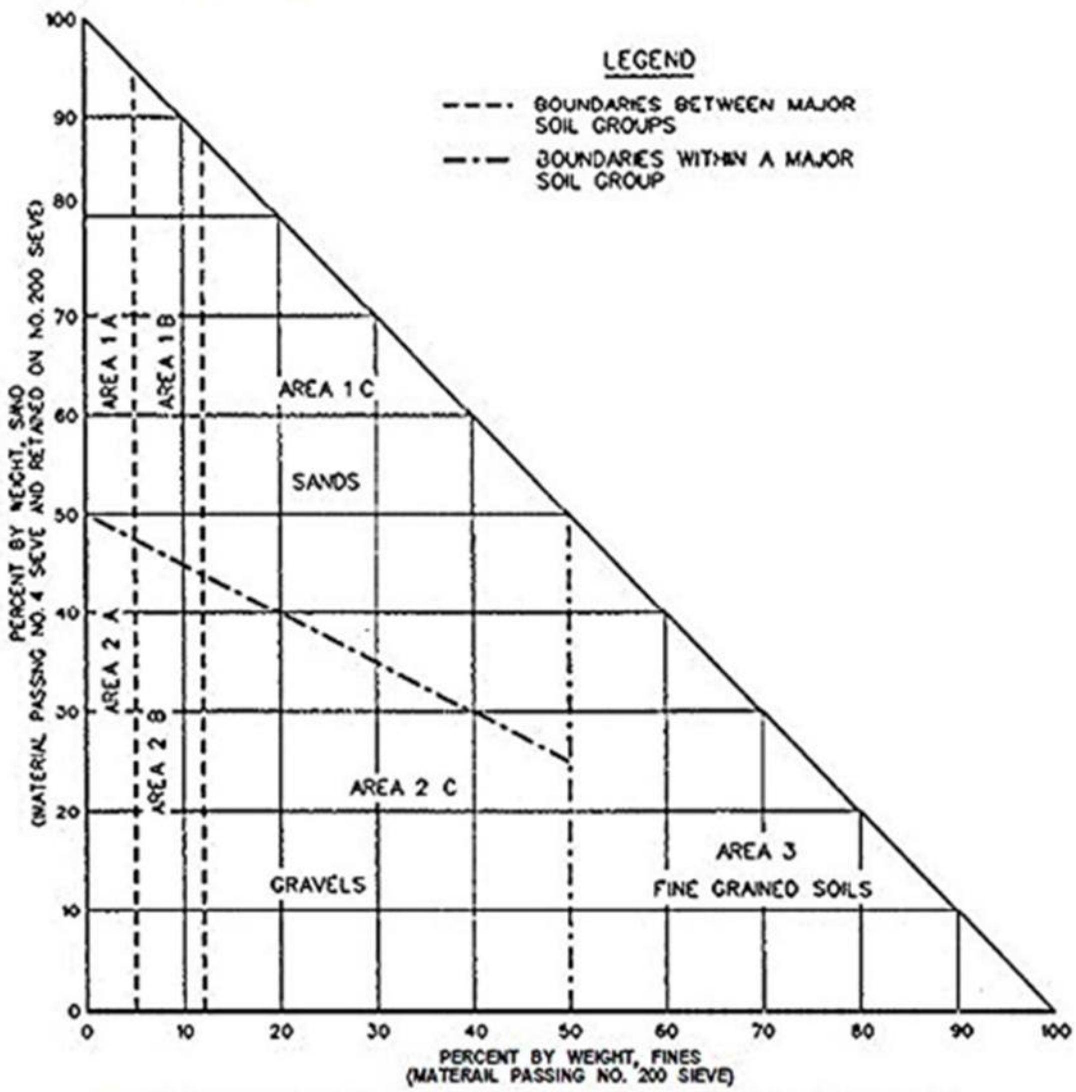


Figure 2.3 Gradation triangle for aid in selecting a commercial stabilizing agent [ref 10]

Table 2.3 Guide for Selecting Stabilizing Additive [ref 10]

Area	Soil Class (a)	Type of stabilizing additive recommended	Restriction on LL and PI of soil	Restriction on percent passing No. 200 sieve (a)	Remarks
1A	SW or SP	1. Bituminous			
		2. Portland cement			
		3. Lime-cement-fly ash	PI not to exceed 25		
1B	SW-SM or SP-SM or SW-SC or SP-SC	1. Bituminous	PI not to exceed 10		
		2. Portland cement	PI not to exceed 30		
		3. Lime	PI not to exceed 12		
		4. Lime-cement-fly ash	PI not to exceed 25		
1C	SM or SC or SM-SC	1. Bituminous	PI not to exceed 10	Not to exceed 30% by weight	
		2. Portland cement	b		
		3. Lime	PI not less than 12		
		4. Lime-cement-fly ash	PI not to exceed 25		
2A	GW or GP	1. Bituminous			Well-graded materials only.
		2. Portland cement			Material should contain at least 45% by weight of material passing No. 4 sieve
		3. Lime-cement-fly ash	PI not to exceed 25		
2B	GW-GM or GP-GM or GW-GC or GP-GC	1. Bituminous	PI not to exceed 10		Well-graded materials only.
		2. Portland cement	PI not to exceed 30		Material should contain at least 45% by weight of material passing No. 4 sieve
		3. Lime	PI not less than 12		
		4. Lime-cement-fly ash	PI not to exceed 25		
2C	GM or GC or GM-GC	1. Bituminous	PI not to exceed 10	Not to exceed 30% by weight	Well-graded materials only
		2. Portland cement	b		Material should contain at least 45% by weight of material passing No. 4 sieve
		3. Lime	PI not less than 12		
		4. Lime-cement-fly ash	PI not to exceed 25		
3	CH or CL or MH or OH or OL or ML-CL	1. Portland cement	LL less than 40 and PI less than 20		Organic and strongly acid soils falling within this area are not susceptible to stabilization by ordinary means.
		2. Lime	PI less than 12		

a. Soil classification corresponds to MIL-STD-619B. Restriction on liquid limit (LL) and plasticity index (PI) is in accordance with Method 103 in MIL-STD-621A

b. PI ≤ 20 + (50 – percent passing No. 200 sieve)/4

It can be noted that the various stabilizer selection criteria are mainly based on the index properties of soils particularly plasticity index and liquid limit, and soil class. These properties can easily be identified using simple laboratory tests. The effectiveness of the particular stabilizer that is selected based on the index properties and soil class shall, however, be evaluated using additional strength and/or durability tests.

2.3 Cement Stabilization

According to American Concrete Institute definition, soil-cement is a mixture of soil and a measured amount of cement and water mixed to a high degree[9]. The term cement stabilization is applied to compacted soil-cement and cement modified soils. In compacted soil-cement, the cement is used to get sufficiently strong and well compacted materials while in cement-modified soils, the cement effects reduction in plasticity, volumetric shrinkage and the like[21]. Cement-modified soils may gain moderate strength improvement owing to the addition of slight amount of cement. It was suggested that seven days UCS of 0.8MPa can be a borderline between cement-modified and cement-stabilized soils. That means, if a cement treated soil gains more than 0.8MPa improvement, it can be called as cement-stabilized and vice-versa[29].

Cement stabilization to produce hardened mixture is carried out by adding Portland cement to a pulverized soil, and allowing the mixture to harden by hydration of the cement. The physical properties of the soil-cement are affected by the soil type, the cement quantity, degree of mixing and compaction, curing time, degree of pulverization, PH of soil, and dry density of compacted mixture[21,31].

The soil-cement interactions are influenced by hydration, cation exchange, carbonation, and pozzolanic reactions, of which the first two are the main factors. Hydration reaction takes place by the reaction of tricalcium silicate (C3S) and dicalcium silicate (C2S) with water. Tricalcium silicate and dicalcium silicate constituent 45% and 27% of ordinary Portland cement respectively. The hydration reaction is written as:

$$C3S + 2H = C2SH + CH$$

$$C2S + 2H = CSH + CH$$

Or

$$3CaO.SiO_2 + 2H_2O = 2CaO.SiO_2.H_2O + Ca(OH)_2$$

$$2CaO.SiO_2 + 2H_2O = CaO.SiO_2.H_2O + Ca(OH)_2$$

Free lime, CH, is formed following the formation of the insoluble silicate gel which crystallizes very slowly into an interlocking matrix. The free lime helps to modify the soil through cation exchange.

Based on previous experience a soil is considered to be suitable for cement stabilization provided that it fulfills the requirement in table 2.4[21].

Table 2.4 Physical Requirements of Soils for Cement Stabilization

Parameters	Limits
% ≤ 0.075mm	< 35 %
% > 0.075 mm	> 55%
Maximum grain-size	< 75mm
Liquid Limit	< 50%
Plastic Limit	< 25%

2.3.1 Test Methods

The soil-cement mixtures are subjected to laboratory tests so as to determine the optimum cement content, the optimum moisture content, the strength of compacted mixture, and the level of compaction that is required to attain the target qualities.

According to Portland Cement Association and US Army and Air Force publications[10,23] the soil-cement specimen parameters are identified by the four AASHTO/ASTM test methods in table 2.5. Road Note 31 on the other hand suggested only moisture-density and strength tests to determine the soil-cement mixture with optimum binder content[28]. The durability tests are not required when the soil-cement mix is designed based on the Road Note 31 approach. The laboratory test methods that are widely applicable in the design of soil-cement mixture are shown in table 2.5.

Table 2.5 Laboratory Test Methods for Soil-Cement Mixtures

Test Type	Test Method
Moisture-Density Relation	ASTM D 558/AASHTO T 134
Wetting and Drying	ASTM D 559/AASHTO T 135
Freezing and Thawing	ASTM D 560/AASHTO T 136
Compressive Strength	ASTM D 1633
Test Methods for Cement-Stabilized and Lime-Stabilized Materials	BS 1924

2.3.2 Determination of Design Cement Content

The design cement content for cement-stabilized soil can be determined using the following steps which were discussed in the manual, TM 5-822-14/AFMAN 32-8010, prepared by Joint Departments of the Army and Air Force, USA, 1994. These steps are:

i. the gradation and classification of the untreated soil are determined using the procedures in ASTM D 2487 and ASTM D 422;

ii. the initial cement content is determined based on the soil classification test result from table 2.6;

Table 2.6 Initial Cement Requirements for Various Soils

Soil Classification	Initial Estimated Cement Content (% dry weight)
GW, SW	5
GP, GW-GC, GW-GM, SW-SC,SW-SM	6
GC, GM, GP-GC, GP-GM, GM-GC, SC, SM, SP-SC, SP-SM, SM-SC, SP	7
CL.ML, MH	9
CH	11

iii. soil-cement mix is prepared using ASTM D 558 and moisture-density tests are conducted using ASTM D 1557 to determine the maximum dry density and optimum moisture content;

iv. triplicate samples of soil-cement mixture are prepared for unconfined compression and durability tests at cement content selected in step 2 and at the cement contents 2% above and 2% below that determined in step 2. The samples are prepared at the density and water content that are expected in field construction. The samples should be prepared in accordance with ASTM D 1632 except that when more than 35% of the material is retained on the No. 4 sieve, a 4-inch diameter mould is used to prepare the specimens. The specimens are cured for 7 days in a humid room. Three specimens are tested using the unconfined compression test in accordance with ASTM D 1633 and the other three specimens are tested for durability using either wet-dry method (ASTM D 559) or freeze-thaw (ASTM D 560) as appropriate.

v. The results of the unconfined compressive strength and durability tests are compared with the requirements given table 2.7 and 2.8. The lowest cement content which meets the required unconfined compressive strength requirement and satisfies the required durability is the **design cement content.** If the results of the specimens tested do not meet the strength and durability requirements, then a higher cement content should be selected and the steps **i** through **iv** shall be repeated.

Table 2.7 Minimum Unconfined Compressive Strength for Cement Stabilized Soils

Stabilized Soil Layer	Minimum Unconfined Compressive Strength, MPa*	
	Flexible Pavement	Rigid Pavement
Base course	5.17	3.44
Sub base course, select material or subgrade	1.72	1.38

* *unconfined compressive strength determined at 7 days for cement stabilization and 28 days for lime, lime fly ash, or lime-cement-fly ash stabilization.*

Table 2.8 Durability Requirements

Type of Soil Stabilized	Maximum Allowable Weight Loss After 12 Wet-Dry or Freeze-Thaw Cycles Percent of Initial Specimen Weight
Granular, PI < 10	11
Granular, PI >10	8
Silt	8
Clays	6

A general guideline was provided in the amount of cement required to stabilize a soil, depending on past experience, as shown in table 2.9[21]. Gravelly and sandy soils require less cement for adequate hardness than silt and clay soils. The cement requirement of soils increase with increase in the amount of the silt and clays contents. However, sandy soils carrying certain amounts of silt and/or clay fractions require lesser amount of clean poorly graded sandy soils[23].

Table 2.9 General guideline on the amount of cement required to stabilize a soil[5(p 28)]

Soil Type	Amount of Cement	
	% by weight	% by volume
A-1-a	3 – 5	5 – 7
A-1-b	5 – 8	7 – 9
A-2	5 – 9	7 – 10
A-3	7 – 11	8 – 12
A-4	7 – 12	8 – 13
A-5	8 – 13	8 – 13
A-6	9 – 15	10 – 14
A-7	10 - 16	10 - 14

According to the Soil-Cement Laboratory Handbook[23] of PCA, the optimum binder content shall be determined based on the durability test results. The durability test can be either freez-thaw test or wet-dry test. The cement content that is required to produce a soil-cement mixture capable of resisting the stress induced by the durability tests is the optimum cement content. Two criteria were set to identify the soil-cement mixture suitable for highest quality base-course construction. These are:

1. Soil-cement losses after 12 cycles of either wet-dry test or freeze-thaw test shall conform to the following limits:

   For Soil Groups A-1, A-2-4, A-2-5, and A-3, not over 14 percent;

   For Soil Groups A-2-6, A-2-7, A-4, and A-5, not over 10 percent;

   For Soil Groups A-6 and A-7, not over 7 percent.

2. Compressive strength should increase both with age and with increase in cement content in the ranges of cement content producing results that meet requirement 1.

In the PCA approach, compressive strength is secondary criteria that is used to check the influence of cement in the soil-cement mixture. That is if the strength of the soil-cement mixture increases with increasing age and increasing cement content, then the second criteria is proved.

2.3.3 Specifications for Cement-Treated Base Course

Many pavement design standards specified the requirement for cement-treated base course in terms of 7-days unconfined compressive strength. Some standards set additional durability requirements based on the wet-dry or freeze-thaw test results.

The Portland Cement Association in its Soil-Cement Laboratory Handbook[23] put only durability requirements as the main specification for cement-stabilized base layer. The US Army and Air Force, on the other hand, set both strength and durability requirements for cement-stabilized bases[10]. Road Note 31[28], South African Code of Practice for the Design of Road Pavements[25], and the Tanzanian Pavement and Materials Design Manual[26] were all specified strength requirements for the cement-treated base course. There are two types of stabilized base layers in Tanzania Pavement and Materials Design Manual and Road Note 31. The two types have different strengths and serve the same purpose but under different traffic loading condition. The one with lower strength will be applied when the traffic load is light and the other with higher strength will be applied in heavy traffic zones.

Summary of the cement-treated base materials specifications is given in table 2.10.

Table 2.10 Specifications for Cement-Treated Base Layer in Flexible Pavement

Standards	Strength Requirement [MPa]	Durability Requirement [% Loss]
Road Note 31 [ERA PDM]	3.0 – 6.0 for CB1	-
	1.5 – 3.0 for CB2	
US Army and Air Force	5.2	Max. 11 for Granular Soil with PI <10
		Max. 8 for Granular Soil with PI > 10
		Max. 8 for Silt
		Max. 6 for Clay
Portland Cement Association	-	Max. 14 for A-1, A-2-4, A-2-5, and A-3 soils
		Max. 10 for A-2-6, A-2-7, and A-5 soils
		Max. 7 for A-6 and A-7 soils
SATCC	1.5 – 3.0	-
Tanzania PMDM	1.0 for C1	-
	2.0 for C2	

The strength requirements are based on the 7-days unconfined compressive strength test in cylinderical specimens. The exception to this is the Road Note 31 strength requirement, which was set based on the 150mm cube strength. The SATCC strength requirement is based on cylinderical speciment with diameter-to-height ratio of 1:1.

The durability requirements refer to the 12-cycle wet-dry or freeze-thaw tests.

2.3.4 Problems Associated with Cement Stabilization

Despite the many benefits, there are problems associated with cement stabilized materials that entail due considerations. The main problems that will have pronounced negative effects if not controlled are cracking and carbonation. These problems are happened in the compacted stabilized layer after construction.

In cement-stabilized bases, cracking is attributed to materials charcteristics, construction procedures, traffic loading, and restraint imposed on the base by the subgrade[1]. The most common type of crack in cement-stabilized base is shrinkage crack[6]. Shrinkage cracks are related to loss of water, cement content, density of compacted material, method of compaction, and pretreatment moisture content of the material to be stabilized.

Cement treated materials begin to lose their moisture through evaporation immediately after they are placed if proper curing is not excercised. The loss of moisture then will lead to the drying and subsequent development of shrinkage cracks. Further, the final strength of the cement treated materials will be reduced as hydration of the cement is hampered due to lack of sufficient moisture in the mix.

The contribution of cement hydration in the development of shrinkage cracks is less as compared to water loss. Nevertheless, excessive amount of cement aggravates the development of cracks in two ways:

1. Higher amount of cement in the mix causes greater water consumption during hydration which in turn increases the drying shrinkage;

2. Increased amount of cement increases the rigidity and tensile strength of the treated materials. As a result, widely spaced wide cracks are developed. The wider spacing of the cracks is attributed to the higher tensile strength and the wider width of individual cracks is due to the distribution of total shrinkage of the material within smaller number of the widely spaced cracks.

In many soils the shrinkage is reduced at beginning with the addition of smaller amount of cement. However, it increases to peak upon further addition of the cement content as shown in fig 2.4. In this figure, it is also shown that the cement content at which shrinkage is lowest is below the minimum amount that required for freeze/thaw durability (F/T req). Therefore, in order to alleviate the contribution of cement hydration in the development of shrinkage cracks, the amount of cement should be limited to a maximum of that required for durability requirement.

Compaction and method of compaction play vital role on shrinkage characteristics of cement-treated materials. In poorly compacted materials, there will be higher shrinkage and wider cracks as there is sufficient space that allows the movement of the soil-cement particles. In a well-compacted material, however, the particles are densely packed and restricted from further movement. As a result there is lower risk of shrinkage in a densely compacted materials. The effect of compaction and moisture content on the shrinkage potential of cement-treated soil is shown in fig 2.5. On the other hand, it was proved in the laboratory that samples compacted by impact (vibratory) compactor shrink more than those compacted by static loading or kneading compactors. Therefore, in order to minimize shrinkage cracking, it is preferrable to use sheeps foot or pneumatic-tire rollers[17,28].

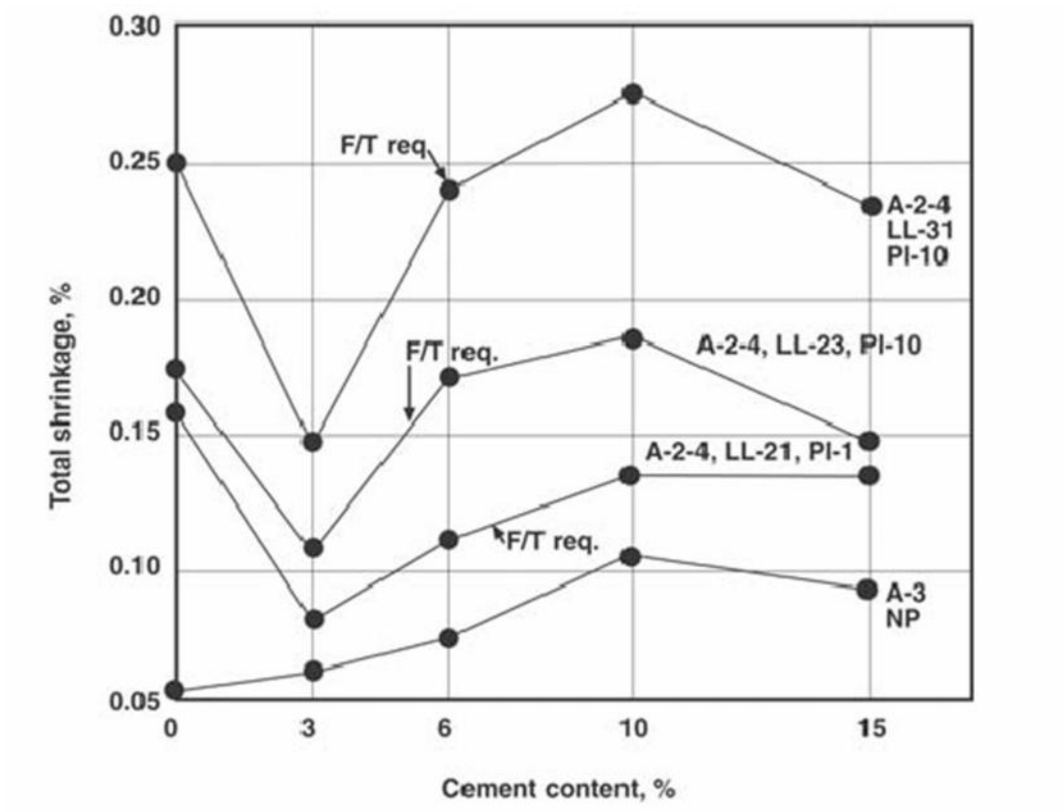


Figure 2.4 Effect of cement content on shrinkage (ref 1)

In gravels containing plastic fines, the development of shrinkage cracks can be minimized by controlling the initial moisture content of the material to be stabilized. Shrinkage problems, in these materials, can be mitigated by using air-dry gravel and completing the entire construction operation within two hours. Lower initial moisture content and quicker mixing and compaction substantially reduce the development of shrinkage strain in these type of materials[28].

Shrinkage cracks, if not controlled, cause reflective cracking in the pavement surfacing which inturn causes premature failure of the pavement. The development of shrinkage cracks can be controlled by: using slow setting binders, adequate mixing of the soil-cement, appropriate compaction, strict control of the mixing water content, and proper curing of the soil-cement mixture. Reflective cracking can be effectively prevented by the inclusion of stress-relief material between the stabilized base course and the pavement surfacing. The stress-relief materials can be a bituminous surface treatment (chip seal), geotextile, or unbound granular material[1,22,28].

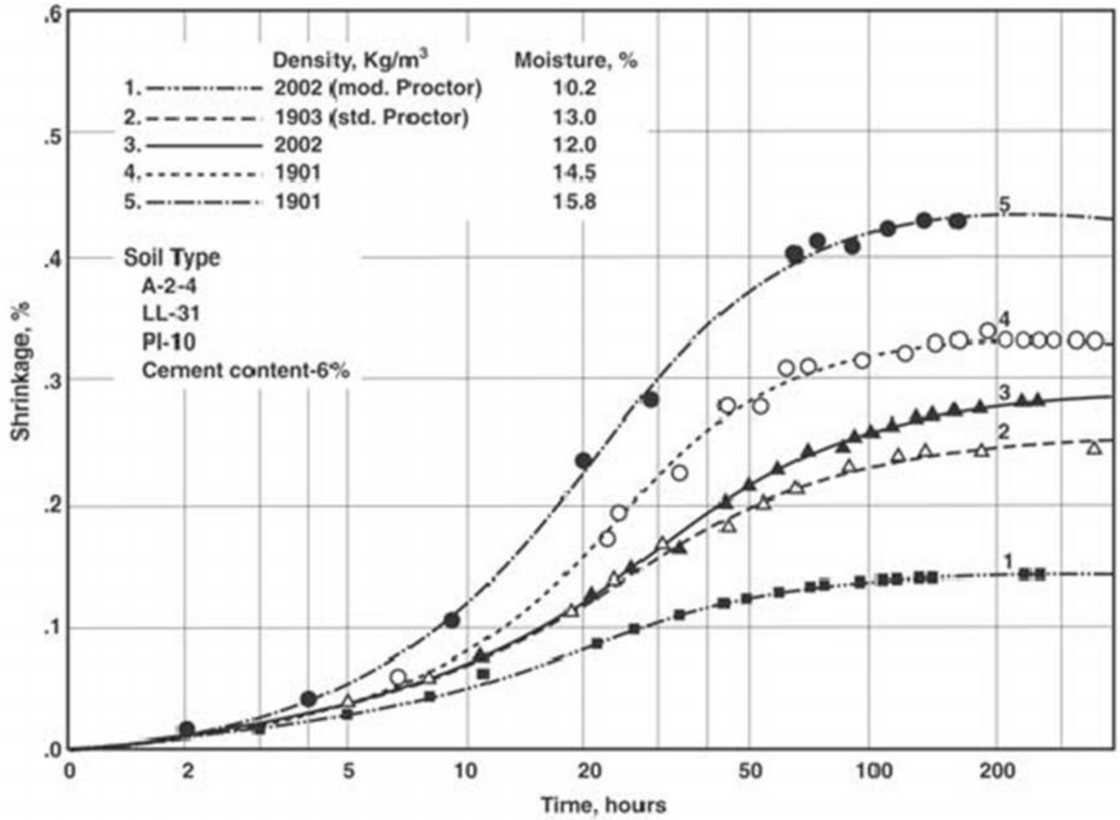


Figure 2.5 Effect of density and moisture on shrinkage (ref 1)

Carbonation is another problem that is associated with cement-treated materials. If cement-stabilized materials exposed to air, the hydration products, hydrated calcium silicate (CSH) and hydrated calcium aluminate (CAH), will react with carbon dioxide and converted into carbonates.

$$CSH + CAH + CO_2 \longrightarrow CaCO_3 + SiO_2 + Al_2O_3 + H_2O$$

This phenomena reduces the strength of the stabilized materials by an average 40% of the unconfined compressive strength. It also causes the reduction in the PH value of the stabilized material from above 12 to about 8.5. The PH of the cement-stabilized materials have to be highly basic, above 12.4, so that the silica and alumina in the soil are dissolved and form silicates and aluminates of calcium.

The occurrence of carbonation can be detected using PH tests and testing the presence of carbonates in the stabilized soils. The PH of the stabilized soil is identified with the help of phenolphthalein and phenol red indicators whereas the presence of carbonates is directly tested using dilute hydrochloric acid. Phenolphthalein remains colorless at a PH less than 8.4 and turns red when the PH is above 11. Phenol red, on the other hand, remains yellow at a PH less than 6.8, suggesting the absence of carbonate, lime, or cement, and turns red when the PH is above 8.0 indicating the presence of the same. Dilute hydrochloric acid effervesce when come in contact with calcium carbonate. Effervescence will not occur with $Ca(OH)_2$, CSH, and CAH. It is specified in BS 1924-2 that the combination of these three tests on the exposed surface of stabilized materials helps to identify the occurrence of carbonation. The determination of the presence of carbonation in stabilized materials based on these tests are summarized in table 2.11.

