EVALUATION OF COLD ASPHALT PATCHING MIXES

by

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Approved by

Prof. Kim J. Jenkins

DECLARATION

I, the undersigned hereby declare that the work presented in this thesis is my own original work and has not previously in its entirety or in part been submitted for a degree to any other University.

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ABSTRACT

Title: EVALUATION OF COLD-MIX ASPHALT FOR PATCHING
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Cold mixed asphalt concretes consist of bituminous binder, either cutback or emulsion, and aggregates that have not been heated. Cold mix asphalt is often used due to unavailability of hot mix asphalt in the vicinity of the project and also used for temporary patches. The poorer performance of the materials associated with expensive cold mixes will results in greater overall cost for patching due to increased cost of labour, equipment and traffic control.

The main objective of this study was to evaluate performance of proprietary cold mix asphalts available in South Africa. Five products were used in this study, which are Roadfix, Tarfix, Much-Asphalt mix, Asphalt King and Glenpatch. Engineering properties of products were investigated, including volumetric properties, permeability and Indirect Tensile Strength. In addition, for performance properties, accelerated pavement testing using Model Mobil Load Simulator (MMLS3) was carried out. The testing was done dry at 50° C up 20,000 load repetitions.

It was found out that all five products have high void contents which range between 15.1% and 23.5%. This makes these cold mix asphalts to be highly permeable. Indirect Tensile Strength values were found to be very low compared to minimum value of 800 kPa specified for Hot Mix Asphalt. Products with emulsion as binder (Asphalt King and Glenpatch) were found to be more susceptible to water damage compared to other mixes with cutback binder. MMLS3 test results showed that Asphalt King was less susceptible to rutting compared to the other four products. In general all products are very highly susceptible to rutting compared to Hot Mix Asphalt.

SINOPSIS

Titel: EVALUERING VAN KOUE MENGSELS ASFALT VIR HERSTELWERK
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Koue mengsels van asfalt-beton bestaan uit bitumineuse bindmiddel, hetsy versny of emulsie, en aggregaat wat nie warm gemaak is nie. Koue mengsels word dikwels gebruik omdat warm mengsels nie beskikbaar is op ’n projek nie en ook vir tydelike herstelwerk. Die swakker vertoning van materiale geassosieer met duur koue mengsels lei tot groter totale onkoste van herstelwerk as gevolg van verhoogde uitgawe aan arbeid, toerusting en verkeersbeheer.

Die hoofdoelwit met hierdie ondersoek was om die vertoning van kommersiële koue mengsels wat in Suid Afrika beskikbaar is te evalueer. Vyf produkte is in die ondersoek gebruik, naamlik Roadfix, Tarfix, Much-Asphalt mix, Asphalt King en Glenpatch. Die ingenieurseienskappe van die produkte wat ondersoek was, het volumetriese eienskappe, deurlatendheid en indirekte treksterkte ingesluit. Verder is die gedragseienskappe geevalueer deur versnelde toetsing met behulp van die Model Mobiele Lasnabootser (MMLS3) uit te voer. Die toetse is onder droë toestande uitgevoer teen 50^0C en tot 20,000 siklusse is aangewend.

Daar is bevind dat al vyf produkte hoë ruimteverhoudings gehad het wat gewissel het tussen 15.1% en 23.4%. As gevolg hiervan was die asfalt-mengsels hoogs deurlatend. Indirekte treksterkte was baie laag in vergelyking met ’n gespesifiseerde minimumwaarde van 800 kPa vir warm mengsels en produkte met emulsie as bindmiddel (Asphalt King en Glenpatch) was meer vatbaar vir waterskade as die mengsels wat versnyde bitumen bevat het. MMLS3 toetse het getoon dat Asphalt King minder geneig tot spoorvorming was as die ander vier mengsels. Oor die algemeen was al die mengsels baie meer vatbaar vir spoorvorming as warm asfalt-mengsels.

I would like to dedicate this thesis to
my parents, Mathew and Margaret Munyagi;
my brothers, Gaspar and Edmund

I'm grateful to have you as part of me

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LIST OF SYMBOLS AND ABBREVIATIONS

APT	Accelerated Pavement Testing
BRD	Bulk Relative Density
HMA	Hot Mix Asphalt
ITS	Indirect Tensile Strength
kPa	kilo Pascal (1 kPa = 1 x 10^3N/mm^2)
LVDT	Linear Variable Displacement Transducer
MMLS3	Model Mobile Load Simulator Mk3
MPa	Mega Pascal (1 MPa = 1 x 10^6N/mm^2)
NMAS	Nominal Maximum Aggregate Size
P200	Particle Passing Sieve No. 200
PennDOT	Pennsylvania Department of Transportation
RD	Rut Depth
SANRAL	South Africa Road Agency Limited
TMH1	Technical Methods for Highways (Standard Methods of Testing Road Construction Materials
TSR	Tensile Strength Ratio
VTM	Voids in Total Mix

Chapter 1

INTRODUCTION

1.1 BACKGROUND

1.1.1 Potholes

Potholes and surface repair of asphalt pavement is one of the most commonly performed maintenance operations for most highway agencies; especially in areas where cold winters and warm, wet springs contribute to accelerated, perpetual pavement break-up every year (Estakhir and Button, 1997).

Potholes are bowl-shaped holes of various sizes in the pavement surface. These are generally a secondary form of distress that develops from cracking or extreme loss of aggregates. They are traffic induced and normally develop from structural cracking in the wheel paths. Moisture ingress into the pavement layers can result in the total loss of the structural capacity of the pavement and in the formation of the potholes (TMH 9, 1992).

Potholes may also be created in freeze/thaw situations, when water in the pavement or the base layer freezes. The expansion of the base layer tends to push the pavement or the base layer upwards and the swelling expansion forces can cause pavement to weaken resulting in potholes. Potholes are not just a nuisance for drivers but also they constitute to hazards that can produce substantial damage to vehicles and force drivers to veer suddenly in traffic or lose control of the vehicle after contact. (Better Roads, 2004).

1.1.2 Patching

Patching is the common method used to repair potholes. Patching can be described as the filling of deteriorated areas on a road to keep traffic moving safely or to prevent rapid deterioration of an area that could become unsafe. Miller and Bellinger (2003) described

patching as portion of pavement surface, greater than $0.1m^2$ that has been removed and replaced or additional material applied to the pavement after original construction.

Patches can either be full depth, which means the patch can go down to the subgrade or an intact subbase layer or partial depth which means the patch is only on asphalt surface.

Patching may be done on an asphalt pavement either for maintenance purposes or rehabilitation purposes. Maintenances patching is convenient and can be used as temporary repair and sometimes done in cold weather, with cold-mixed patching mixtures and without careful construction procedures. As a result, maintenance patches generally do not exhibit the same long-term stability and durability as hot mixed patching mixtures, carefully constructed as part of a rehabilitation strategy.

1.2 PROBLEM STATEMENT

For years there have been many proprietary cold asphalt patching materials on the market that claim to be "wonder products" or the "perfect" fix to the repair of asphalt concrete pavement potholes. Nevertheless, these products do not perform anywhere close to their claims. And, also it is difficult to determine the quality of product by looking at the product literature. Today, more Municipalities and other users of cold-mix asphalt patching materials are looking for suppliers/manufacturers who offer an added measure of quality.

The primary problems with cold asphalt patching materials can be poor workability in stockpile/storage and on road, moisture susceptibility as evidenced by ravelling and stripping, poor stability and overall inconsistent behaviour during mixing preparation and application according to standard specifications and guidelines (Estakhri and Button, 1997). There have been several studies around the world which have contributed to some solutions of these problems. This is including SHRP project (Evans et al, 1992), which is said to be most extensive pavement maintenance experiment ever conducted. The most recent works on these mixes, known to author at time of writing are studies done by Pappagiannakis et al (2004) and Chatterjee et al (2006). As per author knowledge at time

of writing, there has not been any intensive research on the cold asphalt patching materials in South Africa.

1.3 OBJECTIVE OF THE STUDY

The primary objective of this study is to evaluate the performance of the commonly used cold mix asphalt used for patching of potholes in South Africa. The study investigates engineering and performance properties of five cold asphalt patching mixes available in the market.

There are areas of distress which can result in loss of performance, these are; fatigue cracking, rutting/permanent deformation, thermal cracking, friction, moisture susceptibility and ravelling. Among these distresses, rutting/permanent deformation is probably the most important property to be controlled during mix design and QC/QA. This is due to the fact rutting is most likely to be a sudden failure as a result of unsatisfactory asphalt mix (Brown et al, 2001) and typically result in the need for major repairs whereas other distresses take much longer to develop.

Rutting of the asphalt mixes can be evaluated by using various test methods, which have been used in the past and are being used currently. Few of these methods are; uniaxial and triaxial tests, diametral tests, Marshall tests, Hveem tests and simulative tests. Simulative tests can be done by using Asphalt Pavement Analyser (APA), Hamburg Wheel-Tracking Device (HWTD), French Rutting Tester (FRT), Model Mobile Load Simulator (MMLS), Dry Wheel Tracter (Wessex Engineering) and Rotary Loaded Wheel Tester (Rutmeter). In this study, to evaluate rutting responses of the mixes, simulative tests were done by using MMLS3.

1.4 SCOPE AND EXTENT OF THE STUDY

The scope and extent of the study includes literature review, laboratory testing and MMLS3 testing on five cold asphalt patching mixes. Results from this study will provide some insight on how these mixes perform or how will they perform in the field.

Chapter 2 presents a literature review, which covers materials and design consideration of the mixes, repair techniques, labour efficiency and implications of these mixes on environment. Chapter 3 explains laboratory testing for volumetric and engineering properties. This chapter describes materials used, test methods, specimens preparation, results and discussion of the results of the engineering tests done. Chapter 4 explains performance tests done by using MMLS3; the chapter explains short background of MMLS3, test set up, results and discussion of results. Chapter 5 completes the report with summary of the findings, conclusions and recommendations for further works.

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Chapter 2

LITERATURE REVIEW

2.1 INTRODUCTION

Cold mixed asphalt concretes are becoming common asphalt pavement repair materials. Cold mix asphalt is often used due to unavailability of the hot mix asphalt in the vicinity of the project and also used for temporary patches. The performance of the cold mix patching materials has been generally unsatisfactory (Kandhall and Mellot, 1981). The poorer performance of the materials associated with expensive cold mixes results in greater overall cost of patching due to increased cost of labour, equipment and traffic control.

Many highway agencies including SANRAL are more concerned about evaluation of the effectiveness of these patching materials and techniques that can lead to more economical and long-lasting solutions. This literature review will be focusing on the existing information regarding cold mix patching materials and their design consideration, repair techniques and labour efficiency, plus their implications in the environment.

2.2 MATERIALS

2.2.1 Aggregates

Since the cold mix patching materials will be subjected to the same environmental and traffic associated stress as hot mix asphalt, it would appear that the aggregate quality specifications should be stringent the same for all type of mixes. The type and quality of aggregates that are used for patching mixtures are the same as for normal bituminous construction; including crushed stones and gravels, natural sands, stone sands and mineral fillers. Aggregates properties critical to good patching mixtures are grading, maximum

size of aggregates, angularity, and shape of aggregates, surface texture and compatibility with the binder.

2.2.1.1 Grading

Ganung and Kloskowski (1981) found that aggregate grading plays major role in the performance of bituminous patching mixture. In addition, they concluded that an ideal bituminous patching material should consist of an open grading with 9.5mm maximum aggregate size and less than 2% fines.

In the research done by Anderson et al (1986), it was also found that aggregate grading and aggregate crush count are important factors in determining mixture stability. The research further showed that an open-graded crushed aggregate mix with less than 2% passing No. 200 sieve and with maximum particle size of 9.5mm is required for an optimum mix.

Anderson et al (1988) also found that aggregate must contain crushed angular particles and a maximum of 1% to 2% fine and the maximum aggregate size must be less than 13mm. An open graded mix provides sufficient space for thick binder films that contribute to workability and water resistance.

The single-size grade mix is very course aggregate and contains very little material finer than the 4.75mm sieve. In single-size aggregate patching mixtures, strength is obtained primarily through aggregate interlock like macadam, with the binder used essentially as a water-proofed. This grading has reasonable workability. It cures fairly fast after compaction, because it contains a large amount of voids, which allows relatively quick escape of the volatiles (NCHRP, 1979, 64). Table 1 is an example of typical grading of one-size mix:

Table 1: Grading of Single-Size Graded Patching Mixtures (NCHRP, 1979)

Sieve size (mm)	Percent Passing	
	A	B
19.0	100	
12.5	90 - 100	100
9.5	60 - 90	95 - 100
4.75	0 – 25	35 - 65
2.36	0 – 15	0 - 25

A finer dense grading will not have good workability and can reduce adhesiveness of the mix. However, if it is made of predominantly of single-sized aggregate the problem is reduced. Kandhal and Mellot (1981) has mentioned the advantages of a grading consisting of 100% passing the 9.5- or 4.75mm sieve, these are as follows:

a. The mix is pliable and workable
b. Due to increased surface area, more bituminous binder can be incorporated into the mix to improve the durability.
c. The mix remains pliable for a prolonged period of time and continues to densify easily under traffic and will continue to adapt to the changing geometry of the pothole. This characteristic enhances its chances of survival.

2.2.1.2 Maximum size of stone

The majority of all patching mixtures have a 9.5mm or 12.5mm maximum aggregate size. Limiting maximum aggregate size optimizes workability. Large maximum-size aggregates are preferred when good stability is desired and when they can be used in the hole to be patched. This means that a 19mm aggregate must be used in a hole that is at least 38 to 50mm deep or deeper. When used in shallow holes, the large aggregate must be raked out of the mixture during finishing having a smooth patch. In such instances, the contribution of the large aggregate to the stability of a mixture is lost.

When shallow holes are to be filled, small top-size aggregates should be used. Even finer mixtures are needed when the mixture must be feathered at the edges of the hole. For such thin edges, a sand mixture is preferred.

Ideally, two size of patching should be available, a large size aggregate to fill the deeper hole and a small size aggregate mixture for shallow holes and feathered edges. This is not usually done because the maintenance forces want to handle only one patching mixture. The mixture normally used is a compromise, with an average maximum aggregate size of 9.5mm. This mixture is stable, can be used satisfactory in shallow holes, and can be feathered easily with little loss of large aggregates.

2.2.1.3 Shape and Surface texture

Shape and surface texture of the aggregates influence the workability of the mixture. Angular, rough-surfaced aggregates will reduce workability. Preferred patching mixtures often are composed of angular, rough-surfaced, coarse aggregate and rounded natural sands. In these mixtures, the angular coarse aggregate contribute to the stability, while the smoother rounded natural sands help to improve the workability.

Kendhal and Mellot (1981) argue that angular crushed-stone aggregate is an ideal material for the patching mixtures and when the finer single-sized grading is used, the effect of aggregate angularity on the workability of the mix is minimal.

2.2.1.4 Other aggregate characteristics

In asphalt concrete mixtures, the binder and aggregates must be compatible. For patching mixtures, stripping is a much greater problem than in hot mix asphalt, so greater importance is placed on designing for compatibility.

If economically feasible, selected aggregates should be compatible with the binder. But when satisfactory aggregates are not economically available, another binder may be used

that is compatible with locally available aggregates. More likely, anti-stripping agents will be used, and no change will be made in the aggregates.

If highly absorptive aggregates are used in the mix, problems of stripping and drainage will be increased in the stockpile. Also the mix will need longer time to cure since the highly absorptive aggregates need more binder.

It is very important to keep the dust content (minus 200 fraction) in the mixture as low as possible to impart tackiness. This would significantly improve the adhesive and cohesive properties of the mixture (Kendhal and Mellot, 1981). They have also suggested that the aggregate water absorption should be limited to approximately 1%.

2.2.2 Bituminous Binders

Bituminous binders in the cold mix patching materials must be able to maintain workability without the aid of warm temperatures above ambient. It must also be able to increase its consistency quite rapidly once the mixture is compacted and it should have self-adhesiveness. There are two types of bituminous binders that are commonly used in mixing of cold mix patching materials; these are bitumen emulsion and cutback bitumen.

2.2.2.1 Bitumen Emulsions

Emulsions are made up of two components with one dispersed through the other. The dispersed component is not soluble in the continuous component. Bitumen emulsions are normally of the oil-in-water type. The bitumen is dispersed throughout the continuous water phase in the form of discrete globules, typically 0.1 micron to 5 microns in diameter, which are held in suspension by electrostatic charges stabilized by an emulsifier. (Shell Bitumen, 1990)

The bitumen content depends on the intended application of the emulsion, but is rarely lower than 30% or higher than 70%. How course or fine the emulsion is, depends on the

method of manufacture, the bitumen, the emulsifier and the storage and handling. Figure 1 below shows example of coarse and fine bitumen:

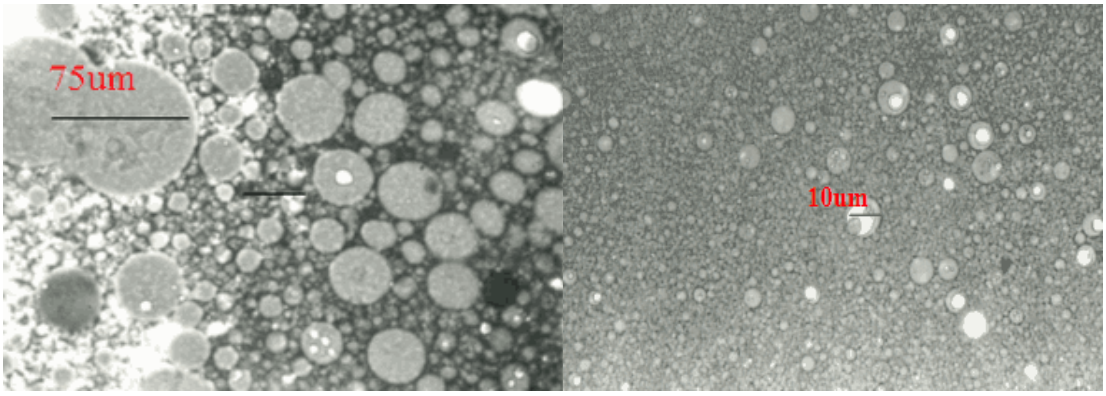

Figure 1: Coarse bitumen emulsion and Fine bitumen emulsion

There are four classes of bitumen emulsions, which are; anionic emulsion, cationic emulsion, non-ionic emulsion and clay-stabilized emulsions. For anionic emulsions, the emulsifier is of anionic type and bitumen droplets are negatively charged while for cationic emulsions, the emulsifier is of cationic type and bitumen droplets are positively charged (Shell Bitumen, 1990).

The bitumen in non-ionic emulsions is neutral. These types of emulsions are sometimes used when extremely stable emulsions are required, primarily for cold mixes containing large quantities of fines (Akzo Nobel, 1999).

Clay-stabilized emulsions are mostly used for industrial applications such as roofing and underbody sealing and use activated clay and bentonites as the emulsifier system. They are essentially anionic emulsions.

Emulsifiers of both the cationic and anionic type are based on salts of fatty long chain molecules, these may be synthetic or derivatives of fatty acids as found in oils and fats. The emulsifier ion orients itself on the surface of the bitumen droplet such that the hydrocarbon chain is firmly bound in the bitumen with the ionic portion located at the surface (Shell Bitumen, 1990).

Anionic emulsifiers are based on fatty acids; these are reacted with a base such as caustic potash or caustic soda (KOH or NaOH) to form a salt. This salt is the active emulsifier. Cationic emulsifiers are based on acid salts of amines prepared from fatty acids. These may be fatty diamines, fatty quaternary ammonium compounds or ethoxylated derivatives. The type of emulsifier determines the number of charges that are on the surface of the bitumen.

The stability of the bitumen emulsion is characterized by how well it is dispersed, the surface charge and the size of bitumen particles. The stability mechanism for bitumen emulsions are settlement, flocculation, coalescence and inversion (water entrapment).

Settlement of an emulsion determines how long it may be stored. Settlement in an emulsion is due to the gravity force and the difference between the two phases. Even if the difference is small, settlement may still take place if the emulsion contains big droplets and the bitumen content is lower than 65% (Akzo Nobel, 1999). Stokes' Law described the rate of settlement as proportional to the density of the dispersed phase and square of the particle radius. The rate of settlement decreases with reducing particle size as a squared function, i.e. if the particle size is halved the settlement will reduced by a factor of 4.

Flocculation is a process where the droplets start adhering to each other. When two particles approach each other several types of interactions may occur. There are two main ways that the colloidal interactions influence flocculation. The first is collision efficiency which is the probability that a pair of colliding particles will form an aggregation and the second is the strength of such aggregations. Flocculated particles act as if they are larger particles and settle faster as per Stokes law.

Coalescence is a process where bitumen droplets in an emulsion merge to form bigger droplets. The coalescence of flocculated particles is a function of the surface charge, shearing and temperature. Coalescence can be started because of mechanical action such as agitation, pumping or vibration. Coalescence occurs in the breaking process and is dependant on the aggregate type (Akzo Nobel, 1999).

In inversion the particles of the emulsion come together so quickly that the water is trapped. This occurs in only rapidly coalescing system, especially very high binder emulsions.

The emulsion should be stable during storage and transport but when applied to mineral aggregate or pavement surfaces, it should break at a predetermined rate. Emulsion break by destruction of the double layer. The main breaking mechanisms are flocculation/coalescence and aggregate interaction, the later is important since it describe the chemical interaction between the emulsion and the aggregate.

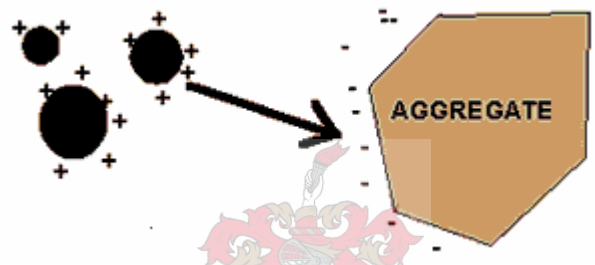

Figure 2: Reaction with the stone

The rate of breaking is mostly influenced by type and content of emulsifier; other factors are type of aggregate, temperature and other climatic conditions.

The cure of bitumen emulsion is often confused with break. Cure is simply the loss of water from the emulsion and the mix. The rate of curing is dependent on water content, rate of evaporation and the diffusion of water through the curing binder. In systems with strong energy differences between the aggregate surface and the emulsified binder, an extra driving force is present to push water away from the aggregate/ binder interface. Cement is often used to enhance this.

2.2.2.2 Cutback Bitumen

Cutback bitumen is the bitumen that has been liquefied through blending with petroleum solvents (diluents). This is done so as to reduce the viscosity of the bitumen for

application at lower temperatures. Upon application to aggregate or pavement, the solvents evaporate, leaving the bitumen residue on the surface.

Cutback bitumen is graded by the type and amount of solvent used to liquefy them. They are divided into three types, which are:

- Rapid curing cutback bitumen (RC), which are produced by blending bitumen with a naphtha-type solvent (diluents of high volatility). They are used primarily for tack coat and surface treatment.
- Medium curing cutback (MC), these are produced by blending bitumen with a kerosene-type solvent (diluents of intermediate volatility). These are used for prime coat, stockpile patching mixture and road mixing operations
- Slow curing bitumen (SC), they are made by blending bitumen with diesel or other gas oils (oils of low volatility). They can also be made by reducing the crude oil directly to grade by distillation. They are used for prime coat, stockpile patching mixtures and as dust palliatives.

