ANALYSIS OF AIR VOID VARIATIONS WITH TRAFFIC IN HOT MIX ASPHALT WEARING COURSE MIXTURES USED IN SRI LANKAN ROADS – A CASE STUDY

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ABSTRACT

Over the past decades, road construction with hot mix asphalt (HMA) has significantly increased. However, many issues have risen with respect to the durability of recently constructed asphalt concrete roads. Therefore, the importance of introducing new quality control measures is a current concern.

Properties of the asphalt mixture and the construction practices are important criteria for quality and durability of asphalt pavements. Present Sri Lankan practice is to measure and control, (1) Thickness, (2) Density, (3) Bitumen content and aggregate gradation, and (4) Roughness index (IRI) of the laid asphalt mat.

Objective of the present research is to find the importance of measuring air void of the laid asphalt mat and the need of a combined index of significant parameters to improve quality and durability of asphalt concrete roads.

Core samples were tested at 12 locations with various initial compaction levels at two aging levels, as 100 days and 225 days. In addition, performance of road sections was evaluated with various levels of initial compaction after 5 years to check the long-term aging of asphalt concrete. It revealed that initial air void content decreased significantly under traffic in a short period and it is not possible to evaluate the long-term performance of HMA roads by initial air void content alone.

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Key words: *Air voids, Marshall density, Degree of compaction*

DEDICATION

This research study is dedicated to my wife Shamalee, my son Thejana, and my daughter Sithuli.

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LIST OF ABBREVIATIONS

Wherever the following abbreviations of titles, terms, and units of measurement are used in the Specifications or on the Drawings, the intent and meaning shall be interpreted as described hereunder.

HMA- Hot Mix Asphalt

CMA- Cold Mix Asphalt

GTM- Gyratory Testing Machine

VFA- Voids Filled with Asphalt

VMA- Voids in the mineral aggregate

ESAL-Equivalent Single Axle Load

ASTM-American Society for Testing and Materials [an international standards organization that develops and publishes voluntary consensus technical standards]

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CHAPTER 1: INTRODUCTION

1.1 Introduction

Asphalt pavements are a significant part of the transportation infrastructure system in Sri Lanka. Road construction with hot mix asphalt (HMA) has significantly increased during last decade. Only five HMA plants were in Sri Lanka before year 2005, but it has increased to over 50 plants in year 2015. Many newcomers enter into HMA production and laying industry in recent past. However, alternatively, quality and durability issues are commonly observed in recently constructed roads in our country. Therefore, improving the quality and durability of HMA road industry is a prime requirement in Sri Lanka.

Therefore, one of the main objective of this research is to add several findings to improve the quality control system of HMA.

HMA is composed of three components: aggregate, asphalt binder, and air. The properties and the proportions of those components can vary in the process of plant production and of onsite construction and the variability can further affect the performance of HMA.

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This research identifies air void content as one of the fundamental parameters, which influence HMA performance.

The present research is based on the asphalt concrete testing. There are two distinct roads considered for the study, which have used HMA from the same production plant with same mix design.

1.2 Research Objective

Objectives of this research are to,

- Estimate the air void variation with traffic
- Check the performance of HMA roads with various initial air voids contents
- Check the impotency of measuring the initial air void content of the laid asphalt mat as a quality control measure

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CHAPTER 2: LITERATURE REVIEW

This chapter comprises a wide range of topics: Introduction to hot mix asphalt (HMA), common failures observed in HMA, The fundamental mixture parameters and their importance, and present quality control practices.

2.1 Hot Mix Asphalt (HMA)

Hot mix asphalt (HMA) is a composite material that contains asphaltic binder, aggregate, and air. HMA is mixed, placed, and compacted at elevated temperatures, hence the name hot mix asphalt. Phase diagram of composition of HMA is shown in Figure 2.1