Table 2.11 Determination of the presence of carbonation according to BS 1924-2

Indicators			Result interpretation according to BS-1924-2
Phenolphthalein	**Dilute hydrochloric acid**	**Phenol Red**	
Red	No Effervescence	-	No carbonation has occurred and carbonates were absent from thhe original materials
Clear	Effervescence	Red	If it is known that carbonates were not present in the original soil, then the result indicates that carbonation of the stabilizer has taken place. However, if carbonates were present in the original material, it is not possible to distinguish between whether carbonation has occurred or whether the stabilizer was not added.
Clear	No Effervescence	Red	Stabilizer was not added. This shall further be assessed using qualitative test for calcium. If it is proved that calcium is not present, then it can be assured that stabilizer is not added.
Red	Effervescence	-	Carbonates were probably present in the original materials. However, if it is known that carbonates were not present in the original material some partial carbonation must have occurred.

The rates of carbonation reportdly ranges from 0.5mm – 2mm/day in air to 2mm – 50mm/year at the bottom and sides of stabilized layers in the road[29]. The theoretical and observed effects of carbonation on the stabilization process are summarized in table 2.12.

Table 2.12 Summary of the effects of carbonation (ref 29)

Process	Observed effect
$Ca(OH)_2$ destroyed, PH decrease from 12.4 (lime) to 8.3 ($CaCO_3$)	$CaCO_3$ formed, plasticity increases
CSH and CAH destoryed	Decreae in UCS, CBR or tensile strength
Shrinkage of calcium silicates	Cracking and micro-cracking
Reduced relative compaction	Road surface deformation and rutting

These risks of carbonation can be prevented by avoiding the wetting-and-drying of the stabilized material during the curing phase, by immediately sealing the compacted stabilized materials, by applying compaction as early as possible after mixing, and by reducing the possibility of reflection cracks[28].

2.3.5 Curing of Cement Treated Materials

Curing is a very important process that prevent loss of moisture from the stabilized materials. Cement treated materials have to be cured at least for the first seven days after they are placed. Curing has multidimensional advantages to the stabilized materials among which the followings are worth mentioning. Curing helps to:

- ensure that sufficient moisture is retained in the layer so that the stabilizer continue to hydrate;
- limit drying shrinkage while the hydration reaction is proceeding and the material is getting strong;
- reduce the risk of carbonation from the top of the stabilized layer.

There are different methods of curing of stabilized materials in the field. Among these, spraying of the water on the surface of the stabilized materials, and covering of the stabilized layer by water-tight medium are the most common methods.

If curing is to be carried out by spraying of water, it is important to cover the surface of the stabilized layer by 30 – 40mm thick sand so as to reduce leaching of the stabilizers. Water-tight covers, on the other hand, can be applied using bituminous materials, water-proof paper, or plastic sheets. In this method, very light spray of water is first applied on the surface of the stabilized layer and viscous cutback bitumen like MC-3000 or slow setting emulsions, or the plastic sheets are applied afterwards. There should be no traffic movement on the covered surface for seven days.

For laboratory specimens, curing is carried out by keeping the samples in a sealed, air-tight bags at a constant temperature[22,23,28].

2.4 Bitumen Stabilization

Soils are stabilized with bitumen to increase their strength and reduce the water absorption of the fine constituents of the soil. Granular soils or coarse grained soils can satisfactorily be stabilized with bitumen. However, there is some difficulty with plastic soils due to problems in mixing unless the soils are pretreated with lime or cement[31].

In non-cohesive granular materials the bitumen adds cohesive strength while in cohesive materials the bitumen waterproofs the soil and help to reduce the loss of strength with increase in moisture content. The mechanics of bitumen stabilization is the formation of bituminous films around the soil grains, which stick them together and prevent the absorption of water.

As bitumen is more expensive than cement and lime, and due attention is required to achieve satisfactory mix, its use as stabilizing agent has been limited[21].

2.4.1 Bituminous Materials

Asphalt cement, cutbacks, and emulsions can be used for soil stabilization. Medium curing cutbacks are the most frequently used soil stabilizers among the cutback bitumen. For sand mixes the properties of the cutback bitumen utilized will depend on the sand gradation, application temperature, and type of mixing plant. Binders of higher viscosity are recommended for sands with small amount of materials passing sieve No. 200 and for plant mixes, while binders of lower viscosity are preferable for soils with larger amount of fines and for mix-in-place methods[21]. As a rule of thumb, the most viscous liquid asphalt that can be readily mixed with the soil shall be utilized in the bituminous stabilization[10].

The suitable type of bituminous material for bitumen stabilization can be selected using the gradation of the soil to be stabilized. The different types of bitumen that are recommended for soils of specified gradation are listed in table 2.13. It is understood from this table that bitumen with lower viscosity are appropriate for soils containing considerable amounts of fines and vice versa.

2.4.2 Soil Requirements

Based on the granulometric composition and physical properties of the soil and the function of the bitumen added, the soil-bitumen mixtures commonly used in highway engineering are broadly classified into four types[21].

1. **Soil bitumen:** is a waterproofed cohesive soil system. The grain size distribution of the soil shall meet the grading requirements in table 2.14 and the maximum size of the soil should not exceed one-third of the compacted lift thickness. The amount of bitumen in this system ranges from 4-7 percent of the dry weight of the soil.

Table 2.13 Recommended Type of Bitumen based on Soil Gradation[11p(3-7)]

No	Soil Type (based on gradation)	Bitumen Type
1	Open-graded aggregate.	a. Rapid- and Medium-Curing liquid asphalt RC-250, RC-800, and MC-3000
		b. Medium-setting asphalt emulsion MS-2 and CMS-2.
2	Well-graded aggregate with little or material passing No. 200 sieve.	a. Rapid and Medium-Curing liquid asphalts RC-250, RC-800, MC-250, and MC-800.
		b. Slow-curing liquid asphalts SC-250 and SC-800.
		c. Medium-setting and slow-setting asphalt emulsions MS-2, CMS-2, SS-1, and CSS-1.
3	Aggregate with considerable percentage of fine aggregate and material passing No. 200 sieve.	a. Medium-curing liquid asphalt MC-250 and MC-800.
		b. Slow-curing liquid asphalts SC-250 and SC-800.
		c. Slow-setting asphalt emulsions SS-1, SS-01h, CSS-1, and CSS-1h.

2. **Sand bitumen:** Beach, dune, river, or pit sand which are free from vegetable matter, clay lumps and adherent films of clays can be used in this mix. Admixtures may be required to meet the stability requirements. The amount of materials passing No. 200 sieve should not exceed 12%. The bitumen content ranges from 4 – 10 percent and the optimum should be determined by compaction, strength, and water resistance and should not exceed the pore space of the compacted mineral mix.

Table 2.14 Soil Characteristics for Soil-Bitumen System[5 (p 72)]

Sieve number	% passing
No. 4	75
No. 40	35 – 100
No. 200	10 – 50
Liquid Limit	< 40%
Plastic Limit	< 15%

3. **Waterproofed clay concrete:** In this system a soil with good gradation and containing from coarse to fine particles, and with high potential density is waterproofed by adding 1 – 2 percent of bitumen.

4. **Oiled earth:** In this system, surface soil containing silt-clay material is made water and abrasion resistant using slow and medium curing bitumen cutbacks or emulsions.

2.5 Sand Asphalt Base

Bitumen stabilization of sand has been successfully implemented all over the world. In many tropical and arid area, sands stabilized with bitumen is usually provide good wearing surface[31]. In 1960, Road Research Laboratory of UK had built 64km experimental road in Northern Nigeria that extended from Maiduguri to Bama, and proved that the sand-bitumen mixture can serve as good base course material[30].

The Asphalt Institute Manual, MS-2[4], stated that sand, either manufactured or natural, with proper gradation can be used for a **base** or surface mixtures. Sand-asphalt mixes are recommended for light and medium trafficked roads only, as high stability values are not obtained due to the absence of large sized aggregate.

Road Note 31 of TRL[28], specified design criteria in table 2.15 for sand-bitumen roadbase mixes for tropical roads with medium to light traffic. According to this publication, the optimum bitumen content shall be determined using the Marshall Test procedures of the Asphalt Institute Manual Series No. 2, MS-2, 1988.

Table 2.15 Marshall Criteria for Sand-Bitumen Roadbase [ref 28]

	Traffic Classes	
	T1 (<300,000 esa)	**T2 (300,000 – 700,000 esa)**
Marshall stability at 60°C (min.)	1000N	1500N
Marshall flow value at 60°C (max.)	2.5mm	2mm

Similarly, the asphalt institute advisory[3] and model construction specification[5] had set criteria in table 2.16 for test limits on hot-mix sand asphalt base mixes.

Table 2.16 SS-1 Suggested Criteria for Test Limits on Hot-mix Sand Asphalt Base Mixes

Design Method	Minimum	Maximum
Marshall-Number of compaction blows, each end of specimen	50	
Stability, N	890	-
Flow, 0.25mm	-	20
Percent Air Voids	3	18
Hveem:		
Stabilometer	20	-
Percent Air Voids	3	18
Swell, mm	-	0.762

Both of the TRL's road note 31 and Asphalt institute's specifications were developed considering sand-asphalt base serves light or medium traffic volume. Sand-asphalt base is not recommened for road-base construction where heavy traffic load is expected.

2.6 Sand-Sulfur-Asphalt Mix

The inclusion of sulfur in a hot mix asphalt has a multifaceted benefits. The findings of laboratory investigations and full-scale paving trials indicated that sulfur imparts good workability, increased strength, and better fatigue life to the hot-mix-asphalt. It was suggested that sulfur can be used to produce high quality bituminous mixes from low quality aggregates including single sized sand[7,20,24].

2.6.1 The Role of Sulfur in Sand-Sulfur-Bitumen Mixes

Addition of sufficient amount of molten sulfur to the hot sand-bitumen mixture increases the fluidity of the mixture thereby decreasing the compaction/consolidation effort, required to keep the mix in-place. When the sand-sulfur-bitumen mixture cools, the sulfur solidifies within the void spaces of the bitumen-coated sand grains as shown, white-band, in fig 2.6 and fig 2.7. Accordingly, the sulfur crystals reduces the air-void in the mixture and interlock the sand-grains. As a result, the sand-sulfur-bitumen mixes possess improved mechanical stability and strength[7].

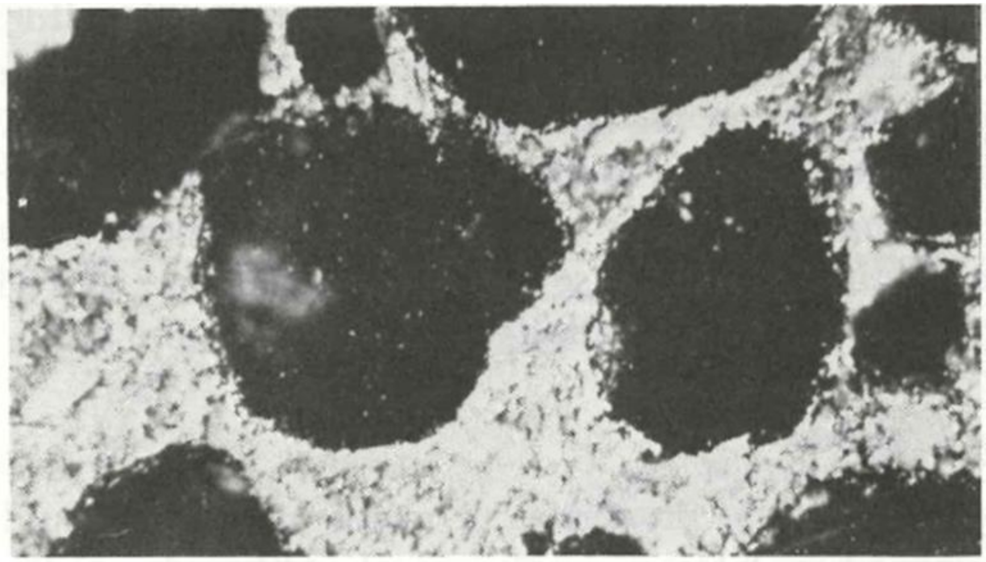


Figure 2.6 Photomicrograph of a polished sand-sulfur-bitumen mix surface

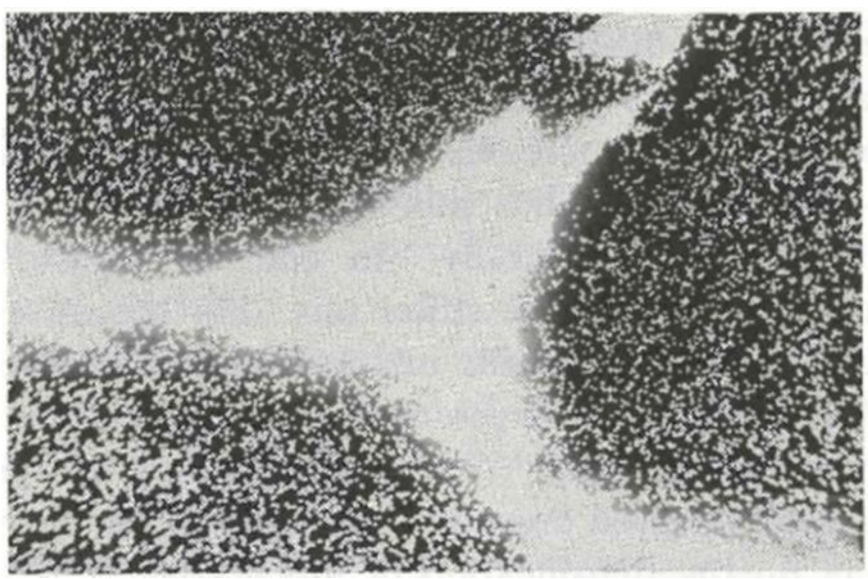


Figure 2.7 X-ray analyzer plot of a sand-sulfur-bitumen mix

2.6.2 Properties of Sand-Sulfur-Bitumen Mixes

A number of studies had been carried out in sand-sulfur-bitumen mixes in 1970's and 1980's. In these studies the mechanical properties of the SSB mixes were identified and comparison was made with the respective properties of the conventional asphalt concrete mixes. The mechanical characteristics that had been studied include marshall stability, tensile strength, flexural strength, fatigue behaviour, and creep/deformation under static and dynamic loading condition. The findings of some of the researches are summarised and briefly presented subsequently.

At higher sulfur content, the hot sand-sulfur-bitumen mixes become fluid and attain maximum density without due compactive energy. At lower sulfur contents, however, the mixture is stiffer and require considerable compaction to expel the air from the loose mix. The density of the stiffer mixes increases with compaction energy[7]. The effect of compaction on the sand-sulfur-bitumen mixes is shown in fig 2.8.

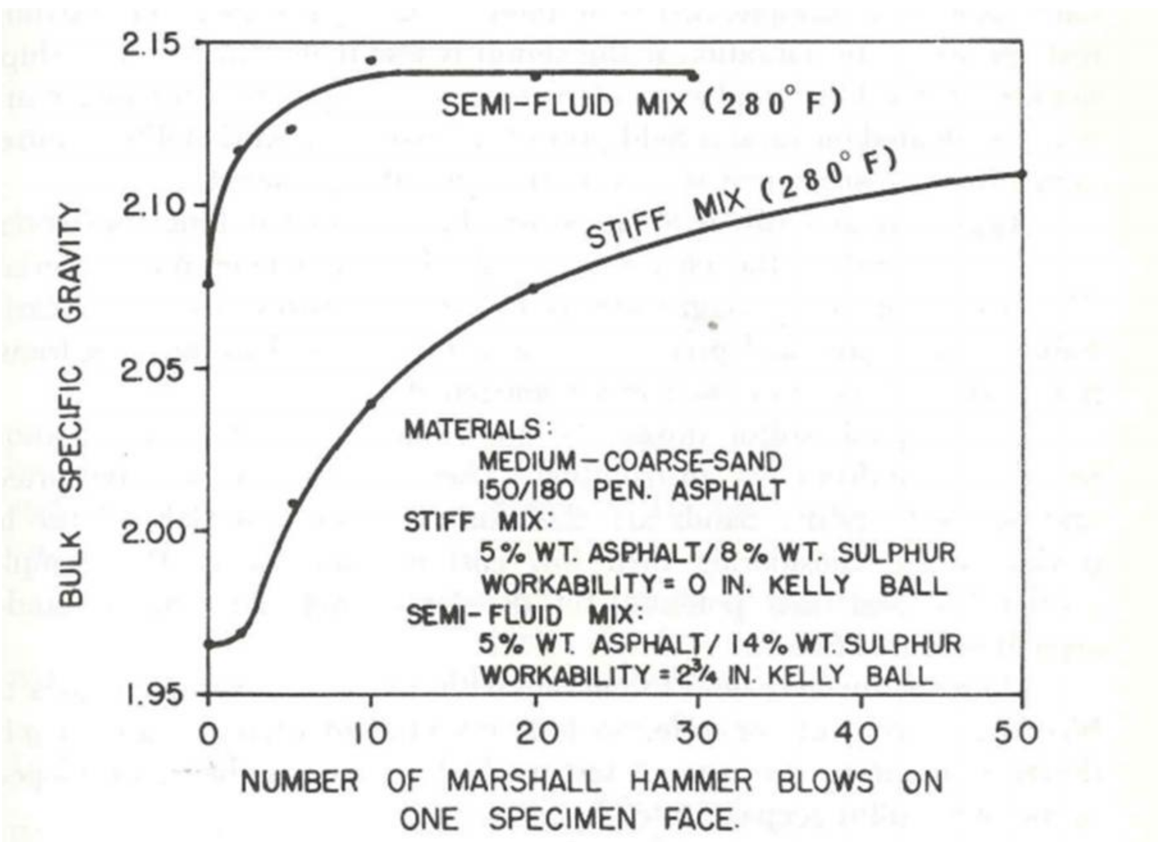


Figure 2.8 Effects of compaction on semi-fluid mix and stiff mix densities [ref 7]

The Marshall stability tests that had been undertaken according to ASTM D 1559 indicated that the stability of the sand-sulfur-asphalt mixes increase to a peak value with increasing sulfur content for all bitumen contents. The test results also shown that the stability values decrease with increasing bitumen content[7]. In most of these tests, the marshall specimens were compacted by one or two blows of marshall hammer in one face. In SSB mixes, the maximum stability depends on the type of sand, viscosity of bitumen, and the mixing sequences. The maximum stability of the SSB specimens vary from 13kN to 32kN. These stabilities were obtained at the correponding optimum sulfur contents ranging from 13 – 20% and optimum binder content ranging from 4 – 6%[20]. Plot of stability values at different bitumen and sulfur contents is presented in fig. 2.9; and summary of the marshall stability, optimum sulfur content and optimum binder content of various SSB mixes is shown in table 2.17.

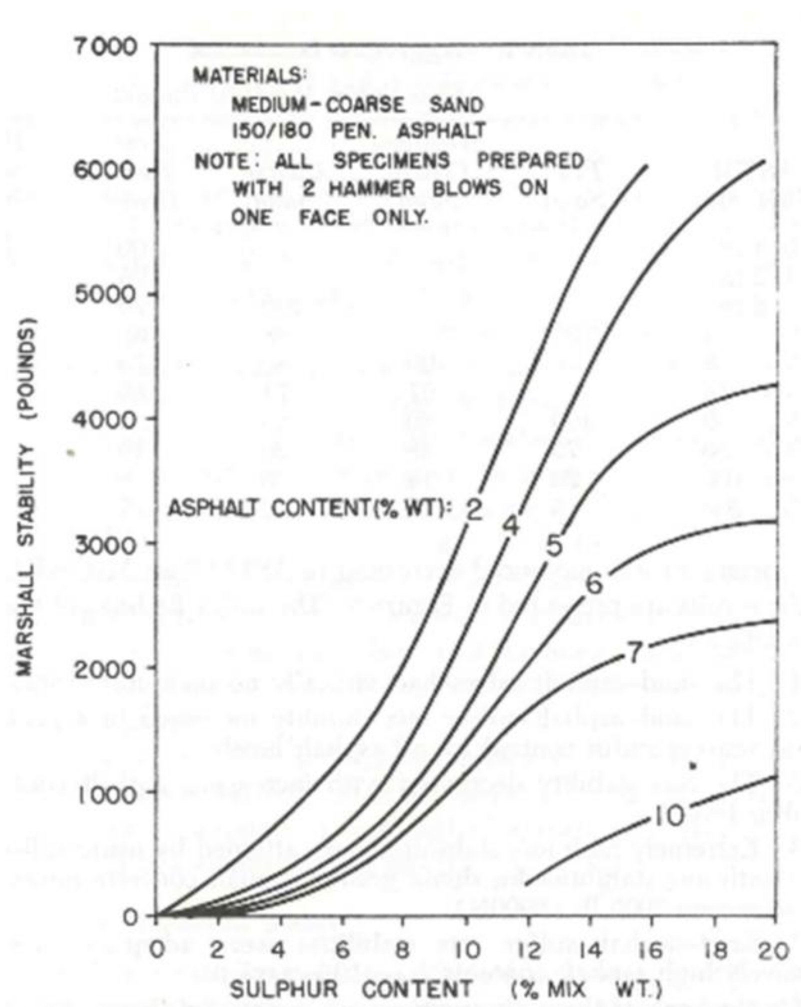


Figure 2.9 Effects of sulfur content on mix stability

Table 2.17 Summary of Laboratory Studies of SSB Mixes (Khodaii, et al. 1986)

	Author					
	Burgess and Deme (1975)	Saylak et al. (1975)	Sullivan et al (1975)	Glascock (1976)	Fatani (1980)	Aboaziza (1981)
Bitumen Content, %	2 - 10	0 - 6	various	various	3 - 7	0 – 8
Sulfur content, %	0 - 20	10 - 20	various	10 - 16	5 - 20	5 – 20
Bitumen grade, pen	150-180	105	i. 85-100 ii.60-70	80	60-70	60-70
Sand Type	Fine, medium, and coarse	Uniform graded	Dune sand and well graded	Uniform sand	Dune sand	Dune sand
Mixing Sequence	Regular	Regular	Simultaneous	Regular	Regular Simultaneous Reverse	Simultaneous
Marshall Stability, kN	>13	>25	i. 18, ii. 19	15	25-32	20-31
Optimum Sulfur, %	14 (fatigue)	18 (Hveem)	i.13.5, ii.17.0	15	15	15-20
Optimum bitumen, %	6	4,6,7	6	5-7	5	4-6

Note: i. coarse sand, ii. Wind blown sand

The flexibility and fatigue property of SSB mixes had been assessed using a three-point bending apparatus. The test was conducted on costant stress mode in a mixture prepared by 6% bitumen content and varying amounts of sulfur. The results indicated that the fatigue life of the sand-sulfur-bitumen increases with sulfur content and then decreases. The relation between sulfur content and fatigue life is shown in fig 2.10[7]. According to fig 2.10, the mixtures with sulfur content of 14% had exhibited longer fatigue life. The fatigue test, conducted (by Khodaii 1986) on cylinderical specimens under controlled axial stress condition had shown that the fatigue life of the SSB mixture is better than the fatigue life of the conventional mixes. The SSB mixes, in this case, were composed of 80% sand, 15% sulfur, and 5% bitumen. According to this research the strain-fatigue life relationship of the SSB mix was found to be

$$N = 2.52 * 10^{-8} * e_t^{-3.142}$$

where N= the number of load repetitions to failure

e_t = the initial tensile strain

The relationship that had been developed for asphalt concrete using similar test method was found to be

$$N = 1.8 * 10^{-8} * e_t^{-3.077}$$

Comparing the two relations, it can be understood that the fatigue behavior of the SSB mixes is better that that of the conventional asphalt concrete. The fatigue behaviour of the SSB mixtures and the comparison of the fatigue behaviour of SSB mixtures with other types of mixes are shown in fig 2.11 and fig 2.12 respectively.

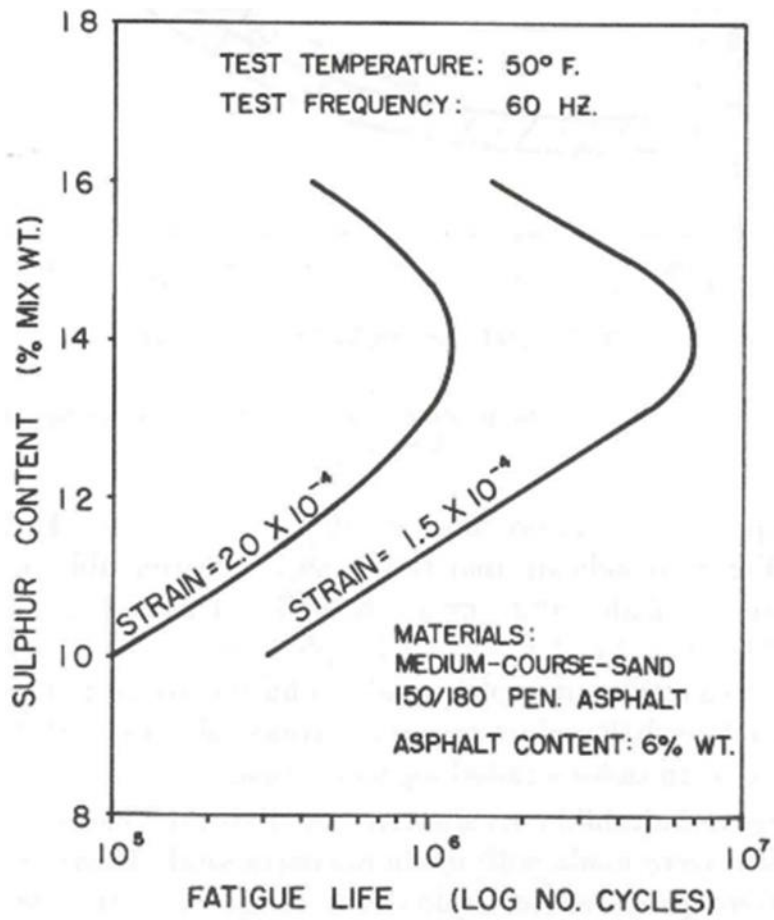


Figure 2.10 Relation between sulfur content and fatigue life

The permanent deformation of sand-sulfur-bitumen mixes under dynamic loading condition is much lower than that of the conventional DBM mixes. This was shown by the study of Khodaii (1986) and Snaith (1973) as depicted in fig 2.13. According to these studies, the axial permanent strains of the SSB mixes are smaller and gradual as compared to those of DBM.