Roberts et al (1996) have mentioned the reasons, which make emulsion bitumen to be increasingly used in lieu of cutback bitumen. These are as follows;

- Emulsions are relatively pollution free. Unlike cutbacks bitumen there are relatively small amounts of volatiles to evaporate into the atmosphere other than water.
- Loss of high energy products. When cutback bitumen cures, the diluents which are high energy are wasted into the atmosphere.
- Emulsions are safe to use. There is little danger of fire as compared to the cutback bitumen, some of which have very low flash points.
- Emulsions can be applied at relatively low temperatures compared to cutbacks, thus saving fuel costs. Emulsions can also be applied effectively to damp aggregates, whereas dry conditions are required for cutbacks.

2.2.3 Additives

Additives are mostly used in bituminous patching material to increase resistance to stripping or to improve workability at low temperatures. These additives may be liquid or fine powder (NCHRP, 1979) and they can be are either hydrated lime or anti-stripping agents.

Additives are frequently used to provide extra water resistance and increase the life of the patch. This is so, since patching materials are subjected to extensive, vigorous water action. The stockpile patching mixture is more pervious than dense-graded HMA and thus more susceptible to severe weather and traffic effects.

The anti-stripping agents are very important part of the formulation of the patching materials. A mixture should retain its coating in the stockpile or during storage under adverse weather conditions, during handling, and in the pothole after placing (Kandhal and Mellot, 1981).

2.3 DESIGN OF COLD MIX PRODUCTS

There is no accepted mix design procedure for cold-mix stockpiled patching materials (Anderson et al, 1988). Kandhal and Mellott (1981) reported challenges in designing stockpiling patching mixtures as the properties required in stockpiling and handling and after material is placed in the pothole are contradictory. Some of the contradictory were reported as follows;

For good mixture workability, an open grading is desired while after the mix is placed in a pothole, a denser grading is needed to improve durability. Also, to get good workability, angular shaped aggregates should be avoided, however once the mix is in place, a high degree of angularity is desirable for better stability. Lower binder viscosity is desired for storageability and workability, but after placement higher viscosity is desirable as soon as possible for better cohesion of the mix.

A study done by Anderson et al (1988) reported the criteria for designing a cold-mix stockpiled patching mixtures as follows:

- First, establish the maximum allowable binder content so that the mix will not drain excessively
- Second, ensure adequate low temperature workability by means of a workability test conducted at the lowest mix temperature expected in the field
- Third, ensure water resistance

A summary of the design considerations required with cold mixes is given in the Table 2. In addition, worker safety, environmental implications and cost must be considered (Anderson et al, 1988)

Table 2: Design Considerations for Cold Mixes (Anderson et al, 1988)

S/N	Design Consideration	Effect on Mixture
1	Binder Consistency (before and during placement)	• Too stiff may give poor coating during mixing • Too stiff makes mix hard to shovel, compact • Too soft causes drainage in stockpile or hot box • Too soft may cause stripping in stockpile • Too soft may contribute to tenderness during compaction
2	Binder Consistency (after placement)	• Too soft accelerates stripping, moisture damage in service • Too soft accentuates rutting, shoving • Too soft may lead to bleeding, which causes poor skid resistance • Must cure rapidly to develop cohesion • High temperature susceptibility causes softening and rutting in summer
3	Binder Content	• Maximize to improve workability • Excess causes drainage in stockpile or hot box • Excess may lower skid resistance (bleeding) • Excess may cause shoving and rutting • Low binder content gives poor cohesion
4	Anti-stripping Agent	• Correct type and quantity may reduce moisture damage
5	Aggregate shape and texture	• Angular and rough aggregate gives good resistance to rutting and shoving but it is hard to work • Rounded and smooth gives good workability but poor resistance to rutting and shoving

6	Aggregate grading	• Reduced fines improves workability • Excess fines can reduce “stickiness” of mix • Coarse (>12.7mm) mixes are hard to shovel • Open-graded mixes can cure rapidly but allow water ingress • Well-graded mixes are more stable • Dirty aggregate may increase moisture damage • Too dense a grading will lead to bleeding or thin binder coating, and a dry mixture with poor workability • Open or permeable mix may be poor in freeze-thaw resistance
7	Other Additives	• Short fibers increase cohesion, decrease workability

2.3.1 Binder Selection

The selection of the proper type and grade of bituminous material to use in the mixing is most important. First consideration should be given to the type and grade performing most satisfactorily with aggregate s and traffic conditions similar to those on project under study (The Asphalt Institute, 1977).

Most stockpiled patching materials are produced with cutback bitumen. However, much of the solvent remains with the material for a long time, thereby imparting some flexibility to the patch and resulting in relatively slow gain in stiffness. Bitumen emulsion is sometimes used as alternatives to cutback bitumen. Emulsions may be modified with surfactants to produce high-float products which exhibit thixotropic characteristics. A thixotropic emulsion forms a gel when static but becomes thin upon stirring or shearing. This thixotropic attribute permits the retention of much thicker films of residual bitumen and aids in workability since the bitumen becomes more fluid as it worked.

The PennDOT 485 (2003) specifications have specified the type of the bituminous materials that shall be used for bituminous stockpile patching material. Table 3 below shows the type of material to be used and the time of the year;

Table 3: Type of material to be used and the time of the year (PennDOT, 2003)

Class of Material	Type of Material	Time of the Year
MC-400	Cutback Petroleum Asphalt	November 1 to March 1
MC-800	Cutback Petroleum Asphalt	March 1 to October 31
E-10	Emulsified Asphalt	Throughout the year
CMS-2s	Cationic Emulsified Asphalt	Throughout the year

There are mainly three factors which influence the selection of the appropriate binder for cold-mix patching materials, these are; consistency, curing rate and characteristics of residue.

2.3.1.1 Consistency

Viscosity of the bitumen binder varies with temperature, becoming more viscous as temperature decreases. The binder used in cold mixes must readily be workable at temperature encountered during construction, thus ambient temperature is a key factor in selecting binder of a viscosity that will produce desirable, uniform mixtures (The Asphalt Institute, 1977).

Another factor that contributes to the consistency of the binder selected is the grading of the aggregates. Open-graded mixes require higher viscosity binder which will reduce drainage problems of the binder from the aggregate particles.

The low viscosity binders are required for good workability during storage and placement but they contribute to binder drainage problems. Resistance to drainage is the function of the consistency of the binder. The ideal binder must remain soft and flexible at low temperatures, during placement and compaction, and set-up rapidly after compaction.

The study by Anderson et al (1988) reports that an ideal binder is one, which has low temperature susceptibility, will shear thin during working and will cure rapidly after placement without excessive hardening.

2.3.1.2 Curing Rate

The bituminous material present in the patching mixtures has the most influence in the time available between mixing and the use of the patching mixture (NCHRP, 1979).

The relative curing rates of the cutback or emulsion bitumen are affected by the quantity of applied bitumen, the prevailing humidity and wind, rain and the prevailing range of ambient temperatures of the area during application.

Dense-graded mix requires relatively longer time during mixing in order to coat the particles thoroughly and therefore they should be mixed with fairly slow curing bitumen binder. On the other hand, an open-graded mixes, requires less time for thorough mixing and they may be mixed with a faster curing bitumen binder (The Asphalt Institute, 1977).

2.3.1.3 Characteristics of Residue

The compacted patching mixture should have high cohesive strength in the pothole. This strength is related to the viscosity of the residue of the cured binder. The most desired cold mix patching material is the one with good workability, i.e. should have slow curing rate and at the same time the binder residue needs to have high viscosity for a strong cohesive mixture. These requirements are generally too onerous for currently available materials.

2.3.2 Aggregate Selection

The study done by Ganung and Kloskowski (1981) concluded that aggregate grading plays a major role in the performance of bituminous patching mixtures. Aggregates can control workability and stability as well as influence all other properties in the mixture. Aggregates with proper grading, shape and other characteristics should be very carefully selected (NCHRP, 1979).

2.3.2.1 Workability and Stability

For good workability, an inter-mediate graded aggregate is often selected, whereas a dense-graded aggregate is more often chosen when stability is of major consideration. A different choice has to be made in selecting the shape of aggregates in cold-mix patching material. For a good workability, the very angular aggregates are avoided, yet the good angular, rough-surfaced aggregates are desired for better stability (NCHRP, 1979).

2.3.2.2 Curing

The amount of air voids in a compacted mixture influences the rate of hardening. Large quantities of air voids will allow quick escape of volatiles, which will result in faster curing. More air voids exist in the more open, intermediate and single-size graded than in the dense grading. Thus when fast escape of the volatiles is desired, the more open-graded mixture is selected. Dense-graded mixtures will cure slowly (NCHRP, 1979).

Intermediate-graded aggregates are used primarily for stockpiled patching mixtures. They are selected for workability. Some of the most widely used proprietary mixtures are intermediate-graded with medium to small top size. If a highly stable, though workable mixture is desired, much coarser, one-size aggregate is used.

Dense-graded aggregates are usually chosen for stockpiled mixtures when good stability is needed. This grading provides greater resistance to water penetration as it has smaller of unfilled voids, but it has potentially poor workability characteristics after being stockpiled. Workability can be improved by heating these mixtures just prior to use.

Highly absorptive aggregates are often avoided when the patching material is mixed cold. These aggregates can contain large amounts of moisture, which will increase the tendency for stripping. Also, then often create drainage problems in the stockpile and usually absorb more binder, which can result in a longer curing time for the patching mixture.

2.3.3 Adequate Binder Content

There are no formal, step by step procedures for the design and selection of the binder content of cold-mixed patching materials. A trial and error procedure is used (NCHRP, 1979). High residual binder content in the mixture is needed to increase film thickness on the aggregate for adhesive properties, cohesion and durability but this can lead to drainage problems in stockpiles. Also, high binder content can produce unstable mixture.

Kandhal and Mellott (1981) reported that, at least 4.5% residual bituminous binder is required in a stockpile patching mixture made from aggregate whose water absorption is less than 1%. If aggregate absorbs water in excess of 1%, the residual binder content should be increased in a similar amount. Thus, an aggregate that has 1.5% water absorption should have 5.0% minimum residual bituminous binder. The factor limiting the maximum amount of the bituminous binder is drainage in the stockpile just after manufacture. The drainage can be minimized by using lower mix temperature and limiting the stockpile height to 1.2m during the first 48 hours.

PennDOT 485 (2003) has specified the minimum bitumen residue values that shall be used in the bituminous stockpile patching mixtures. This is shown in the Table 4 below.

Table 4: Minimum Bitumen residue (PennDOT, 2003)

Aggregate Type	Water Absorption (Coarse Aggregate) (%)	Bitumen Residue (%)
Stone and Gravel	Less than 1.0	4.5
	1.1 to 1.5	5.0
	1.6 to 2.0	5.5
	2.1 to 2.5	6.0
	2.6 to 3.0	6.5
Slag	Less than 4.0	7.0
	4.1 to 5.0	8.0
	5.1 to 6.0	9.0
	6.0 to 7.0	10

2.3.4 Additives

The use of additives varies considerably. Most specifications require additives be used when the patching mixture does not meet the minimum specified criteria. In addition, some specifications allow the use of additives whenever the producer of the patching mixture wants to use them. The exact time of anti-stripping additive to be used in patching mixture is usually left up to the producer (NCHRP, 1979).

There are many commercially available anti-stripping agents in the market for use with the medium curing (MC) cutback bitumen. Extensive testing has shown that there is no single additive that works with all aggregate types. Thus, it is essential that type of anti-stripping agent and its rate of application be selected after testing with the specific aggregate used in the mix (Kandhal and Mellot, 1981).

2.4 PATCHING MIXTURES PROBLEMS AND THEIR CAUSES

When unsatisfactory mixture or poor construction practices are used, the patch will not perform as intended (NCHRP, 1979). There are several problems with the patching mixtures which are currently in use. These deficiencies are reflected in poor performance or premature failure, which may be initiated in the stockpile, during handling and placement or in service. A list of inadequate performance and their probable causes is given in the Table 5 below (Anderson et al, 1988)

Table 5: Problems and Probable Causes in Cold Mix Patching Material (after Anderson et al, 1988)

Problem or Symptom of Failure	Probable Causes
In Stockpile	
Hard to work	Binder too soft; too many fines in aggregate; dirty aggregate; mix too coarse or too fine
Binder drains to bottom of pile	Binder too soft; stockpiled or mixed at high temperature
Loss of coating in stockpile	Inadequate coating during mixing; cold or wet aggregate
Lumps – premature hardening	Binder cures prematurely
Mix too stiff in cold weather	Binder too soft for climate; temperature susceptibility of binder too great; too many fines in aggregate, dirty aggregate; mix too coarse or fine.
During Placement	
Too hard to shovel	Binder too stiff; too many fines; dirty aggregate; mix too coarse or fine
Softens excessively upon heating (when used with hot box)	Binder too soft
Hard to compact (appears "tender" during compaction)	Insufficient mix stability; too much binder; insufficient voids in mineral aggregate; poor aggregate interlock; binder too soft
Hard to compact (appears stiff during compaction)	Binder too stiff; excess fines; improper grading; harsh mix – aggregate surface texture or particle
In Service	
Pushing, shoving	Poor compaction; binder too soft; too much binder; tack material contaminates mix; binder highly temperature susceptible, causing mix to soften in hot weather; in-service curing rate too slow; poor aggregate interlock; insufficient voids in mineral aggregate
Dishing	Poor compaction; mixture compacts under traffic
Ravelling	Poor compaction; binder too soft; poor cohesion in mix; poor aggregate interlock; moisture damage-stripping; absorption of binder by aggregate; excessive fines, dirty aggregate; aggregate grading too fine or too coarse
Freeze-thaw deterioration	Mix too permeable, poor cohesion in mix; moisture damage
Poor skid resistance	Excessive binder; aggregate not skid resistant; grading too dense

Shrinkage or lack of adhesion to sides of hole	Poor adhesion; no tack used or mix not self-tacking; poor hole preparation
Note: In some instances, items appear as both symptoms and causes. It is difficult to separate the symptoms from the causes in some cases	

2.4.1 Patching Mixtures Problems in Stockpile and Stockpile life

The most commonly encountered mixture deficiencies at the stockpile are poor workability, stripping and drainage of the binder (Anderson et al, 1986). Mix workability is affected by a number of factors including the grading of the aggregate, the stiffness of the binder, the quantity of the binder and premature curing. Curing characteristics of the binder are also very important in the stockpile (Chatterjee et al, 2006). Although some skinning or crusting may be expected, it should not be so pronounced that the mix is lumpy or hard to work.

Stripping can occur in the stockpile because the aggregate is not properly coated during mixing or through the washing action of rainfall or snow. Although stripping could be minimized by covering the stockpile with a tarpaulin, if the mix strips in the stockpile, it is likely that it will strip in service.

Drainage, which occurs when the binder drains from the aggregate and becomes concentrated on the bottom of the pile, can be caused by improper stockpiling temperatures, excessive binder in the mix or the selection of a binder that is too soft (Anderson et al, 1986). Stockpiles that are higher than 2 m may result in an excess of drained binder at the bottom of the pile (Kandhal and Mellot, 1981) as well as compaction of the mix under the surcharge.

The term “stockpile life” refers to the time a patching mixture can be stored and still have satisfactory workability. For patching mixtures produced with cutback binders, stockpile life is a function of the curing rate of the volatiles from the patching material. The rate of curing in turn is dependent upon the type of volatiles used to thin cutback and the conditions under which the material is stored. Cutbacks with thinners that volatize slowly

such as MC and SC, will have long stockpile lives. Any weather condition, such as heat, that tends to speed the rate of volatile evaporation will shorten the stockpile life (NCHRP, 1979).

The stockpile life of patching materials made with bitumen emulsions depends upon whether or not the emulsion has broken and the stiffness of the emulsion's residue. Emulsion patching mixtures that are cold-mixed usually are not broken when stockpiled. These mixtures must not be allowed to freeze. As soon as they freeze, they can no longer be used as a patching material (NCHRP, 1979).

Stripping problems will be eliminated by bagging cold mix patching material during the storage/stockpile period and reduction of workability during storage will be minimized since evaporation of hydrocarbons or water will be to a minimum level. In addition, the effect of weather condition which tends to reduce storage/stockpile life of the patching mixture will be reduced.

2.4.2 Patching Mixtures Problems during Transport and Placing

Workability of the mix is the primary concern during transport and placement for stockpile patching material (Anderson et al, 1988). For proprietary bagged patching materials, workability is not much problem during transport.

Workability and compactability are related. Although a workable mix is not necessary easy to compact, a mix with poor workability is generally difficult to compact. A workable mix can usually be compacted without difficulty unless the workability is gained by using an excessive amount of binder or a very soft binder; neither case will result in a stable repair.

Immediately after compaction, the mix must be stable and not susceptible to pushing or shoving even though there is no appreciable curing of the binder. Therefore, the stability immediately after compaction is primarily obtained through careful attention to aggregates.

2.4.3 Mix Problems during Service

The most regularly encountered failures are pushing or shoving, ravelling, dishing and debonding. Other patching mixtures failures may include freeze-thaw deterioration, poor skid resistance and the lack of the adhesion to the side or bottom of the patch repair.

2.4.3.1 Pushing and Shoving

Shoving is a longitudinal displacement of a localized area of the pavement surface. Pushing and shoving under traffic may be caused by a number of factors, all of which reduce to stability of the mix. Inadequately compacted mixes also are susceptible to pushing and shoving, because compaction is required to develop the aggregate interlock that is primarily responsible for mixture stability (Anderson et al, 1988) and shear strength.

The main factors that contribute to shoving are binder and aggregate type and proportions. A soft binder can contribute to mixture instability. The binder should not be too soft nor should it soften excessively in hot weather. In addition, too much binder will soften the mix and insufficient binder will not coat all of the aggregate, leaving loose aggregate in the mix. Aggregate should be crushed, open-graded and contain maximum of 2% fines.

Pushing and shoving can be caused by a non-stable mix resulting from the contamination of tacking materials that have migrated into the mix. Unless the tacking material is applied in a very thin film, it will contribute to the binder content of the patch. Ideal cold mix patching materials should be self-tacking, thus eliminating the need for tacking material and equipment and in so doing precluding its misapplication.

Ideally, once the mix is placed in the hole, the binder should cure immediately, leaving a stiff binder. A cured patch that is more flexible than the existing pavement is more

desirable to accommodate shrinkage, cracking and frost heaving (in cold areas) in the pavement. To this extent, cold mixes are to be preferred over conventional hot mixes.

2.4.3.2 Dishing

Dishing occurs as a result of a mix compacting under traffic, thereby leaving a depression in the repaired surface. It is most caused by inadequate compaction or instability of the mix. The dishing failure is not responsive to new and improved binders but is properly addressed through mixture design and proper compaction

2.4.3.3 Ravelling

Ravelling is a loss of aggregate from the surface of the repair due to inadequate cohesion within the mix. Ravelling is usually caused by loss of adhesion between the binder and aggregate, excess fines in the binder, stripping of the binder from the aggregate, inadequate aggregate interlock and poor compaction.

Absorptive aggregate, or aggregate that selectively absorbs the cutter stock from the binder, can reduce the adhesiveness and self-tacking character of the mix, which may lead to ravelling. Excess fines in the binder may have the same effect, causing the mix to become stiffer and less tacky.

2.4.3.4 Debonding

If the patch possesses insufficient adhesion or if the pothole in which the material was to be placed was not properly cleaned then the patch material will become debonded from the hole. Once this occurs, the patch will progressively deteriorate under the effects of moisture and traffic.

2.4.3.5 Freeze-Thaw Resistance

Freeze-thaw deterioration has been reported as a problem by some researchers. The most commonly cited mechanism is the delaminating of the patch from the original pavement as a result of freezing of water at the bottom of the repair. The deleterious effects of freezing water in open mixes have been cited as a potential problem, but this is not well supported by field observations. Much of the freeze-thaw damage is undoubtedly due to the improper adhesion of the patch to the bottom of the hole, which in turn may be the result of improper compaction, tacking or hole preparation.

2.4.3.6 Skid Resistance

Poor skid resistance can result from a flushed or bleeding surface or from polished aggregate. Non-polishing aggregates that retain adequate micro texture during service should be employed where high levels of skid resistance are needed. The 9.5mm to 12.7mm maximum aggregate size and the open grading specified for cold patching mixtures should ensure adequate micro texture. Micro texture may deteriorate in service because of flushing or bleeding, which can be caused by excess bitumen binder, inadequate voids, or stripping. These factors can be controlled with an appropriate mix design.

2.5 PERFORMANCE REQUIREMENTS

Most failures in stockpile patching mixtures are often related to the lack of desirable properties. The desired performance requirements are discussed as follows (Roberts et al, 1996, Chatterjee et al, 2006):

2.5.1 Workability

Workability is required during handling operations at the stockpile and during placement. The patching mixture must be sufficiently soft and pliable for easy shoveling and raking, especially in cold weather. The mix should be free of lumps that are hard to break. The mix with poor workability usually produces poor patches.

Temperature has a pronounced effect on workability because it controls the hardness of the bituminous binder. During cold weather the bituminous binder can be relatively stiff and improvements can be made by using low viscosity binders.

2.5.2 Resistance to Water action

Stripping is the separation of bitumen binder from aggregate, usually in the presence of water. If the mix is susceptible to stripping, the cold patch mix can ravel, causing the patch to fail. The stripping will be more significant if the cold mix patching material is used to patch potholes which were formed due to poor water drainage. Stripping can also result when the mix is not compacted adequately. Cold patching mixtures also may strip due to poor coating of the aggregate during mixing. This problem is evident from the extensive use of anti-stripping agents.

Emulsions on the other hand, may be used fairly well when water is present on the surface of the compatible aggregates. When bitumen emulsions are used, aggregates and emulsions must be compatible.

2.5.3 Adhesive Properties

Adhesiveness causes the mixture to adhere to the underlying pavement and sides of the pothole. Both the type and amount of binder influence the adhesiveness of the patching mixture. Soft binders usually adhere better than harder binders. Adequate adhesiveness is obtained normally by increasing the quantity of the binder in the mixture. Quite often maintenance crews do not take time to clean and dry the hole thoroughly so that proper tack coat can be applied. In such cases a stickier patching mix is helpful.

2.5.4 Stability

A patching mixture should be stable after placement and compaction to resist vertical and horizontal displacement due to traffic loads, especially right after compaction when the material is still uncured. Stability is also enhanced by proper attention to aggregate grading and aggregate properties. Lack of stability causes dishing and shoving of the mixture.

2.5.5 Skid Resistance

Adequate skid resistance is desirable for long, large patches. Slipperiness can be caused if the mixture contains aggregates which are easily polished and/or excessive bitumen binder which causes flushing and bleeding. Patching mixture should be designed to have adequate skid resistance, this means good, low-polishing, coarse and fine aggregate should be used and the bitumen content closely controlled to prevent excess binder.