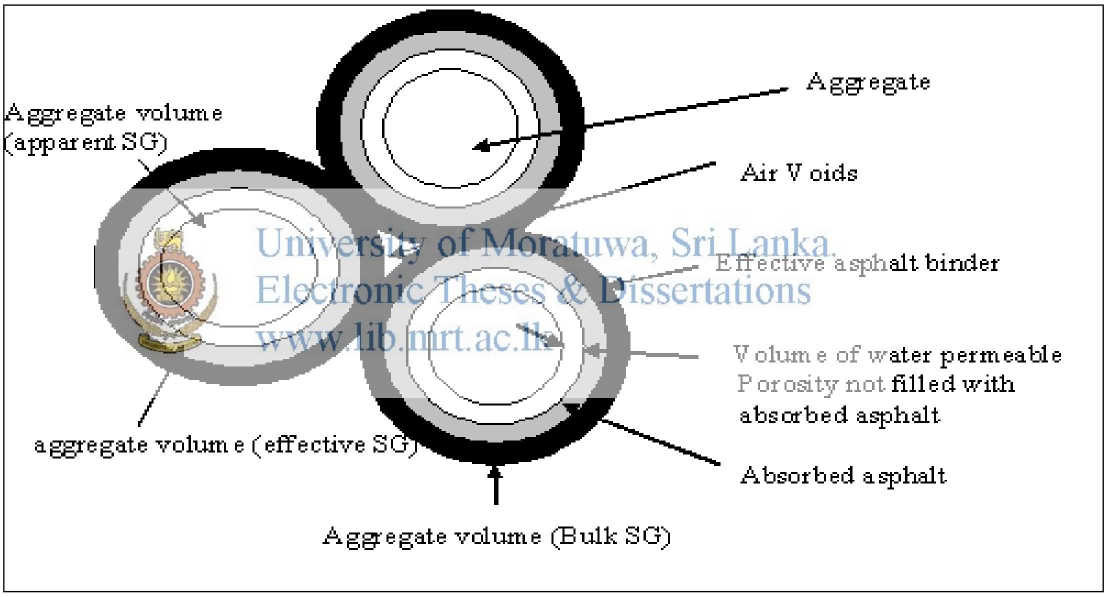

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Figure 2-1: Composition of HMA

Hot mix asphalt (commonly abbreviated as HMA) is produced by heating the asphalt binder to decrease its viscosity, and drying the aggregate to remove its moisture prior to mixing. Mixing is generally performed with the aggregate at about 150 °C and binder at about160 °C. Paving and compaction must be performed while the asphalt is sufficiently hot. HMA is the form of asphalt concrete most commonly used on high traffic pavements such as those on major highways, racetracks, and air fields.

The hot mix asphalt is loaded into trucks for transporting to the paving site. The trucks dump the HMA in to hoppers located at the front of paving machines. The asphalt is placed, and then compacted using a heavy roller, which is driven over the asphalt. Traffic is generally permitted on the asphalt, immediately after asphalt has cooled. HMA paving process is shown in Figure 2.2.

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Figure 2-2: Laying of HMA (original in colour)

HMA is mixed, placed, and compacted at elevated temperatures. Typically, it is applied as 50mm to 200mm thick layers, with the lower layers acting to support the top layer, known as the surface or friction course. The aggregates in the lower layers are chosen to prevent rutting and failure, while the aggregates in the surface course are chosen for their friction properties and durability.

When designing a HMA concrete, the aggregate used must be strong and durable, and with a good angular shape that facilitate to resist rutting. The fine aggregate (mineral filler) is used to fill in the voids between coarse particles, which increases the density of the asphalt concrete and provides load transfer between the larger particles.

2.2 Cold Mix Asphalt (CMA)

Cold mix asphalt, or cold placed mixture, is generally a mixture made with emulsified or cutback asphalt. Emulsified asphalts may be anionic or cationic. Aggregate material may be anything from a dense-graded crushed aggregate to a granular soil having a relatively high percentage of dust. At the time of mixing, the aggregate may either be damp, air dried, or artificially heated and dried.

Mixing methods may be performed either in the roadway, on the side of the roadway, or in a stationary mixing facility. The resultant mixtures are usually spread and compacted at atmospheric temperatures.

Cold mix asphalt may be used for surface, base, or sub-base courses if the CMA is properly designed. Cold mix surface courses are suitable for light and medium traffic; however, they normally require a seal coat or hot asphalt concrete overlay as surface protection. When used in the base or sub base, they may be suitable for all types of traffic.

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2.3 Volumetric of HMA

Volumetric phase diagram of a compacted HMA is shown in figure 2.3. Definition of the terms are given below.