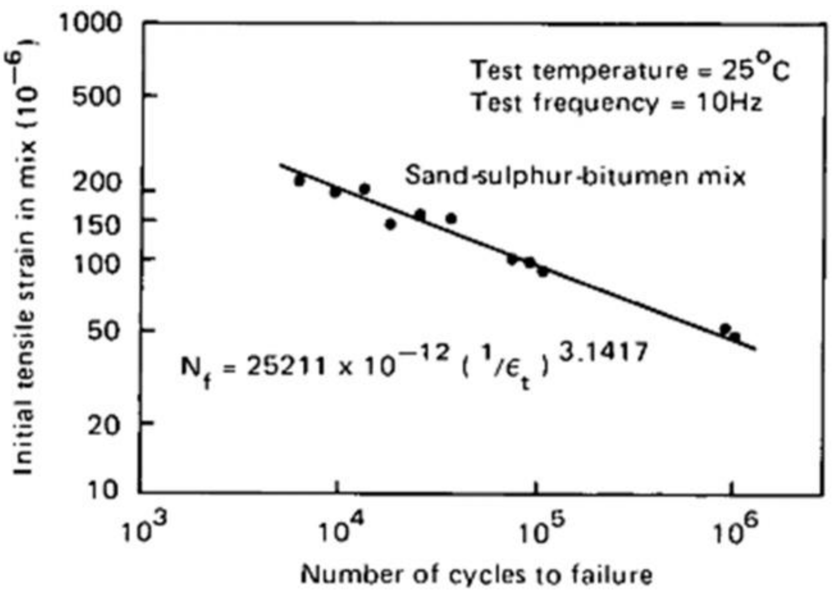


Figure 2.11 Tensile strain-fatigue life relationship for the dune sand/sulfur/bitumen mix

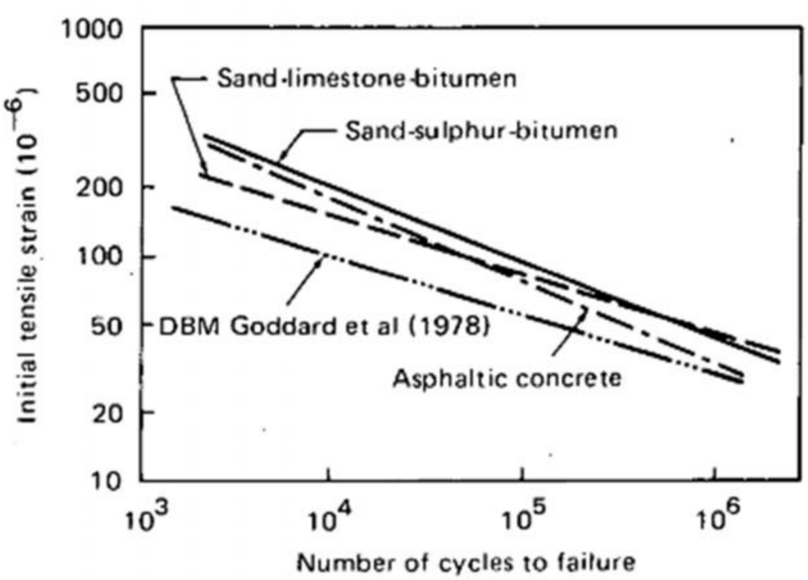


Figure 2.12 Comparison of fatigue behaviour of sand/sulfur/bitumen with SLB, AC, and DBM mixes

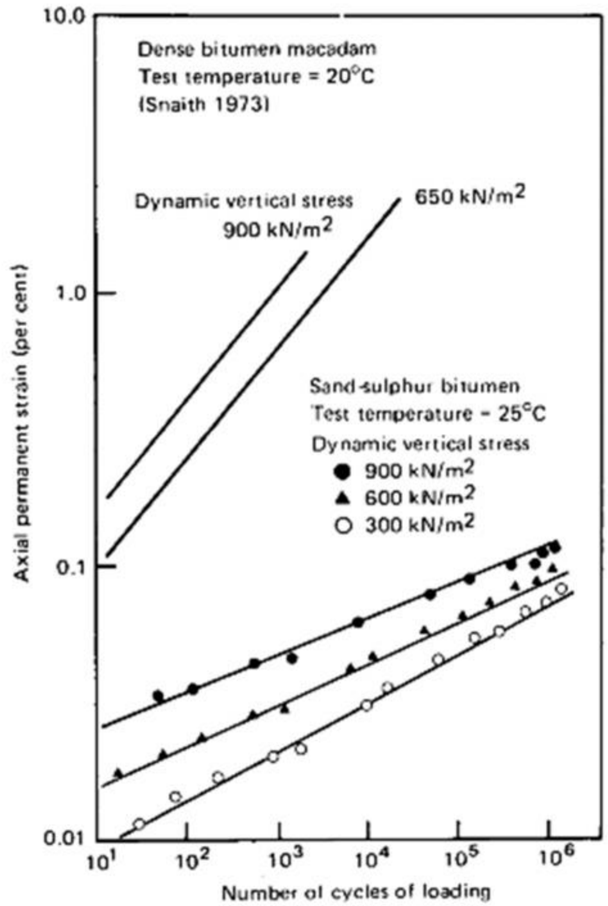


Figure 2.13 Comparison of permanent strains in DBM and SSB under different dynamic vertical stress

In sand-sulfur-bitumen mixes the requirements for workability, stability, and other mechanical properties are fulfilled before the voids in the sand are completely filled by the binders. Accordingly, the air-void content of the compacted mixture is often above 10%. However, the air-voids are not interconnected as the sulfur network isolate them. As a result, the compacted mixture is not readily permeable despite the high air-void. This was proved by the permeability tests, conducted using constant head air permeameter. According to this test, the coefficient of permeability of the sand-sulfur-bitumen mix with 16% air-void is 10^{-8}cm/s. The coefficient of permeability of 10^{-8}cm/s is considered as the threshold below which the mixture is considered impermeable.

Thus the high air-void content in sand-sulfur-bitumen mixes does not create the problem it will create in the conventional hot-mix-asphalt. The plot of the permeability test is shown in fig 2.14.

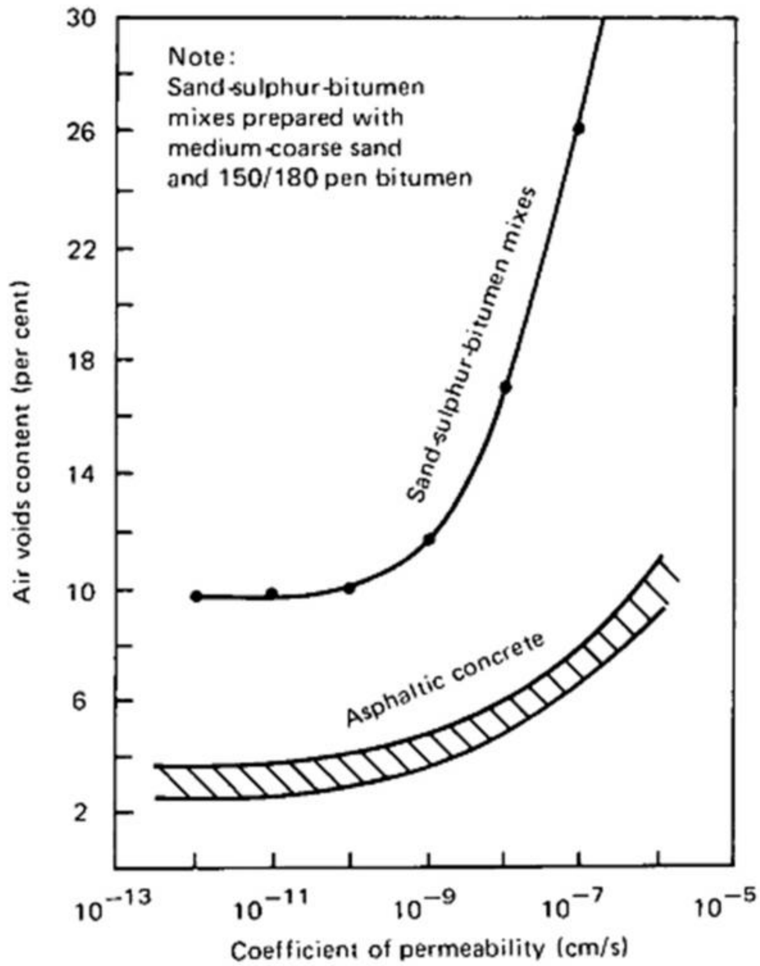


Figure 2.14 Relation between sulfur content and permeability [ref 20]

2.6.3 Effects of Aging on Sand-Sulfur-Bitumen Mixes

The effects of aging on the SSB mixes had been investigated so as to assess the long term performance of the mixtures. In the research by Rolt et al (1999) the influences of aging and weathering on the mechanical properties of the SSB mixtures were studied. The SSB specimens, in this research, were prepared using the same materials that had been used by Khodaii (1986) to evaluate the mechanical properties of fresh SSB mixes. Three types of SSB mixtures were prepared by varying the proportions of sulfur and bitumen while keeping the sand proportion to a constant value of 80% by weight of the total mix. This was done so as to observe the influence of the sulfur/bitumen ratio on the mechanical properties of the aged and weathered mixes. Further, to identify and quantify the specific effects of sulfur on the sand/bitumen mixtures, control specimens were prepared by replacing the sulfur in the mixes with equal volume of limestone. The proportions of the three SSB mixes and the control mixtures are shown in table 2.18.

Table 2.18 Composition in percentage by weight of the SSB Specimens [ref 24]

SSB Mix	Sand	Sulfur	Bitumen	Proportion of Lime in SLB mix
A	80	13	7	17.9
B	80	15	5	20.7
C	80	17	3	23.4

The SSB mixes were prepared by heating each components of the mixes to 145^{o}C and mixing them following the regular sequence, in which the sand and bitumen are mixed first and then mixed together with the molten sulfur. The total mixture was then placed in a 300mm square and 204mm deep mould and compacted on one face with 12 blows of a purpose designed Marshall hammer.

In order to identify the time at which the compacted SSB mixture attain its maximum stability, core had been cut from the compacted specimens on the 1st, 3rd, 6th, 28th, and 56th days and subjected Marshall stability test. The test results indicated that the Marshall stability of the **compacted mixes gradually increases up to 28th days** and indicates negligible increment afterwards as shown in fig 2.15. Therefore, the compacted specimens were cured for 28 days before being exposed to weathering.

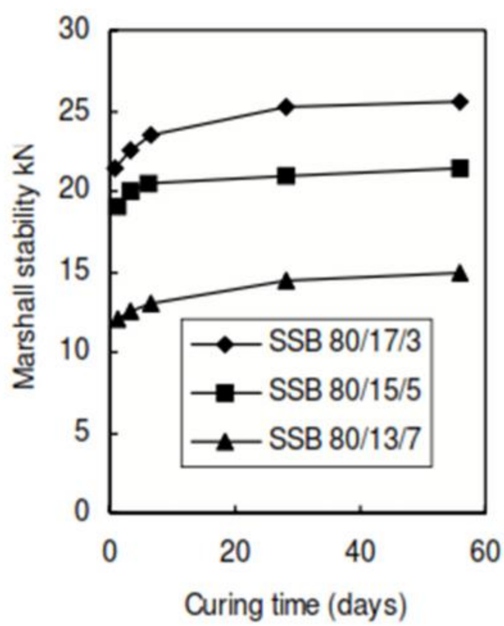


Figure 2.15 Effect of curing time on Marshall stability of SSB specimens

The specimens that had been cured for 28 days were then made to weather in a purpose designed container for 6, 12, 18, 24, and 30 months and subjected to respective tests to identify the change in the mechanical properties[24].

Marshall stability test was conducted on the samples that had been cut from the top (surface) and middle (subsurface) part of the compacted specimens. The test results revealed that both of the SSB and SLB mixes exhibit very little, if any, variation in the stability values over the 30 months weathering period. The only exception is the 80:15:5 SSB specimens in which the Marshall stability kept on gradual increment throughout the weathering period. Here it is worth noting that the Marshall stability of SLB specimens is 12-20% of the Marshall stability of the SSB specimens. Alike the Marshall stability, the flow of the SSB specimens also show very small difference throughout the weathering period. The effect of weathering on the Marshall stability and flow of the SSB and SLB mixes is shown in fig 2.16 and 2.17.

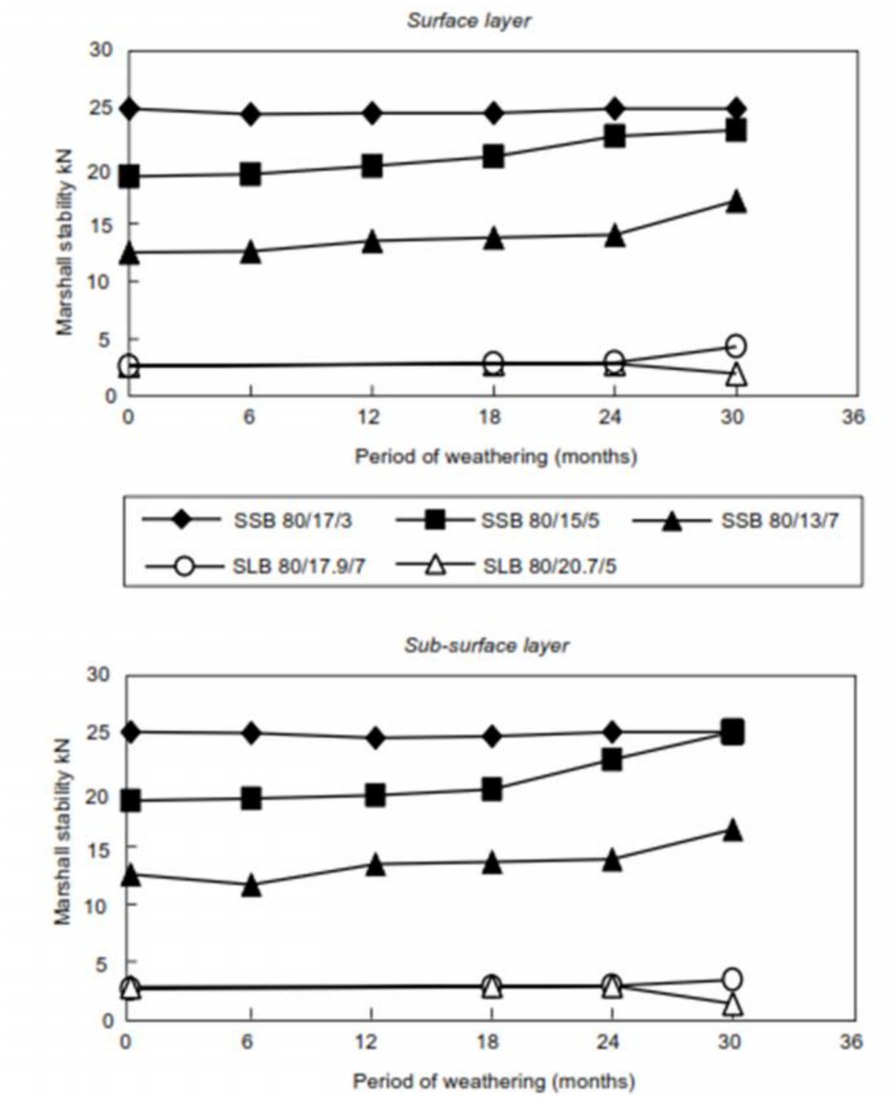


Figure 2.16 Effect of weathering on Marshall stability of SSB and SLB specimens

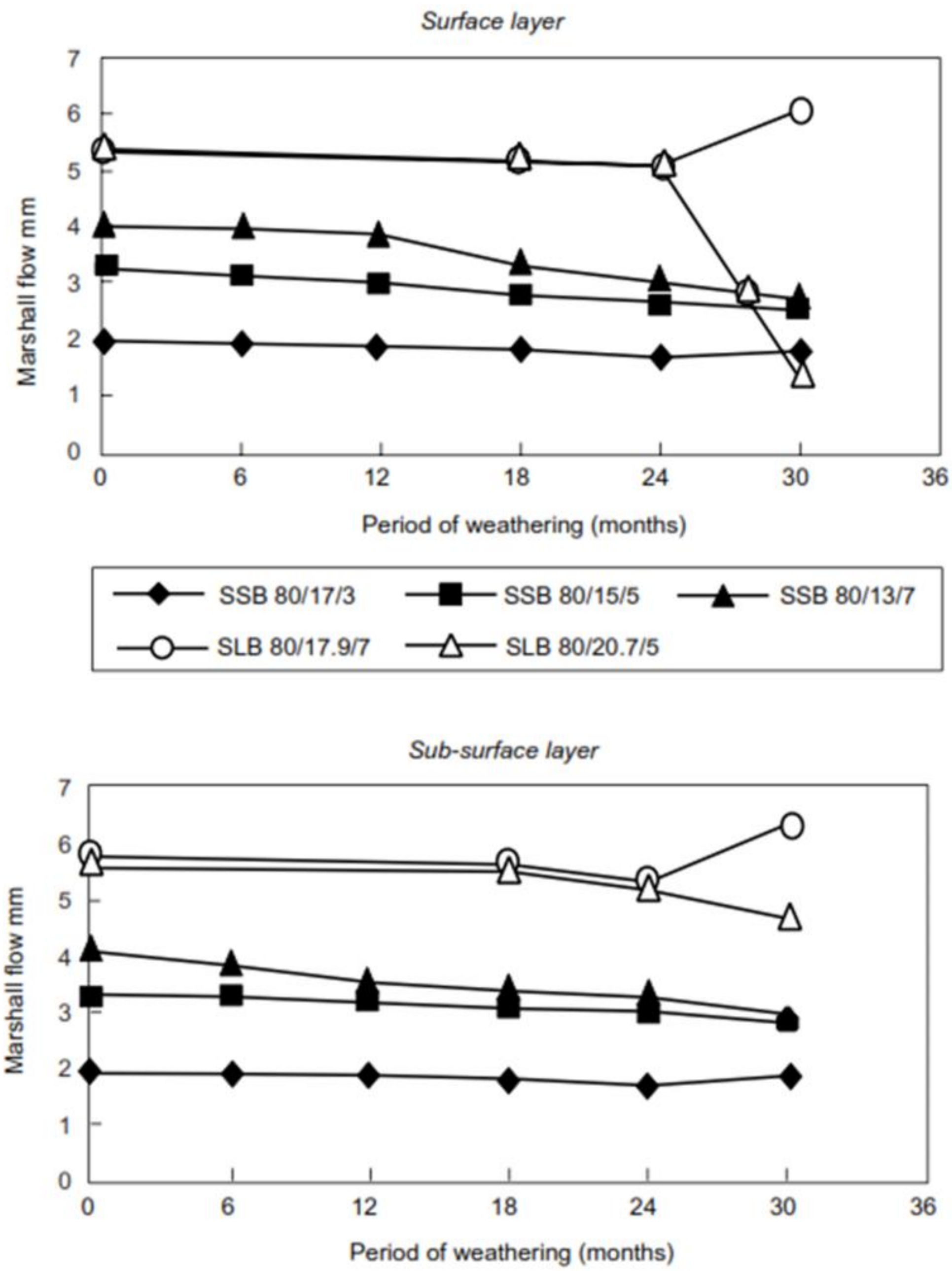


Figure 2.17 Effect of weathering on Marshall flow of SSB and SLB mixes

Tensile strength tests had been conducted using the double punch test method. The test results indicated that the change in the tensile strength of the SSB mixes is dependent on the amount of sulfur in the mixes. According to this test, the tensile strength of the 85:15:5 SSB mix increases by about 40% over the weathering period of 30 months. The control SLB specimens have very low tensile strength which was not significantly changed over the weathering period. The results of the tensile strength tests on the weathered specimens are shown in fig 2.18.

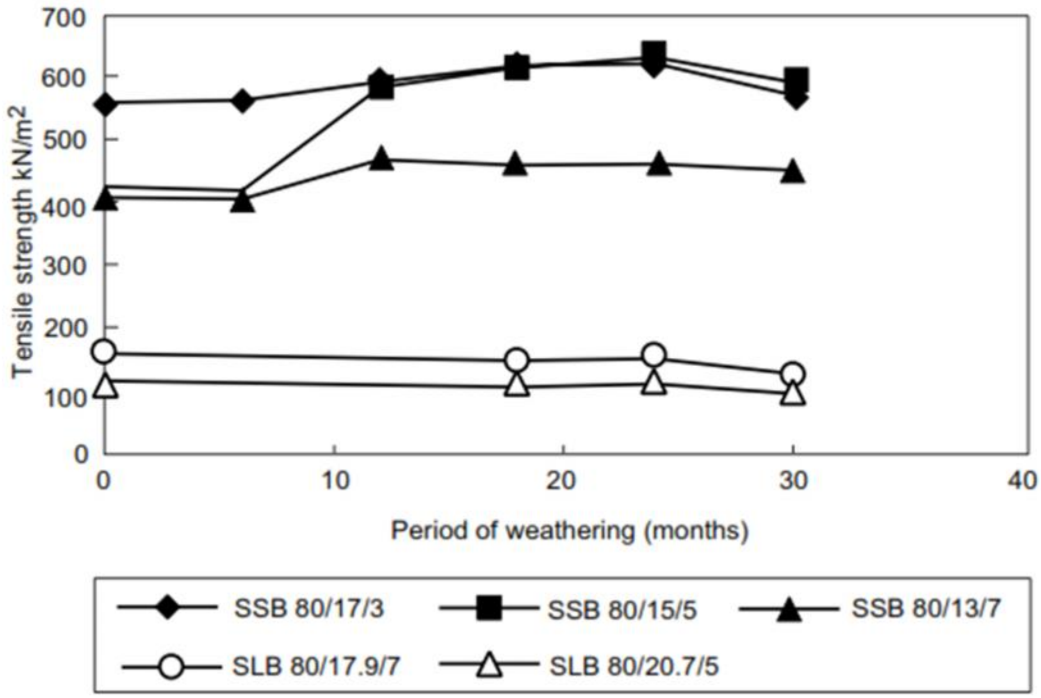


Figure 2.18 Effect of weathering on the tensile strength of SSB and SLB specimens (test temperature = 25°C)

The results of flexural strength test had revealed that the SSB specimens have much superior flexural strength than that of SLB specimens. The flexural strength of the SSB specimens increase to a peak during 18 – 24 months of weathering period. SSB 80:15:5 mix showed 10% increase in the flexural strength after 24 months of weathering. The flexural strength test was carried out according to ASTM D 1635-63 using simply supported beams with three point loading at a deformatiion rate of 1.2mm/minute. The results of the flexural strength test is shown in fig 2.19.

The fatigue test that had been undertaken under controlled stress condition indicated that the fatigue properties of the SSB mixes exhibited little change over the 30 months weathering period with the 80:15:5 SSB mix showing the best fatigue properties. The dynamic modulus of the 80:15:5 SSB mix increased continuously within the weathering period while the moduli of the other two SSB mixes stop increasing after 18 months of weathering. The SLB specimens, on the other hand, had dynamic moduli which were 5% or less of the equivalent SSB mixes. The plot of the fatigue life versus initial strain of the specimens is shown in fig 2.20 and the effect of weathering on the dynamic modulus is shown in fig 2.21.

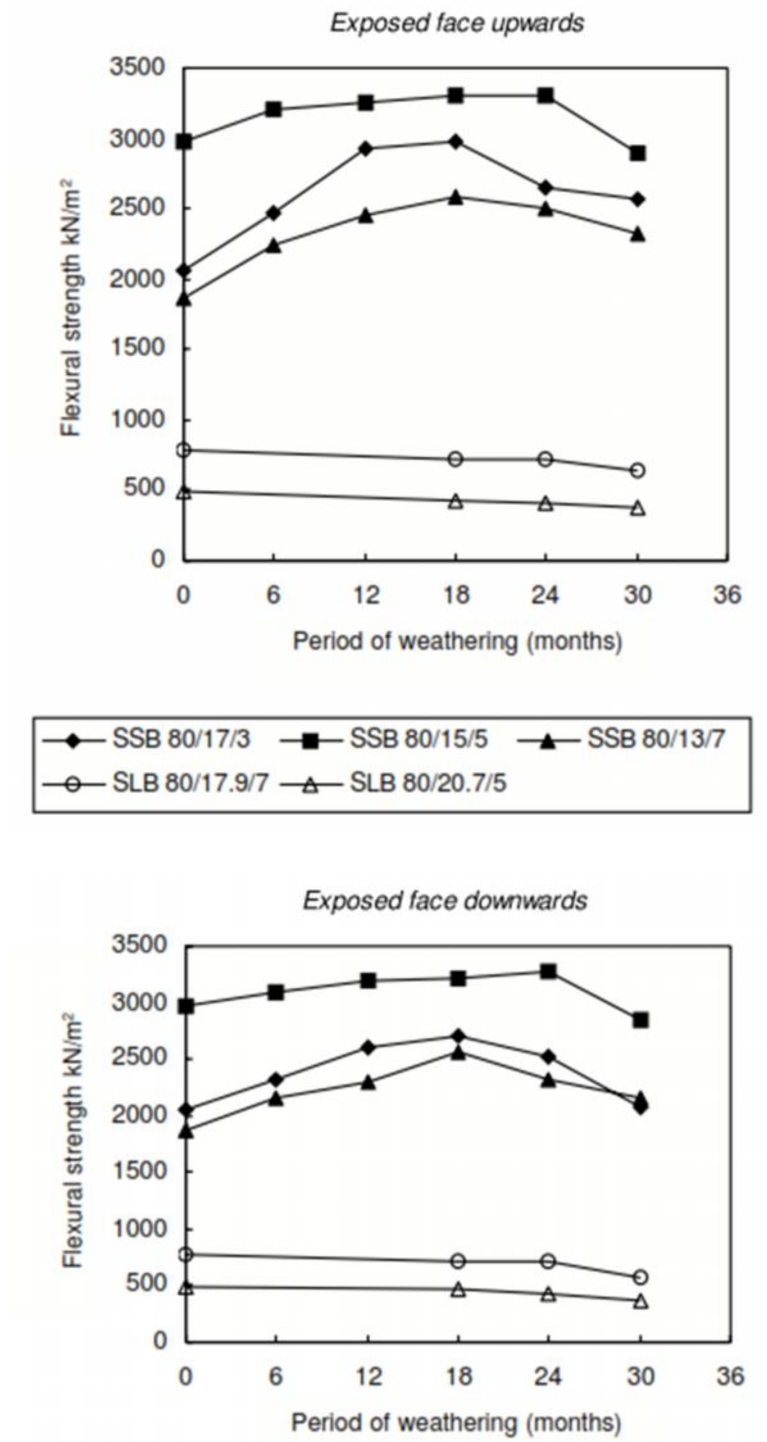


Figure 2.19 Effect of weathering on the flexural strength of SSB and SLB specimens (test temperature 25^{o}C)

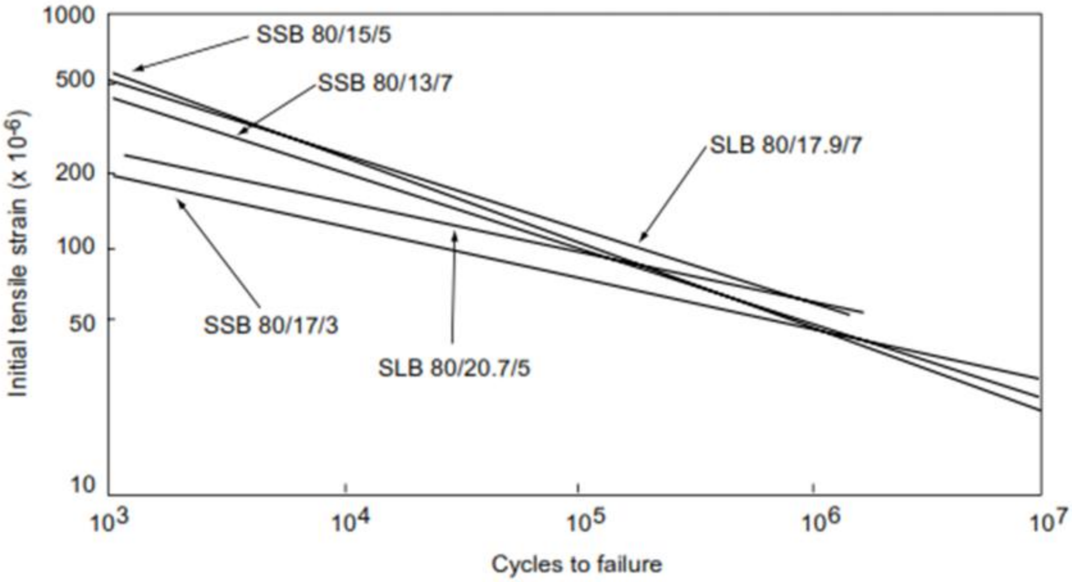


Figure 2.20 Tensile strain - fatigue life relationship for SSB and SLB specimens (period of weathering = 24 months, laboratory test temperature = 25°C, loading frequencey = 10HZ)