2.5.6 Durability

Durability is the resistance of the patching mixture to deterioration due to traffic and weathering forces. Durability problems usually appear in the form of raveling, which defined as the progressive loss of aggregate from the surface of a repair (NCHRP, 1979).

The durability of patching mixtures is closely related to the quantity and type of binder. Greater durability is achieved with optimum binder contents. The viscosity of the binder also can significantly influence durability. Soft binders with soft resides are often considered more desirable. Aggregate grading also influences the rate of hardening

2.5.7 Complete Curing

It is important that the mix to should completely cure after it has been placed in the hole. The patch which has not completely cured will be susceptible to shoving, dishing and significant rutting.

2.5.8 Storageability

A stockpile patching mixture should remain workable when stored for a desired period of time (6 to 12 months). If the mixture does not have the right type of bitumen binder, it can lose volatiles too rapidly and become harder with time. Covering the mix with tarpaulins or polyethylene extends the storage time.

2.5.9 Resistance to Bleeding

Bleeding is related to excess binder in the patching mixture. The patching mix should have a proper amount of bitumen binder that the aggregate can hold without any measurable drainage. After compaction of the mix in the hole, there should be sufficient

voids within the mineral aggregate to hold this quantity of binder, at least 5% up to 8% air voids to ensure resistance to bleeding.

2.5.10 Freeze-Thaw Resistance

Freeze-thaw resistance is the ability of the patching mixture to withstand the weakening effect of cyclic thermal expansion and contraction forces resulting from freeze and thaw cycles. It undermines the durability of the mix. The lack of freeze-thaw resistance has been cited as one factor that contributes to premature failure in cold areas.

2.5.11 Other Considerations

Several other considerations must be addressed in the development of new mixes. Workers safety is of paramount importance. The binder materials and solvents must not create a fire hazard nor be toxic as defined by safety and environmental agencies. This requirement precluded the use of such materials as asbestos fibres, sulphur and other organic additives. The patching material must not also release any toxic chemicals to the environment that can cause roadside pollution or other environmental degradation.

Satisfying the above requirements necessitates a trade-off among many different factors. For example, the open grading that facilitates workability also causes the mix to be permeable. A binder that improves workability may lower mix stability. The design of cold patching mixes is a continuing compromise among the desired engineering properties (see Table 6).

Table 6: Pavement Patching Material Characteristics (Kandhal and Mellot, 1981)

Property	Parameter	Workability	Durability
Grading	Open grading	Good	Poor
	Dense grading	Poor	Good
Aggregate shape	Angular	Poor	Good
	Round	Good	Poor
Binder Viscosity	Low viscosity	Good (good storageability)	Poor
	High viscosity	Poor	Good
Aggregate size	Larger grading	Poor	Good with ideal conditions
	Finer mix	Good	Good if depth <76mm
Grading	One size	Good (effective cure)	
Anti-strip			Good if compatible

2.6 POTHOLE REPAIR EFFICIENCY

A major emphasis of the pothole repair researches has been to document the productivity of different pothole patching operations (Wilson and Romine, 1993). Material costs are a small percentage of the total cost for pothole repair, which implies that newer, more expensive materials that can provide greater repair longevity will be cost-effective (Thomas and Anderson, 1986).

In this section, different pothole repair procedures and their role in the productivity are described. Also it describes the adverse effect the pothole patching crew has on the efficiency.

2.6.1 Repair Methods

There are four different pothole repair methods that have been used by many maintenance agencies. These methods are; throw-and-roll, edge seal, semi-permanent and spray injection. They are explained as follows;

2.6.1.1 Throw-and-Roll

The throw-and-roll method is often used for temporary patches. This is appropriate when weather conditions are too poor for a semi-permanent patch to be placed or the road is due to be rehabilitated soon.

In the throw-and roll method, cutting is eliminated, and compaction is performed with a truck. Because there is no cutting or cleaning operation, the actual repair time for this method is considerably less compared to other standard method (Anderson et al, 1988). Although not considered the best to patch potholes, it is commonly used method because of its high rate of productivity.

The throw-and roll method consist of placing material into unprepared pothole (which may or may not be filled with water or debris, see Figure below), the patch is compacted using truck tyres (see Figure 3). No additional effort was required after the truck had compacted the patch, but if the compacted patch appear to be low extra material was added.A minimal crown is left in the compacted patches (between 3 and 6mm) to allow for further compaction. The repaired section is opened to traffic as soon as maintenance workers and equipment are cleared from the area.

In this method, the only major costs of equipments are for trucks carrying the material and the traffic control vehicles and signs. Other equipments that are needed for placing of material are shovels, rakes or other hand tools.

Figure 3: Throw-and-Roll Procedure, material placement and compaction of patch

The one difference between this method and the traditional throw-and-go method is that some effort is made to compact the patches. Compaction provides a tighter patch for traffic than simply leaving loose material. The extra time to compact the patches (generally 1 to 2 minutes per patch) will not significantly affect the productivity. This is especially true if the areas to be patched are separated by long distances and most of the time is spent travelling between potholes (Wilson and Romine, 1999).

In the study done by Maher and others (2001), it was observed that the throw-and roll technique proved just as effective as the semi-permanent procedure (to be described later) for those materials with which the two procedures were compared directly. The semi-permanent procedures has higher labour and equipment costs and lower productivity, thus, the throw-and-roll procedure would be more cost-effective in most situations, when quality materials are used. This is valid when longevity of the patch repair is not taken into consideration.

The patches that last longer and require very little re-patching, greatly reduce the labour and equipment costs for the overall repair operation. This lead to the conclusion reached by Thomas and Anderson (1986), which stated; rigorous procedures that involve cutting, cleaning and compacting are the most cost-effective way to repair potholes. Non-standard throw-and-go procedures cost about three times more than rigorous procedures.

2.6.1.2 Semi-Permanent

The semi-permanent repair method is considered to be one of the best for repairing potholes, short of full-depth removal and replacement (Wilson and Romine, 1999). The semi-permanent procedure as described by Wilson (1993) includes saw-cutting or jack hammering the edges of the pavement around the pothole to achieve straight, sound sides for the patch; removing the water and debris from the pothole; placing the material (in lifts, where pothole depths were excessive); and compacting the patches with single-drum vibratory rollers or vibratory plates. The repair is open to traffic as soon as maintenance workers and equipments are clear. Figure 4 shows compaction using vibratory plate compactor.

Figure 4: Semi-Permanent procedure – Compaction using vibratory-plate compactor (Wilson and Romine, 1999)

The repair procedure provides a sound area for patches to be compacted against and results in very tightly compacted patches (Wilson and Romine, 1999). This method requires additional manpower and equipments and has lower productivity compared to either throw-and-roll or the spray-injection method (to be described later), but it produces more durable patches by improving surrounding support.

2.6.1.3 Edge Seal

To improve the durability of the patch, the edge of the patch should be sealed to prevent the intrusion of water and other debris.

In the edge seal procedure, the material is placed into the pothole without any prior preparation or removal of water debris. The material is then compacted using truck tyres. The compacted patch is then checked for levelness. The compacted patch should have minimal crown of between 3mm to 6mm (Wilson and Romine, 1999) to allow for further compaction by traffic. The patch is then left to dry for one day after installation and a band of bituminous tack material on top of the patch edge is placed (tack material should be placed on both patch and pavement surface). A layer of sand is then placed over the tack material to prevent tracking by vehicle tyres.

The second visit to the repaired section for tack placement reduces the productivity of this method. The placement of the tack material prevents water from getting through the edge of the patch and can stick together pieces of the surrounding pavement, improving support for the patch. Figure 5 shows example of edge sealed patch.

Figure 5: Edge Seal Application

2.6.1.4 Spray Injection

In the spray injection method the water and debris are first removed from the pothole and a tack coat of binder is sprayed into the pothole on the sides and bottom. The asphalt and

aggregate are then blown into the pothole and the patched area is then covered by an aggregate layer (Maher et al, 2001).

This method requires no compaction after the cover aggregate has been placed. Figure 6 below shows the spray-injection device cleaning the hole and applying aggregate and asphalt bled.

Figure 6: Hole cleaning and asphalt and aggregate blend application (Maupin, 2003)

In the study done by Maupin and Payne (2003), it was concluded that, placing spray injection patches is much safer than placing other types of patches because the operation is performed from the truck cab with a minimal number of personnel exposed to traffic and it can be successfully placed throughout the year.

2.6.2 Cost-Effective Use of Manpower

Productivity of pothole repair crew is the objective that must be balanced against the need to produce a long lasting repair. To achieve durable repairs, the repair crew must be provided with high-quality material and adequate equipment.

From the study done by Thomas and Anderson (1986), it was concluded that, the factors that have the greatest influence on the total repair are repair procedures, daily production and crew deployment. Material costs account for less than 20% of the total cost when standard procedures are used.

Table 7 below shows the average productivity values for various repair operations (Maher et al, 2001). As seen on the Table 7, without taking into consideration repair longevity and consider only average productivity and number of labourers required, the spray-injection method is cost-effective with high productivity and still requires only 2 labourers.

Table 7: Average Productivity values for various Repair Operations (Maher et al, 2001)

Procedure	Average Productivity (tons/hr)	Labourers Recommended	Average Productivity (tons/person-day)
Throw-and-roll	1.6	2	3.2
Edge Seal	1.4	2	2.8
Semi-Permanent	0.3	4	0.3
Spray Injection	1.7	2	3.4

1 ton = 907 kg

2.6.2.1 Times for Basic Operations

A durable repair requires that the work be divided into several basic operations: cutting, cleaning, tacking, filling, levelling and compaction (Thomas et al, 1985). Table 8 below shows the ideal time for pothole repair processes excluding delays.

Table 8: Ideal Time for Pothole Repair Processes (Excluding Delays) (Thomas et al, 1985)

Operation	Probable Time[a] (Minutes)
Cutting	
Air Compressor	4.66
Pionjar[b]	7.26
Cleaning	
One HMW[c]	7.61
Two HMWs	5.58
Tacking	0.77
Filling	
One HMW	3.72
Two HMWs	2.22
Leveling	1.90
Compaction	
4-to-6-Ton Roller	8.64
Essick Roller[d]	4.79

[a]Weighted average based on the pothole volume distribution

[b]Gas-operated pavement breaker [c]Highway maintenance worker

[d]Walk-behind vibratory roller

Pothole repair is controlled by the cutting, cleaning and compaction operations. One of the ways to improve cutting efficiency is to mark the holes prior to cutting and this will minimize the need for the operator to wait for instructions. It has been concluded by Thomas et al (1982) that, compaction operations can be improved by overfilling the hole and sending the filling crew to the next hole without delay.

2.6.2.2 Delays

The extent of delays is an indicator of the relative efficiency of the operation. There is also a direct correlation between the absence of delays and effective management leadership (Thomas et al, 1982).

Delays are an inherent part of any operation. Although they cannot be avoided entirely, many can be controlled. Controlling and avoiding delays are at heart of the management effort. Foremen must exercise leadership in this area. In the absence of leadership, delays will be uncontrolled (Thomas et al, 1985). Table 9 shows delays that affected pothole repair on a daily basis, according to each basic operation.

The data in the Table 9 highlights what is perhaps the most difficult task in controlling the productionized operation: the challenge to the foremen to keep the various operations progressing. Improving the efficiency of the cutting operation would have a positive effect on the other operations because the major source of lost time is waiting for a hole to be available to work on (Thomas et al, 1985). The afore-mentioned research was done in Pennsylvania, United States.

Table 9: Percentage of Total Delay Time for Each Operation (Thomas et al, 1985)

Activity	**Cutting**		**Cleaning**	**Filling**	**Compaction**
	Air Comp.	Pionjar			
Receive Instruction	3	1	1	0	0
Wait for Instruction/ Observe	29	33	3	10	3
Wait for Another Operation	4	1	2	7	9
Wait for Mix to Arrive	-	-	-	9	-
Wait for Pothole	4	-	39	37	59
Move to Another Pothole	2	2	0	3	2
Change Tools	-	-	1	-	-
Start/Refuel/Adjust Equip.	6	2	-	3	1
Clean Hole with Air	5	-	2	-	-
Wait for Traffic	0	0	0	-	0
Personal Delays	4	1	1	2	-
Undetermined Delays	3	9	2	3	2
Productive Work (Efficiency)	40	51	49	26	24

A crew cannot achieve high levels of production unless the work is productionized and continuous throughout the day. Delays in crew deployment, which reduce crew efficiency, have a great effect on production. Thomas and colleagues (1985) recommended that; on average, the crew should arrive at the job site within 30 min of the beginning of the work shift. The crew shouldn't leave the job site less than 30 min before the end of the shift. This time frame should provide a minimum of 335 min (5:35 hr) of actual work time and 390 min (6:30hr) of time at the job site.

Having the crew arrive at the job site on time is only one aspect of crew deployment. For the operation to run smoothly, the material should arrive no later than 15 – 20 min after the crew begins work. The filling of the holes should begin almost immediately, because the crew is waiting and the holes have been prepared. There should not be too much time between the cleaning and filling operation because this could create safety problem. Table 10 summarized the recorded on the study done by Thomas and others in 1985.

Table 10: Time Data on Crew Deployment (Thomas et al, 1985)

Work Location	Work Began	Material Arrived	Delay	Filling Began	Delay	Material
1	8:45	9:35	50	9:45	10	Cold Mix
2	8:35	9:15	40	9:20	5	Cold Mix
3	9:15	9:41	16	10:53	72	Cold Mix
4	9:53	Before 9:53	0	11:36	103	Cold Mix
5	9:00	10:00	60	10:57	57	Cold Mix
6	12:55	Before 12:55	0	2:00	65	Cold Mix
7	9:18	10:20	62	10:28	8	Hot Mix
8	10:07	9:50	0	10:15	25	Hot Mix
9	10:00	Before 10:00	0	10:30	30	Hot Mix
10	8:47	10:05	78	10:15	10	Hot Mix
11	8:10	9:15	65	9:25	10	Hot Mix
12	8:40	10:15	35	10:55	40	Hot Mix
13	9:11	10:40	89	10:40	0	Hot Mix
Average[a]	9:13	9:52	39	10:25	33	

[a]Excluding work location no. 6

2.6.2.3 Maintenance Crew Size

For the throw-and-roll method, the labour cost can be as little as two workers who do the actual patching, plus traffic control. One of the two workers shovels the material from the truck into the pothole, and the other drives the truck over the section to compact the patch, the driver of the vehicle is able to shovel material when patching large areas. This generally improves the productivity of the overall operation (Wilson and Romine, 1999).

The edge seal procedure requires the same two workers and traffic control as the throw-and-roll procedure, but requires an extra pass to place the tack and sand material

The semi-permanent patching operation has proven to be the most efficient when four workers are used, along with the appropriate traffic control. Two workers clear out debris and square-up the edges while other two workers follow behind placing material and compacting the patches (Wilson and Romine, 1999).

The single unit spray-injection device requires a single operator. Two operators are recommended when using the trailer unit equipment (one to operate the vehicle and the other one to place the material). In both cases, traffic control is required (Wilson and Romine, 1999).

In the study done Thomas and others (1985) presented ideal times for pothole repair processes, shown in Table 8. The Table 8 shows the relative inefficiency of using more than one crew member in the cleaning and filling operations. Although total production may be increased slightly by adding and additional worker to the cleaning operation, this will have adverse effect on productivity. The same study indicated that, for 18 crews involved in the repair operations on roads with a high frequency of holes, the average crew size was seven. The performance standard specifies five. Both figures include the foreman but not those highway maintenance workers required for traffic control. The size of 15 out of 18 crews (83 percent) exceeded the performance standard.

Thomas, Anderson and Kilareski (1982) reported about crew balance, which means that the size of the crew should adjusted to the pace of the work so that each crew member is utilized to the maximum degree. They mentioned two measures of reducing idle time for crew members;

1) Reduce the crew size. This action will tend to improve productivity (tons per man-hour), but production (tons per day) will probably remain the same.

2) Divide the operation into its component parts. Idle workers can be assigned to these tasks.

Both actions will probably increase productivity; however, reorganizing the crew is usually the more effective measure the crew is greatly over-manned. Utilization of all crew members is one of the most important and challenging aspects of crew management.

2.6.3 Patch Life (Smith et al, 1991)

Permanent vs Temporary Patching Procedures

Permanent patching procedures (assuming dry hole conditions) improve the life of asphalt concrete (AC) hot mix patches under cold conditions by a factor of 5.9. For conventional cold mixes, the improvement for a wet hole condition is 3.1 percent.

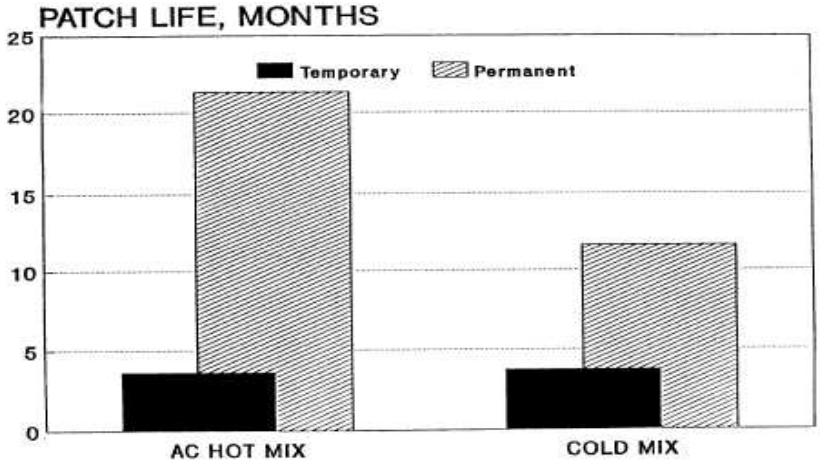

Figure 7: Life expectancy of hot mix and cold mix patch materials placed in permanent and temporary conditions

Cold vs Warm Temperatures

Warm temperatures contribute to an increased service life of conventional mixtures in wet holes by a factor of 2.4 for AC hot mix and 1.2 for cold mix, and also for proprietary cold mixtures by a factor of 1.1. Heating the mixtures may be one way of achieving this life service, even during cold weather.

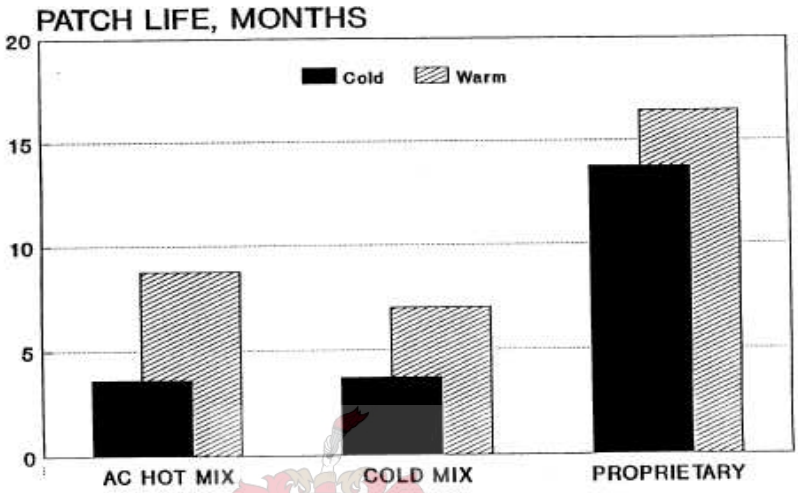

Figure 8: Life expectancy of patch materials placed in wet potholes in cold and warm temperatures

Wet Hole vs Dry Hole

One of the most effective ways to improve on the service life of patches is to dry the hole prior to placement of the material. Conventional patching materials[1] (using temporary procedures in cold temperatures) showed an improvement in life of 2.5 times for cold mix to 4.7 times for hot mix. All proprietary materials, usually designed for wet conditions, showed a smaller improvement of 1.4 times.

[1] Conventional cold mixes – these mixes are produced according to specifications set by the agency that will use the mix. The specifications normally include the acceptable types of aggregate and asphalt.

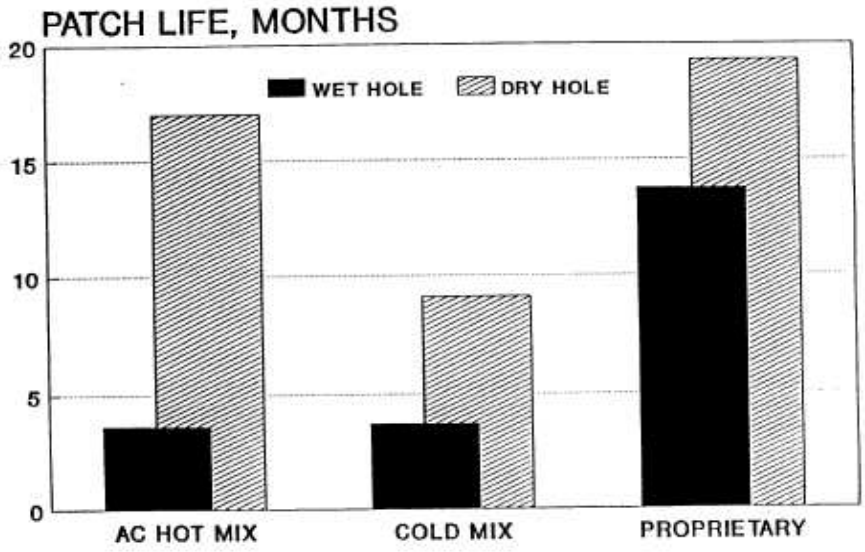

Figure 9: Life expectancy of patches in wet and dry potholes, in cold temperatures

Summary of Findings

Bituminous hot mix has shown the longest service life of all materials when it is placed using permanent procedures in a dry hole (1.8 to 4.4 years on average, depending on the temperature). This conclusion has been supported by many studies. However, if hot mix is placed in a wet hole using temporary or permanent procedures, its life is reduced by a factor of more than 10.

Large improvement in the service life of conventional cold mix patch materials can be attained if such patches are placed using permanent patching procedures and the hole is dry. Increased life in the order of 3 to 4 times was reported. However, when these conventional cold mixes is placed into a wet hole in either cold or warm temperatures conditions, its life will vary from few days to at most a few months.

2.7 ENVIRONMENTAL CONSIDERATIONS

As seen in previous sections, cold-mixed asphalt concrete use cutbacks and emulsions during mixing of the mixtures. Cutbacks harden through the evaporation of the solvent into the air, while emulsions harden through the evaporation of water and the emulsifying agents. The large use of these cold mix patching materials can have significant environmental implications.

Understanding the atmospheric emissions of asphalt constituents into the environment and associated health risks continues to be major challenge for occupational and environmental health scientists (Kuhn et al, 2003).

Other atmospheric contaminants, such as dust, develop during mixing of bituminous materials, but these are now adequately controlled. The hydrocarbons thinners in cutbacks escape into the air when the cutbacks are mixed with the aggregates. Evaporation also takes place in the stockpile, but the amount of volatiles released may be quite small once the crust is formed (NCHRP, 1979). The problem of evaporation of volatile during stockpile can be eliminated or reduced to a minimum by containerizing or bagging the cold mix patching materials. Most of proprietary cold-mixed patching materials used in South Africa are bagged. The large amount of the volatiles is given off at the patching site while the mixture cures.