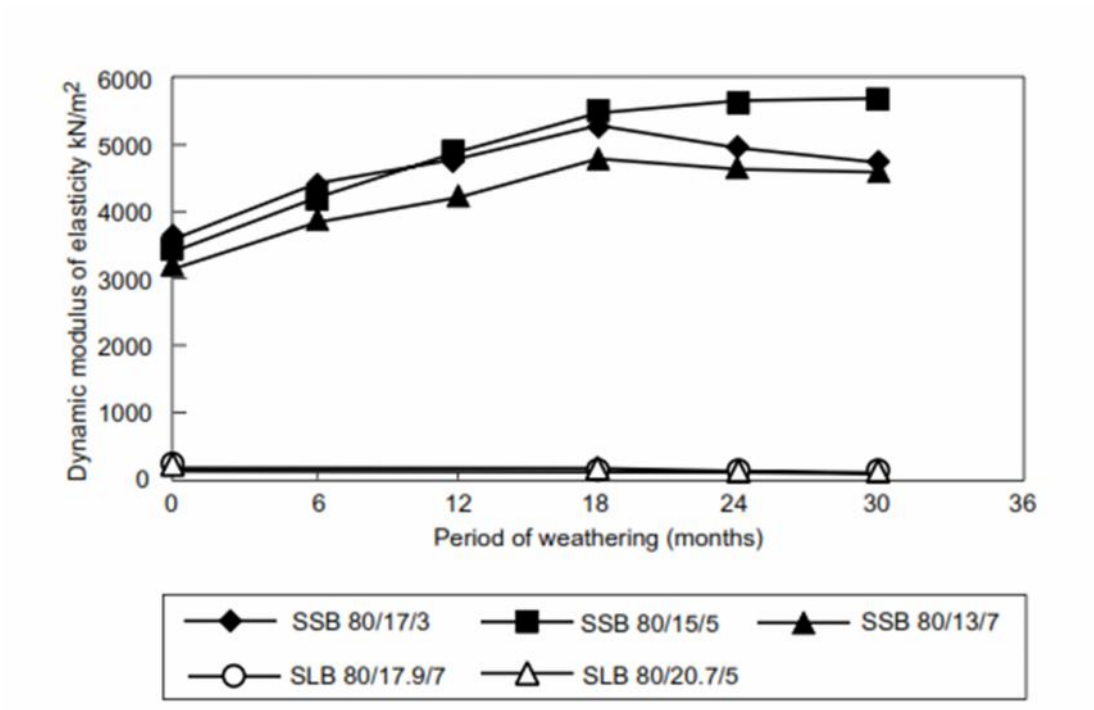


Figure 2.21 Effect of weathering on the dynamic modulus of elasticity of SSB and SLB mixes (testing temperature = 20°C, loading frequency = 10HZ)

In order to determine the influence of weathering on the physical propteris of bitumen, free bitumen from each of the mixes were recovered using a rotary evaporator of the type described in IP 105/BS 598. The viscosity of the bitumen that had been recovered from different depths of the samples was measured using microviscometer at 60^oC and a shear rate of 0.05 sec^{-1}. The results of the viscosity tests indicated that the viscosity of the recovered bitumen depends on th sulfur-to-bitumen ratio in the mix: the higher the ratio, the harder was the recovered bitumen. The cause of hardening of the recovered bitumen was believed to be the chemical reaction of sulfur with bitumen. The viscosity of bitumen that was recovered from the first layer was found to be higher than that recovered from the second and third layers in a similar fashion as the bitumen-aggregate mixture. On the other hand, the viscosity of the bitumen that was recovered from all of the SSB mixes, with the exception of the 80:13:7 SSB mix, is harder than that from SLB mixes. These phenomena enforces the premise that the hardness of the bitumen in the SSB mixes is attributed to the reaction of sulfur with bitumen. The results of the viscosity tests are shown in fig 2.22.

The permeability of the weathered specimens were measured using the falling head method with a modified cell that employ a seal to ensure a tight fit. The test results revealed that the permeability of the specimens was not significantly affected by weathering. The mean coefficient of permeability of the specimens over the weathering period is shown in table 2.19.

Table 2.19 Coefficient of permeability using the falling head test [ref 24]

Mix designation	Voids in the Mix (%)	Coefficient of permeability (m/s), mean value over the weathering period
SSB (80:17:3)	14	$6.5*10^{-9}$
SSB (80:15:5)	13	$3.2*10^{-9}$
SSB (80:13:7)	15	$9.3*10^{-10}$
SLB (80:17.9:7)	22	$5.5*10^{-7}$
SLB (80:20.7:5)	28	$7.7*10^{-7}$

2.6.1 Safety in Using Sulfur

Studies that had been conducted to assess the safety issues while using sulfur in the road construction industry pointed out that the detrimental effects of sulfur can be mitigated provided that appropriate precautionary measures were taken[24]. Some of the recommended precautionary measures are mentioned as follows:

- By keeping the mixing temperature of the SSB mixes below 160^oC, the concentration of all gaseous pollutants can be kept below the allowable threshold limits;

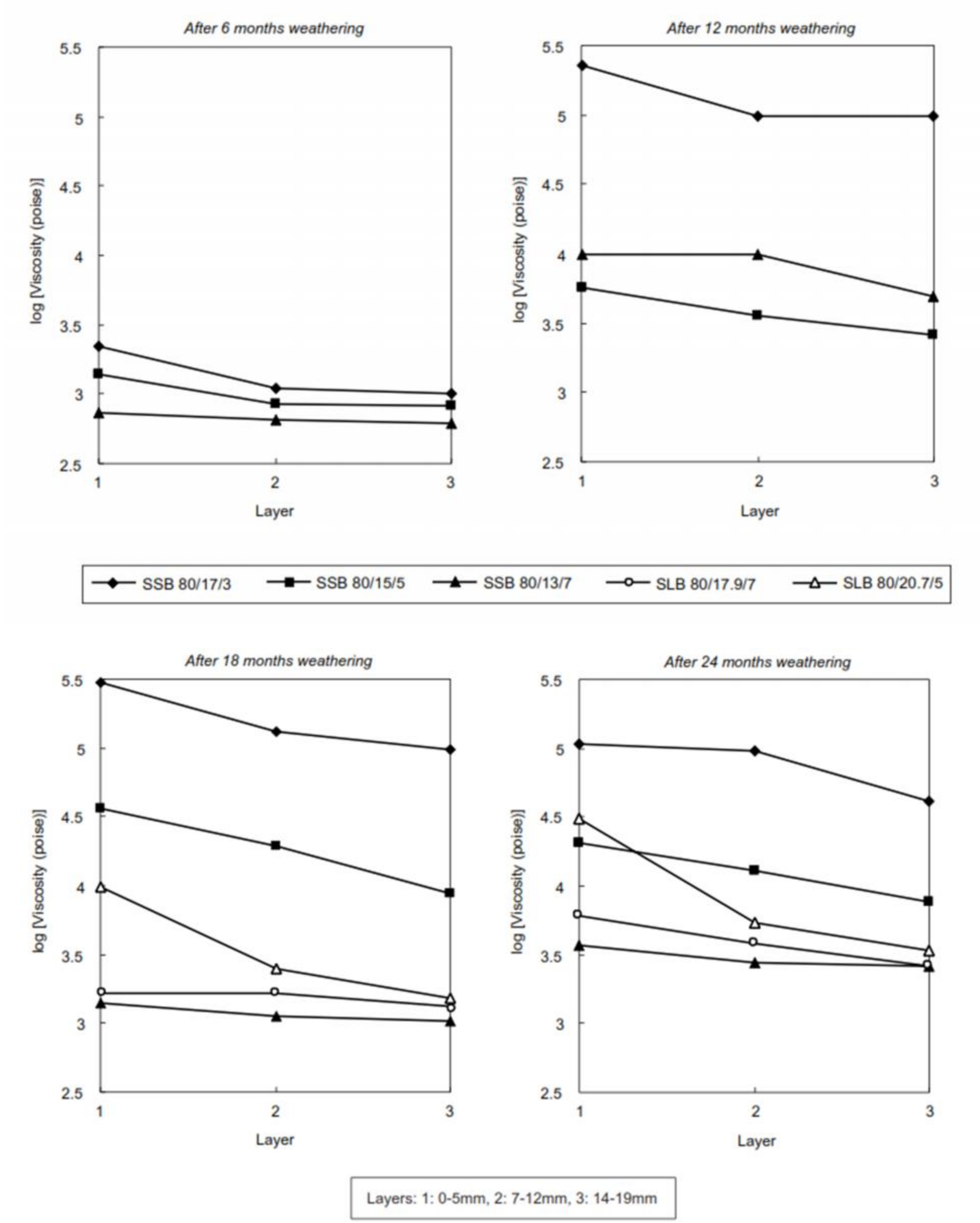


Figure 2.22 Effects of weathering on viscosity at various depths in SSB and SLB specimens (test temperature = 60oC)

- The emission of hydrogen sulfied can be controlled by using additives like cupric oxide, calcium carbonate, activated carbon, lime, sodium carbonate, aluminium oxide, and calcium chloride.

- Overnight or longer storages of SSB mixes shall be avoided so as to prevent accumulation of hydrogen sulfied and subsequent explosion.

In general it can be understood from the studies of the mechanical behaviours and physical properties of SSB mixes that a well designed sand-sulfur-bitumen mixtures can perform as good as, if not better than, the conventional asphalt concrete mixtures provided that they (SSB mixes) are found economical viable. Furthermore, the problems associated with the inclusion of hazardous chemicals like sulfur can be avoided by adopting proper safety regulations and taking precautionary measures.

3 RESEARCH METHODS, MATERIALS, AND PROCEDURES

3.1 Study Area

The study area is found in western Ethiopia, Gambella regional state. The topography of this region is predominantly flat with altitude ranging from 405m – 435m above mean sea level. According to the geological map of Ethiopia, 1996 ed., the geology of the area is mainly composed of alluvial and lacustrine deposits.

This study was done on the natural sand that had been collected from the spots along **Adura – Burbey** road which is still under construction. This segment was selected because of the following reasons:

1. gravel and rock are very scarce along the stretch;
2. the stretch can be accessed easily;
3. excavation machines can be obtained from the on-going road projects; and
4. it is possible to find vehicles for shipment of the collected samples.

The project beginning village, Adura (E559877,921387), is located at 137km from the regional capital Gambella town whereas the project end village Burbey (E-525577,N-930599) is located at the Ethio-Sudan border 44.1km from Adura. Both of the controls are found within zone 36 of the UTM coordinate system.

Along the Gambella – Adura – Burbey road segment there is only one fresh rock and one natural gravel sources, which are located at 8km and 47.5km from Gambella town respectively. The road traverses vast flat land that is covered with black cotton soil. The black cotton soil is underlain by natural sand deposit as shown in fig 3.4. It was observed that the sand deposit is found at depths of 0.5m – 3.0m from the surface of the natural ground.

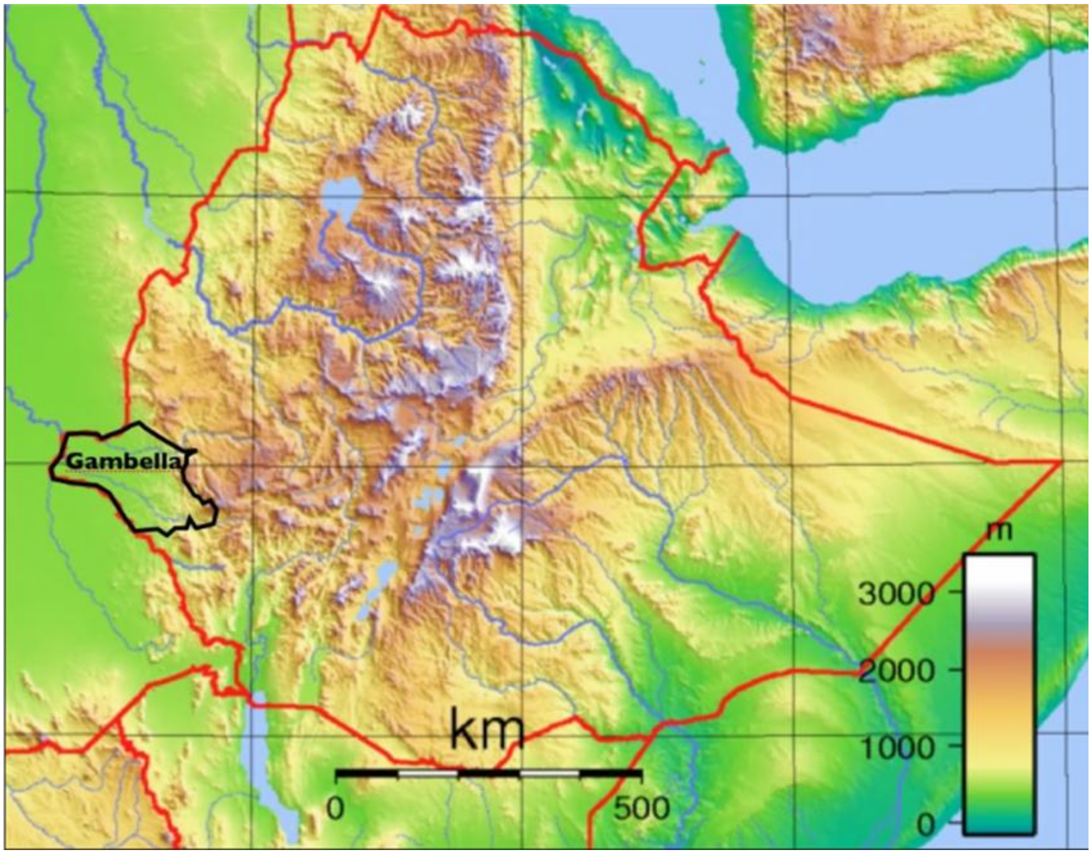


Figure 3.1 Location and topography of Gambella region, (www.google.com/imgres, 28th April, 2013)

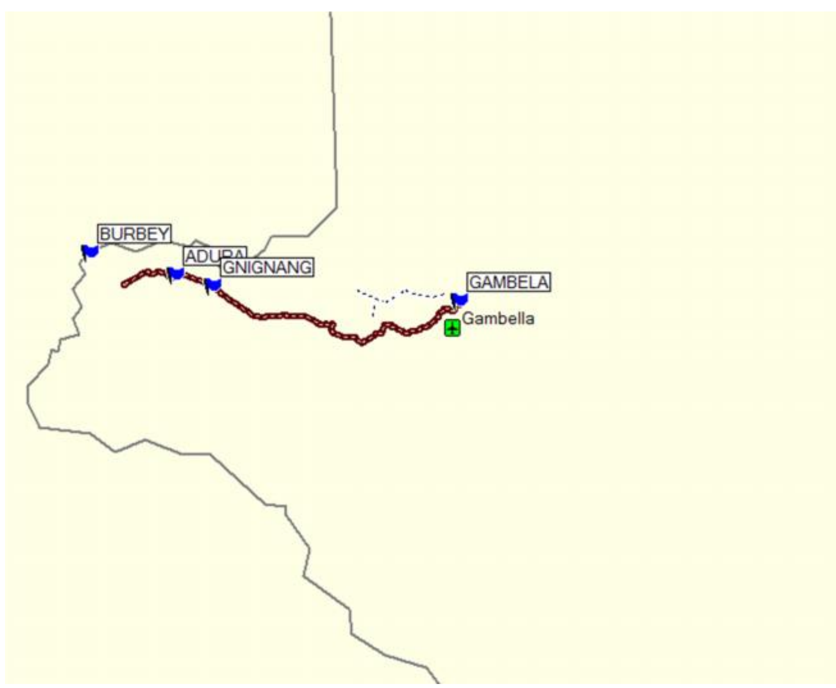


Figure 3.2 Hand-held GPS track of Gambella-Adura-Burbey Road

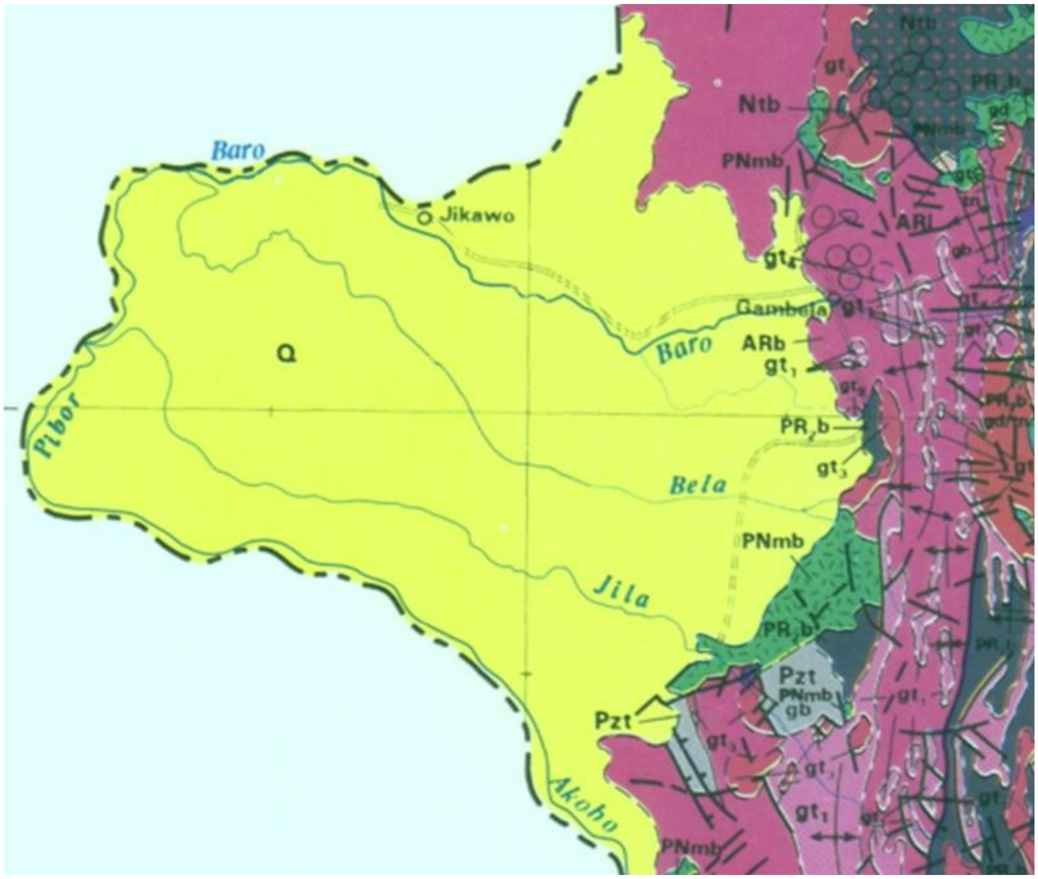


Figure 3.3 Homogenous geological formation of Gambella (Alluvial and Lacustrine Deposit) (Geological Map of Ethiopia, 1996 ed.)

Figure 3.4 Sand deposit underneath the black cotton soil

3.2 Study Design

This research was designed to answer the research questions and meet its objectives based on experimental findings.

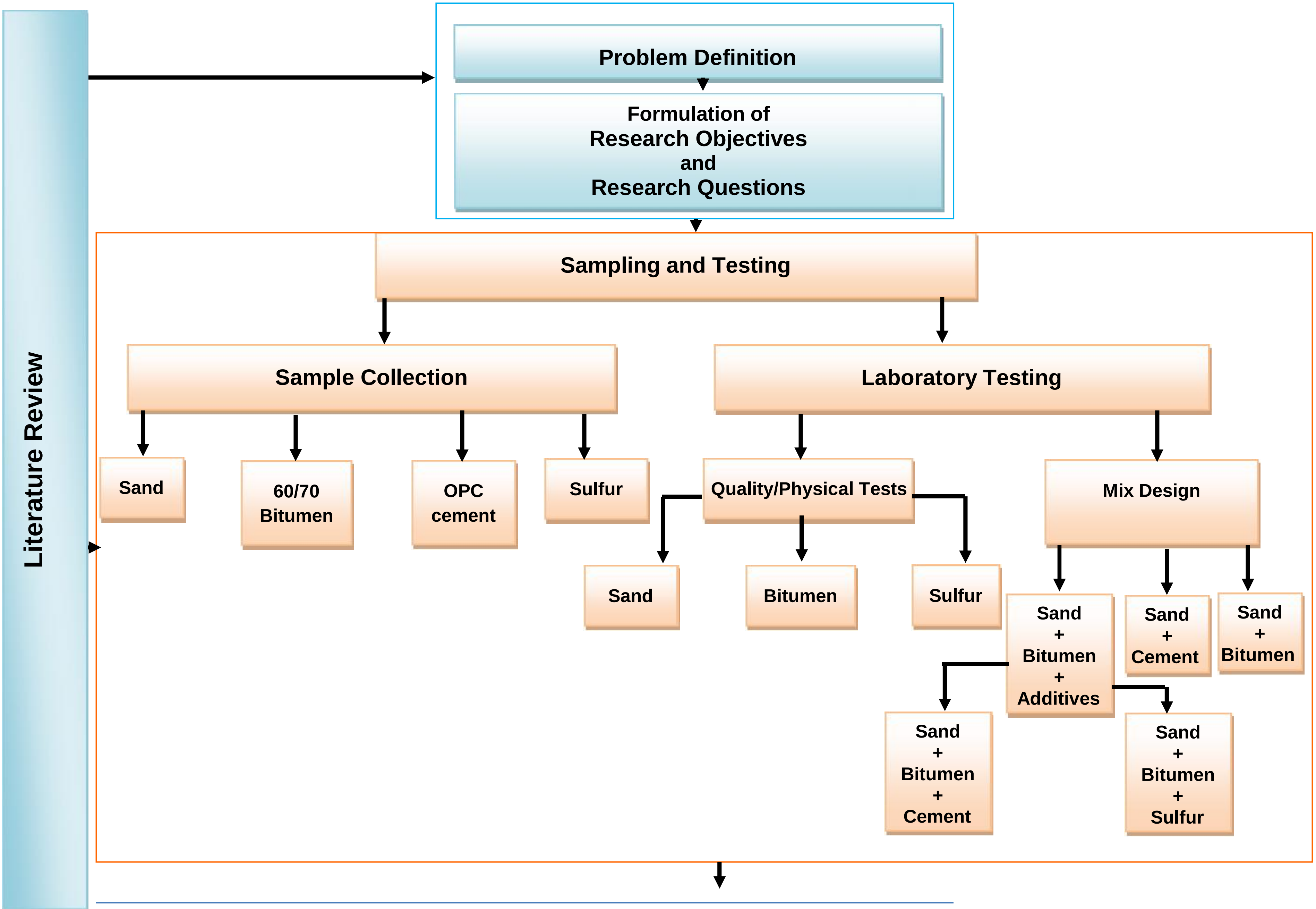
The first step in the research work was sample collection. At this stage the samples of the component materials viz. sand, bitumen, ordinary portland cement, and sulfur were collected.

The second step was laboratory testing. This strep is comprised of two major phases: Phase I-Quality testings and Phase II-Mix design. Qualilty tests were undertaken on each of the component materials so that their physical and/or chemical properties are identified. During the mix design phase, four types of mixtures were designed and their properties were assessed to check whether they are suitable as road-base materials or not. Three of the four mixtures were bituminous mixtures that were composed of sand and bitumen; sand, bitumen, and ordinary portland cement; sand, bitumen, and sulfur. The bituminous mixtures wer tested and evaluated according to Marshall method of mix design. The fourth type of mixture was composed of sand and ordinary portland cement. This was tested and its suitability was assessed based on the requirements set by Joint Departments of the Army and Air Force of USA, in their publication USA TM 5-882-14/AF MAN 32-8010,1994.

The third step of the research was analyses and interpretation of the laboratory test data. In this step, the laboratory test data were analysed and interpreted so that the optimum mixture compositions are identified, the effects of additives like cement and sulfur are addressed, and the sensitivity of bituminous mixes to slight variation in the mixtures' bitumen content is understood. Further, in this step, the suitable bituminous mixture was compared, based on economic criteria, with the suitable sand-cement mixture so as to identify the cheaper option.

The fourth and final step was declaration of the research findings and recommendations based on those findings.

The entire research process is shown by the flow chart in figure 3.5.