In a way of protecting the environment, environmental agencies is some areas prohibit the use of cutbacks either in densely populated areas or during the summer warm season. Because of environmental and energy concerns, the use of cutbacks patching material is being reduced. Therefore, the use of patching mixtures made with bitumen emulsions is increased (NCHRP, 1979).

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Chapter 3

TESTING: VOLUMETRIC AND ENGINEERING PROPERTIES

3.1 INTRODUCTION

This chapter describes volumetric and engineering properties of five cold mix patching materials available in the market. Some background information like grading of the product, type and content of the bitumen binder of some products were obtained from products respective suppliers. Tests for determination of moisture content and volumetric properties were part of this study.

ITS tests for unconditioned and conditioned specimens were done at ambient temperatures for all products. Indirect Tensile Strength test results were averaged from three specimens for each test, except in one product where there was some difficulties in specimen's preparations. Indirect Tensile Strength tests results for conditioned and unconditioned specimens were analysed and compared with minimum specified Indirect Tensile Strength value and Tensile Strength Ratios were calculated.

Permeability tests were done by using falling-head method and results were discussed. In this chapter also, there is description of some difficulties which arose during specimen preparation.

3.2 MATERIALS

In this study five bagged proprietary cold mix products were used. Proprietary cold mix products are produced by companies that test the local aggregates, design the mixes and monitor production to ensure the quality of the product. When using proprietary materials that are already mixed, some acceptance testing must be done before purchasing the material (Wilson and Romine, 1999).

All products sealed in bags were delivered in Stellenbosch University. The products tested and their manufacturers are shown in the Table 11. Henceforth, the products will be referred as product A to E as shown in the Table 11. All of these products are available in South Africa.

Table 11: Mixes Used in the Study

Product ID	Product	Manufacturer	Weight in Bag (kg)
A	Roadfix	Roadfix International	25
B	Tarfix	Tarfix (Pty) Limited	25
C	Much-mix	Much Asphalt (Pty) Limited	25
D	Asphalt King	Asphalt King (Pty) Limited	30
E	Glenpatch	Glenpatch	30

The five products are described as follows as per information from suppliers or website of the manufactured company as cited;

Roadfix: It is a proprietary material produced with graded aggregates and cutback bitumen as binder. It has specific properties which render it particularly suitable for the filling of potholes and reinstatement of trenches.[2]

Tarfix: It is a proprietary cold mix material, manufactured very stringent quality conditions using quality graded aggregates and cutback bitumen as binder. The Tarfix formulation contains five highly specialized additives that are imported from Sweden.[3]

Much-Mix: It is cold mix asphalt made with fine continuously graded aggregates. It is produced as Hot Mix Asphalt to 120°C then diesel is added to make it workable at low temperatures.[4]

[2] www.roadfix.co.za
[3] www.tarfix.co.za
[4] Supplier verbal information

Asphalt King: It is a proprietary material produced by using graded aggregates and 60% anionic emulsion as bitumen binder. It is available in 30kg bags that will cover 0.8m^2 at a thickness of 25mm.[5]

Glenpatch: It is proprietary product, produced by mixing carbonated shale, aggregate and bitumen emulsion. The grading of the mix complies with requirements of medium continuously graded asphalt according to various road authorities specifications.[6]

3.3 BACKGROUND ENGINEERING PROPERTIES

3.3.1 Moisture Content

The moisture contents of the Products D and E in sealed bags were determined by oven drying three samples from each mix for 24 hours. It was found that, Product D has a moisture content of 7.4% and Product E has a moisture content of 5.7%. Product A, B and C don't contain moisture after mixing as they only incorporate cutback as binder.

3.3.2 Grading

Grading of the aggregates for Products B, C and D were provided by their respective manufacturers. For Products A and E, the grading were obtained by extracting binders from the mix then grading analysis was done as per method B4 as recommended by TMH1 (1986). This test was done at Much Asphalt Central Laboratory. Table 12 and Figure 10 show the grading for all of the products. As seen on the Table, the maximum aggregate size is 6.7mm for all products except Product E with maximum aggregate size of 9.5mm.

[5] Product brochure from supplier
[6] Product brochure from supplier

Table 12: Aggregate Grading for the Products

Sieve size (mm)	Percent Passing (%)				
	A	**B**	**C**	**D**	**E**
0	0	0	0	0	0
0.075	5.6	7.9	6.5	9.88	7.7
0.150	7	10	10	13.7	11.7
0.300	9.8	13	19	18.44	16.9
0.600	13.9	18	26	27.1	22.8
1.180	19.8	28	35	38.66	28.2
2.36	30.3	47	50	59.76	36.7
4.75	63.4	71	76	92.04	52.9
6.70	91.5	**93**	**97**	**98.25**	65.3
9.50	**99.7**	100	100	100	**98.2**
13.20	100	100	100	100	100

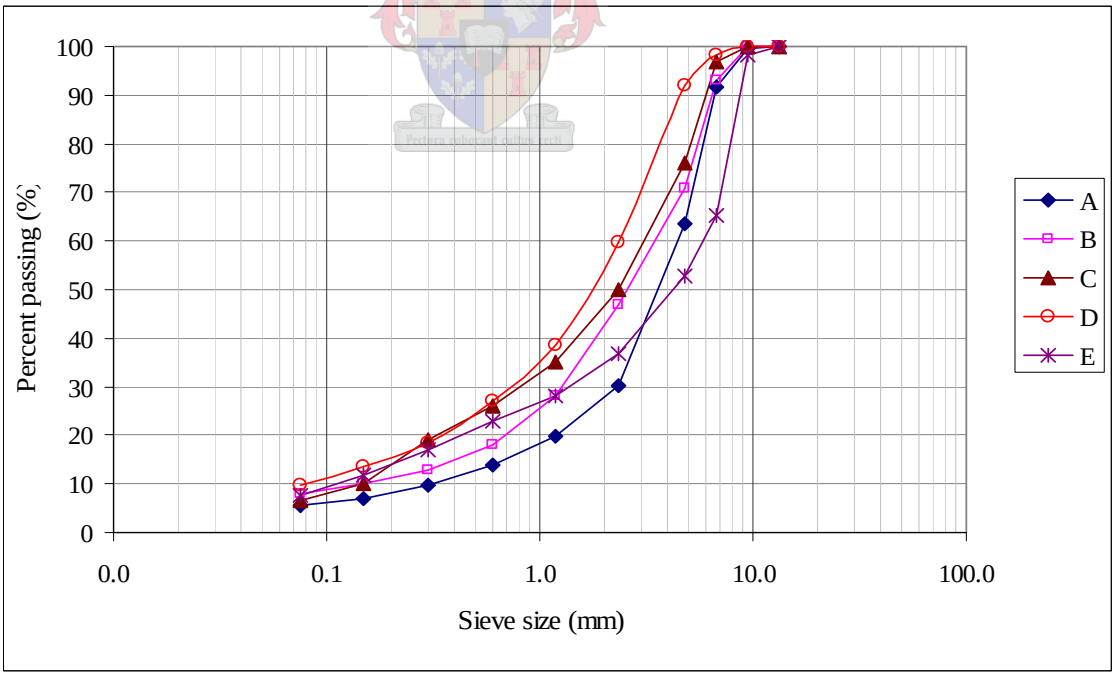

Figure 10: Aggregate Grading for the Products

3.3.3 Bituminous binder

As per information provided by manufacturers, Products A and B have cutback bitumen as mixing binder and Products D and E have emulsion bitumen as the mixing binder. For Product C, the mix is prepared as hot mix at 120°C and then diesel was added to the mix to provide workability in ambient temperature. The type of binder for Product C will be referred as cutback bitumen. Binder contents for the Products B, C and D were given by respective manufacturers, for Products A and E, their binder contents were obtained extracting binder from the mix as per method C7(b) of TMH1 (1986) and one test per was done. Type and contents of binder of the products are tabulated as shown below;

Table 13: Type and Content of Binder by mass of dry aggregate

Product	Type of binder	Binder content
A	Cutback	5.9%
B	Cutback	6.1%
C	Cutback	5.5%
D	Emulsion	5.8%
E	Emulsion	5.1%

3.3.4 Volumetric Properties

3.3.4.1 Specimen Preparation and Curing

The Hugo hammer was used for compaction during preparation of specimens. The Hugo hammer is the modification of Marshall hammer. These modifications include indents on the face of the hammer and turning of hammer during compaction after every blow, providing a “kneading” effect in the compaction of the material. An LVDT was attached on top of the hammer to measure change in height and hence the densification of specimens during compaction. Figure 11 shows the Hugo hammer.

For each specimen 2 blows were applied on each side to condition the specimen before starting taking the readings. Afterward, 75 blows were applied to one face and then the

mould was turned over and an additional 75 blows were applied. The size of the moulds used was 150mm.

Figure 11: Hugo Hammer

All specimens were compacted at ambient temperature; this was done since patching of pothole using cold-mix asphalt usually is done at ambient temperature in the field. Compaction temperatures range from 19°C to 23°C and aggregates temperature before compaction were taken as shown in the Table 14. The specimens were left in the moulds to cure for 3 days in the oven at 30°C.

Figure 12: Products Specimens

3.3.4.2 Relative Density and Compaction Curves

The maximum relative densities (Rice's density) were determined by using C4 method as per TMH1 and bulk relative densities were determined by using method C3 according to TMH1. The void contents in the mixes were determined and summarized in the Table 14.

Table 14: Summary of Volumetric Properties

Product	UNTRAFFICKED			Compaction Temp. (°C)	TRAFFICKED	
	BRD (g/cm^3)	Rice's Density (g/cm^3)	Air Voids (%)		BRD (Hot, 20k) (g/cm^3)	Air Voids (%)
A	2.330	2.719	15.4	20	2.305	15.6
B	2.246	2.647	15.1	20	2.282	14.8
C	1.976	2.453	19.4	20	2.137	17.9
D	2.072	2.458	15.7	19	2.018	16.2
E	*1.688*	2.205	23.5	20	*1.930*	20.4

By using the Hugo hammer compaction method to compact cold asphalt patching mixes, the mixes could not be compacted to achieve final air voids of less than 10 percent. The compaction curves for compaction done on the mixes are shown in the Figure 13. The compaction curves indicate that all the compacted specimens have air voids between 15 and 24 percent. As seen in Table 14, Mix E has the highest air voids content of 23.5% and Mix B has voids content of 15.1%, which is the lowest of all five patching mixes. Kandhal et al (1995) concluded in their research that, voids in the total mix (VTM) is most affected by bitumen content, P200 (0.075mm sieve) and the relative proportions of coarse and fine aggregates. They also said that, VTM can be increased by reducing asphalt content or P200 or both.

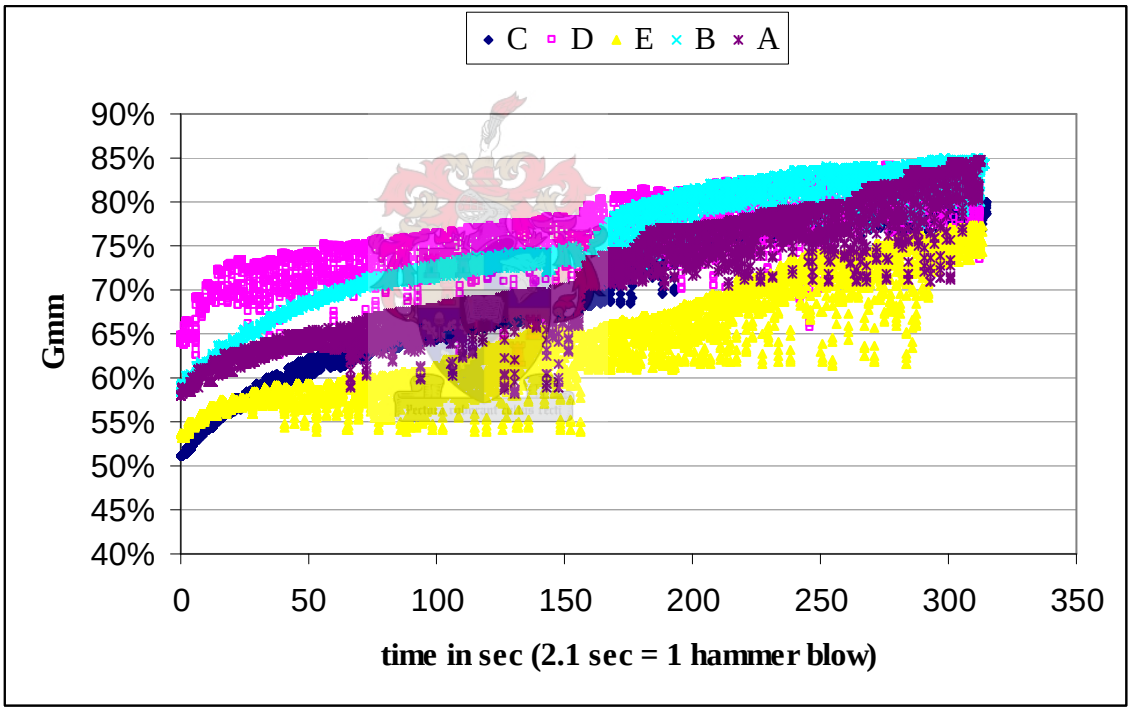

Figure 13: Products Compaction Curves

In the Table 14 there are also BRD tests results after MMLS3 trafficking (MMLS3 trafficking will be discussed in Chapter 4). It can be seen that there was a relative increase in density after trafficking for Product B, C and E, of about 2%, 8% and 13% respectively. This increase was due to the decrease in voids due to densification under

MMLS3 trafficking. But there was also decrease in BRD on Products A and D, of about 1% and 3% respectively.

3.3.5 Indirect Tensile Strength Testing

The Indirect Tensile Strength (ITS) test is used to evaluate the cohesive strength of asphalt mixes. This property can be used to evaluate tensile strength and is also an important component of rutting resistance in the medium temperature range (Taute et al, 2000). In South Africa the minimum value of ITS is 800 kPa for HMA; however it has suggested that rutting potential tends to increase for ITS values below approximately 1000 kPa. And at the same time ITS value above 1700 kPa may indicate a tendency of brittleness and low flexibility (Taute et al, 2000).

3.3.5.1 Specimens Preparation and Curing

Specimens were prepared the same as for volumetric tests but with different curing procedure. All specimens were cured using two curing conditions to represent early and medium-term field curing condition. In addition, curing process was different for specimens with emulsion and specimens with cutback. This was done since solvents in the bitumen cutback take longer to evaporate compare to rate of evaporation of water in the emulsion binders.

For short term curing; specimens with bitumen emulsion as binder (Products D and E) were left to cure in compaction moulds for 3 days in the oven at 30°C. Products with cutback bitumen (Products A, B and C), specimens were also left in the moulds for 3 days at 40°C.

For medium term curing, specimens with emulsion were left in the compaction moulds for 3 days at 40°C and then they were extracted from moulds. After that, specimens were sealed and cured for 3 days at 40°C. For specimens with bitumen cutback; they were left in moulds for 5 days in the oven at 50°C and then taken out of oven and were left to cool

for 24 hours at ambient temperature. Then, specimens were cured for 7 days by blowing hot air within a temperature range between 28°C to 30°C.

3.3.5.2 Specimen Conditioning

All specimens were subjected to moisture conditioning before ITS testing and ITS results of conditioned specimens were compared to those of unconditioned specimens. Then the tensile strength ratio (TSR) of the product, which is ratio of indirect tensile strength of the moisture-conditioned specimens divided by the indirect tensile strength of unconditioned specimens were calculated. The TSR value shows how susceptible the mix will be to stripping or is an indication of loss of strength caused by the moisture conditioning. Roberts et al (1996) cited that a high number of TSR indicates that good performance is expected while a low number indicates that poor performance is expected. Conversely, Dukatz and Phillips (1987) reported that the use of average tensile strength to calculate a TSR value used for acceptance determination may results in asphalt mixtures with poor moisture resistance being accepted and a mixture with a low moisture resistance being excluded. They continue to argue that, the reason for this is, a ratio does not take into account the relative strength of the asphalt mix. A mix with a low unconditioned tensile strength and a conditioned tensile strength that is similar mathematically gives a high TSR. The data results from this study will show this controversy. Dukatz and Phillips (1987) said that, “The possibility of an undesirable mix passing the test is just as high as good mix failing”. To minimize the possibility of good mixes being rejected and poor mixes being accepted, a minimum tensile strength should be specified along with the TSR (Dukatz and Phillips, 1987).

In this study, specimens moisture conditioning was done as follows; specimens were vacuum saturated for one hour and then removed from the vacuum container and immediately were immersed into the 25 ± 2.0^{0} C water bath, and were left for one hour before testing. The Lottman moisture conditioning process of vacuum saturation followed by freeze-thaw cycle is said by some investigators (Gilmore et al, 1985 and Dukatz and Phillips, 1987) to be too severe and have proposed some variations that eliminate the

freezing step. The study was done on the hot mix asphalt specimens, which would have made results to be even worse for cold mix asphalt.

In the Interim Guidelines for the Design of HMA in South Africa (Taute et al, 2000), a minimum TSR value of 0.7 is specified for routine mix design. But for mixes in high rainfall areas and high traffic applications, a minimum TSR value of 0.8 is recommended. Table 15 shows TSR criteria based on the permeability of the mix and the climate in which the mix will operate.

Table 15: TSR criteria based on mix permeability and climate (Taute et al, 2000)

Climate	**Permeability**		
	Low	Medium	High
Dry	0.60	0.65	0.70
Medium	0.65	0.70	0.75
Wet	0.70	0.75	0.80

The Interim Technical Guideline: The Design and Use of Foamed Bitumen Treated Materials (Asphalt Academy, 2002), also provides guidelines for the TSR for roads constructed in different regions based on climatic regions using Weinert's N-value, as shown in the Table 16. The Weinert's N-value is calculated by the formula shown below. The higher the N value the drier the area. As seen in the Table 16 in the dry areas with rolling terrain type, the minimum TSR value of 0.5 is specified and with wet areas with flat terrain the minimum TSR value is 0.75.

$$N = \frac{12E_j}{P_a}$$

where E_j = evaporation during the month of January (the hottest month in South Africa)

P_a = annual rainfall

Table 16: Guidelines for TSR (%) Requirements Based on Climatic Regions using Weinert's N-value (Asphalt Academy, 2002)

Terrain Type and Drainage	Dry (N > 5)	Moderate (5 > N > 2)	Wet (N < 2)
Rolling - well drained	50	60	70
Flat - poorly drained	60	65	75

3.3.5.3 Test Method

All ITS tests were done at ambient temperature, in the work done by Mamlouk and Wood using asphalt-emulsion treated bases argued that, high test temperatures decrease ITS. The Zwick machine (see the Figure 14) was used for the testing. The test was performed by applying a single compressive load across the vertical diametric plane of a cylindrical specimen. The specimen is loaded at a rate of 50mm/min of the vertical ram and the load is applied until failure, which is the maximum load. The tensile strengths were calculated using the following formula;

$$ITS = \frac{2P}{\pi \cdot t \cdot D}$$

$$\begin{aligned} where: \quad & ITS \rightarrow \text{Indirect tensile strength } (kPa) \\ & P \rightarrow \text{maximum vertical compressive load } (kN) \\ & t \rightarrow \text{specimen thickness } (m) \\ & D \rightarrow \text{specimen diameter } (m) \end{aligned}$$

Figure 14: Specimen on the Zwick machine

3.3.5.4 ITS Test Results

The indirect tensile strength varied between 16.8 kPa and 266.8 kPa for both conditioned and unconditioned specimens, see Table 17 to Table 20 and Figure 15 to Figure 18. These values are far below compared to the minimum specified value of 800 kPa. Table 17 shows the ITS test results for short term curing condition for unconditioned specimens, the results were plotted on Figure 15. Product D has the highest value of 116.6 kPa and product A with lowest value of 17.8 kPa

Table 17: ITS Values for Short Term Curing Condition for Unconditioned Specimens

ITS Testing Results for Short Term Curing - Unconditioned Specimens					
Product	A	B	C	D	E
No. of Specimens	2	3	3	3	3
Condition of specimen	dry	dry	dry	dry	dry
Average Maximum Load (kN)	0.313	1.432	1.320	1.641	0.965
Average specimen height (m)	0.0747	0.0674	0.0663	0.0599	0.0649
Average Tensile Strength (kPa)	17.8	90.1	85.0	116.6	63.2

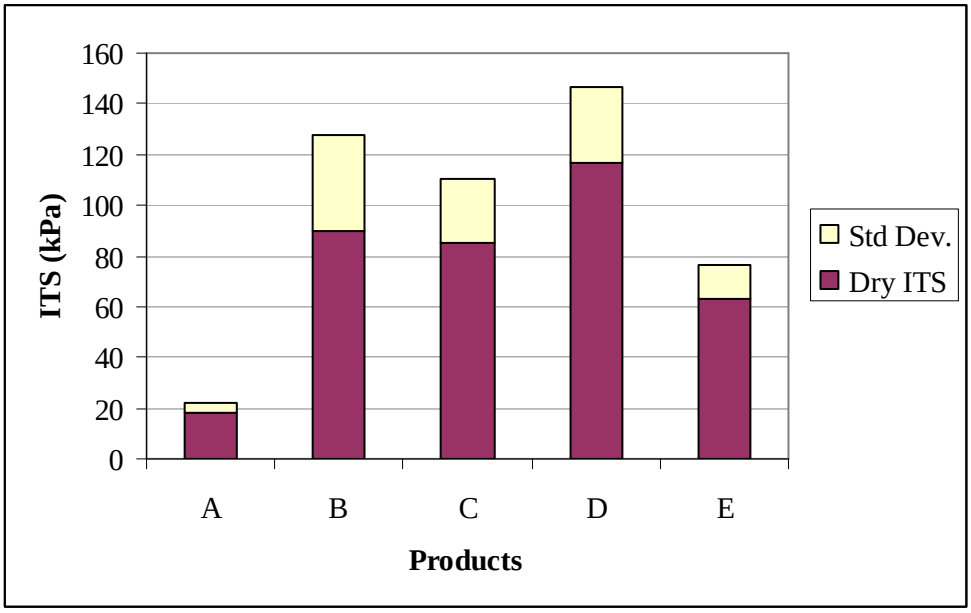

Figure 15: ITS Results for unconditioned specimens cured at short period

Table 18 and Figure 16 shows ITS summary of ITS results for conditioned specimens, which were cured on short term curing condition. Product A shows that there is lost of strength of 1 kPa, which is approximately 6%. This implies that, even though Product A has lowest ITS values but is not very much affected by moisture. It can also be seen that, Product B lost 26% of its strength, while Product C, Product D and Product E, lost their strength by 8%, 84% and 52% respectively. Product D and Product E, which have bitumen emulsion as binder, have lost more than 50% of their strength. This means they are much more susceptible to moisture damage than mixes with cutback binders.

Table 18: ITS Values for Short Term Curing Condition for Conditioned Specimens

ITS Testing Results for Short Term Curing - Moisture Conditioned Specimens					
Product	A	B	C	D	E
No. of Specimen	2	3	3	3	2
Condition of specimen	wet	wet	wet	wet	wet
Average Maximum Load (kN)	0.300	0.976	1.235	0.251	0.458
Average specimen height (m)	0.0756	0.0619	0.0669	0.0586	0.0647
Average Tensile Strength (kPa)	16.8	66.7	78.5	18.2	30.1

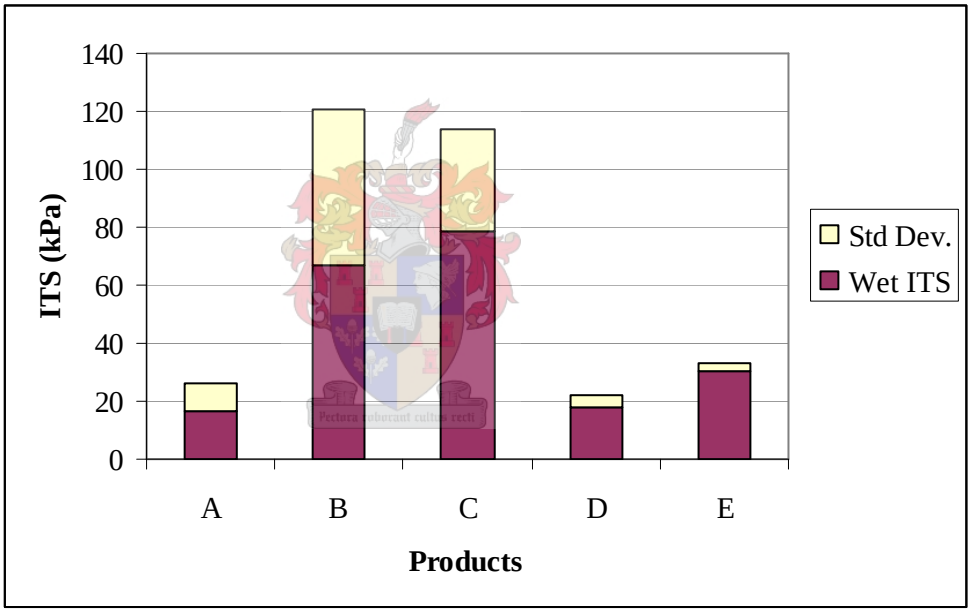

Figure 16: ITS Results for conditioned specimens cured at short period

Table 19 and Figure 17 show ITS results for medium curing unconditioned specimens. It can be seen that Product D still has highest ITS value and Product A the lowest. However, there is much increase in strength in all products except Product E, which has strength increase of approximately 5 kPa only.

Table 19: ITS Values for Medium Term Curing Condition for Unconditioned Specimens

ITS Testing Results for Medium Curing - Unconditioned Specimens					
Product	A	B	C	D	E
No. of Specimen	3	3	3	3	2
Condition of specimen	dry	dry	dry	dry	dry
Average Maximum Load (kN)	0.832	3.683	1.779	3.710	1.073
Average specimen height (m)	0.0660	0.0681	0.0660	0.0590	0.0662
Average Tensile Strength (kPa)	53.5	228.9	114.5	266.8	68.8

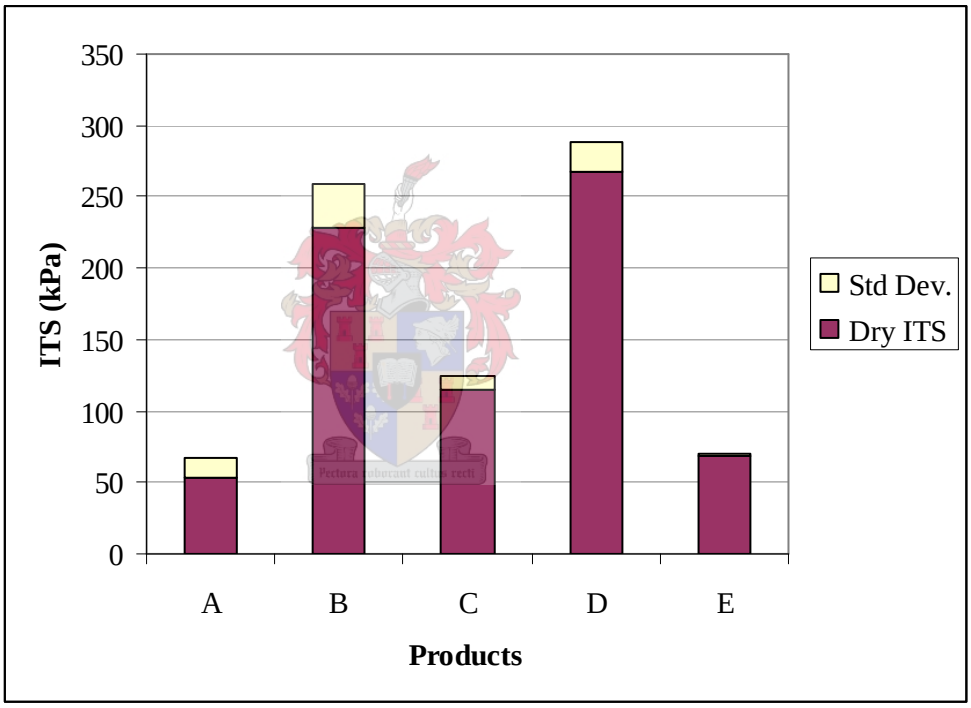

Figure 17: ITS Results for unconditioned specimens cured at medium term period

Table 20 shows increasing curing period makes cold mix patching material little less susceptible to moisture damage. The data shows that product Indirect Tensile Strength has increased after conditioning of the specimens instead of decreasing; this might be due to discrepancy during testing. The data also shows that, Product D is still more susceptible to moisture damage compare to other products, even though it had the highest ITS value before conditioning of specimens.

Table 20: ITS Values for Medium Term Curing Condition for Conditioned Specimens

ITS Testing Results for Medium Term Curing - Moisture Conditioned Specimens					
Product	A	B	C	D	E
No. of Specimen	2	3	3	3	2
Condition of specimen	wet	wet	wet	wet	wet
Average Maximum Load (kN)	0.841	2.949	1.562	1.666	0.755
Average specimen height (m)	0.0645	0.0671	0.0684	0.0584	0.0645
Average Tensile Strength (kPa)	55.4	186.3	97.0	120.9	49.6

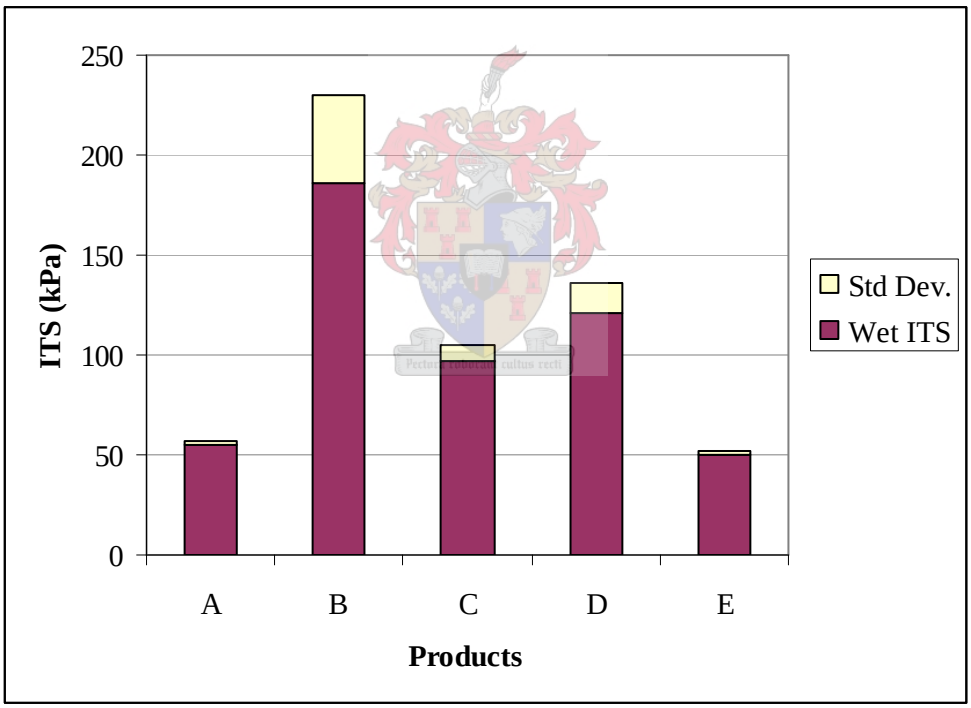

Figure 18: ITS Results for conditioned specimens cured at medium term period

Figure 19 shows the summary for ITS test results for short term and medium term curing period for both conditioned and unconditioned specimens.

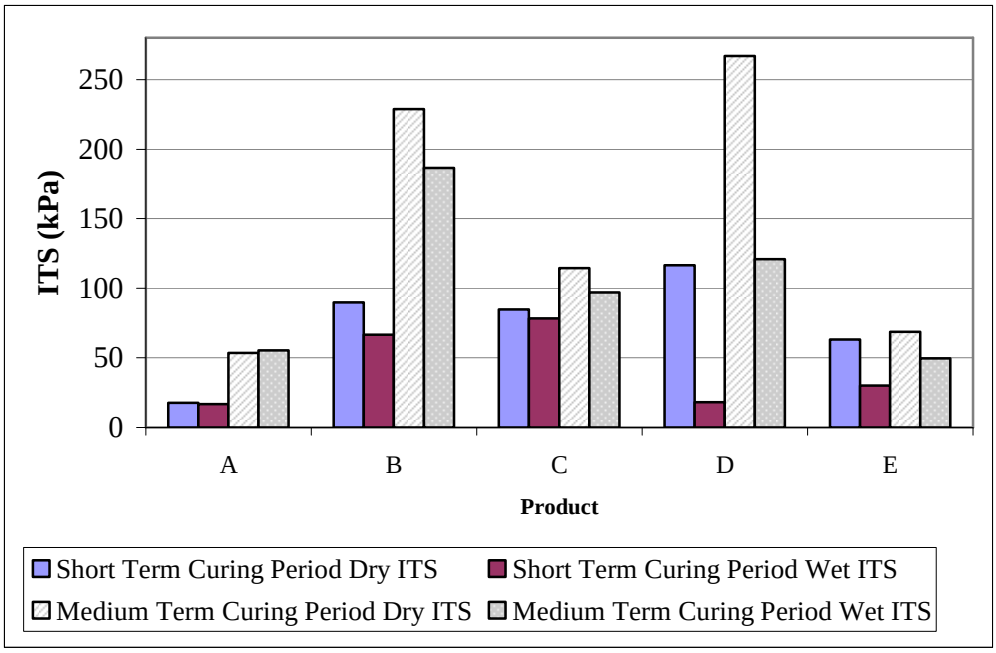

Figure 19: Summary of ITS Results for Short Term and Medium Term Curing Period

Tensile Strength Ratio

Table 21 and Figure 20 show TSR values for short term and medium term curing conditions for all products. As seen on the Table 21 Product A has TSR value of more than 1 for medium term curing condition, this is so, since average ITS value after conditioning of the specimens was little bit higher than that of unconditioned specimens. This can conclude that according to data, Product A is not affected with moisture or water. There is decrease in TSR on Product C, even though the tensile strength increased with increased curing period. Product A, Product B and Product C have TSR more than 0.7 for both curing conditions, while Product E has TSR value 0.48 for short term curing condition. Product D has the lowest value of 0.16 for short curing period and it increased with curing time. It can also be seen that, products with bitumen emulsion as binder showed significant increase in TSR values with longer curing period, compared with products with cutback bitumen as binder.

Table 21: Tensile Strength Ratio

Product	TSR	
	Short Term Curing Period	**Medium Term Curing Period**
A	0.94	*1.03*
B	0.74	0.81
C	0.92	0.85
D	0.16	0.45
E	0.48	0.72

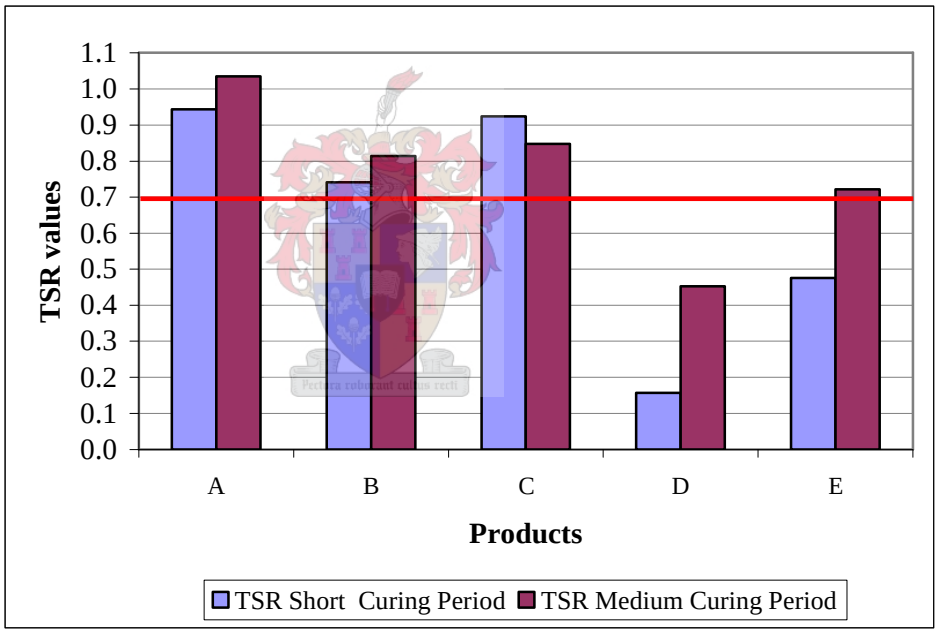

Figure 20: Tensile Strength Ratio

3.3.6 Permeability Test

Permeability of asphalt concrete pavements to water and air is an important factor that has to be considered in the design of asphalt mix. Asphalt layers with higher permeability allow water to enter readily the voids and infiltrate into underlying granular layers, resulting in a reduction in pavement support.

Permeability to water or air is directly related and influenced by asphalt content, compactive effort and type of aggregate and aggregate grading. Permeability of a compacted asphalt mixture is due to the presence of air voids in the mix. However, the total volume of air voids in a specific mix cannot be taken as a direct measure of the permeability of that mix. Mixes with identical air voids can have significantly different permeability. Size, continuity and total volume of air pockets are the important factors influencing the permeability of a specific mix (Abdullah et al, 1998). This is to say that air void structure and not air void content is the major contributor to asphalt mixes.

In the same research, Abdullah et al (1998) showed that the water permeability index increased significantly as the grading of the asphalt mix was made coarser. Kanitpong et al (2001) did a research using specimens of various grading, which showed that permeability of HMA is primarily controlled by air void content but also sensitive to grading and thickness. Also it has been concluded that mixes with finer blends tend to have higher permeability with all other factors being equal.

Work by Cooley et al (2002) concluded that nominal maximum aggregate size of the mix affects the permeability characteristics of the pavement. Mixes with larger nominal maximum aggregate sizes have more potential for high permeability than mixes of smaller nominal maximum aggregate sizes, at the same air voids. Also the work by Mallick et al (2003) has shown that at a given in-place air void content, permeability increased by one order of magnitude as the NMAS increased.

Researches by Mallick et al (2003), Hainin et al (2002) and Cooley et al (2002) have showed that there is decrease in permeability with an increase in thickness.

Cooley et al (2002) studied comparison between laboratory and field permeability measurements and they found that at permeability values less than about 500 x 10^{-5} cm/sec, there was an almost one-to-one correlation. And with permeability values above 500 x 10^{-5} cm/sec, the laboratory test method provided higher values. But they indicated that this trend of results was not expected. It was anticipated that field permeability results would provide higher values, since in the field water can flow from field device in any direction, while in the laboratory water is restricted to one direction. The possible explanation for the deviation was given as; at permeability values above 5 x 10^{-3} cm/sec, asphalt mixes have high percentage of interconnected air voids. In the field, an interconnected air voids may or may not be of a length it allows water to flow. However, the laboratory permeability device may allow a single interconnected air void that extends within the asphalt specimen which leads to higher results.

Choubane et al (1998) found that when the amount of air voids is less than 6%, the pavement is "virtually impermeable" (permeability level is negligible) and they suggested that pavement permeability limit should not exceed 1 x 10^{-3} cm/sec. In another research done by Cooley et al (2001) concluded that, pavements with coarse-graded 9.5 and 12.5mm NMAS[7] becomes excessively permeable at approximately 7.7% voids content and they suggested a critical field permeability value of 1 x 10^{-3} cm/sec. At this critical value the pavement becomes excessively permeable.

In summary, literature supports that permeability is an important factor in asphalt mix design. Researchers agree that the void content is a most significant factor but not on its own, also void structure plays important part. Type of grading, thickness and nominal maximum aggregate size influence the permeability. Finally, it has been shown that laboratory permeability test results can give good indication or estimate of field permeability.

[7] Nominal Maximum Aggregate Size: is the largest sieve that retains some of the aggregate particles but generally not more than 10% by weight

3.3.6.1 Test Method

There two test methods of measuring permeability, a constant head method and a falling head method. The constant head method is more appropriate for measuring high permeable materials ($K > 10^{-3}$ cm/sec), and falling head method is more appropriate for measuring materials ($K < 10^{-3}$) (WisDOT, 2004). Asphalt mixtures have typical values of permeability in the range of $10^{-3} – 10^{-5}$ cm/sec, which makes falling head test method a better choice.

The method used to test permeability in this case was *falling-head* test, for reasons explained in the previous paragraph. The compacted sample in the mould was sealed with a rubber membrane and bolted with a steel cover on both ends; to prevent leaking of the water. The top of the cylinder was connected to a stand-pipe, as the water from the stand-pipe flows through the sample, the level in the stand-pipe falls. The two readings of water head, h_1 and h_2, are taken at a time interval of t. The permeability was calculated with the formula below;

$$k = \frac{a}{A} \cdot \frac{L}{t} \ln\left(\frac{h_1}{h_2}\right)$$

where a = cross-sectional area of the stand-pipe

A = average cross-sectional area of the sample

L = length of the sample

t = duration between h_1 and h_2

h_1 = initial head

h_2 = final head

Figure 21: Specimen during Permeability Testing

3.3.6.2 Analysis of Permeability Test Results

Permeability testing was done on two specimens from each of products, which were prepared as all other specimens by using Hugo hammer and cured on short-term curing condition. Table 22 and Figure 22 shows permeability test results, which suggest that Product C is the most permeable of all products. It was expected that Products A, B and D to have low close permeability results compare to Product C and E; and Product E to have highest permeability of all products due to highest voids content. Voids structure and type of grading might be the reason for this deviation. Results also agree with the data from research done by Hainin et al (2003), which showed that, for coarse- and fine-graded mixes, the permeability begins to increase at greater rate at approximately 9% air voids. At this air void content, the pavement is expected to have permeability of 2 x 10^{-3} cm/sec for coarse-graded mix and 5 x 10^{-4} cm/sec for fine-graded mix. But they cited that, at air voids larger than 11%, the fine-graded mix becomes more permeable than coarse-graded mix. All products have voids content higher than 11% and product 3 is fine continuously graded asphalt mix.

All results were higher than the value suggested by Cooley et al (2001) that coefficient of field permeability should be less than 1 x 10^{-3} for the mixes with NMAS of 9.5mm or 12.5mm.

Table 22: Permeability results

Product	Coefficient of Permeability (cm/sec)
A	0.00624
B	0.00702
C	0.00818
D	0.00511
E	0.00509

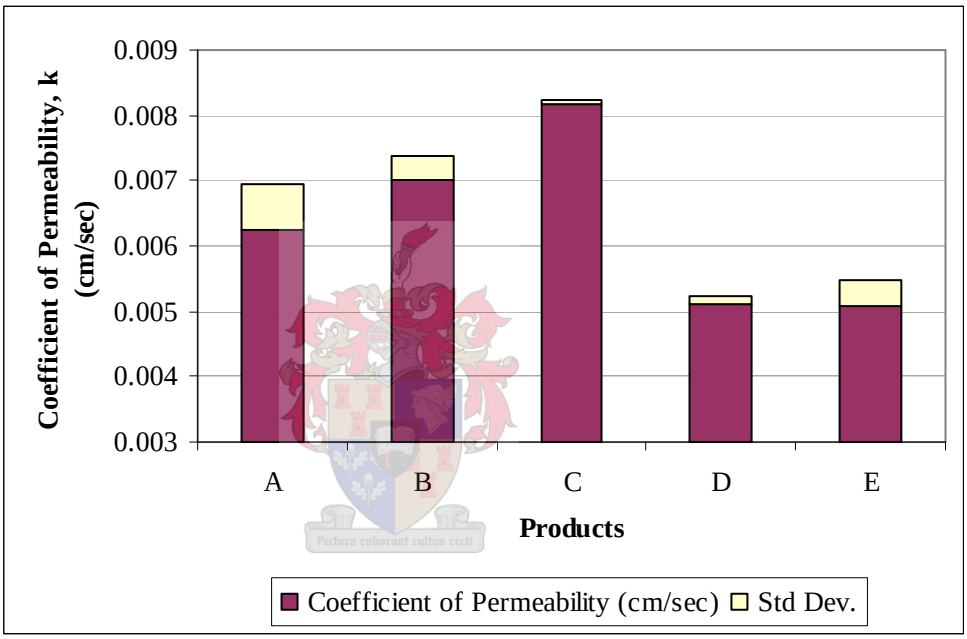

Figure 22: Coefficient of Permeability

3.4 PROBLEMS FACED WITH SPECIMENS PREPARATION AND CURING

There were several problems encountered during specimens' preparation and specimens curing. The major problems with specimen compaction for cold patching mixes which has bitumen emulsion as binder (Product D and E) using the Hugo hammer compaction method is that, the indents in the hammer leaves loose materials on the top surface of the specimen. So this leaves the specimen with a rough surface. Figure 23 shows one the sample after compaction.

Figure 23: Product D – Compaction Problem

One of the main problems for the Products A and B (they have cutback bitumen as binder) was that, just after compaction when removed from moulds the mixes could not contain their integrity, the only way to resolve the problem was to leave specimens in the moulds for five days or more. Figure 24 shows two specimens which collapsed under their own weights after extracted from moulds.

The second problem for Product A was the sticking of the mix to the bottom surface of the oven. For medium curing condition, Products A, B and C were left in the oven for five days at 50°C. During this time Product, A has a problem of binder draining-down; this made the mix to stick to the bottom of the oven and made it difficult to remove afterwards. Figure 25 shows the bottom of the specimen still in the mould and part of it left out in the oven.

Figure 24: Problem with integrity of specimens of Product A

Figure 25: Curing Problems with Product A

3.5 SUMMARY

This chapter has shown that all tested products have relatively high voids content after being compacted using the Hugo hammer at ambient temperatures. As will be discussed in Chapter 4, MMLS3 testing resulted in a significant increase in BRD for products C and E due to MMLS3 compaction effects.

ITS tests have shown that type of binder plays a major role in water damage susceptibility. Emulsion based products showed significant increase in strength within a short period of time, but they are highly susceptible to water damage than cutback based products. For cutback based products, they take time to gain strength but less susceptible to water damage. ITS tests results showed that products tested have low ITS as to minimum specified value of 800 kPa.

With regard to permeability, all products are excessively permeable as per researches done by Choubane et al (1998) and Cooley et al (2001).