Literature Review
Problem Definition
Formulation of Research Objectives and Research Questions
Sampling and Testing
Sample Collection
Laboratory Testing
Sand
60/70 Bitumen
OPC cement
Sulfur
Quality/Physical Tests
Mix Design
Sand
Bitumen
Sulfur
Sand + Bitumen + Additives
Sand + Cement
Sand + Bitumen
Sand + Bitumen + Cement
Sand + Bitumen + Sulfur

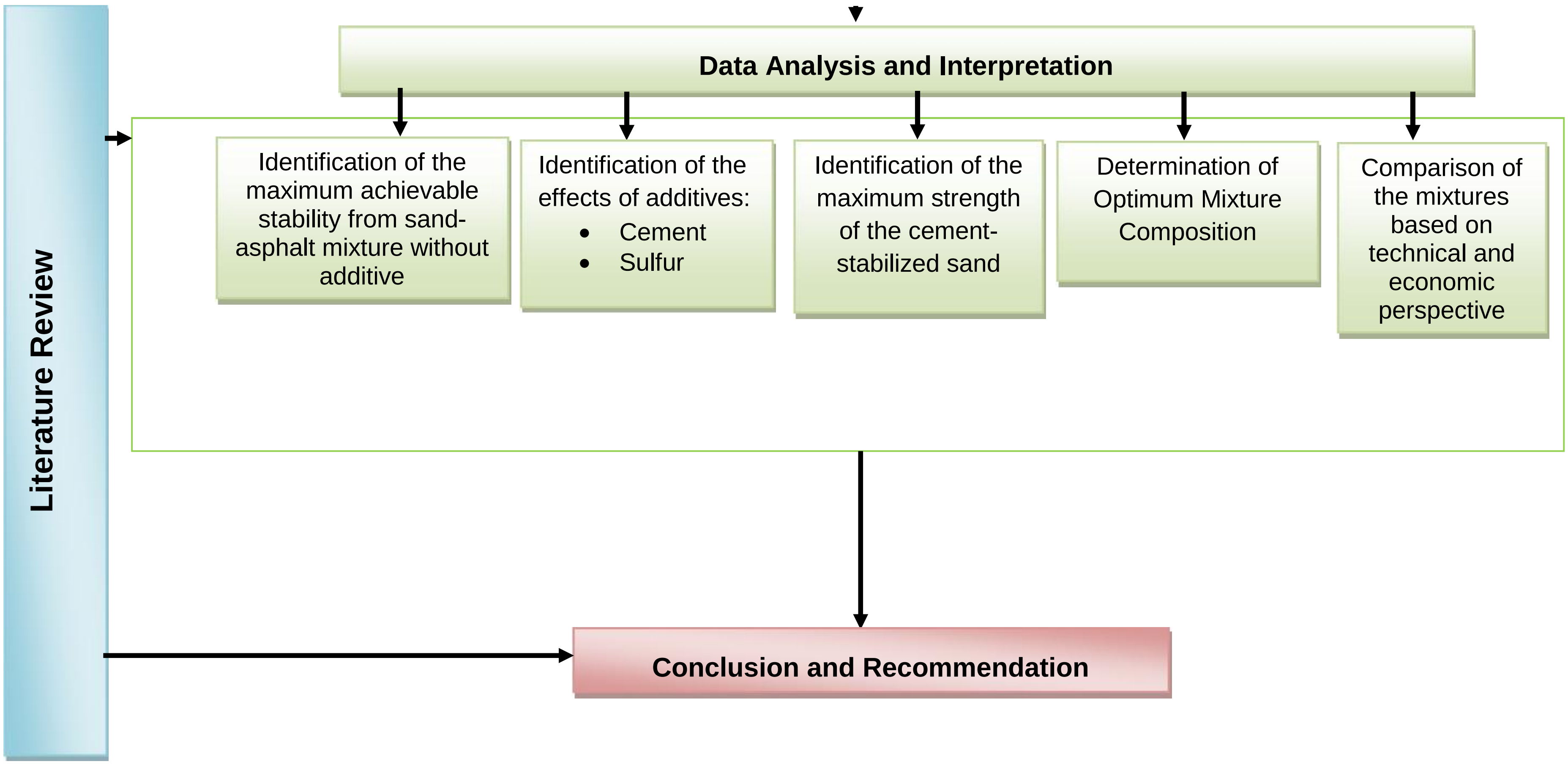


Figure 3.5 The research design

3.3 Materials Sampling

The sand samples were collected from three spots along the Adura – Burbey road segment. The sand samples were collected on the 26th of June 2013. The sample locations as shown in fig 3.6 are concentrated within the first 10km of the project. This was because the other borrow pits were inundated and were not accessible during the sampling period.

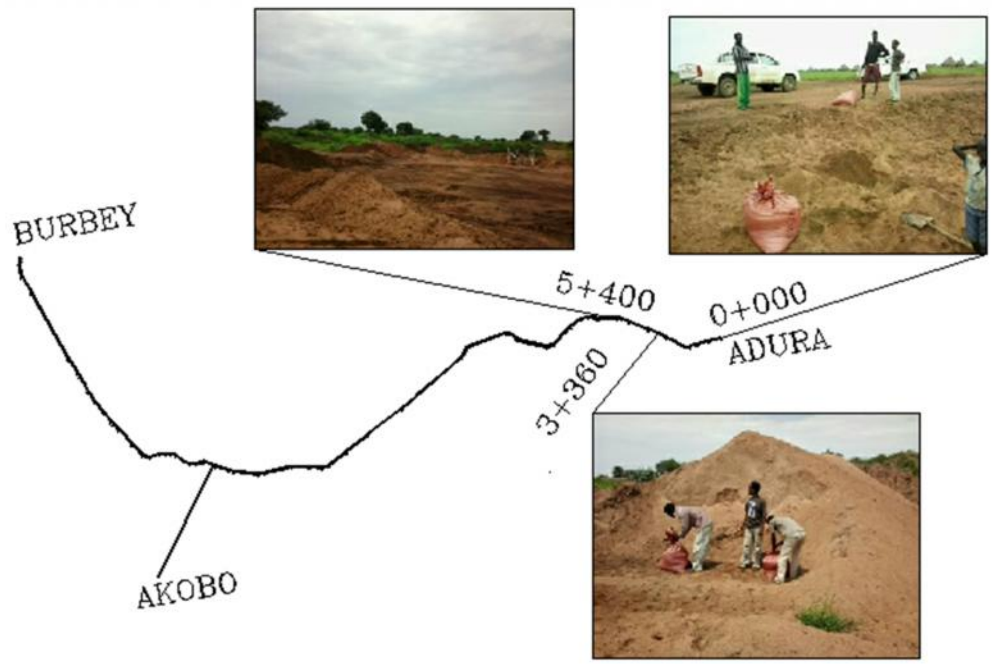


Figure 3.6 Natural sand sample locations

The ordinary portland cement and pen 60/70 bitumen were obtained from the central laboratory of CORE Consulting Engineers. The ordinary portland cement manufactured by Derba Cement Factory was utilized for the research. The sulfur was purchased from the local market. Twenty kilogram first class commercial sulfur was used for the study.

3.4 Data Acquisition

This is a controlled experimental study. The analyses were carried out based on primary data that had been obtained from laboratory tests. The following test methods were employed in the research process.

I-Quality Tests

1. Tests on the Natural Sand

No.	Test	Method
1	Atterberg Limits	AASHTO T 89 and T 90
2	Sieve analysis of fine and coarse aggregate	AASHTO T 27
3	Specific gravity and water absorption of fine aggregate	AASHTO T 84
4	Organic impurities in fine aggregate	AASHTO T 21
5	Effect of organic impurities in fine aggregate on strength of mortar	AASHTO T 71
6	Soundness of aggregate by use of sodium sulfate	AASHTO T 104
7	Clay lumps and friable particles in aggregate	AASHTO T 112
8	Plastic fines in graded aggregate and soils by the use of sand equivalent test	AASHTO T 176
9	Complete silicate analysis	LIBO2 Fusion, HF attack, Gravimetric, and AAS

2. Tests on Bitumen

No.	Test	Method
1	Penetration of bituminous material	AASHTO T 49
2	Flash and fire point by clevland open cup	AASHTO T 48
3	Ductility of bituminous material	AASHTO T 51
4	Softening point of bituminous material by ring and ball apparatus	AASHTO T 53
5	Solubility of bituminous material	AASHTO T 44
6	Specific gravity of semi solid bituminous material	AASHTO T 228

3. Tests on Sulfur

No.	Test	Method
1	Complete silicate	LIBO2 Fusion, HF attack, Gravimetric, and AAS

II-Mix Design

1. Bituminous Mixes

No.	Test	Method
1	*Marshall Tests*	*ASTM D 1559*
2	*Maximum specific gravity of paving mixture*	*AASHTO T 209*
3	*Bulk specific gravity of compacted hot-mix saturated surface dry specimens*	*AASHTO T 166*

2. Sand-Cement Mixes

No.	Test	Method
1	*Laboratory compaction characteristics of soil using modified effort*	*ASTM D 1557*
2	*Standard test method for wetting and drying of compacted soil-cement mixtures*	*ASTM D 559*
3	*Compressive strength of molded soil-cement cylinders*	*ASTM D 1633*

3.5 Data Quality Assurance

The quality of the data obtained from each of the tests were assessed using the precision and bias statements of the respective test methods.

3.6 Analysis Plan

The data obtained from the laboratory tests of bituminous mix were analysed and interpreted using the Marshall Mix Design Criteria, specified in the latest version of Asphalt Institute Manual Series No.2, MS-2, 1997. These criteria are shown in table 3.1 and table 3.2.

On the other hand, for cement treated sand, the criteria set by Joint Departments of the Army and Air Force of USA (Joint Departments of the Army and Air Force, USA TM 5-822-14/AFMAN 32-8010, 1994) were used. These criteria were tabulated in table 2.7 and 2.8.

Table 3.1 Marshall Mix Design Criteria (MS-2, 1997)

Marshall Method Mix Criteria	**Light Traffic**		**Medium Traffic**		**Heavy Traffic**	
	Surface & Base		**Surface & Base**		**Surface & Base**	
	Min.	**Max.**	**Min.**	**Max.**	**Min.**	**Max.**
Compaction, number of blows each end of specimen	35		50		75	
Stability, N	3336	-	5338	-	8006	-
Flow, mm	2	4.5	2	4	4	3.5
Percent Air Voids	3	5	3	5	3	5
Percent Voids in Mineral Aggregate	See Table 3.2					
Percent Voids Filled with Asphalt	70	80	65	78	65	75

Table 3.2 Minimum Percent Voids in Mineral Aggregate (VMA) (MS-2, 1997)

Nominal Maximum Particle Size		**Design Air Voids [%]**		
mm	**in.**	**3.0**	**4.0**	**5.0**
1.18	No. 16	21.5	22.5	23.5
2.36	No. 8	19.0	20.0	21.0
4.75	No.4	16.0	17.0	18.0
9.5	3/8	14.0	15.0	16.0
12.5	½	13.0	14.0	15.0
19.0	¾	12.0	13.0	14.0
25.0	1.0	11.0	12.0	13.0
37.5	1.5	10.0	11.0	12.0
50	2.0	9.5	10.5	11.5
63	2.5	9.0	10.0	11.0

4 LABORATORY TEST RESULTS

In this section, summaries of the results of laboratory tests that had been conducted for the study were presented. The detailed laboratory test results are appended at the end of the paper and the analyses and interpretation of the test results were presented in next chapter, chapter 5.

4.1 Quality Test Results

Material quality tests were carried out on the natural sand, bitumen, and sulfur that were used for this research. The results of the laboratory tests are discussed subsequently.

4.1.1 Natural Sand

The natural sand were subjected to physical and chemical tests so as to understand its physical properties and composition. The physical tests were conducted in the central laboratory of CORE Consulting Engineers Plc. These tests were done on each of the three samples, collected from the site, and on the sample that had been prepared by combing the three samples by **one**-to-**one**-to-**one** ratio on volume basis. The chemical test, which is complete silicate analysis test, was carried out in the Geochemical laboratory of Geological Survey of Ethiopia. This test was done only on the combined sand sample.

The sieve analysis tests were carried out in accordance with AASHTO T 27 and AASHTO T 11. This test methods are used to determine the grading of the materials and develop relationships concerning porosity and packing. The test is conducted by separating a sample of dry aggregate of known mass through a series of sieves of progressively smaller openings. The results of these tests indicated that the sand from all sources are poorly graded with coefficient of uniformity of 3.7 – 5.4 and fineness modulus of 3.0 – 3.6. Gradation of the sand samples is shown in fig 4.1 and the particle size parameters are presented in table 4.1.

Table 4.1 Particle Size Parameters of the Natural Sand

Sample Station	D10 (mm)	D30 (mm)	D60 (mm)	Cu	Cc	FM
0+000	0.74	1.28	3.2	4.3	0.7	3.6
3+360	0.22	0.48	1.12	5.0	0.9	3.0
5+400	0.21	0.64	1.12	5.4	1.8	3.0
Combined	0.30	0.60	1.12	3.7	1.1	3.2
FHWA 0.45-power gradation	0.075	0.83	3.97	52.9	2.3	5.0

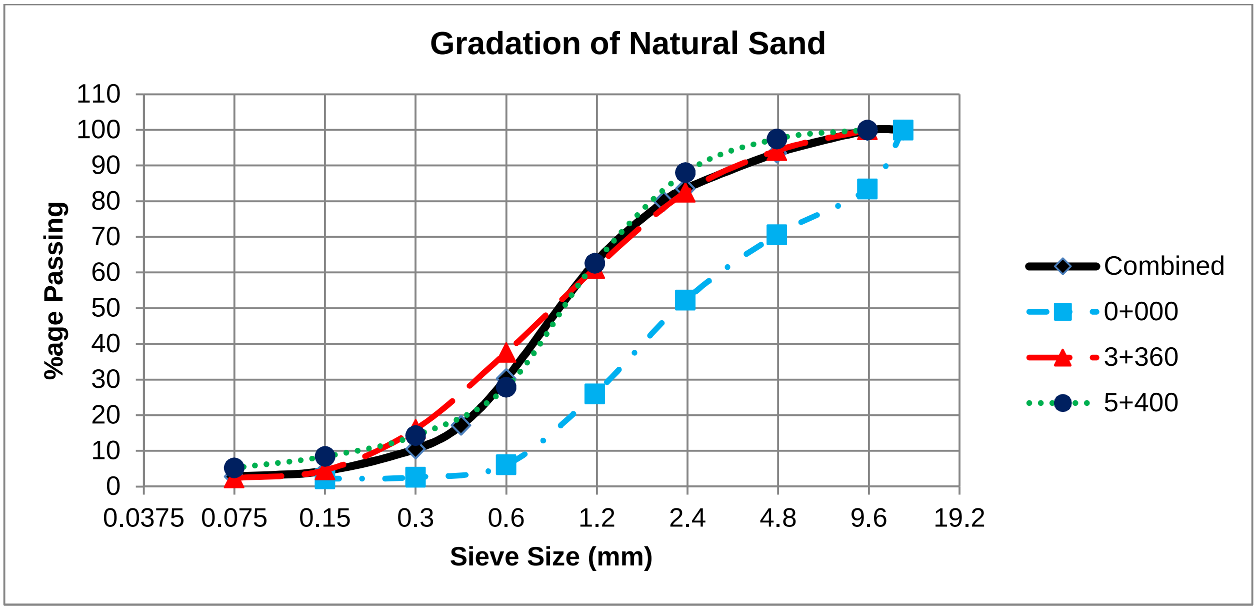


Figure 4.1 Gradation of natural sand samples

Comparing with the FHWA 0.45-power gradation curve, the gradation of all of the natural sand samples has the shape of uniform grading curve. Plot of the combined sand gradation and the FHWA 0.45-power gradation is shown in fig 4.2. The 0.45-power gradation has coefficient of uniformity of 52.9 and coefficient of curvature of 2.3.

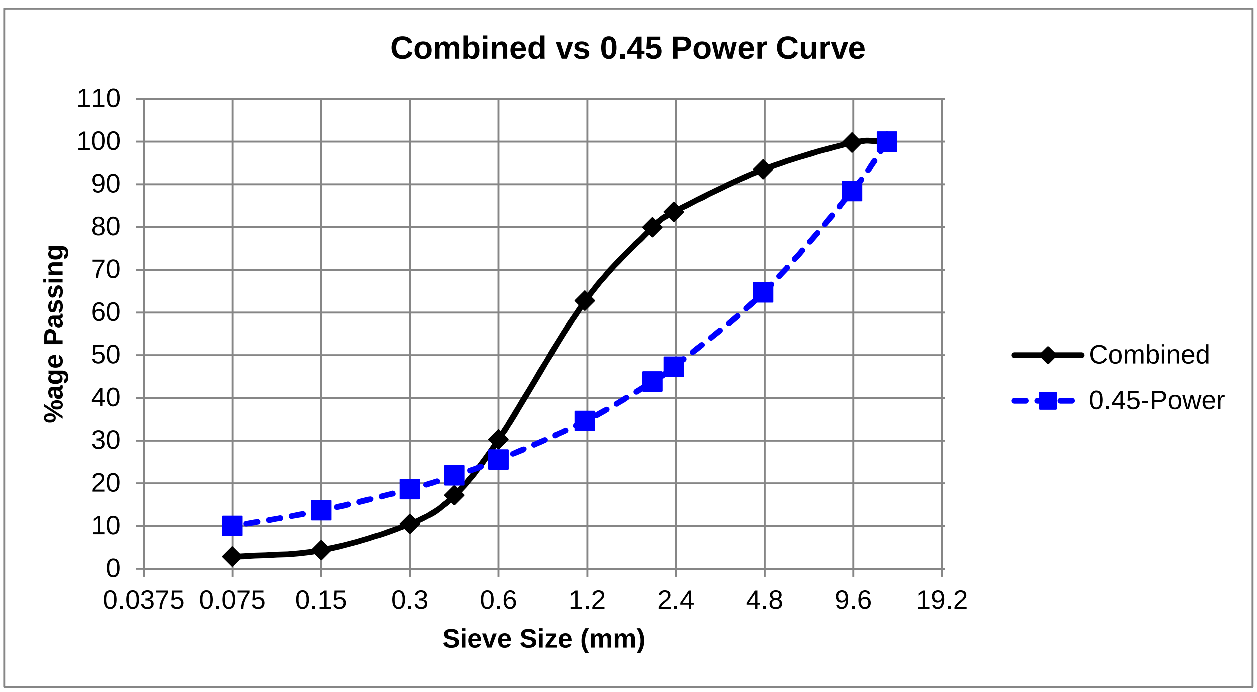


Figure 4.2 Comparison of the combined sand gradation against 0.45-power curve

The specific gravity and water absorption of the sand were determined according to AASHTO T 84. The bulk specifc gravity is the characteristic that is used to calculate the volume occupied by the aggregate in various mixtures. It was determined by the pycnometer tests using the following relation.

$$G = \frac{A}{(B + S - C)}$$

where:

A= mass of oven-dry specimen in air, g;

B= mass of pycnometer filled with water, g;

C = mass of pycnometer with specimen and water to calibration mark, g;

S= mass of saturated surface-dry specimen, g.

In this research the bulk specific gravity of the natural sand was used to compute its volume in the sand-cement and sand-cement-bitumen mixtures. The water absorption value indicates the volume of the interconnected voids within the aggregate grains. It is computed by the following relation:

$$WA = \left[\frac{(S - A)}{A}\right] * 100$$

where:

WA = water absorption, %;

A= mass of oven-dry specimen in air, g;

S= mass of saturated surface-dry specimen, g.

The specific gravities and water absorption values of the four sand samples are presented in table 4.2.

Table 4.2 Specific Gravity and Water Absorption of the Natural Sand

Parameter	0+000	3+360	5+400	Combined
Bulk Specific Gravity [oven-dry]	2.662	2.627	2.562	2.633
Bulk Specific Gravity [SSD]	2.681	2.646	2.589	2.657
Apparent Specific Gravity	2.713	2.677	2.634	2.697
Water Absorption [%]	0.7	0.7	1.1	0.9

The organic impurity is used to check the presence of organic impurities in the fine aggregate. This test was conducted in accordance with AASHTO T 21. In this method, the glass bottle is filled with fine aggregate sample and 3% Na(OH) solution is shaked vigorously and allowed to still for 24 hours. The color of the supernatant liquid is then compared with standard color in the glass color standard. The organic impurity tests that had been done on the four natural sand samples indicated that the amount of injurious organic compounds is within tolerable limits. In each of the four cases, the color of the supernatant liquid is lighter than the color of the standard organic plate no. 3. The results of organic impurity test are presented in table 4.3.

Table 4.3 Organic Impurity Test Results

	0+000	3+360	5+400	Combined
Color of Supernatant Liquid	1	1	2	2
Color of Standard Organic Plate	3			

The effect of injurious organic compounds in the fine aggregate was tested using AASHTO T 71. In this method a mortar with water cement ratio of 0.6 and consistencey of 100±5 is prepared and molded in 50mm cubes.The molded specimens are then subjected to compression tests and the mortar strength is determined by dividing the breaking load with the area of the loaded face of the cube specimens. This test is performed on fine aggregates that produce supernatant liquid darker in color than the color of the standard plate no 3. In this study, however, the test was conducted merely to identify to what extent those trace organic compounds affect the strength of the sand-cement mixture. The effect of organic impurity tests revealed that the organic compounds in the sands from 0+000 and 3+360 have hardly any effect on the mortar strength value. However, the organic impurities in the sand from 5+400 have substantial detrimental effect as they lessen the mortar strength of the sand-cement mixture by about 15%. The effect is minimal in the combined sand-cement mixture. The results of the test are summarized in table 4.4.

Table 4.4 Effects of Organic Impurities in Mortar Making Properties of Sand

Sample Station	Unwashed:Washed Sand Cement Mixture Strength [%]	
	3-days	7-days
0+000	90.9	95.0
3+360	89.2	95.1
5+400	80.0	85.3
Combined	88.1	93.6

Figure 4.3 Organic impurity test

The soundness test was undertaken so as to judge the resistance of the sand to the action of weathering. The test was done in accordance with AASHTO T 104 using Sodium Sulphate solution. The test is carried out by soaking predetermined amount of sample in the sodium sulfate solution for 16-18 hours and drying to constant mass for five cycles. After five cycles of soaking and drying the loss in the fractions in each of the test sieves is recorded and reported as the weighted average of the original gradation of the sample. The results of soundness test are shown in table 4.5.

Table 4.5 Soundness Test Results

Sample Station	**0+000**	**3+360**	**5+400**	**Combined**
Soundness Loss [%]	7	7	8	7

Clay lumps and friable particles test was carried out according to AASHTO T 112 to identify the amount of clay lumps and weaker grains in the sand samples. This test was accompanied by Sand Equivalent test which was done in accordance with AASHTO T 176. The Sand Equivalent test is used to determine the proportion of deterimental plastic fines in the portion passing 4.75mm sieve. The results of these two tests indicated that the sample from 5+400 contains substantial amount of clay lumps and plastic fines. The amount of clay lumps and plastic fines on the other samples are very low as compared to the amount in the sample from 5+400. The results of the two tests are summarized in table 4.6.

Table 4.6 Clay Lumps and Friable Particles and Sand Equivalent Test Results

Sample Station	Clay Lumps and Friable Particles [%]	Sand Equivalent [%]
0+000	1.9	95
3+360	2.2	90
5+400	15.1	85
Combined	4.4	90

The Atterberg limits tests, which were done as per AASHTO T 89 and AASHTO T 90, revealed that the natural sand from all sources are non-plastic.

All of the four natural sand samples are grouped under **A-1-b(0)** soil class when classified according to AASHTO M 145 and under **SP** (poorly graded sand) soil class when classified according to ASTM D 2487.

Based on the complicate silicate analysis report, the sand is predominantly composed of Silicon dioxide and contains certain amounts of Aluminium Oxide, Potassium Oxide, and Iron Oxide (Ferric Oxide). The results of the complete silicate analysis is shown in table 4.7.

Table 4.7 Composition of Combined Sand Sample [%]

Sample	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O	LOI
Combined Sand	85.28	6.66	2.50	0.54	0.12	0.64	2.94	0.08	0.03	0.36	0.23	0.58

The detailed results of all of the laboratory tests, conducted on the natural sand, were annexed as Appendix I-1.

4.1.2 Bitumen

Six physical quality tests had been conducted on the samples of the bitumen that were used in this study. These are penetration (both on the original and aged bitumen), ductility (both on the original and aged bitumen), softening point, solubility, flash and fire points, and specific gravity.

The penetration test was performed in accordance with AASHTO T 49. The test is used to measure the consistency of the bitumen and classify the bitumen according to penetration grading system. This test was done on the original bitumen and on the aged residue so as to identify the hardening effect of oxidation of the bitumen.

The ductility test is the measure of the tensile property of the bituminous materials. The test was conducted as per AASHTO T 51. Similar to penetration test, this test was also conducted on both of the original and aged bitumen.

Softening point is the test which is used to indicate the temperature at which the bitumen becomes softer and tends to flow. The test was carried out according to AASHTO T 53.

Solubility test is purity test, which is used to check if foreign materials other than active cementing constituents are present in the bitumen. This test was done according to AASHTO T 44.

The flash and fire point test is safety test that is used to determine the temperature at which vapor of the bituminous materials catch fire and ignites. The test was conducted as per AASHTO T 48.

The specific gravity of the bitumen was determined based on AASHTO T 228. The specific gravity of bitumen is used to convert the unit mass into volume and vice versa.

The results of the laboratory tests that were carried out on the bitumen are attached as Appendix I-2 and summarized in table 4.8. The test results, as shown in table 4.8, indicated that the bitumen fulfills all of the requirements specified for pen 60-70 in AASHTO M 20.

Table 4.8 Summarry of Bitumen Quality Test Results

Test Type	Test Method	Result	Spec [AASHTO M 20]
Penetration of Bituminous Materials, dmm	AASHTO T 49	65	60 – 70
Flash and Fire Point by Cleveland Open Cup, ^{o}F	AASHTO T 48	635	Min. 450
Ductility of Bituminous Materials, cm	AASHTO T 51	100+	Min. 100
Softening Point of Bituminous Materials,^{o}C	AASHTO T 53	58.7	-
Solubility [%]	AASHTO T 44	99.8	Min. 99.0
Specific Gravity	AASHTO T 228	1.022	-
Tests on the aged residue			
Penetration of Bituminous Materials, dmm	AASHTO T 49	84.7	Min. 54
Ductility of Bituminous Materials, cm	AASHTO T 51	100+	Min. 50
Loss on heating, %	AASHTO T 240	0.2	Max. 0.8

4.1.3 Sulfur

The first class commercially available sulfur was used for SSB mixture. The chemical composition of the sulfur was checked in the Geochemical laboratory of Geological Survey of Ethiopia. The test result indicated that material contains 99.7% pure sulfur. The test result is shown in table 4.9.

Table 4.9 Results of Complete Silicate Analysis on Sulfur

Sample	S	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O
Commercial Sulfur	99.7	<0.01	0.19	<0.01	<0.01	<0.01	<0.01	0.04	<0.01	<0.01	<0.01	<0.01

4.2 Bituminous Mix Test Results

Bituminous mix in this research context refers to sand-bitumen, sand-cement-bitumen, and sand-sulfur-bitumen mixes. All of these mixtures were prepared and tested according to ASTM D 1559: Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus to determine the Stability and Flow of the compacted mixture. For the volumetric computation of the compacted mix, further Theoretical Maximum Specific Gravity and Bulk Specific Gravity tests were carried out in accordance with ASTM D 2041 and ASTM D 2726 respectively. The Theoretical Maximum Specific Gravity [TMSG] Test was conducted on each of the trial mixes.

4.2.1 Sand-Bitumen Mix

The mixtures of sand and bitumen were prepared using the combined sand and 60/70 penetration grade bitumen as ingredients. The mixtures were prepared by four different bitumen contents (5.0%, 5.5%, 6.0%, 6.5%). The first trial was conducted by 5.0% bitumen as this is the least amount to keep the sand grains bound. The results of the tests conducted on these mixtures are shown in table 4.10. The detailed laboratory test results is presented in Appendix I-4.

Table 4.10 Sand-Bitumen Mix Test Results

Bitumen Content [% total mix]	Stability [kN]	Flow [mm]	Bulk Sp. gr	TMSG
5.0	1.42	1.20	2.038	2.445
5.5	1.27	1.31	2.046	2.437
6.0	1.20	1.47	2.054	2.429
6.5	1.02	1.78	2.052	2.423

4.2.2 Sand-Cement-Bitumen Mix

The sand-cement-bitumen mixes were prepared by combining 2% (of total mix) ordinary portland cement with the combined sand and 60/70 penetration grade bitumen. These mixtures were prepared to check whether the addition of cement improves the strength of the sand-bitumen mixture. In preparing the mixture, the moderately heated cement is thoroughly mixed with the heated sand first and then the heated bitumen was added and mixed together. The amount of cement is limited to 2% because this is the maximum amount recommended by a number standards [ERA STS-6402d, SATCC-4202c, Tanzania-4202c] so as to control shrinkage cracking in the compacted mixture. The sand-cement-bitumen trial mixes were prepared at six different binder contents. The first trial was made with 4.5% and the last with 6.5% bitumen contents. The summary of the test results is shown in table 4.11 and the detailed is presented in Appendix I-5.