Table 23 shows summary of engineering properties based on the type of binder used in the tested products.

Table 23: Summary of Engineering Properties

FACTOR	BINDER	
	CUTBACK	EMULSION
ITS_{DRY}	Variable: Product A very low	Variable
$ITS_{SATURATED}$	Relatively good	Very Poor to Poor
Influence of Curing Time	Small	Significant
Permeability	Very High	High

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Chapter 4

TESTING: PERFORMANCE PROPERTIES (MMLS3)

4.1 INTRODUCTION

This chapter describes a short background of MMLS3 testing, materials used during testing; testing set-up and it also present tests results and discussion of the results. The comparison of rut results between products was made. MMLS3 testing was performed on laboratory prepared specimens at 50°C.

4.2 BRIEF BACKGROUND OF MMLS3 TESTING

Accelerated pavement testing (APT) may be defined as "the controlled application of a prototype wheel loading, at or above the appropriate legal load limit to a prototype or actual, layered, structural pavement system to determine pavement response and performance under a controlled, accelerated, accumulation of damage in a compressed time period." For small scale APT, the definition must include wheel loading below the legal load limit (Douries, 2004). APT enables pavement engineers to gain insight into pavement-related problems in a relatively short period of time.

MMLS3 was developed in South Africa and it has been used as an APT tool to evaluate asphalt pavement performance in terms of rutting and fatigue since 1997 (Hugo et al, 2004). MMLS3 is a third scale unidirectional vehicle-load simulator used for trafficking of model or full-scale dry or wet pavements. The device consists of four re-circulating axles, each with a single 300mm diameter pneumatic tyre wheel. These wheels can be laterally displaced across 80mm in a triangular distribution about the centre line to simulate traffic wandering, if desired. The machine has maximum wheel load of 2.7 kN and maximum tyre pressure of 700 kPa (Muller, 1999). It can apply about 7200 wheel loads per hour. Its nominal wheel speed is about 9km/hr, which corresponds to a loading

frequency of 4 Hz and equal the loading time for a truck wheel travelling at 27 km/h. Figure 26 shows schematic diagram of MMLS3.

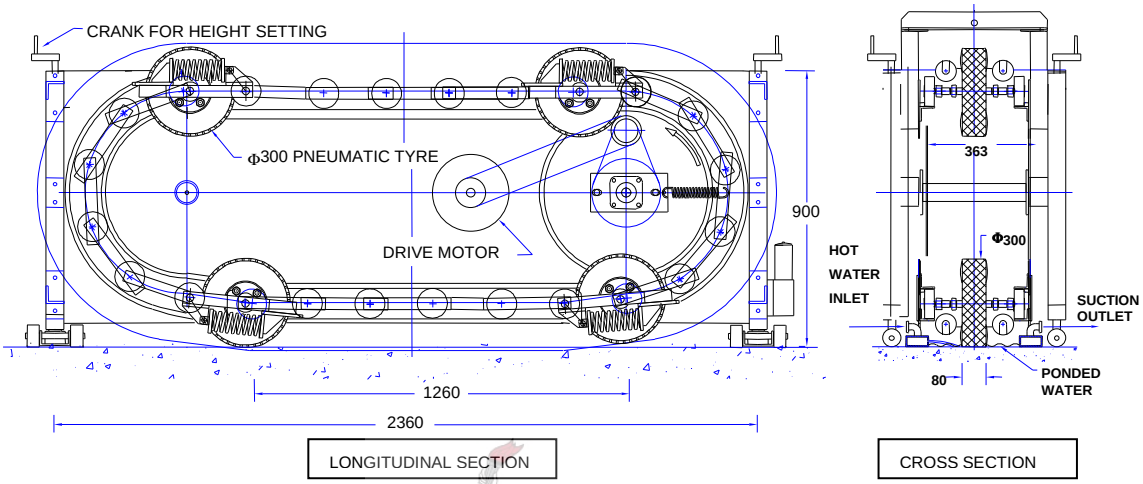

Figure 26: MMLS3 Schematic Diagram

MMLS3 testing can be applied in the following;

- Field test
- Laboratory tests
- Cores extracted from the field
- Laboratory prepared briquettes (cylindrical specimens)
- Laboratory prepared slab specimens
- Laboratory prepared scaled pavements

MMLS3 machine can simulate and easily control different environmental conditions. The test temperature can be controlled within the range of -5°C up to 60°C depending on the abilities of the testing facility.

Main advantages of MMLS3 machine are; it has high rate trafficking: more than 7,200 simulated axle loads per hour can be applied; the load is moving in one direction and it

can be used in the laboratory as well as be transported to fields for full scale testing of asphalt pavements.

MMLS3 can be used for the evaluation of performance of new pavement materials as done in this research, also can be used for evaluation of distress mechanisms such as water susceptibility and selection of rehabilitation strategies.

4.3 MATERIALS

Five cold asphalt patching mixes were tested; these are Products A to E (as explain in previous chapter). Their aggregate grading and other engineering properties were discussed in previous chapter. In this study two MMLS3 tests were performed; the first test had three mixes which all of them have cutback as binder i.e. Product A, Product B and Product C. The second test included two mixes, which are Product D and Product E; these two products had bitumen emulsion as binder

4.4 SPECIMEN PREPARATION AND CURING

Cylindrical specimens with 150 mm diameter were used for this testing. All specimens were compacted at ambient temperature using the Hugo hammer, the same as specimens used for other tests. The curing condition for the specimens was medium curing condition which is the same as the one used for curing specimens used for ITS testing, see section 3.3.5.1.

After curing all specimens were cut to width of 105mm and height of 60 ± 1mm, Figure 27 shows the plan of the specimen after cutting.

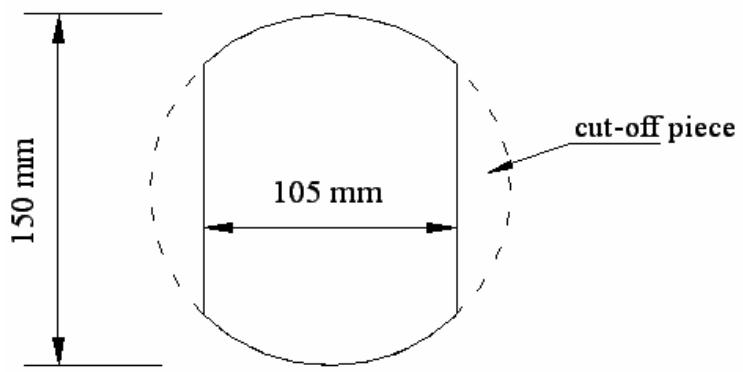

Figure 27: Schematic Plan of Specimen after Cutting

4.5 MMLS3 TEST SET-UP

The MMLS3 tests specifications were as follows; the wheel load was 2.7 kN and tyre pressure was 700 kPa. The rate of loading of 6480 repetitions per hour was applied. The wheel lateral wander was not considered in this study.

The test set-up consisted of nine (9) specimens including 2 dummy specimens on each side. Figure 28 and Table 24 shows the set-up arrangement of the two tests.

Table 24: Mixes arrangement during testing

Position	Test 1 Mixes	Test 2 Mixes
1	*Dummy*	*Dummy*
2	*B*	*D*
3	*B*	*D*
4	*C*	*D*
5	*C*	*D*
6	*C*	*E*
7	*A*	*E*
8	*A*	*E*
9	*Dummy*	*Dummy*

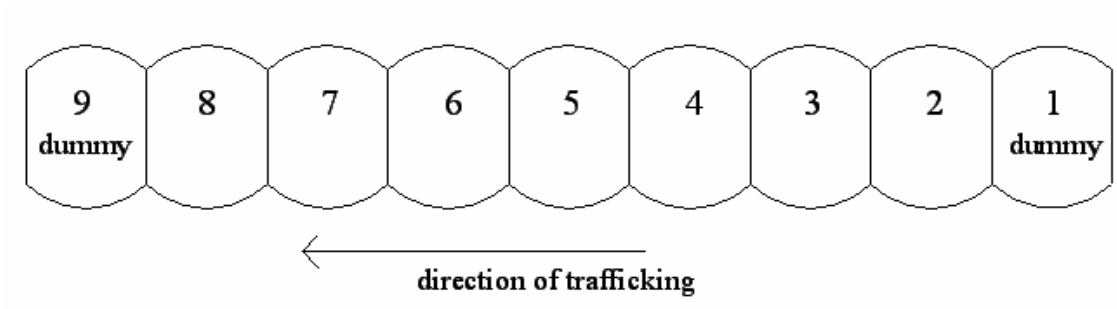

Figure 28: Schematic plan of test set up

The heating process was done by blowing hot air using heaters from both sides of the MMLS3 machine and at the same time the machine was covered by insulating material to prevent heat loss. The test temperatures were measured using thermocouples placed between specimen 3 and 4 at mid-depth. The surface temperature was maintained between 50°C to 55°C. After every profile measurements, specimens were reheated with heaters until the temperature was 50°C.

Figure 29: Laser Profilometer

The transverse profilometer measurements were taken before trafficking i.e. at 0 load repetitions, and from then on at specific intervals i.e. 2 500, 5 000, 10 000, 20 000, and 50 000. The maximum number of load repetitions was 20,000 for test 1 and 50,000 for test 2.

This was done so, since the rut depths were significantly high to continue the test up to 100,000 load repetitions. All transverse surface profiles were measured by using a laser profilometer. Figure 29 shows laser profilometer used during MMLS3 testing.

4.6 MMLS3 TEST RESULTS AND DISCUSSION

When the load is applied, the maximum deformation occurs directly under the center of the load contact area (Lee, 2003). The *reference method of analysis* (Epps et al, 2001) was used in calculating rut depth. By this method, the initial profile before trafficking and a second profile after specific amount of trafficking are recorded and compared to obtain the maximum difference between two profiles.

In the research done by Epps et al (2001), the rut depth criteria MMLS3 after approximately 100,000 load repetitions with two to five replicate measurements for 95 percent reliability level were presented. These criteria were based on MMLS3 testing at critical temperature for rutting over an extremely hot period during summer. Table 25 shows rut depth criteria for MMLS3 on Hot Mix Asphalt after approximately 100,000 load repetitions.

Table 25: Rut Depth Criteria for MMLS3 after Approximately 100,000 Load Repetitions (Epps et al, 2001)

Sample Size (n)	Acceptable Mean Rut Depth (mm)
2	3.0
3	3.5
4	3.7
5	3.9

Figures 30 to 34 present progressive surface deformation at different number of load repetitions for all of five products. Figure 30 shows progressive surface deformation for Product A, it can be seen that there was no densification during testing due to shoving of the mix. The mix has heaving of 5.8 mm on right (there was no heaving on left on all specimens since the wheel load was not at the centre of the specimens). For Product B; as

it can be seen in the Figure 31, the mix exhibit the same trend of shoving as Product A, but with little densification and heaving of 6.5 mm.

Figure 32 shows progressive deformation for Product C, which indicates significant densification due to MMLS3 trafficking and heaving of 3 mm. Product D (Figure 33) has little densification as compare to other products and very little heaving. Figure 34 shows that Product E had significant high densification during MMLS3 testing and has heaving of 2.7 mm.

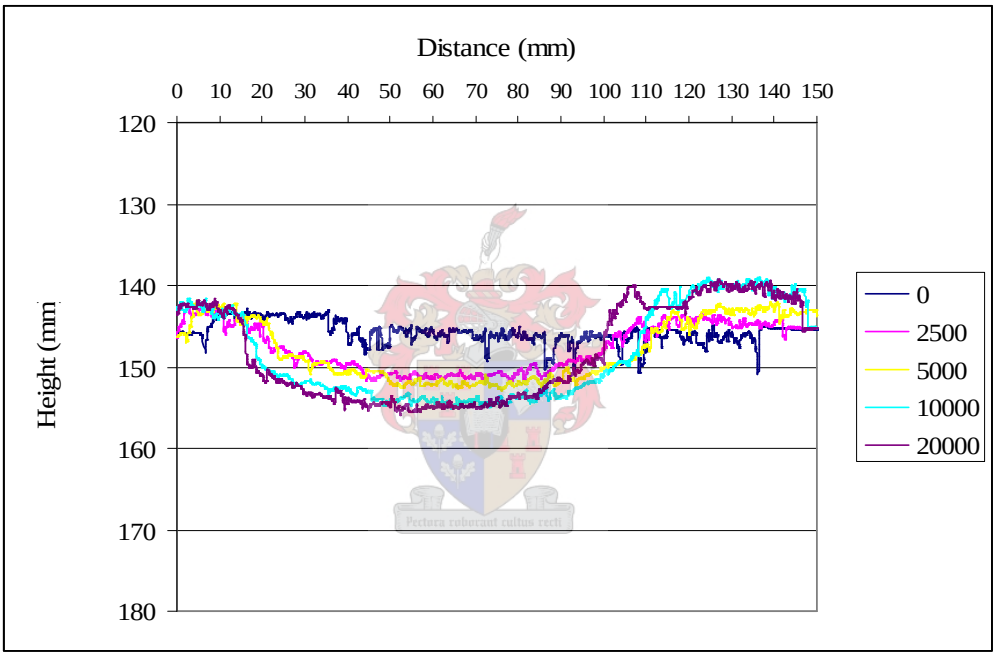

Figure 30[8]: Texture and Rut Profile Comparison for Product A using MMLS3 (Dry) at 50^0C

[8] Note: 1. Height is measured from an outside reference point
2. MMLS3 wheel trafficking between 30 and 110mm in the horizontal distance location

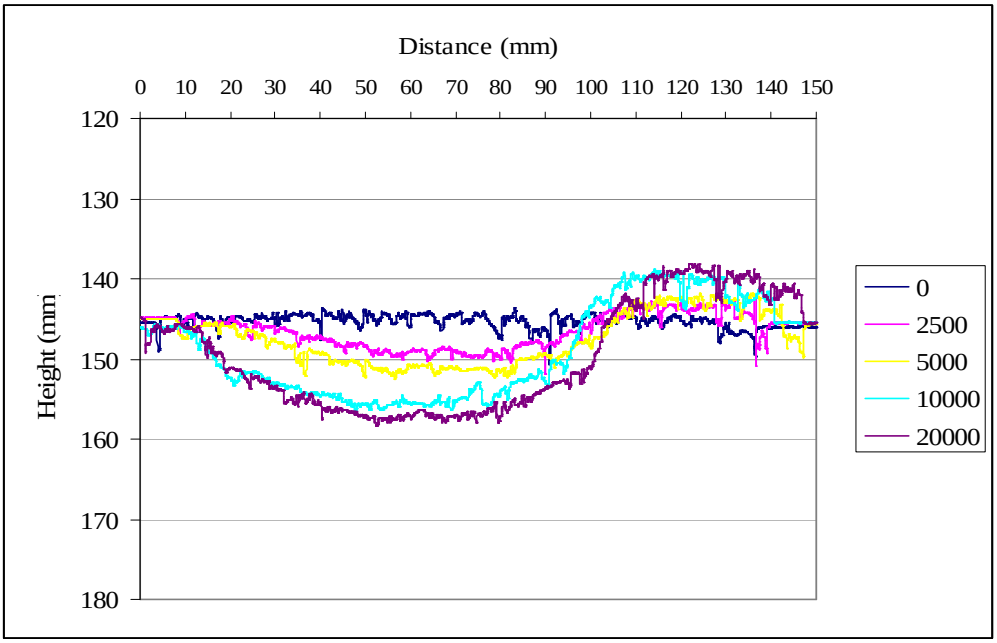

Figure 31: Texture and Rut Profile Comparison for Product B using MMLS3 (Dry) at 50^0C

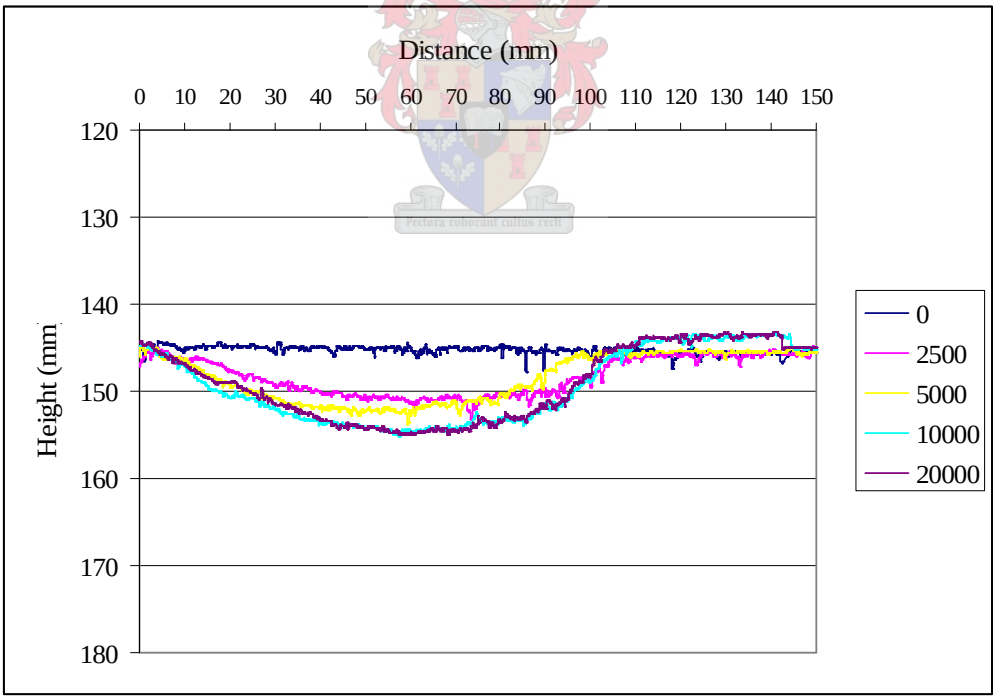

Figure 32: Texture and Rut Profile Comparison for Product C using MMLS3 (Dry) at 50^0C

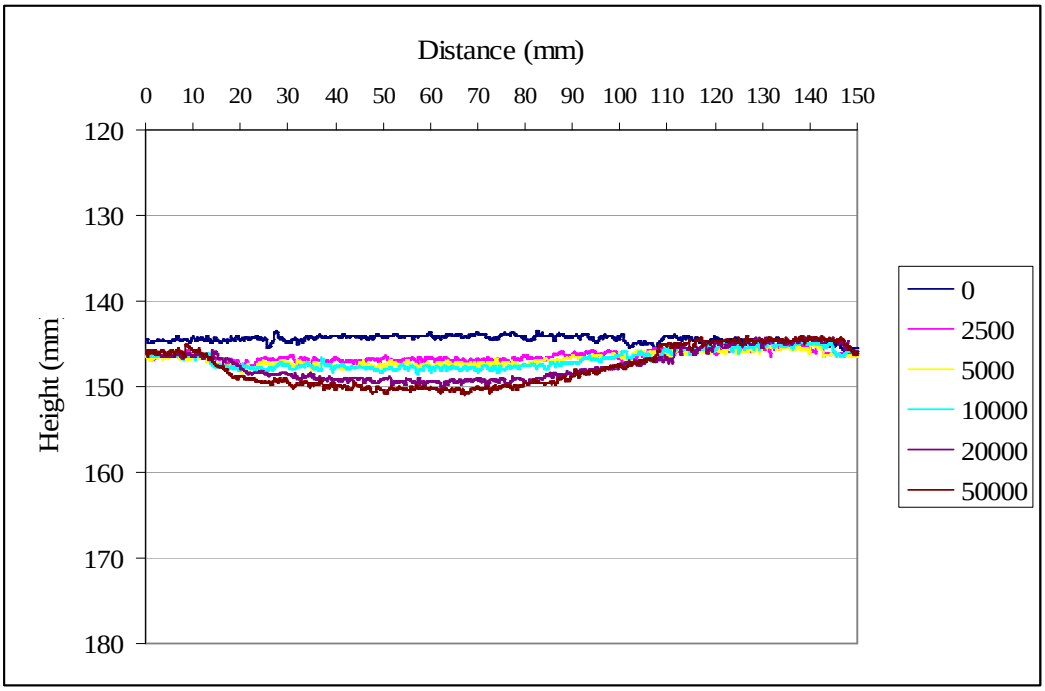

Figure 33: Texture and Rut Profile Comparison for Product D using MMLS3 (Dry) at 50^0C

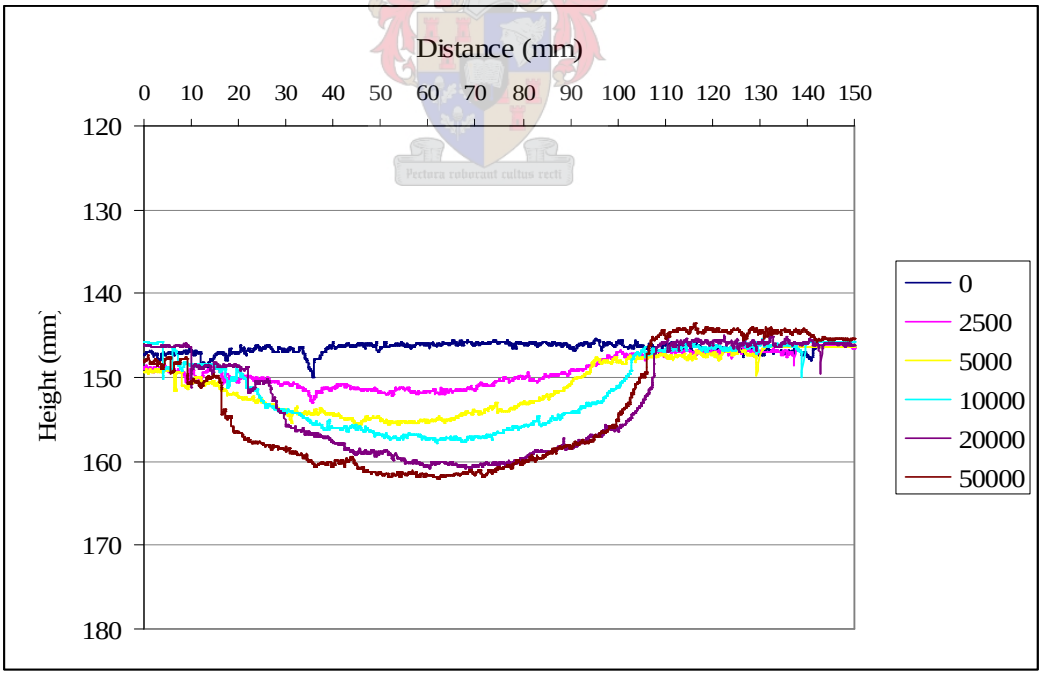

Figure 34: Texture and Rut Profile Comparison for Product E using MMLS3 (Dry) at 50^0C

Table 26 summarizes the MMLS3 rutting results for both tests (i.e. for all products). The cumulative rutting curves for all of the products are shown in Figure 35.