Table 4.11 Sand-Cement-Bitumen Mix Test Results

Bitumen Content [% total mix]	Stability [kN]	Flow [mm]	Bulk Sp. gr	TMSG
4.0	1.13	1.55	2.018	2.532
4.5	1.30	1.75	2.045	2.498
5.0	1.37	1.81	2.068	2.460
5.5	1.39	1.75	2.093	2.422
6.0	1.49	1.82	2.083	2.349
6.5	1.72	2.09	2.072	2.278

4.2.3 Sand-Sulfur-Bitumen Mix

The sand-sulfur-bitumen mixtures were prepared using the combined sand, 99.7% pure commercial sulfur lumps, and 60/70 bitumen. The sand and the bitumen were heated to 150^{o}C – 160^{o}C separately before mixing while the sulfur was heated to a temperature of 145^{o}C-150^{o}C so that it is melted and converted into liquid state. The predetermined amount of the heated sand and the hot bitumen were thoroughly mixed first. The molten sulfur was then added to the sand-bitumen mixture and thoroughly mixed together.

The trial sand-sulfur-bitumen mixtures were prepared and tested in two stages. In the first stage, the mixtures contained 5% bitumen and 5% - 25% sulfur. At this stage, the optimum mixture compositions that meets the base course requirements were identified. In the second stage, the mixtures were prepared at 5.5% and 6.0% bitumen contents, and 15% and 20% sulfur contents. The second stage tests were done to identify the sensitivity of the optimum mixtures to slight changes in the bitumen content. The test results are summarized in table 4.12 and the detailed results presented in Appendix I-6.

Table 4.12 Sand-Sulfur-Bitumen Mix Test Results

Bitumen Content [% total mix]	Sulfur Content [% total mix]	Stability [kN]	Flow [mm]	Bulk Sp. gr	MTD
5.0	5	2.38	1.75	2.005	2.417
	10	3.26	1.67	2.074	2.386
	15	8.10	1.44	2.149	2.355
	20	8.94	1.97	2.173	2.302
	25	10.29	1.91	2.213	2.310
5.5	15	7.92	2.02	2.151	2.329
	20	8.76	2.00	2.170	2.300
6.0	15	7.57	2.14	2.154	2.320
	20	8.19	2.04	2.173	2.291

4.3 Cement Stabilized Sand Test Results

According to the USCS, the natural sand that had been used for this study was poorly graded sand and falls in SP soil group. For this type of soil, the initial cement content, recommended for soil-cement mixtures, is 7% of the weight of the dry soil. In this study, the first trial mixes were prepared by 7% and 10% cement contents and based on the observation of the results of the two trials, two further trials were conducted by 8.5% and 9% cement contents. The test that had been conducted on the trial mixes and the corresponding results were discussed in the following sections.

4.3.1 Soil-Cement Mix Preparation

The soil-cement mixture was prepared in accordance with ASTM D 558, method B. Accordingly, the natural sand was quartered and reduced to 3 to 4 kilograms. The fractions retained on No.4 (4.75mm) sieve were separted and soaked for 24-hours and those passing No.4 sieve were oven-dried to constant mass. The soil-cement mixture was then prepared by first blending the oven-dry fraction with the ordinarly portland cement in the predetermined proportion. The blend was thoroughly mixed until uniform color had been observed. Then predetermined amount of water was added to the blend and mixed thoroughly. The soaked fraction was then surface-dried using trowel and added to the mixture at this point and mixed thoroughly again until it is well combined and dispersed within the mixture. Then the soil-cement mixture become ready for compaction.

Figure 4.4 Preparation of soil-cement mixture

4.3.2 Moisture-Density Test

The moisture-density test is used to determine the relationship between the water content and the dry unit weight, and the maximum dry density [MDD] and the optimum moisture content [OMC] of the soil-cement mixture. The test was conducted according to ASTM D 1557, Procedure B. This procedure is used for materials having less than 20% retain on 9.5mm sieve and more than 20% retain on 4.75mm sieve. The materials fulfilling this requirement is compacted in 101.6mm (4-inch) diameter mold in five layers with each layer being compacted with 25 blows of 44.5N rammer falling from a height of 457mm. In this method, 2,700kN-m/m^3 compactive energy is induced and transferred to compact the soil-cement mixture.

The results of the moisture-density tests that were conducted on the four trial mixes are shown in fig 4.5 and table 4.13.

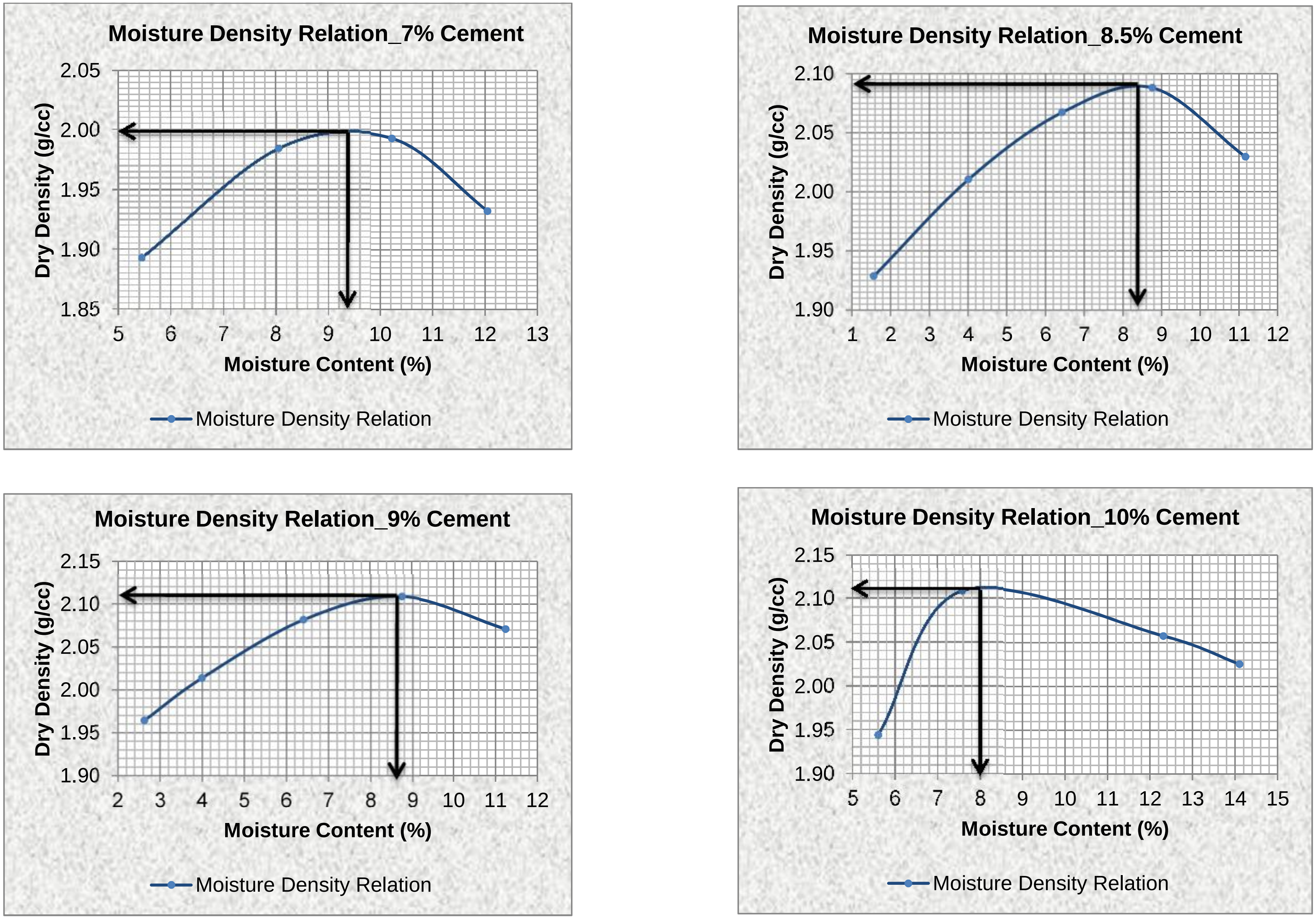


Figure 4.5 Moisture-Density relations at different cement contents

Table 4.13 MDD and OMC of the Soil-Cement Mixtures

Cement Content (% by wt)	MDD (g/cc)	OMC (%)
7.0	2.00	9.4
8.5	2.09	8.4
9.0	2.11	8.6
10.0	2.11	8.0

4.3.3 Unconfined Compression Test

The purpose of the unconfined compression test is to measure the strength of the compacted soil-cement mixture. In this study, the test was carried out according to ASTM D 1633, Method A in which 4-inch (101.6mm) diameter and 4.584-inch (116.4mm) mold is used for preparing the compacted specimen.

The compacted specimens were prepared based on ASTM D 559. The molded specimens were cured by covering them with plastic sheets for seven days. At the end of the curing period, the specimens were soaked in water for 4-hours and then subjected to compression tests. The compressive strength of the specimens was then calculated by dividing the maximum compressive load with the cross sectional area of the specimens.

Since 4-inch mold was used, the length-to-diameter ratio (L/D) of the specimens are less than two. Consequently, the compressive strength of these specimens needs to be reduced using certain correction factors. The reduction factors in ASTM C 32 were utilized to correct the compressive strength of the specimens in this study. According to ASTM C 32, the correction factors in table 4.14 are recommeded for various L/D ratio, and interpolated factors are suggested for values not given.

Table 4.14 Correction Factors [ASTM C 32]

Length-to-Diameter Ratio [L/D]-X	Strength Correction Factors-Y
1.75	0.98
1.50	0.96
1.25	0.93
1.00	0.87
Interpolating Equation	$Y=0.2133X^3-1.04X^2+1.7667X-0.07$

The unconfined compressive strength of the specimens of the four trial mixes are shown in table 4.15.

Figure 4.6 Unconfined Compression Test and Test Specimen Preparation

Table 4.15 Unconfined Compressive Strength of the Compacted Soil-Cement Specimens

Cement Content [%]	Compressive Strength [MPa]_4-inch Specimen	Corrected Compressive Strength [MPa]
7.0	2.6	2.4
8.5	4.7	4.3
9.0	5.1	4.7
10.0	6.5	5.9

4.3.4 Wetting and Drying Test

This is one of the durability test which are used to determine the resistance of the compacted soil-cement specimen to various weathering conditions. The weathering conditions in this test method are simulated by frequent wetting and drying of the specimens. The test was conducted according to ASTM D 559. By this test the soil-cement losses, water content changes and volumteric changes (swell and shrinkage) of the soil-cement specimen can be detected.

For this study, this test was preferred to the Freeze-Thaw because of the characteristics of the study area. The study area is characterized by hot and humid climate and large wet season. Hence, the wet-dry test better simulate the conditions in the study area.

The test is carried out in, at least, two compacted soil-cement specimens after seven days curing period. One of these specimens is control specimen and the remaining is/are soil-cement loss specimens. The specimens are submerged in potable water for 5-hours at room temperature. Then they are weighed and measured, and put in an oven at 71^{o}C for 42 hours. At the end of drying, the specimens are removed from the oven and weighed and measured after they are allowed to cool. The soil-cement loss specimens are then scratched with two firm strokes of a wire-brush within their full height and width. The specimens are then weighed and measured and the process repeated 12-times. After the 12-cycle process, the specimens are oven-dried at 110^{o}C to constant weight. The data collected in due course are then used to compute the volume and water change of the control specimen and the soil-cement loss of the remaining specimens.

In the subject case, wet-dry test was carried out in compacted soil-cement specimens with 9% and 10% cement contents as these specimens have strengths close to the minimum required for base course. Three 9% cement specimens were molded for the wet-dry test, of which one is control and two are soil-cement loss, while four were prepared for the 10% cement mixture with two control and two soil-cement loss specimens. For the 10% cement mixture, the durability tests were conducted on the paired specimen at different times. That means the first test was done on the first pair and the results were noted. Then the second confirmatory test was carried out on the second pair of specimens.

The results of the wet-dry test of the two trial mixtures are summarised in table 4.16 and detail test results are appended as Appendix I-7.

Table 4.16 Summary of the Wet-Dry Test Results

Cement Content (%)	Design		Actual		Max. Vol. Change [%]	Max. Water Content [%]	Soil-Cement Loss [%]
	MDD [g/cc]	OMC [%]	MDD [g/cc]	OMC [%]			
9	2.11	8.6	2.09	8.9	-0.91	7.1	2.7
			2.10	8.9			
			2.08	9.0			
10	2.11	8.0	2.03	7.6	0.73	9.0	2.0
			2.09	7.4			
			2.07	8.2	1.82	8.4	
			2.06	7.9			

5 ANALYSIS AND DISCUSSION

5.1 Bituminous Mixture

5.1.1 Sand-Bitumen [SB] Mix

The Marshall test results for the sand-bitumen mix indicated that the stability of the mixture ranges from 1.02kN – 1.42kN at the testing bitumen content of 5% - 6.5%. These are very low strength values as compared to the target minimum value of 8kN. The stability of this mix decreases continously with increasing asphalt content which is typical of recylced materials.

The flow of the mix is between 1.2mm and 1.8mm. This values fall below the minimum acceptable value of MS-2, which is 2.5mm. The flow of the sand-bitumen mixture increases with increase in the bitumen content, which is the normal trend for HMA.

The mixture is very porous with air void contents varying from 15.3% to 16.6%. This shows that the compacted mix is permeable and prone of quick aging. The air void contents are by far exceeds the maximum limit specified for base course in MS-2, which is 5%. The plot of the air void versus bitumen content indicates that the amount of the air void decreases with increase in the bitumen content, which is again the normal trend for HMA mixes. The story is different when it comes to VMA curve. The VMA increases continously with increasing bitumen content. This means that the bitumen in the mix is pushing the natural sand grains apart even at the lowest testing bitumen content. Thus, there was no grain-to-grain contact of the natural sand in the compacted mixture and that may be the case for the continous drop in stability of the mix with increasing bitumen content. At the testing bitumen contents, the VMA values vary from 26.5% - 27.1%. These values are above the minimum requirements of the MS-2, 14% - 16%, for nominal maximum aggregate size of 9.5mm. The values of the Voids Filled with Binder [VFB] indicates that only 37.1% - 43.6% of the VMA is filled with bitumen, which is very low as compared to the design requirement of 65% - 75%. The plot of the VFB curve depicts that the VFB follows the same trend as HMA by increasing with increasing bitumen content.

In general, the sand-bitumen mix test results indicate that the mixture is very weak and does not fulfill many of the design criteria to serve as a base course material.

The plots of the compacted mixture properties against the mixing bitumen content are shown in figure 5.1.

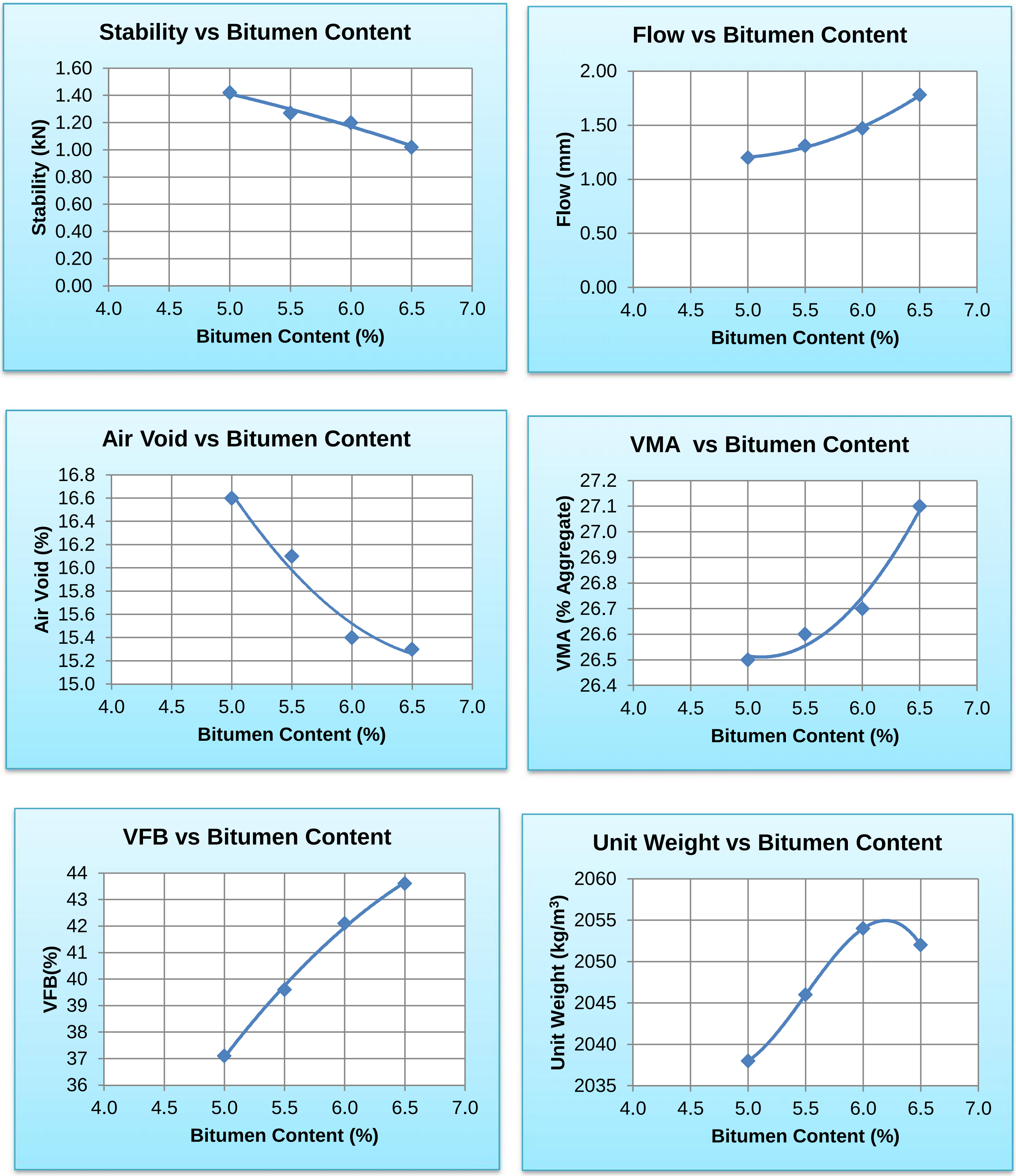


Figure 5.1 Test property curve for sand-bitumen mixture

5.1.2 Sand-Cement-Bitumen [SCB] Mix

In order to observe whether the inclusion of the active filler into the SB mix improve its properties, a mixture composed for natural sand, 2% (by weight of total mix) ordinary portland cement, and bitumen were prepared and subjected to Marshall tests.

The addition of 2% ordinary portland cement to the natural sand had slightly improved the properties of the sand-cement-bitumen as compared to the sand-bitumen mixture.

The stability, flow, unit weight, and VFB of the of the SCB mix are slightly increased while the air-void and VMA values are slightly decreased from the corresponding values of the SB mix.

The trends the SCB mixture has exhibited with the bitumen content are normal of the HMA. That is:

- Stability increases with bitumen content. However, the peak was not reached at the testing bitumen content;
- The flow and VFB values also increase with increase in the bitumen content;
- The air-void continously decreases with increasing bitumen content;
- The VMA curve shows an upward parabola shape; and
- The unit weight increases untill the peak point and decreases there after.

Eventhough the addition of 2% cement slightly improved the mix properties, the mixture in general does not meet all but the VMA criteria, specified in MS-2 for base course mixtures. As shown in the test results, the stability of the SCB mixture increases from 1.13kN at 4.0% bitumen content to 1.72kN at 6.5% bitumen content. These values are by far less than the minimum stability value of 8kN that is required for base course materials. Therefore, it can be concluded that the addition of 2% cement has hardly any effect on the stability of the SB mixture and the SCB mixture can not be used for base course construction.

The plots of the compacted mix properties of the SCB mixture are shown in figure 5.2 and the comparison of the SB and SCB mixture properties are shown in figure 5.3.

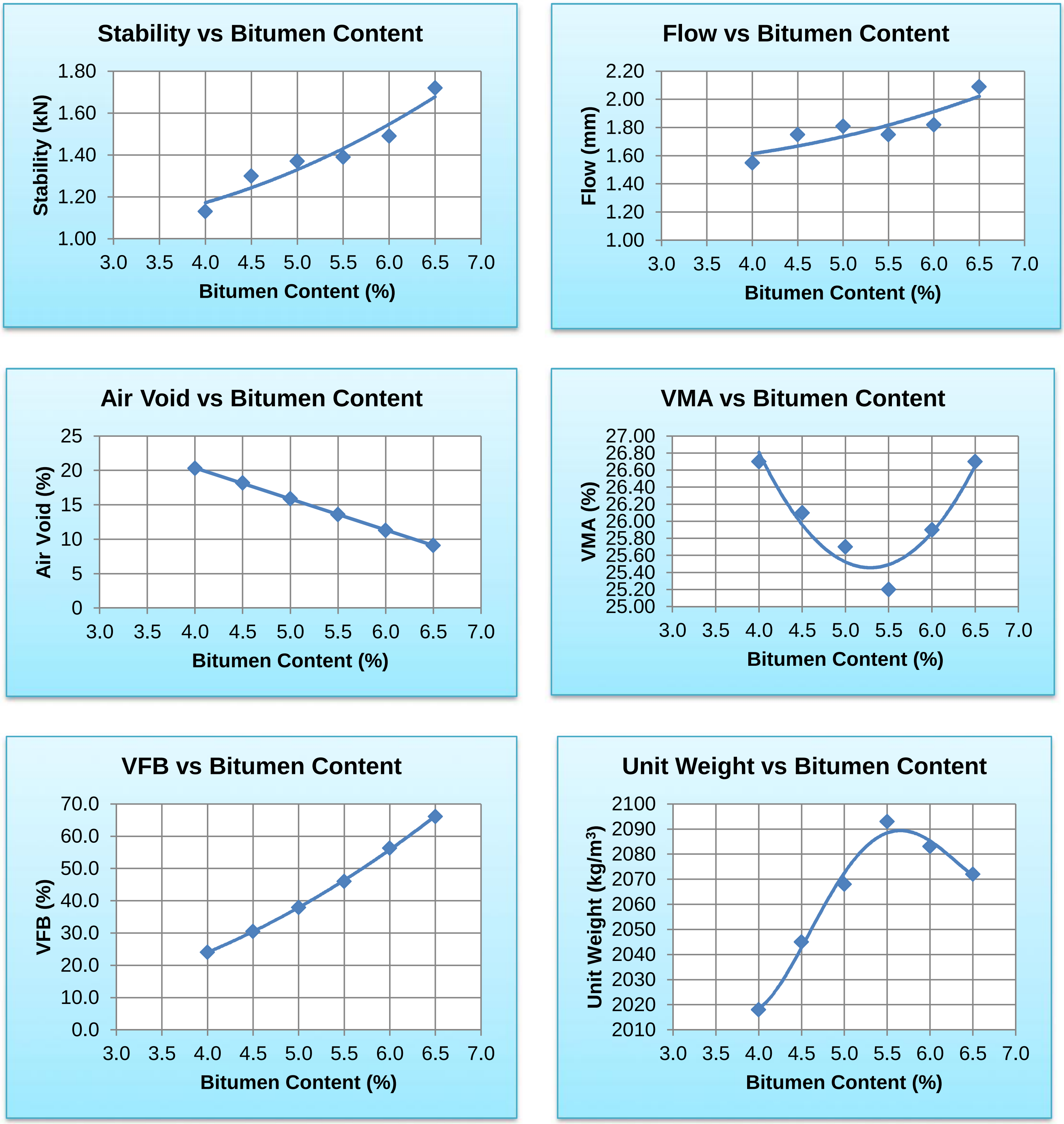


Figure 5.2 Test property curves for sand-cement-bitumen mixture

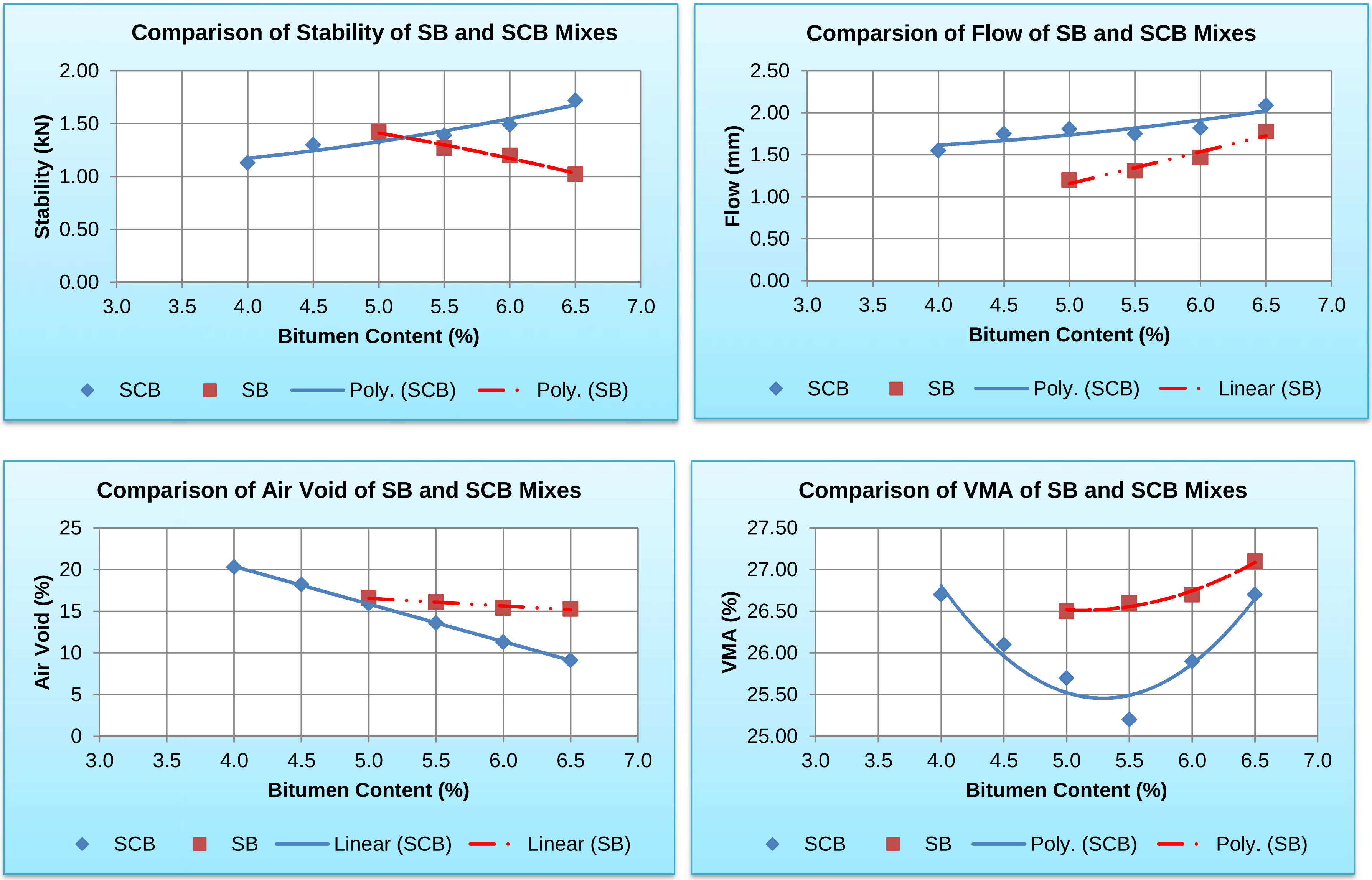
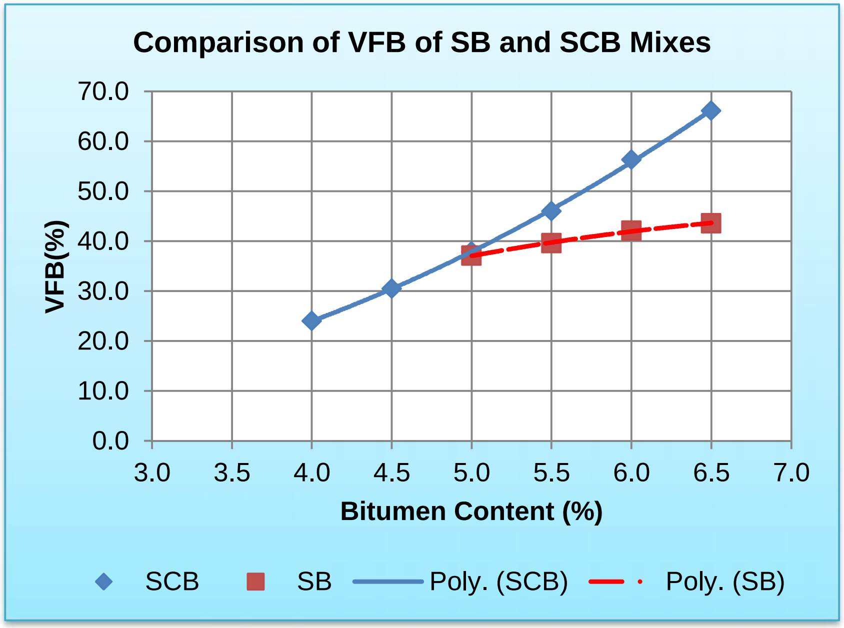


Figure 5.3 Comparison of the properties of SB and SCB mixtures



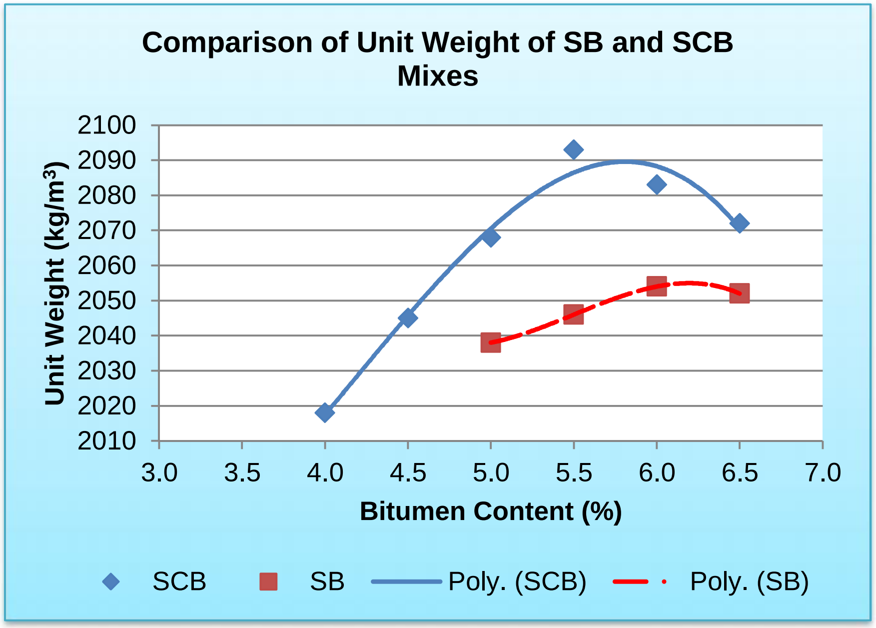


Figure 5.4 Comparison of the properties of SB and SCB mixtures (cntd.)