Table 26: MMLS3 Rutting Results (mm)

Load Repetitions	Product				
	A	***B***	***C***	***D***	***E***
0	0	0	0	0	0
2500	7.8	5.0	5.5	2.7	4.5
5000	10.0	7.5	7.0	3.3	7.5
10000	13.5	13.0	9.2	4.0	11.5
20000	14.5	15.5	9.8	4.7	15.0
50000	--	--	--	5.5	16.7

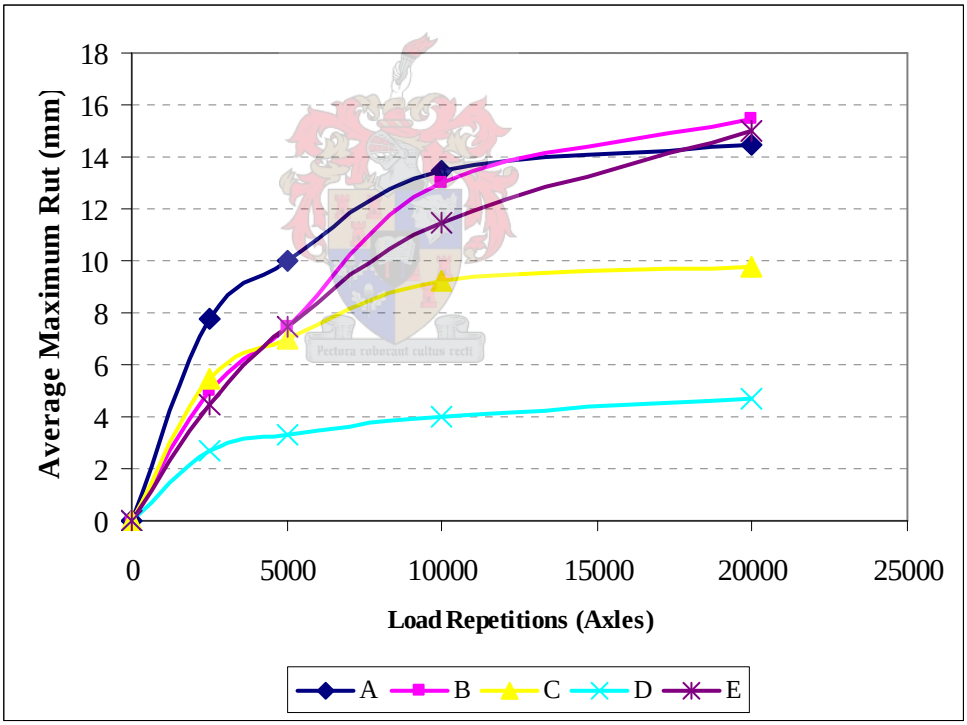

Figure 35: Cumulative Rutting

From Figure 35 it can be seen that Product B has maximum rutting but there is small difference in rut depth between Products A, B and E. Product D has minimal rutting compared to other products. For Products A and C the rate of rutting stabilizes after 10,000 load repetitions, while for Product B and E, their rutting rate continues to increase after 10,000 load repetitions.

Figure 10 shows grading of all products. As seen on the Figure 10 and Figure 35, Product D has finer aggregate grading and minimum rutting. The study done by Brown and Besset (1990) concluded that increasing maximum aggregate size in a mix should increase the mix's resistance to rutting, this didn't apply in the Product E which has higher maximum aggregate size compare to other products grading. Briefly the rutting results did not show any correlation between aggregate grading and rut depth of the products. Kandhal and Cooley Jr. (2002) conducted a research to compare rutting susceptibility of mixes having coarser and fine grading; the results indicated that there are no significant differences in rut potential between the two grading types.

Moreover, there was no any significant correlation between MMLS3 test results and ITS values.

Relating MMLS3 Laboratory Results To Field Performance

The main reason of laboratory testing of an asphalt mixes is to estimate the performance of the mixes in the field. This section presents relation between MMLS3 laboratory results and field performance of the mixes by using Douries (2004) laboratory and field results comparison.

The ratios of laboratory rut to field rut during wet tests were calculated. These ratios were calculated using rut depths at 20,000 and 100,000 load repetitions. The wet tests results were used instead of dry test results since the laboratory dry test was not done. The assumption was made that the rate of rutting during MMLS3 wet testing is equal to the rate of rutting during dry testing. The ratios were calculated as follows by using Figure 36

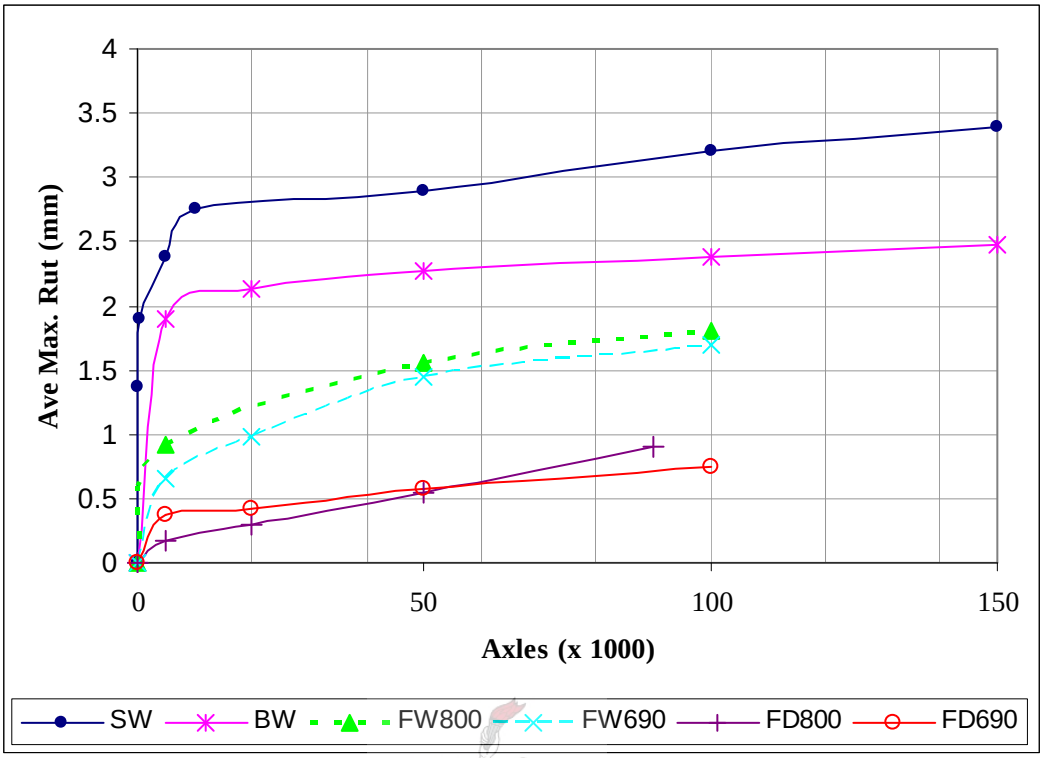

Figure 36: Cumulative Rutting Curves for HMA (Douries, 2004)

Ratio at 20,000 Load Repetitions for MMLS3

$$\frac{\text{Laboratory briq. rut}}{\text{Field rut}} = \frac{2.125}{0.974} = 2.182 \quad \text{..... (4.1)}$$

Ratio at 100,000 Load Repetitions for MMLS3

$$\frac{\text{Laboratory briq. rut}}{\text{Field rut}} = \frac{2.375}{1.7} = 1.397 \quad \text{..... (4.2)}$$

Table 27 and Table 28 shows estimated field rut depths at 20,000 and 100,000 load repetitions respectively. In this study, the first MMLS3 test was stopped at 20,000 load repetitions and the second at 50,000 load repetitions. The rut depths for 100,000 load repetitions were determined by extending lines in the cumulative rutting graph (see Figure 35) up to 100,000 while assuming the constant rate of rutting for all mixes.

Table 27: Expected Field Rut After 20,000 Load Repetitions

Product	Actual Laboratory Rut (mm)	Estimated Field Rut (mm)
A	14.5	6.6
B	15.5	7.1
C	9.8	4.5
D	4.7	2.2
E	15.0	6.9

Table 28: Expected Field Rut After 100,000 Load Repetitions

Product	Extrapolated Laboratory Rut (mm)	Estimated Field Rut (mm)
A	15.8	11.3
B	19.5	13.9
C	11.5	8.2
D	7.0	5.0
E	19.0	13.6

From Table 27, it can be seen that just after 20,000 all products except Product D have field rut depth of more than 3.5mm which is acceptable average RD after 100,000, see Table 25 (Epps et al, 2001). Table 28 shows estimated field rut depths after 100,000 load repetitions, and it can be seen that all products have exceeded acceptable RD of 3.5mm. Product D has minimum rut depth of 5.0mm follows with Product C.

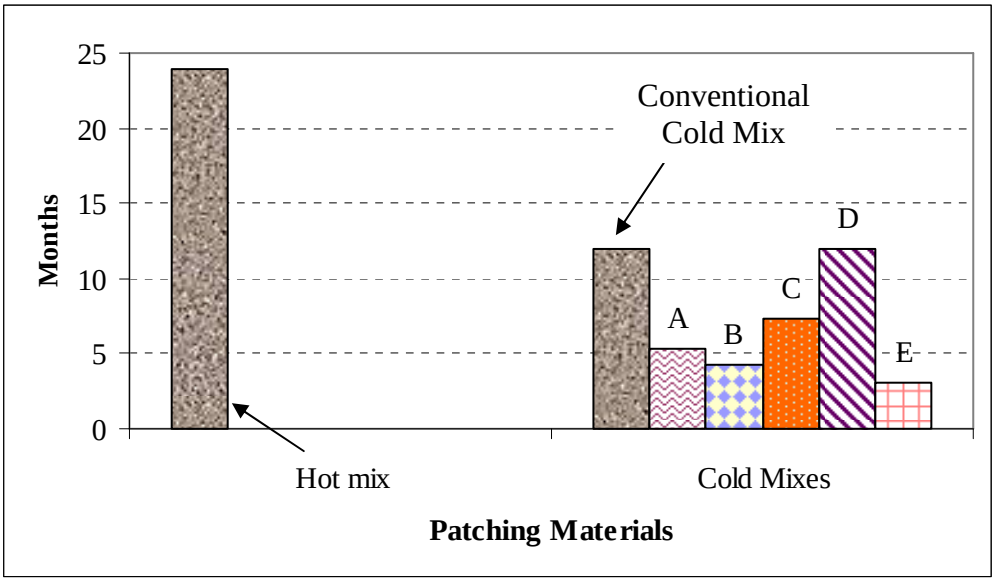

Figure 37: Life expectancy of hot mix and estimated cold mix patching materials

Figure 7 in section 2.6.3 shows life expectancy of hot mix and cold mix patch materials placed in permanent and temporary conditions. Taking the data of patch materials placed in permanent conditions and assuming that the conventional cold mix used in Figure 7 has the same quality (same level of traffic and no effect of lateral wander) as Product D (which has the least rut depth compare other products tested) with 12 months patch life. The aim of Figure 37 is to compare life expectancy of cold mixes and Hot Mix Asphalt and as seen cold mix patch materials have much less patch life compare to HMA.

RUT PREDICTION

The MMLS3 is an accelerated pavement testing device that applies realistic trafficking to the pavement. The load is scaled, but tyre pressures are at the same level as in full scale trucks. Accordingly, the results are transferable to conventional real life trafficking. The device can be used directly to explore performance of the upper 125mm of asphalt pavements in the laboratory and in the field. The MMLS3 can characterize a mix and predict actual performance if factors such as load frequency, temperature, aging and lateral wander of load applications are taken into account (Smit et al, 2004).

Application of the MMLS3 to predict rutting performance is based on the hypothesis that for a simple pavement structure with only a single AC layer and similar tire pressures, the distribution of maximum vertical compressive stress beneath the MMLS3 will be comparable to that beneath a single truck tire with a standard load of 1/4 ESAL, if the scaling factor (one-third) and dimensionless depth are considered. For example, the stress at mid-depth of a 450mm AC layer on a stiff subgrade under full-scale loading is expected to be equivalent to the stress at mid-depth of a 150mm (1/3 * 450mm) layer under the MMLS3 (Epps et al, 2003).

Rut Prediction Procedure (Smit et al, 2003)

A theoretical rutting ratio (TRR) is defined as the ratio of rutting determined using stress analysis under the MMLS3 to that under full-scale trucks:

$$TRR = \frac{\text{Theoretical RD}_{\text{MMLS3}}}{\text{Theoretical RD}_{\text{Trucks}}} \qquad \text{(i)}$$

The first step in determining the TRR is to calculate a stress potential (SP) as the area under the vertical compressive stress with depth curve up to the depth of influence:

$$SP = \int_{z1}^{z2} \sigma_z \cdot dz \qquad \text{(ii)}$$

The procedure used to compare rutting performance relates the relative rutting damage to the vertical compressive stress under full- and model-scale trafficking. Elastic layer analysis may be used to determine the vertical compressive stresses with depth. The influence of modulus is determined by applying a temperature and frequency correction factor (TFC) based on;

1. the ration of G^* values adjusted for both temperature and frequency in the surfacing layer

2. the assumption that the accumulation of permanent deformation is inversely proportional to shear stiffness (G^*):

$$TFC = \frac{G^*_{Trucks}}{G^*_{MMLS3}} \qquad \text{(iii)}$$

The rutting potential ratio (RPR) under the MMLS3 to that under full-scale trucks is used as the best estimate of the TRR calculated as follows:

$$RPR = \frac{TFC \cdot SP_{MMLS3}}{SP_{Trucks}} = TRR \qquad \text{(iv)}$$

By dividing the asphalt layer into sub-layers (n), a more accurate estimate of the TRR may be obtained by calculating RPR using the following equation:

$$RPR = \frac{\sum_{i=0}^{n} TFC_i \cdot SP_i^{MMLS3}}{\sum_{i=0}^{n} SP_i^{Trucks}} \qquad \text{(v)}$$

The field rutting ratio (FRR) is defined as the rut depth (RD) under the MMLS3 to that measured under the full-scale trucks:

$$FRR = \frac{RD_{MMLS3}}{RD_{Trucks}} \qquad \text{(vi)}$$

The ratio of the theoretical and field rutting ratios defines the rutting prediction ratio (PR):

$$PR = \frac{TRR}{FRR} \qquad \text{(vii)}$$

With a PR of 1, a prediction of the full-scale rutting may be determined:

$$RD_{Trucks} = \frac{RD_{MMLS3}}{TRR} \qquad \text{(viii)}$$

Typically a correction factor must be applied to the damage to account for differences between full- and model-scale trafficking. Hence, the rutting prediction ratio (PR) with correction factor (CF) applied becomes:

$$PR = \frac{TRR}{FRR} \cdot CF \qquad \text{(ix)}$$

The factor correction factor (CF) must account for a number of different influences, which follows; (i) temperature and frequency (ii) aging (iii) dynamic effects (iv) density (v) lateral wander and (vi) effective trafficking volumes and microclimate.

From the above-described procedure TFCs [using equation (iii)] was calculated differently for products with cutback binder and emulsion binder and by using Stress Potential from Table 6 of Smit et al, 2003 we have TRR values as shown in the Table 29 and Table 30 below. By using calculated TRR and equation (vi), we get estimated field ruts by using Smit et al procedure. These values are shown in the Table 31 below.

Table 29: Determination of TRR for Cutback based Products

Sub-layer	**SP_{MMLS3}**	**SP_{Trucks}**	**Depth (mm)**	**TFC**	**TFC*SP_{MMLS3}**	**TRR**
0 - 20	19.6	21.1	10	1.3	25.5	
20 - 40	12.1	20.0	30	1.3	15.7	
40 - 60	6.4	17.6	50	1.3	8.3	
Total		**58.7**			**49.5**	0.84

Table 30: Determination of TRR for Emulsion based Products

Sub-layer	SP_{MMLS3}	SP_{Trucks}	Depth (mm)	TFC	$TFC*SP_{MMLS3}$	TRR
0 - 20	19.6	21.1	10	1.2	23.5	
20 - 40	12.1	20.0	30	1.2	14.5	
40 - 60	6.4	17.6	50	1.2	7.7	
Total		**58.7**			**45.7**	0.78

Table 31: Estimated Field Rut

Product	Extrapolated Laboratory Rut (mm)	Estimated Field Rut (Douries) (mm)	Estimated Field Rut (Smit et al) (mm)
A	15.8	11.3	18.7
B	19.5	13.9	23.1
C	11.5	8.2	13.6
D	7.0	5.0	9.0
E	19.0	13.6	24.4

Table 31 compares extrapolated laboratory rut depths with estimated field rut depths calculated by using Douries and Smit et al procedures. As it can be seen on the Table 31, field rut depths by Douries (2004) are lower compared to extrapolated laboratory rut depths. This was explained by Douries (2004) as due to the fact that the confinement in the laboratory specimens is different than in the field. Also, in the laboratory, the temperature can be better controlled. In the field, there is more temperature variation. This variation is more frequently below the desired test temperature. The ageing of the specimens may also have contributed to the differences in the results.

The estimated field rut depths by Smit et al as shown on the Table 31, are higher than extrapolated laboratory ruts, this is the expected trend. As it can be seen on the Table, Product B and Product E are expected to rut with depth of more than 2cm in the field.

In the research done by Hugo at al (2004), it was concluded that under traffic loading of 1.7 million axles applied at high temperature, it was unlikely that downward rut depth would exceed 10mm. It was also estimated that total rut depth would not exceed 15mm.

By using this information and Figure 10 (Hugo et al, 2004), number axles needed for the tested products to have 15mm total rut depth can be estimated, as shown on the Table 32.

Table 32: Estimated Million E80s

Product	**E80s (x 10^6)**
A	0.5
B	0.4
C	0.6
D	0.9
E	0.3

4.7 SUMMARY

Based on the results on this chapter, it has been shown that all products tested are very susceptible to rutting. Product D is more resistant to rut as compare to other four products while Product B is the least resistant to rutting, with no much difference when compared to Products A and E.

During MMLS3 testing, visual observations were also done. Pushing and shoving of the mixes were observed for Product A and Product B just after 10,000 load repetitions. There was little loss of aggregate on Product D and this problem was mainly on joints between two briquettes. No loss of aggregates was observed for other products. At 10,000 load repetitions, closing up of air voids were observed for all Products but for Product D this effect was less compared to other products. No cracking was observed during this MMLS3 testing.

From above mentioned observations, it shows that rutting is the main problem of these mixes which needs more attention compared to other previously mentioned asphalt mixes problems. Table 33 shows summary of observed failures during MMLS3 testing.

Table 33: Summary of Observed Failures during MMLS3 testing

Failure	**Products**	
	Cutback	**Emulsion**
Rutting	Highly Susceptible	Highly Susceptible
Ravelling	No Ravelling was observed	Ravelling was observed on Product D briquettes joints
Skid Resistance	Medium	Medium
Freeze-Thaw Resistance	Not Applicable	Not Applicable
Pushing/Shoving	Less Stable	Varies from Medium to High Stability

4.8 REFERENCES

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Smit, A. dF., F. Hugo, D. Rand and B. Powell; **Model Mobile Load Simulator Testing at National Center for Asphalt Technology Test track**. Transportation Research Record, no. 1832, Transportation Research Board, Washington DC, 2003

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

From the findings of the study, the following conclusions and recommendations were made.

5.1 SUMMARY

Two fundamentally different types of cold asphalt patching mixes were tested.

Table 34: Summary of Laboratory Testing Results

Cold Asphalt Patching Mixes

		Cutback			Emulsion	
	Mix Sample	A	B	C	D	E
Mix Composition	Binder Content (%)	5.9	6.1	5.5	5.8	5.1
	Air Voids (%)	15.4	15.1	19.4	15.7	23.5
	Moisture Content (%)	N/A	N/A	N/A	7.4	5.7
	Grading	Continuous			Finer	Coarser
Engineering Properties of the Mixes	Average ITS -short term cure (kPa)	17.8	90.1	85	116.6	63.2
	Average ITS -medium term cure (kPa)	53.5	228.9	114.5	266.8	68.8
	Average TSR -short term cure	0.87			0.32	
	Average TSR - medium term cure	0.9			0.59	
Permeability of the Mixes	Average Coefficient of Permeability (cm/s)	7.15×10^{-3}			5.1×10^{-3}	
MMLS3	Rut depth after 20,000 (mm)	9.5	11.5	9.8	4.7	14.6

5.2 CONCLUSIONS

It can be concluded that;

- Residual binder content of emulsion mixes ranges from 5.1% to 6.1%. Type of binder of binder did not play role in the binder content.

- All cold asphalt patching mixes tested in this study have high void contents when compacted at ambient temperatures. Void contents of mixes ranges between 15.1% and 23.5%. There was no direct correlation between void content of the mixes and their respective aggregate grading.

- Indirect Tensile Strength values were found to be below the specified minimum value in South Africa of 800kPa for Hot Mix Asphalt. The type of binder played significant role in strength gaining. Mixes with bitumen emulsion as binder, had the advantage of higher rate of gain of strength with time compared to other mixes with cutback bitumen as binder.

- Mixes with cutback had higher values of TSR than mixes with emulsion, which means that they are less susceptible to water damage compared to emulsion mixes.

- The products with cutback as binder had higher values of permeability coefficient than products with emulsion as binder. All mixes have higher coefficient of permeability than specified value of 1×10^{-3} cm/sec (Cooley et al, 2001), this makes these cold asphalt patching mixes to be very permeable.

- The products which contain bitumen emulsion as binder (D and E), had lower permeability coefficient values than other products with cutback bitumen. Their values were close to each other despite the fact that product D had void content of 15.1% and product E had a void content of 23.5%. These results conclude that voids were not directly correlated to permeability of the mixes, i.e. other factors played a role too, such as connectivity of voids.

- MMLS3 testing results concluded that, all patching mixes tested are very susceptible to rutting, with rut depth ranging from 4.7mm to 14.6mm after just 20,000 load repetitions. By comparison, a good HMA mix will only experience 3.5mm after 100,000 load repetitions.

- Type of binder and aggregate grading of the mixes did not show any correlation with rutting. It was also found that there was no correlation between Indirect Tensile Strength and rutting performance of the mixes.

- Estimated field rut depths ranges from 9mm to 24.4mm after application of 100,000 load repetitions. Table 32 shows that just after 0.3 million E80s, Product E will have rutting of 15mm and for the same rut depth Product D can withstand 0.9 million E80s.

5.3 RECOMMENDATIONS

- In this research only laboratory tests were done and so it is recommended that field performance be investigated with the products used in this study and the results be compared with these laboratory tests results. The monitoring of field patches should be done over a period of six months to one year depending on available time. The filling of natural potholes can be done using either methods specified by products respective manufacturer, throw and roll procedure or semi-permanent method. The field tests should be done in wet season and hot/dry season, this is important since products with emulsion as binder has been seen to be very sensitive to water damage while products with cutback as binder are sensitive to high temperatures.

- It is also recommended that laboratory tests be done to check the change of products mix and binder properties during storage/stockpile. An example of mix properties which can be tested would include workability of the mix. Other tests are cohesion mix and coating of the binder.

- During compaction, few difficulties were encountered with compaction method as discussed in section 3.4. Therefore, the investigation of different compaction methods in the laboratory and how it influences the specimens' performance is recommended.
- Further investigation of moisture susceptibility of different patching mixes needs to be made. This could include MMLS3 tests under saturated conditions.

- Standard curing techniques for cold mixes used in CIPR (Cold In Place Recycling) were found to be inadequate to generate sufficient strength for the cold patching mixes. As a result, increased curing times and temperatures were used for cutback patching mixes. Further research is needed to verify which curing procedure is most applicable to these materials.