5.1.3 Sand-Sulfur-Bitumen [SSB] Mix

The sand-sulfur-bitumen mixes were prepared by blending various amounts of the commercial sulfur with the sand and bitumen. The amounts of sulfur added ranges from 5% to 25% with 5% increments.

The sulfur was added to the mixes in solution form after completely melting the lumps of the commercial sulfur. The SSB mixture was prepared by first mixing the hot natural sand with the bitumen and then adding and further mixing with the sulfur solution heated to the temperature of 140^{o}C – 150^{o}C.

The addition of sulfur to the sand-bitumen mixes improved every aspects of the mixture properties. The stability of the mix increased at each increments in the amount of sulfur. The trend the stability curve showed matches with those mentioned in the literature review as the stability increases to a peak value. The plot of the stability versus sulfur content is shown in figure 5.5 and the trend of the stability with sulfur content is shown in figure 5.6.

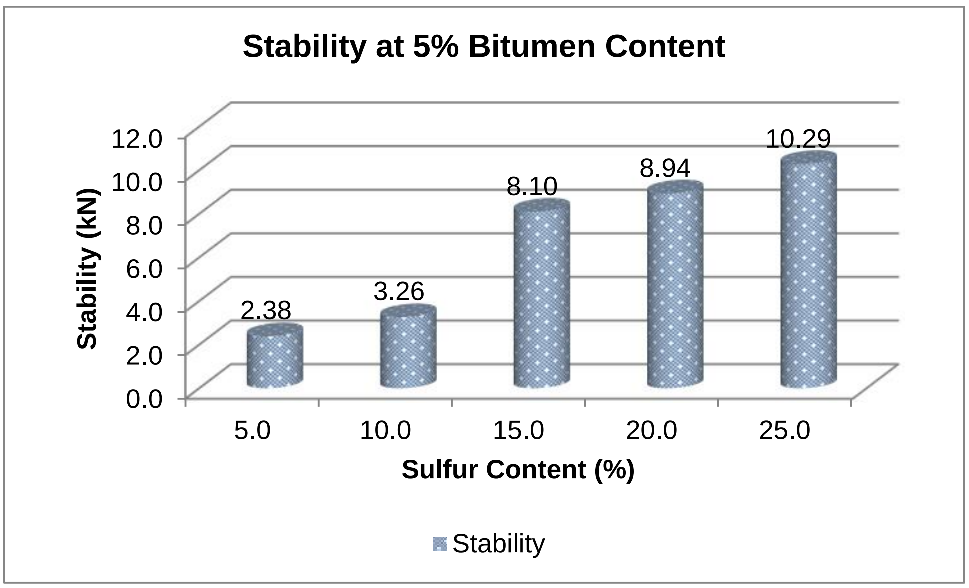


Figure 5.5 Stability of SSB mix at 5% bitumen content

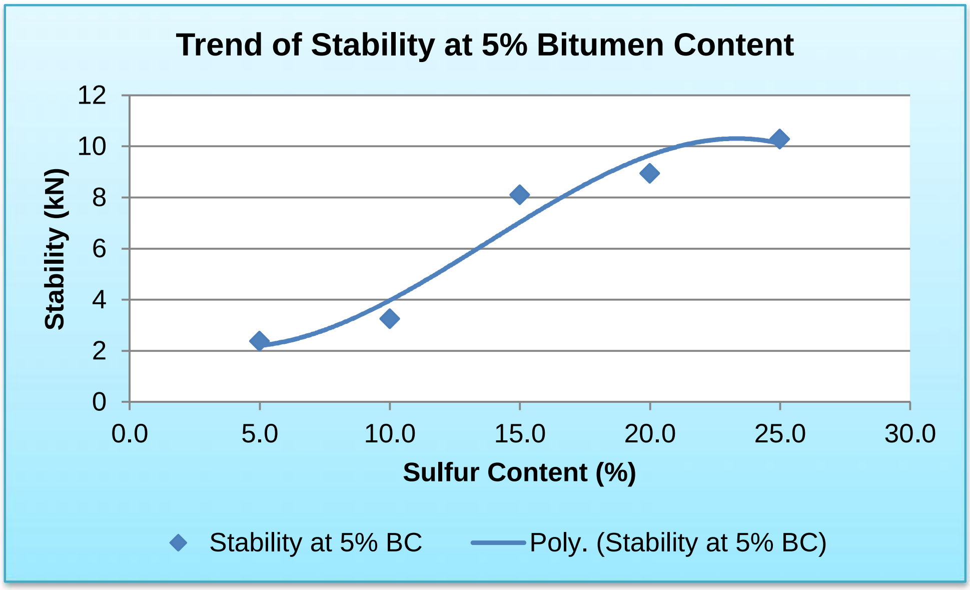


Figure 5.6 Trend of stability of SSB mix at 5% bitumen content

It is shown in fig 5.5 that the stability of the SSB mix increased by 0.88kN, which is 37% of the stability at 5% sulfur, as the sulfur content increased from 5% to 10%. Increasing the amount of sulfur to 15% shot the stability to 8.1kN, which is 148.5% increase in the stability of the mixture. Further increments of the sulfur content to 20% and 25% resulted in 10.4% and 15% increase in stability values respectively.

According to fig 5.5, SSB mixes containing 15% and above sulfur fulfill the MS-2 stability requirement for base course having stability values above 8kN.

In order to observe sensitivity of the SSB mixes to bitumen content, sensitivity analysis was carried out by increasing the bitumen contents to 5.5% and 6.0%. The sensitivity analysis was conducted on mixtures with 15% and 20% sulfur contents. Accordingly it was found out that at 15% sulfur content, the stability of the mix slightly decreased both at 5.5% and 6.0% bitumen contents. Similarly, at 20% sulfur content, the stability of the mixture decreased both at 5.5% and 6.0% bitumen contents. Therefore, it can be concluded that the stability of the SSB mixes decrease with increase in the bitumen content provided that the amount of sulfur in the mix remains constant. The sensitivity of stability of SSB mixes to bitumen content is depicted in fig 5.7.

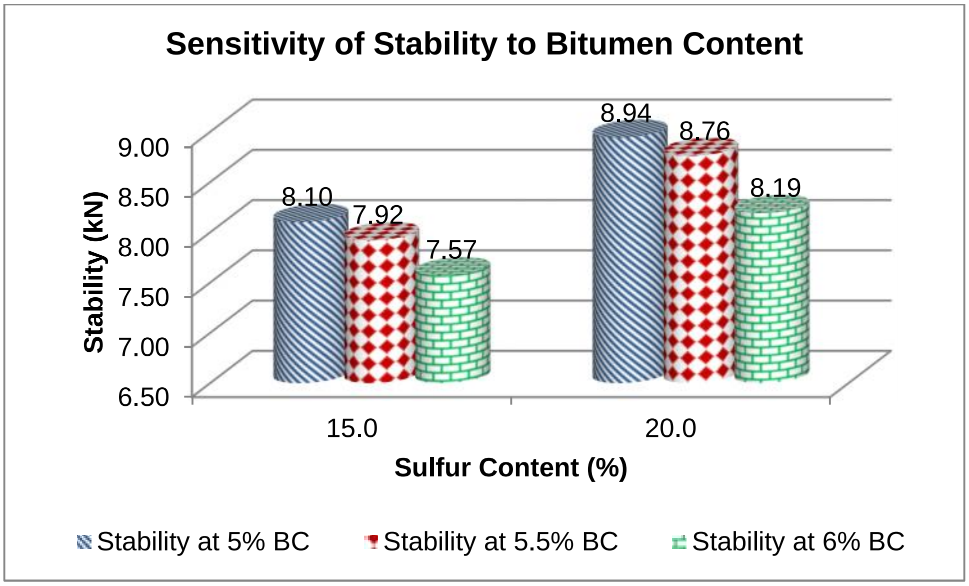


Figure 5.7 Sensitivity of stability of SSB mixes to bitumen content

The flow of the SSB mixtures with 5% bitumen content are all below 2mm at the testing sulfur contents of 5% - 25%. Thus, at 5% bitumen, the flow values are below the acceptable limits, 2mm – 3.5mm, of MS-2 for base course materials. As shown in fig 5.8, the flow values at 5% bitumen content do not follow distinct trend. The sensitivity analysis, carried out at 5.5% and 6.0% bitumen, however indicated that the flow of the SSB mixes has the same trend as the conventional HMA mixes as it increases with increase in the bitumen content. At 5.5% and 6.0% bitumen content, the flows of both of the SSB mixtures with 15% and 20% sulfur are within the target range of 2mm-3.5mm. The sensitivity of SSB mixtures to bitumen content is shown in fig 5.9.

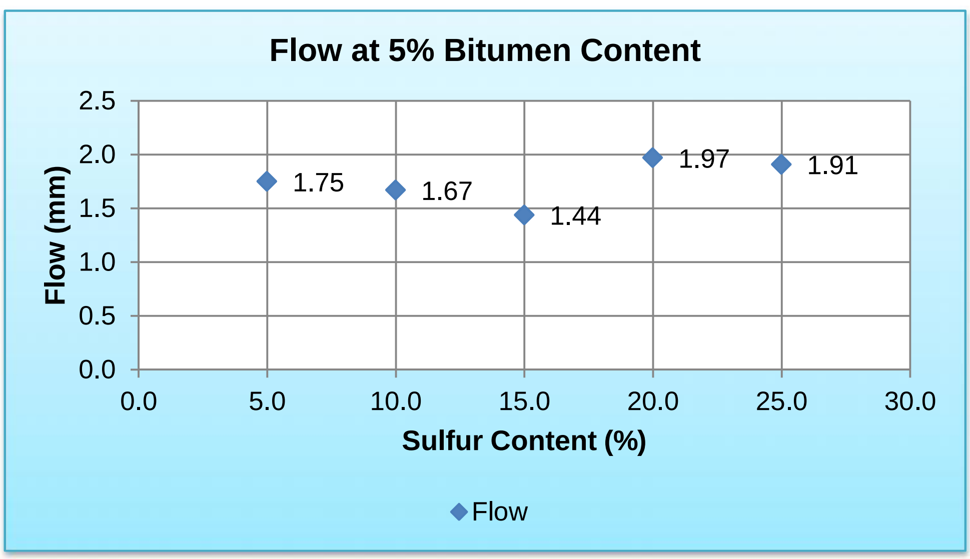


Figure 5.8 Flow values of SSB mixes at 5% bitumen content

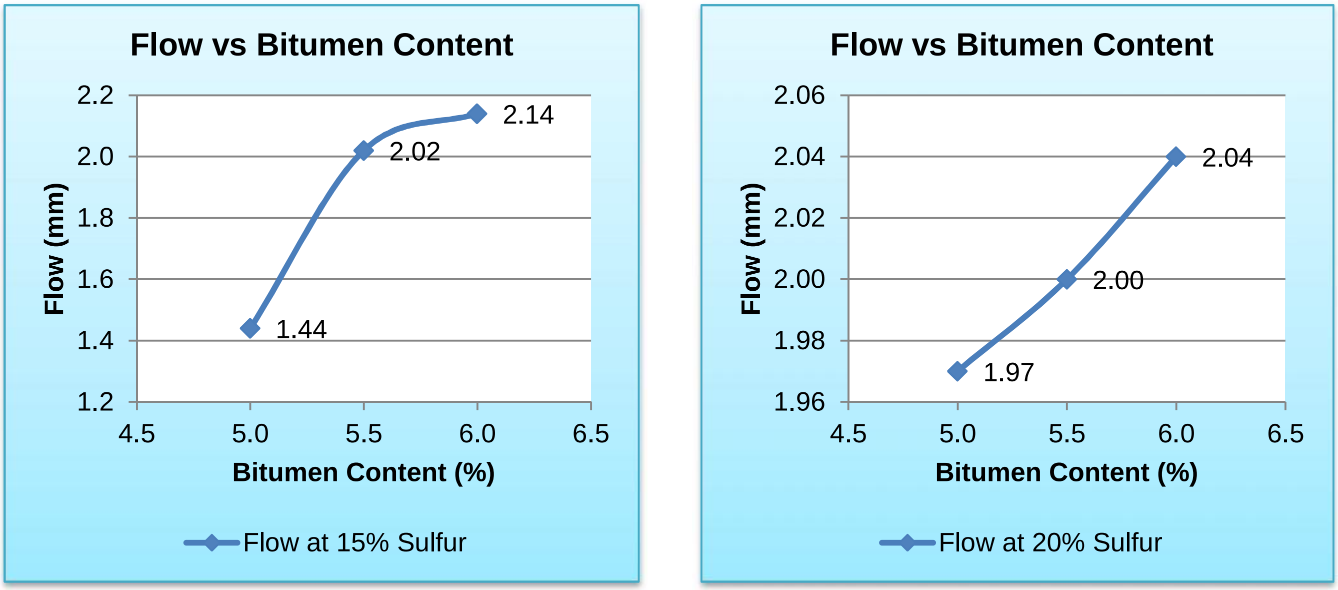


Figure 5.9 Sensitivity of flow of SSB mixes to bitumen content

As stated in the literature review section 2.6.2, there is peculiarity regarding the air void content of SSB mixture. That is even if the SSB mixes have higher air void content, the compacted mixtures have very low permeablilities. This is because the air voids are not interconnected, due to the sulfur network, and hence free movement of water and air through the air voids is restricted.

In the subject case, the air void content of the SSB mix with 5% sulfur content and 5% bitumen content is 17%. This value of the air void is dropped continuously at each successive increments of sulfur contents and reached to a value of 4.2% at 25% sulfur and 5% bitumen contents. This continuous fall in the air void content with incremental binder contents is typical of the conventional HMA mixes. This has happened because the natural sand, utilized in the mixture, contained grains of almost all sieve sizes so that the larger voids within the bigger grains are filled with smaller grains and then with the binders. The mixtures that had been prepared by varying the bitumen content to 5.5% and 6.0% were also exhibit similar trend of decreasing air void with increasing amount of binder content. This property of the mixture is shown in fig 5.10 and 5.11.

In general, two points are clear from the air void analysis:

1. The air void of the SSB mixture in the subject case can be reduced substantially to be within the acceptable band of 3% - 5% by increasing the binder content;

2. The air void contents of the SSB mixture in the subject case is by far less than those mixes described in the literature review at total binder contents above 20%. Thus the mixes in the subject case can have even lower permeability than those mentioned in the literature review.

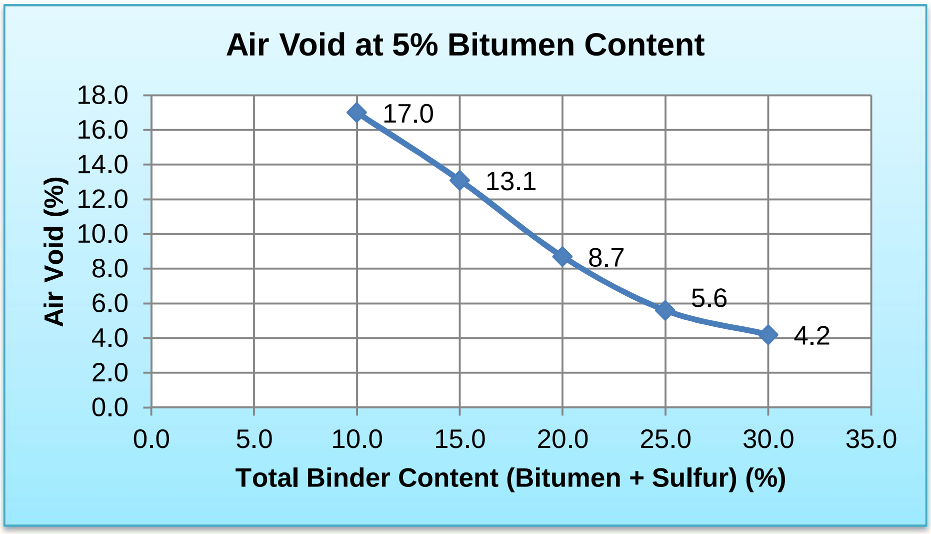


Figure 5.10 The air void of SSB mixes at 5% bitumen content

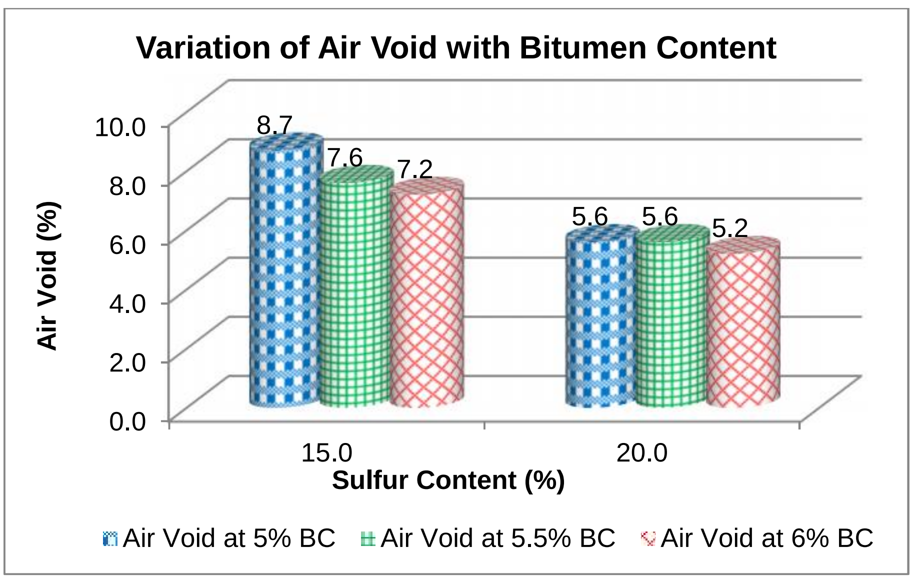


Figure 5.11 Sensitivity of air void of SSB mixes to bitumen content

The void in the mineral aggregate, VMA, values of the SSB mixture at the testing binder contents are well above the MS-2 minimum requirements for 9.5mm nominal size of aggregate. These are 14% at 3% air void, 15% at 4% air void, and 16% at 5% air void contents. At 5% bitumen content, the VMA curve continuously decline with the increase in the amount of sulfur. At the testing binder contents, the VMA curve has the shape of left half of the up ward parabola. This indicates that the binders in the mixes were acting as a lubricant so that the mixes were further densified when the amount of binders were increased. In the subject case, the binder content did not reach its peak value at which it begins to push the grains of the sand apart and increase the VMA within the mix. The VMA values of the mixes that had been prepared at 5.5% and 6.0% bitumen content exceed the corresponding values at 5.0% bitumen content which may be the result of thicker bitumen film thickness in the former cases. The trend of VMA with binder content is shown in fig 5.12, and the sensitivity of the VMA values to bitumen content is presented in fig 5.13.

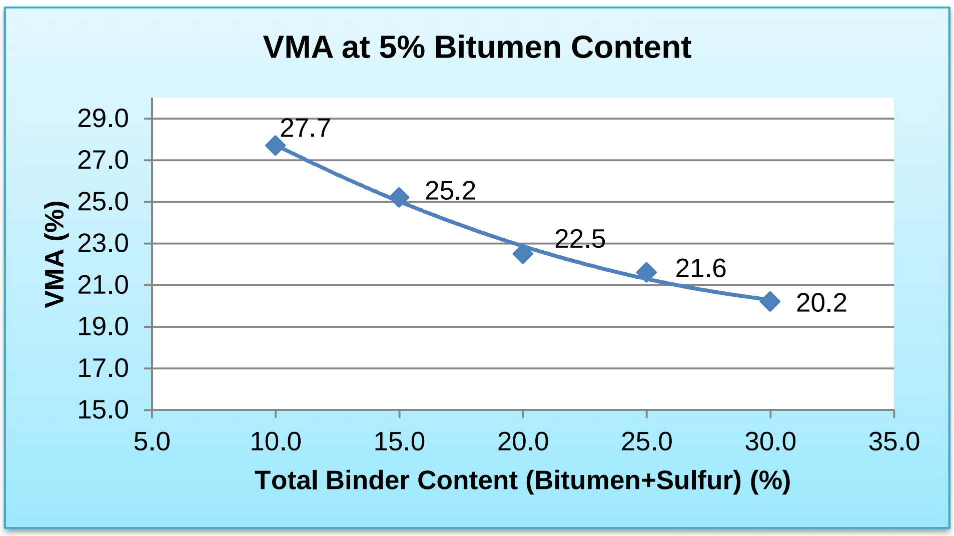


Figure 5.12 The VMA of SSB mixes at 5% bitumen content

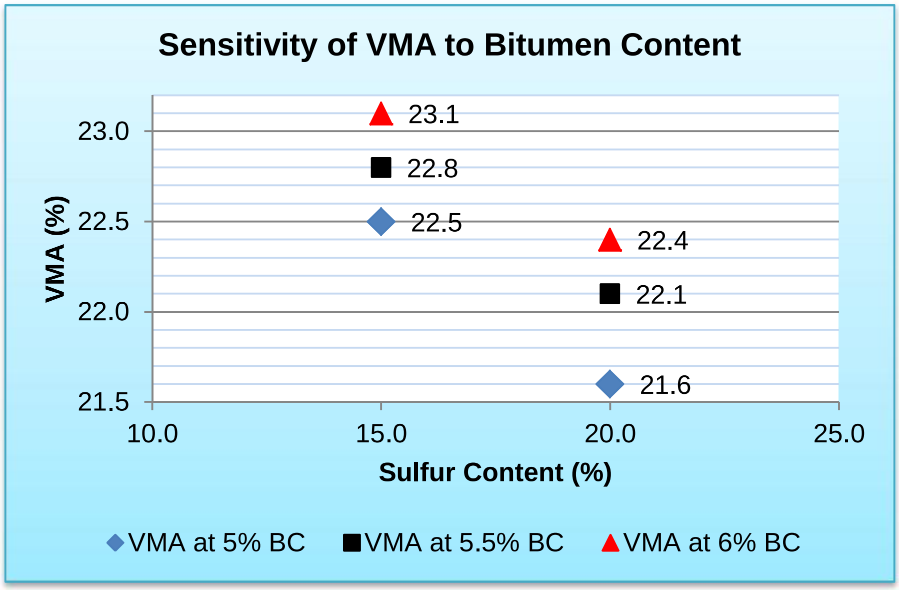


Figure 5.13 Sensitivity of VMA of SSB mixes to bitumen content

The VFB (voids filled with binder) of the SSB mix at 5% bitumen content follow the normal trend, that is it increases with increasing amount of the total binder content (sulfur + bitumen). The reaction of VFB to 5.5% and 6.0% bitumen content is also normal. The VFB requirement for base course, according to MS-2, is satisfied when the sulfur content is 20% and above for mixes with 5% bitumen. However, for mixes with 5.5% and 6.0% bitumen, the VFB criteria can be met at sulfur contents as low as 15%. Figure 5.14 presents the plot of VFB vesrsus total binder content and fig. 5.15 shows the variation of VFB with varying bitumen contents.

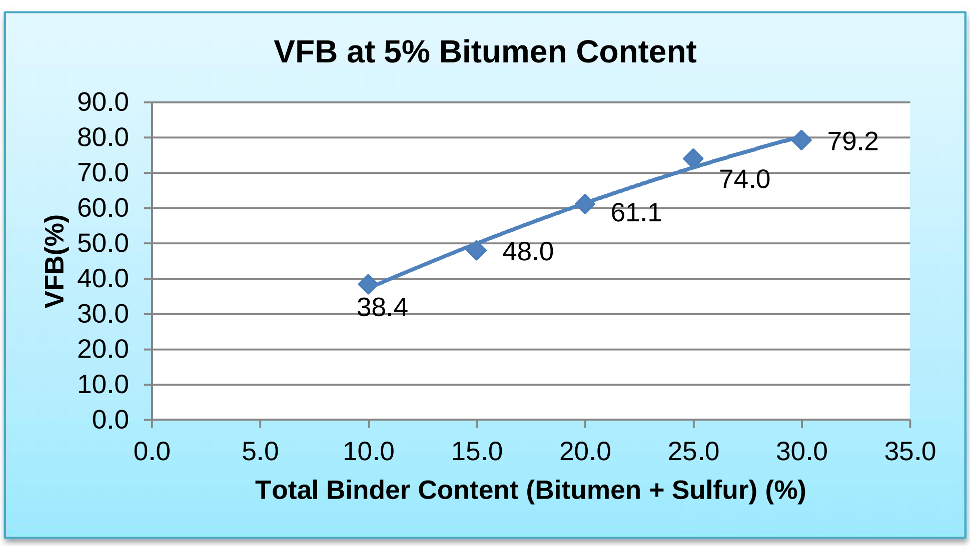


Figure 5.14 The VFB of SSB mixes at 5% bitumen content

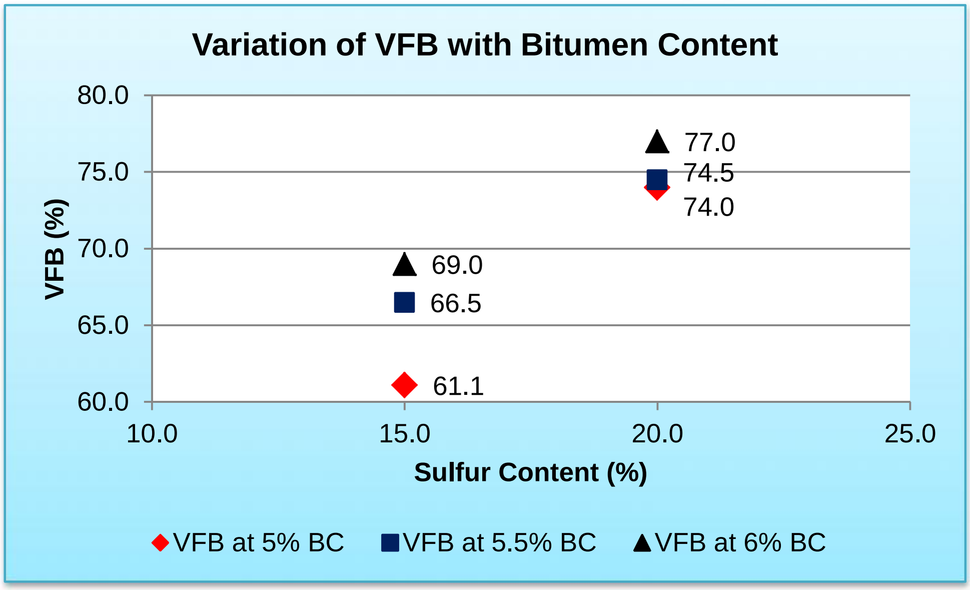


Figure 5.15 Variation of VFB of SSB mixes with bitumen content

The unit weight of the compacted mixtures increases with the total binder content and did not reach to the peak value at the testing binder contents. This condition well matches with the trend of the VMA curve, which did not quit falling at the testing binder contents and the assumption that the binder in the mix is acting as lubricant which helps densification of the mixture. The unit weight of the mixture at 15% sulfur content increased as the bitumen contents of the mixture were increased to 5.5% and 6.0%. However, at 20% sulfur, the unit weight did not increase as the bitumen content increased. The plot of unit weight versus total binder content is shown in fig. 5.16 and the variation of unit weight with bitumen content is presented in fig 5.17.

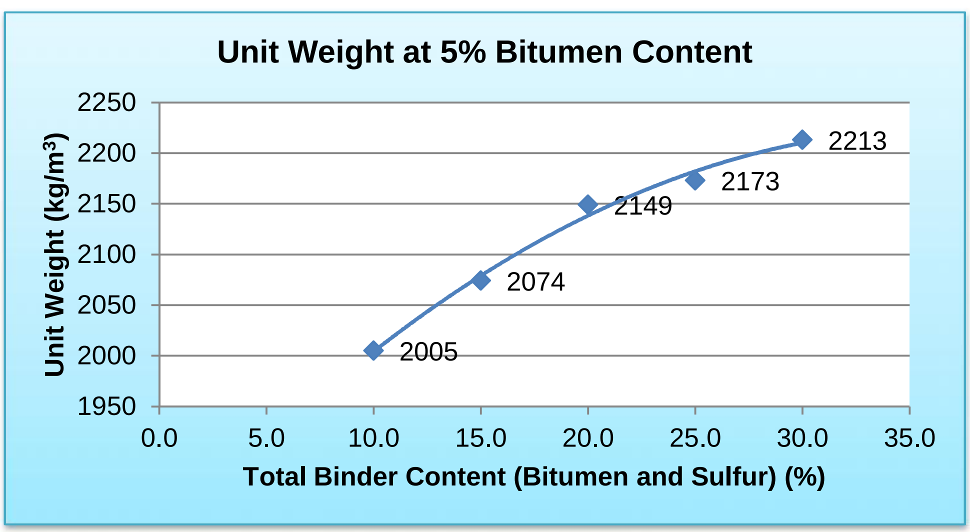


Figure 5.16 Unit weight of SSB mixes at 5% bitumen content

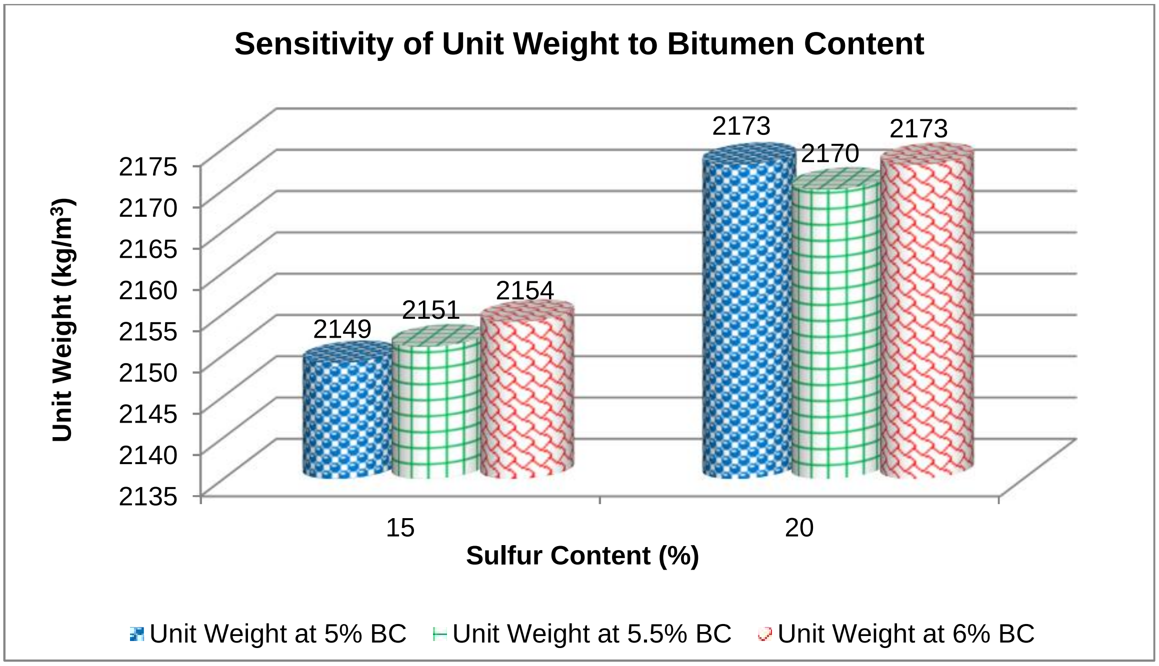


Figure 5.17 Variation of unit weight of SSB mixes with bitumen content

Out of the nine trial SSB mixes, the mixture composed of 74.5% natural sand, 20% sulfur, and 5.5% bitumen satisfy all but the air void requirements of the MS-2 for base course mixes. The air void of this mixture is 5.6%. This value is marginally above the maximum requirement. However, due to the special characteristic of SSB mixes, it can be infered that the coefficient of air permeability of this mix (in compacted state) is below 10^{-8} cm/s, which is the upper margin for mixes to be considered impermeable.

Accordingly the mixture with 74.5% sand, 20% sulfur, and 5.5% bitumen is found to be the best mixture for base course construction in the subject location. Properties of this mixture is tabulated in table 5.1.

Table 5.1 Summary of Properties of 74.5:20:5.5 SSB Mixture

Composition	Stability (kN)	Flow (mm)	Air Void (%)	VMA (%)	VFA (%)	Unit Weight (kg/m^3)
N.Sand - 74.5%, Sulfur – 20%, Bitumen – 5.5%.	8.76	2.0	5.6	22.1	74.5	2170
MS-2 requirement	Min. 8.0	2.0 - 3.5	3.0 – 5.0	Min. 14 - 16	65 - 75	-

5.2 Cement Stabilized Sand

Determination of the optimum binder content (OBC), so that the stabilized soil fulfills both the strength and durability requirements, is the primary objective of the soil-cement stabilization process. Among the various approaches, in this study, the stabilization guidelines and the requirements that had been specified in TM 5-822-14/AFMAN 32-8010 (Joint Departments of the Army and Air Force, USA, 1994) were employed for determining the OBC. This method was chosen because:

i. both strength and durability requirements were specified and the strength criteria is relatively higher than other standards' requirement;

ii. the testing procedures are easy and thus can be performed in ordinary soil laboratory.

5.2.1 Optimum Binder Content

The unconfined compression test results indicated that the unconfined compressive strength (UCS) of the soil-cement specimens increases linearly with the cement content. As shown in table 4.15 and fig 5.18, the UCS values increase from 2.4MPa at 7% cement to 5.9MPa at 10% cement contents. The minimum strength requirement of TM 5-822-14/AFMAN 32-8010 for base course is 5.2MPa. This criteria is fulfilled by the soil-cement mixtures with cement contents of 10% and above. A mixture with 10% cement has a UCS value of 5.9 Mpa.

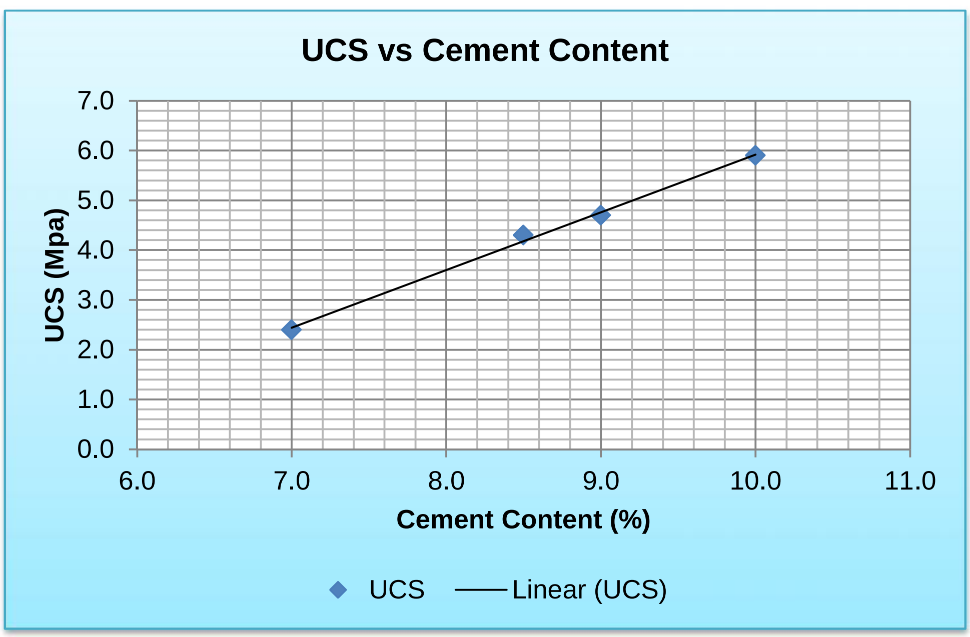


Figure 5.18 Variation of UCS values of soil-cement mixtures with cement content

To compare the strengths of the trial mixes with the requirements of ERA Pavement Design Manual, the UCS values of the trial specimens have to be converted to equivalent value of the 150mm cube specimen. Accordingly, the correction factors set by Road Note 31 were used to derive appropriate correction factor based on the height to diameter ratio of the specimens.

Table 5.2 Correlation factors for Cube and Cylinderical Specimens

Sample Type [Ht.XDia.]	Height/Diameter	Correction Factor
200mmX100mm	2.0	1.25
115.5mmX105mm	1.1	1.04
127mmX152mm	0.8	0.96
116.4mmX101.6mm	1.15	1.05#

 # interpolated factor for the actual sample type

Accordingly, the 150mm cube strengths of the trial mixtures are shown in table 5.3.

Table 5.3 Adjusted Unconfined Compressive Strength

Cement Content [%]	UCS of 116.4mm*101.6mm [MPa]	UCS of 150mm cube [MPa]
7.0	2.4	2.5
8.5	4.3	4.5
9.0	4.7	4.9
10.0	5.9	6.2
ERA PDM Requirement	**CB1**	**3.0 – 6.0**
	CB2	**1.5 – 3.0**

The results in table 5.3 indicate that all of the trial soil-cement mixtures satisfy the strength requirement of ERA Pavement Design Manual for stabilized base course.

The results of the 12-cycle wet-dry test, conducted on specimens with 9% and 10% cement contents showed that the soil-cement losses are 2.7% and 2.0%. These are very low as compared to the allowable maximum limit of 11% which is specified in TM 5-822-14/AFMAN 32-8010.

Depending on the strength and durability test results, the minimum binder content of 10% is required to meet the requirements of TM 5-822-14/AFMAN 32-8010 and 8.5% is needed to satisfy the ERA PDM criteria for CB1. This is summarised in table 5.4.

Table 5.4 Optimum Binder Content

Criteria	Optimum Binder Content [%]
TM 5-822-14/AFMAN 32-8010 of US Army and Air Force	10.0
ERA Pavement Design Manual	8.5

5.3 Economic Evaluation

The economic benefit of the stabilized base was analysed considering utilization of the crushed granular roadbase as base case scenario. The per km cost of the pavement structure with crushed granular roadbase was compared against the same cost of the pavement structures with bituminous and cement-bound roadbases.

For the sake of economic evaluation, four alternative pavement structures, which accommodate T6 [6-10m esa] class traffic load over S4 [8% – 14% dCBR] class subgrade, were selected from structural catalogue of ERA PDM 2013. The T6 traffic class was chosen considering that the development of vast farmlands within the study area induces massive traffic volume, and S4 subgrade class was used based on the assumption that the natural sand - if mechanically stabilized by the overlying clay- can attain a minimum CBR value of 8%, and hence can be used for embankment construction over the flat terrain.

The pavement structures so selected were shown in table 5.5 and table 5.6.

Table 5.5 Pavement Structures for Comparing Granular Base against Bituminous Base

Unbound Granular Roadbase vs Bituminous Roadbase			
Chart C1, ERA PDM 2013		**Chart D1, ERA PDM 2013**	
Subgrade Class: S4		Subgrade Class: S4	
Traffic Class: T6		Traffic Class: T6	
Structural Layer	**Thickness (mm)**	**Structural Layer**	**Thickness (mm)**
Asphalt Concrete	100	Asphalt Concrete	90
Granular Roadbase	200	Bituminous Roadbase	130
Granular Sub base	175	Granular Sub base	150

Table 5.6 Pavement Structures for Comparing Granular Base against Cement-Bound Base

Unbound Granular Roadbase vs Cement-Bound Roadbase			
Chart A1, ERA PDM 2013		**Chart A3, ERA PDM 2013**	
Subgrade Class: S4		Subgrade Class: S4	
Traffic Class: T6		Traffic Class: T6	
Structural Layer	**Thickness (mm)**	**Structural Layer**	**Thickness (mm)**
DBST	-	DBST	-
Granular Roadbase	225	Cement-Stabilized Roadbase, CB2	200
Granular Sub base	275	Cement-Stabilized Sub base, CS	200
		Granular Capping Layer	125

The unit rates for each of the structural layers were computed considering the Materials, Manpower, and Equipment costs that will be invested during material production/purchase, haulage, and placement. While fixing the rates, it was assumed that 500m roadbase is constructed per day for each type of materials (granular or stabilized). The unit rates plugged for each of the base layers and the basis for the determination of the same are discussed afterwards.

5.3.1 Crushed Aggregate Base Course

The unit rate per cubic meter of the crushed aggregate was determined based on the assumption that daily:

- $600m^3$ of quarry rock is produced;
- $320m^3$ of the quarried rock is hauled to the crusher plant, with 75 tonne/hr crushing capacity, which is to be installed at 2km away from the quarry;
- $240m^3$ of rock is crushed;
- $876m^3$ of the crushed aggregate is hauled to the site of placement, which is on average 156km away from the crusher plant;
- $730m^3$ of compacted base layer is placed over 500m stretch.

The cost of each of these activities were estimated based on the latest material, manpower, and equipment rates. The amounts so estimated were increased by 35% to account for the overhead, profit, other contingencies. The unit rate determination for the crushed aggregate base course is detailed in Appendix II-1 and the summary is presented in table 5.7.

Table 5.7 Summary of Unit Rate Determination for Granular Roadbase

Activities	Unit Rate (ETB/m^3)
Quarry rock production	51.20
Quarried rock haulage to crusher	31.47
Rock crushing	407.57
Crushed aggregate haulage to the site	2,642.37
Base course placing	48.14
Total Cost of Granular Roadbase Construction	**3,180.75**

5.3.2 Sand-Sulfur-Bitumen Base Course

The unit rate of Sand-Sulfur-Bitumen roadbase was determined by summing up the materials, manpower, and equipment costs of the activities from material production/purchase to mixing and placing. The rate was estimated based on the assumption that the following activities are performed on daily basis.

- 400m^3 of natural sand is extracted;
- 320m^3 of the extracted sand is hauled to the asphalt batch plant, which is to be located midway along the project stretch. The average hauling distance is taken as 5km;
- 656m^3 of sand-sulfur-bitumen mixture is prepared;
- 656m^3 of sand-sulfur-bitumen mixture is hauled to the site; and
- 525m^3 of the compacted sand-sulfur-bitumen mixture is placed as roadbase

Thirty five percent of the total cost of these activities was surcharged considering overhead cost, profit margins, and other contingencies. The detailed rate determination approach for sand-sulfur-bitumen base course is shown in Appendix II-2 and summary of the cost estimated for each of the activities is presented in table 5.8.

Table 5.8 Summary of Unit Rate Determination for Sand-Sulfur-Cement Roadbase

Activities	Unit Rate (ETB/m^3)
Natural sand production	35.27
Natural sand haulage to the asphalt batch plant	55.82
Sand-Sulfur-Bitumen mix production	26,128.62
Sand-Sulfur-Bitumen mix haulage to the site	69.27
Sand-Sulfur-Bitumen mix placing	72.43
Total Cost of SSB Roadbase Construction	**26,361.41**

5.3.3 Cement Stabilized Sand Base Course

In determining the unit rate of the cement treated sand base course, the in-place mixing of the sand and cement was considered. It was assumed that 1925.4 quintal of cement and 600m3 of sand are hauled to the site to produce 913m^3/day of loose sand-cement mixture, which is then placed to be 730m^3/day of compacted cement treated base course. The rate determination for the cement stabilized base course is detailed in Appendix II-3 and summarized in table 5.9.

Table 5.9 Summary of Unit Rate Determination for Cement Stabilized Sand Roadbase

Activities	Unit Rate (ETB/m^3)
Cement, sand, and MC-250 bitumen purchase/production and hauling	1,011.41
Sand-cement mixture placing and curing	37.13
Total Cost of Cement Treated Sand Roadbase Construction	**1,048.54**

5.3.4 Cost Comparison

Additional unit rate analyses were carried out so as to plug rates for other structural layers of the pavement structures, shown in table 5.5 and table 5.6. These rates are shown in table 5.10 and the analyses are shown in detail in Appendix II.

Table 5.10 Unit Rates of other Structural Layers

Structural Layer	Unit	Unit Rate
Asphalt Concrete	ETB/m^3	9,505.21
Double Surface Treatment	ETB/m^2	120.07
Granular Sub base	ETB/m^3	705.07
Cement-stabilized Sub base	ETB/m^3	917.87
Granular Capping	ETB/m^3	165.27

The costs of the pavement structures that was determined using these rates are shown in table 5.11 and table 5.12.

Table 5.11 Comparison of the Cost of Pavement with Crushed Granular Base against Pavement with SSB Base

Crushed Granular Base vs Bituminous Base (SSB)											
Pavement with Crushed Granular Base						Pavement with Bituminous Base (SSB)					
Structural Layers	Thickness (mm)	Quantity/ km	Unit	Rate	Cost/km	Structural Layers	Thickness (mm)	Quantity/ km	Unit	Rate	Cost/km
Asphalt Concrete	100	700	m^3	9,505.21	6,653,647.17	Asphalt Concrete	90	630	m^3	9,505.21	5,988,282.45
Granular Road base	200	1,460	m^3	3,180.75	4,643,898.82	Bituminous Road base	130	949	m^3	26,361.41	25,016,976.64
Granular Sub base	175	1,313	m^3	705.07	925,410.29	Granular Sub base	150	1125	m^3	705.07	793,208.82
Total Cost/km =					12,222,956.28	Total Cost/km =					31,798,467.91

Table 5.12 Comparison of the Cost of Pavement with Crushed Granular Base against Pavement with Cement-Bound Base

Crushed Granular Base vs Cement-Bound Base											
Pavement with Crushed Granular Base						Pavement with Cement-Bound Base					
Structural Layers	Thickness (mm)	Quantity/ km	Unit	Rate	Cost/km	Structural Layers	Thickness (mm)	Quantity /km	Unit	Rate	Cost/km
DBST	-	7,000	m^2	120.07	840,524.89	DBST	-	7,000	m^2	120.07	840,524.89
Granular Road base	225	1,643	m^3	3,180.75	5,224,386.17	Cement Stabilised roadbase	200	1460	m^3	1,048.54	1,530,872.97
Granular Sub base	275	2,063	m^3	705.07	1,454,216.17	Cement Stabilised Sub base	200	1500	m^3	917.87	1,376,810.94
						Granular Capping	125	962.5	m^3	165.27	159,076.64
Total Cost/km =					7,519,127.24	Total Cost/km =					3,907,285.45

As indicated in table 5.11 the cost of the pavement with bituminous base course is very high as compared to the pavement with crushed granular roadbase. This is due to the expensive cost of the sand-sulfur-bitumen roadbase which in turn is exaggerated due to the expensive cost of sulfur. Sulfur, even if it is cheap byproduct of petroleum, is very expensive in Ethiopia, owing to limited number of manufacturer's and traditional practice of mining and extracting it.

On the contrary, pavement with cement-bound base cheaper than the pavement with crushed granular base. In using pavement with cement-bound base, there is benefit of 48% saving in the pavement cost.

Therefore for the subject study area, it is recommended to utilize cement-treated base when the road is upgraded to a higher standard as there is significant economical advantage without compromising the pavement quality.

6 CONCLUSION AND RECOMMENDATION

6.1 Conclusion

The findings of this research highlighted that the abundantly available sand in study area can be upgraded to serve for road base construction, even for high standard roads. The sand can be stabilized either using bitumen or ordinary portland cement.

As the sand-bitumen mixture is very weak, 15% - 20% of pure sulfur shall be added so that the mixture be qualified for base course construction. The inclusion of ordinary portland cement has hardly any effect in improving the Marshall stability of the sand-bitumen mixture. Even if the sand-sulfur-bitumen mixture satisfactorily fulfills the base course materials criteria, it is the least preferable option for road constructions in the study area due to its excessively high cost. The cost of this mix is exaggerated because of expensive cost of sulfur in the local market.

Cement stabilization is the best option that can be effectively implemented in the research area. The natural sand that is treated with 10% ordinary portland cement can be used for road base construction. This mixture satisfy both the strength and durability requirements that were set by different agencies for base course materials.

To summarize the research findings, the research questions in section 1.3 are answered as follows:

- The answer for the first research question "*Is it possible to use the locally available abundant natural sand for base course construction?*" is **Yes!** The locally available natural sand can be used for base course construction by using either cement or bitumen stabilization techniques.

- The answer of the second question "*Is it possible to stabilize the locally available natural sand with bitumen without additives?*" is **No!** as the stability of the sand-bitumen mixture is very low, this mixture can not be used for base course contruction.

- The answer for the third question "*Is it possible to further increase the stability of the natural sand – bitumen mixture with the help of additives?*" is **Yes** it is possible to increase the stability of the sand-bitumen mixture by using sulfur as an additive.

- The answer for the fourth question "*What composition of bituminous mix optimally fufill the base course requirements?*" is the sand-sulfur-bitumen mixture with 74.5% sand, 20% sulfur, and 5.5% bitumen (the proportions are on weight basis) optimally fulfills the base course requirements specified in American Asphalt Institute's Manual Series, MS-2.

- The answer of the fifth question "*What composition of ordinary Portland cement and natural sand produce materials that meet the road base requirement?*" is the sand-cement mixture with 10% (by weight of sand) portland cement meets both of the strength and durability requirements of base course.

6.2 Recommendation

Based on the results of the research, it is recommended that utilization of the locally available natural sand shall be given due consideration for upcoming road construction projects in the study area or in other locations with similar charcteristics.

Further researches on the following topics are recommended:

1. Stabilization of natural sand for road surfacing construciton

The properties of both of the cement treated sand and sand-sulfur-bitumen mixtures in this paper have given indication that these materials fulfill not only the base course but also the surfacing requirements. Therefore, assessment of the suitability of these materials for road surfacing construction will solve one construction problems in this researche's study area.

2. Evaluation of the performance of cement treated sand roadbase based on full scale tests

Evaluation and affirmation of the performance of the stabilized sand based on the full scale tests will give confidence for the decision makers to allow the application of such materials in the road construction projects. Furthermore, complete and detailed in-service behaviour of the material is also identifed. Thus, performace evaluation based on site trials is recommended for cement treated sand roadbase.

3. Utilization of natural sand for road base and sub-base construction using geocell cellular confinement

Geocells provide confinements to the natural sand and increase the stifness of the system. It is believed that application of geocells reduces the thickness of the pavement structure and saves upto 30% of the pavement cost. Therefore, researches dedicated to the utilization of this material with natural sand for the construction of pavement layers will be of paramount significance.

Figure 6.1 Geocell cellular confinement

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APPENDICES

ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

STABILIZATION OF NATURAL SAND WITH CEMENT, AND BITUMEN AND SULFUR FOR BASE COURSE

By

ESKEDIL ABEBAW MELESE

July, 2014

Approved by Board of Examiners

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