APPENDICES

APPENDIX A

Binder Content Test and Grading Analysis for Product E

LFS - 01

Much Asphalt Central Laboratory

Asphalt Premix Test Work Sheet Report Ref: CL\06\

Cenlab Sample No	ANNA 1	Date Tested	06/09/2006
Site Sample No		Contract Description	
Date Sampled		Mix Type	Glenpatch
Date Received		Submitted by	

(A) Binder Content: TMH1 : C7(b)		**(C) Maximum Theoretical Density: TMH1:C4**	
Equipment Checked		**Equipment Checked**	
(a) No of Flask		(a) No of Flask	
(b) Mass of Flask + Sample	2528.1	(b) Mass of Flask + Sample	
(c) Mass of Flask	1277.4	(c)Mass of Flask	
(d) Mass of Sample (b-c)	1250.7	(d) Mass of Sample (b-c)	
(e) No of Cup	CL/m004	(e) Mass of Flask + Water @ 25ºC	
(f) Mass of Cup + Fines	265.2	(f) Mass of Flask, Sample + Water @ 25ºC	
(g) Mass of Cup	196.6	(g) Volume of Sample(d+e-f)	
(h) Mass of Fines (f-g)	68.6	**(h) Maximum Theoretical Density (d/g)**	
(I) Mass of Aggregate	1118.4	% Voids in Mix	
(j) Total Mass of Aggregate (h+I)	1187	% Voids in Mineral Aggregate	
(k) Mass of Bitumen (d-j)	63.7	% Voids filled with Bitumen	
(I) % Bitumen in Mix (k/d)*100)	5.10%	**Tested By:**	
Tested By:		**(D) BRD Determination : TMH1 C3**	
		Equipment Checked	
(B) Grading Analysis : TMH1 : B4		Briquette Temperature	
Equipment Checked		(a) No of Briquette	

Sieve	Mass Ret.	% Ret.	%Passing	Site Result	SPEC
37.5 mm					
26.5 mm					
19.0 mm					
13.2 mm	0		100		
9.5 mm	20.8	1.8	98.2		
6.7 mm	391.0	32.9	65.3		
4.75 mm	147.4	12.4	52.9		
2.36 mm	192.7	16.2	36.7		
1.18 mm	101.1	8.5	28.2		
0.600 mm	64.0	5.4	22.8		
0.300 mm	69.5	5.9	16.9		
0.150 mm	62.1	5.2	11.7		
0.075 mm	47.2	4.0	7.7		
~~23.0~~ <0.075mm	91.2	7.7	0		
Tested By:					
BRD of Aggregate Blend		Surface Area			
Filler / Bitumen Ratio		Film Th'ness			
Flakiness Index					

(b) Thickness of Briquette			
(c) Dry Mass in Air			
(d) Surface Dry Mass in Air			
(e) Mass in Water			
(f) Volume of Briquette (d-e)			
(g) R.D of Briquette (c/f)			
(h) Average Rel. Density			
(E) Marshall Stability & Flow : TMH1 C2			
Equipment Checked			
Stability Gauge Reading			
Stability (kN)			
Correlation Ratio			
Corrected Stability (kN)			
Average Stability (kN)			
Flow (mm)			
Average Flow (mm)			
Stability / Flow Ratio			
Tested By:			
Reported by:			

2006-06-02

Binder Content Test and Grading Analysis for Product A

LF 5 - 01

Much Asphalt Central Laboratory

Asphalt Premix Test Work Sheet Report Ref: CL\06\

Cenlab Sample No	ANNA2	Date Tested	06/09/2006
Site Sample No		Contract Description	
Date Sampled		Mix Type	Roadflex
Date Received		Submitted by	

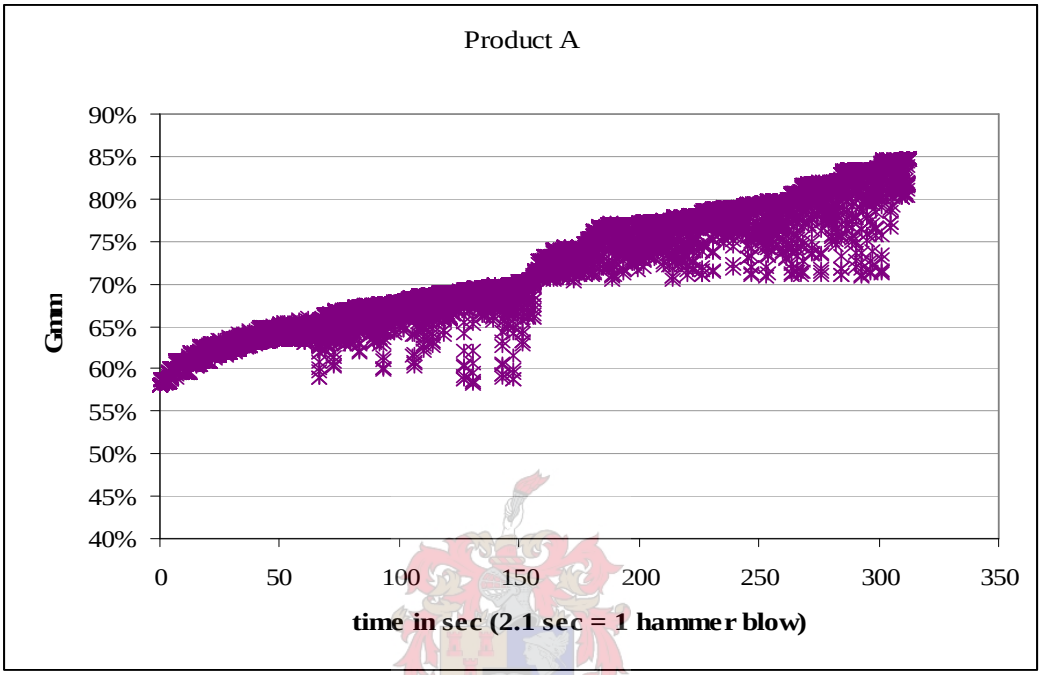
(A) Binder Content: TMH1 : C7(b)		**(C) Maximum Theoretical Density: TMH1:C4**	
Equipment Checked		**Equipment Checked**	
(a) No of Flask		(a) No of Flask	
(b) Mass of Flask + Sample	2542.4	(b) Mass of Flask + Sample	
(c) Mass of Flask	1277.3	(c)Mass of Flask	
(d) Mass of Sample (b-c)	1265.1	(d) Mass of Sample (b-c)	
(e) No of Cup	CL/M005	(e) Mass of Flask + Water @ 25°C	
(f) Mass of Cup + Fines	267.8	(f) Mass of Flask, Sample + Water @ 25°C	
(g) Mass of Cup	215.2	(g) Volume of Sample(d+e-f)	
(h) Mass of Fines (f-g)	52.6	**(h) Maximum Theoretical Density (d/g)**	
(I) Mass of Aggregate	1137.6	% Voids in Mix	
(j) Total Mass of Aggregate (h+I)	1190.2	% Voids in Mineral Aggregate	
(k) Mass of Bitumen (d-j)	74.9	% Voids filled with Bitumen	
(I) % Bitumen in Mix (k/d)*100)	5.9%	**Tested By:**	
Tested By:		**(D) BRD Determination : TMH1 C3**	
		Equipment Checked	
(B) Grading Analysis : TMH1 : B4		Briquette Temperature	
Equipment Checked		(a) No of Briquette	
		(b) Thickness of Briquette	
		(c) Dry Mass in Air	
		(d) Surface Dry Mass in Air	
		(e) Mass in Water	
		(f) Volume of Briquette (d-e)	
		(g) R.D of Briquette (c/f)	
		(h) Average Rel. Density	
		(E) Marshall Stability & Flow : TMH1 C2	
		Equipment Checked	
		Stability Gauge Reading	
		Stability (kN)	
		Correlation Ratio	
		Corrected Stability (kN)	
		Average Stability (kN)	
		Flow (mm)	
Tested By:		**Average Flow (mm)**	
BRD of Aggregate Blend		Stability / Flow Ratio	
Filler / Bitumen Ratio		**Tested By:**	
Flakiness Index		**Reported by:**	

Sieve	Mass Ret.	% Ret.	%Passing	Site Result	SPEC
37.5 mm					
26.5 mm					
19.0 mm					
13.2 mm			100		
9.5 mm	3.4	0.3	99.7		
6.7 mm	97.3	8.2	91.5		
4.75 mm	334.1	28.1	63.4		
2.36 mm	394.5	33.1	30.3		
1.18 mm	125.1	10.5	19.8		
0.600 mm	70.4	5.9	13.9		
0.300 mm	48.9	4.1	9.8		
0.150 mm	33	2.8	7		
0.075 mm	17.1	1.4	5.6		
13.4 <0.075mm	66	5.5			
BRD of Aggregate Blend			Surface Area		
Filler / Bitumen Ratio			Film Th'ness		
Flakiness Index					

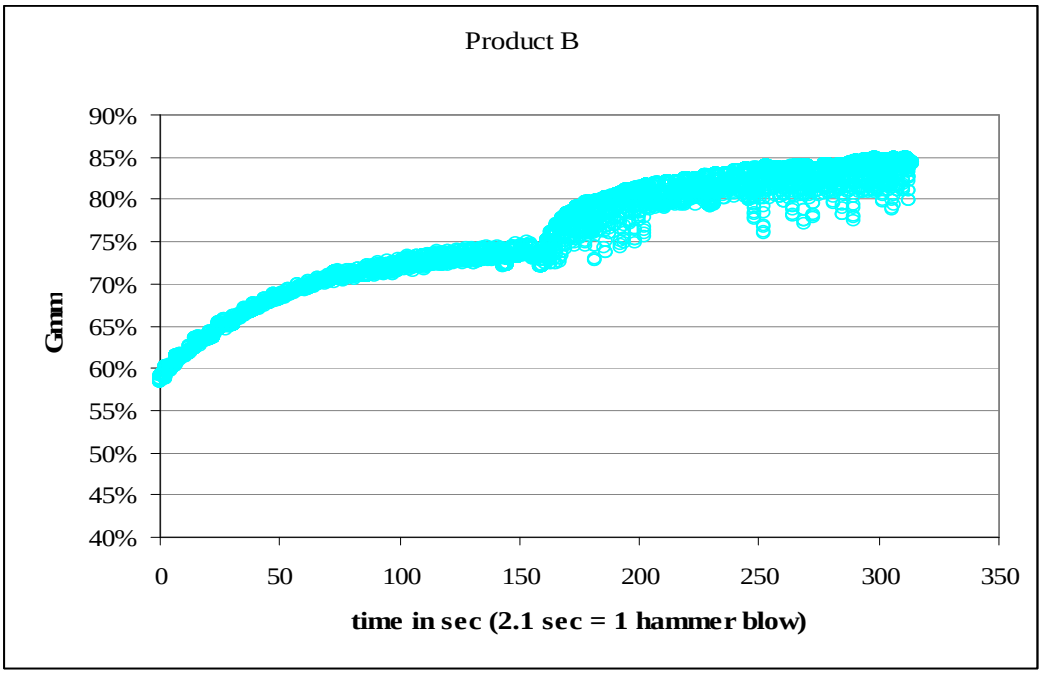
2006-06-02

APPENDIX B

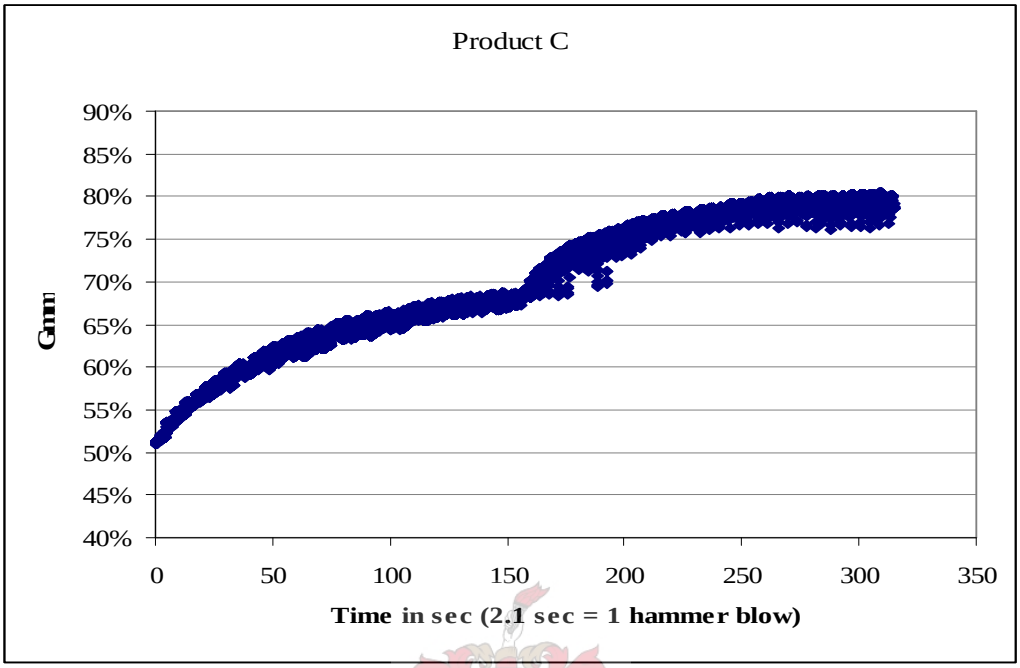
Product A Compaction Curve



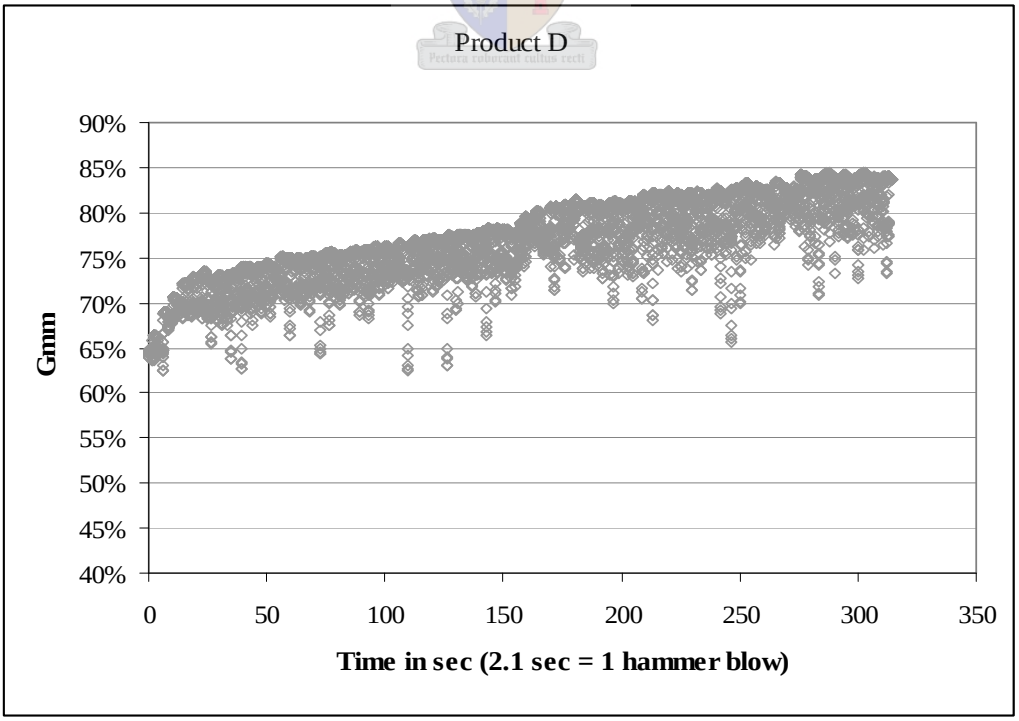
Product B Compaction Curve



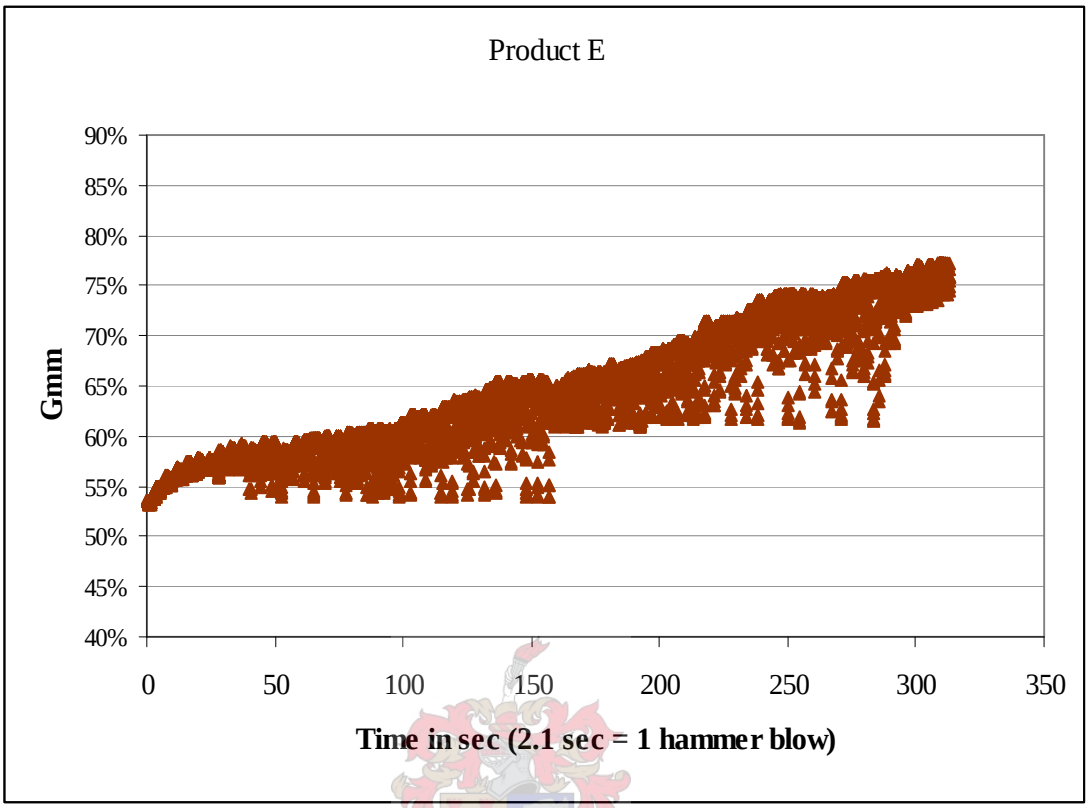
Product C Compaction Curve



Product D Compaction Curve



Product E Compaction Curve



APPENDIX C

INDIRECT TENSILE STRENGTH: TEST RESULTS

Product	Specimen ID	Condition of Specimen	Height (m)	Average Height (m)	Maximum Load (kN)	Average Maximum Load	ITS (kPa)	Average ITS (kPa)	Standard Deviation	TSR
Asphalt King	AK-M-1	dry	0.0594	0.0590	3.787	3.710	270.6	266.8	21.8	0.45
	AK-M-2		0.0586		3.360		243.3			
	AK-M-3		0.0590		3.983		286.5			
	AK-M-4	wet	0.0572	0.0584	1.400	1.666	103.9	120.9	15.1	
	AK-M-5		0.0591		1.753		125.9			
	AK-M-6		0.0590		1.845		132.8			
	AK-S-7	wet	0.0576	0.0586	0.282	0.251	20.8	18.2	4.0	0.16
	AK-S-8		0.0590		0.281		20.2			
	AK-S-9		0.0594		0.191		13.7			
	AK-S-10	dry	0.0595	0.0599	1.574	1.641	112.3	116.5	29.9	
	AK-S-11		0.0594		2.075		148.3			
	AK-S-12		0.0607		1.274		89.0			
Much Mix	MCHX-M-1	dry	0.0664	0.0660	1.863	1.779	119.0	114.5	10.1	0.85
	MCHX-M-2		0.0652		1.868		121.5			
	MCHX-M-3		0.0662		1.605		102.9			
	MCHX-M-4	wet	0.0679	0.0684	1.626	1.562	101.6	97.0	8.0	
	MCHX-M-5		0.0688		1.421		87.7			
	MCHX-M-6		0.0684		1.638		101.6			
	MCHX-S-7	dry	0.0663	0.0663	1.238	1.320	79.2	85.0	25.7	0.92
	MCHX-S-8		0.0645		1.719		113.1			
	MCHX-S-9		0.0681		1.004		62.6			
	MCHX-S-10	wet	0.0664	0.0669	1.666	1.235	106.5	78.5	35.0	
	MCHX-S-11		0.0671		1.418		89.7			
	MCHX-S-12		0.0671		0.622		39.3			

Product	Specimen ID	Condition of Specimen	Height (m)	Average Height (m)	Maximum Load (kN)	Average Maximum Load	ITS (kPa)	Average ITS (kPa)	Standard Deviation	TSR
Tarfix	TFX-S-1	wet	0.0623	0.0619	1.881	0.976	128.1	66.7	53.9	0.74
	TFX-S-2		0.0625		0.653		44.3			
	TFX-S-3		0.0609		0.396		27.6			
	TFX-S-4	dry	0.0671	0.0674	1.385	1.432	87.6	90.1	37.5	
	TFX-S-5		0.0672		0.852		53.9			
	TFX-S-6		0.0679		2.058		128.7			
	TFX-M-7	wet	0.0653	0.0671	2.450	2.949	159.2	186.3	43.8	0.81
	TFX-M-8		0.0677		2.597		162.7			
	TFX-M-9		0.0681		3.801		236.8			
	TFX-M-10	dry	0.0697	0.0681	4.056	3.683	247.0	228.9	30.1	
	TFX-M-11		0.0660		3.019		194.1			
	TFX-M-12		0.0687		3.975		245.6			
Roadfix	RDX-S-1	wet	0.0766	0.0756	0.423	0.3	23.4	16.8	9.4	0.94
	RDX-S-2		0.0745		0.177		10.1			
	RDX-S-3	dry	0.0740	0.0747	0.253	0.313	14.5	17.8	4.6	
	RDX-S-4		0.0753		0.373		21.0			
	RDX-M-5	wet	0.0633	0.0645	0.812	0.841	54.4	55.4	1.3	1.03
	RDX-M-6		0.0656		0.87		56.3			
	RDX-M-7	dry	0.0661	0.0660	0.608	0.832	39.0	53.5	14.2	
	RDX-M-8		0.0662		0.843		54.0			
	RDX-M-9		0.0656		1.044		67.5			
Glenpatch	GPT-S-1	dry	0.0669	0.0649	0.831	0.965	52.7	63.2	13.5	0.48
	GPT-S-2		0.0634		0.873		58.4			
	GPT-S-3		0.0644		1.190		78.4			
	GPT-S-4	wet	0.0640	0.0647	0.483	0.458	32.0	30.1	2.8	
	GPT-S-5		0.0654		0.433		28.1			
	GPT-M-1	dry	0.0658	0.0662	1.085	1.073	70.0	68.8	1.6	0.72
	GPT-M-2		0.0665		1.061		67.7			
	GPT-M-3	wet	0.0648	0.0645	0.735	0.755	48.2	49.6	2.1	
	GPT-M-4		0.0643		0.774		51.1			