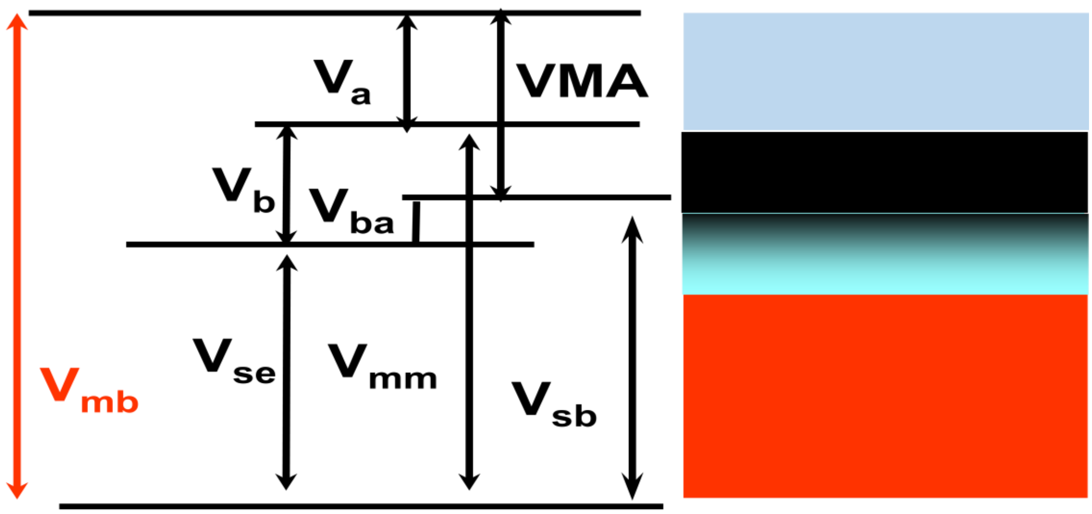

Figure 2-3: Volumetric phase diagram of a compacted HMA

V_{ma}– Volume of voids in Mineral Aggregate

V_{mb}– Bulk Volume of compacted mix

V_{mm} –Void less volume of paving mix

V_{fa} – Volume of voids fill with asphalt

V_a – Volume of air voids

V_b – Volume of asphalt

V_{ba} – Volume of absorbed asphalt

V_{sb} –Volume of Mineral Aggregate (by bulk specific gravity)

V_{se}–Volume of Mineral Aggregate (by effective specific gravity)

2.4 Air voids - Va

The total volume of the small air pockets between the coated aggregate particles throughout a compacted paving mixture, expressed as percent of the bulk volume of the compacted paving mixture. Equation for estimation of air voids are given in Equation 2.1 – 2.4.

Air voids in compacted mixture, Pa

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$$\text{Pa} = \frac{V_a}{V_{mb}} \times 100\% \qquad \text{Eq 2.1}$$

$$\text{Pa} = \frac{V_{mb} - (V_s + V_{be})}{V_{mb}} \times 100\% \qquad \text{Eq 2.2}$$

$$\text{Pa} = 1 - \frac{100/G_{mm}}{100/G_{mb}} \times 100\% \qquad \text{Eq 2.3}$$

$$\text{Pa} = (1 - \frac{G_{mb}}{G_{mm}}) \times 100\% \qquad \text{Eq 2.4}$$

2.5 Voids Filled with Asphalt (VFA)

The VFA is the percentage of voids in the compacted aggregate mass that are filled with asphalt cement. It is synonymous with the asphalt-void ratio. The VFA property is important not only as a measure of relative durability, but also because there is an excellent correlation between it and percent density.

If the VFA is too low, there is not enough asphalt to provide durability and to over-densify under traffic and bleed. Thus, the VFA is a very important design property.

Most DOT specifications require 70-80 during the design phase; this requirement is intended for the mix during the design phase only and is typically not a production requirement. HMA designed for moderate to heavy traffic may not pass the VFA requirement with a relatively low percent of air voids in the field, even though the amount of air voids is within the acceptable range. Because low air void contents may be very critical in terms of resisting permanent deformation, the VFA requirement helps to avoid those mixes that are susceptible to rutting in heavy traffic situations.

VFA also restricts the allowable air void content for HMA that are near the minimum VMA criteria. HMA designed for lower traffic volumes may not pass the VFA requirement with a relatively high percent air voids in the field even though the air void requirement range is met. The purpose for the VFA is to avoid less durable HMA resulting from thin films of binder on the aggregate particles in light traffic situations.

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VFA (Voids filled with asphalt)

$$VFA = \frac{\text{Vol. of eff. asphalt}}{\text{Vol. of eff. asphalt + air}} \quad \ldots\ldots\ldots \text{Eq 2.5}$$

$$VFA = \frac{V_{be}}{V_a + V_{be}} \text{ X } 100\% \quad \ldots\ldots\ldots \text{Eq 2.6}$$

$$VFA = \frac{VMA - P_a}{VMA} \text{ X } 100\% \text{ ; Where } V_{be} = \text{volume of effective asphalt} \quad \ldots\ldots\ldots \text{Eq 2.7}$$

$$VFA = V_b - V_{ba} \quad \ldots\ldots\ldots \text{Eq 2.8}$$

- VFA is related to VMA and air voids (P_a), as can be seen from the equations 2.5 - 2.8.

- A low VFA may result in a high air voids, and a high VFA may result in a low air voids

2.6 Voids in the Mineral Aggregate (VMA)

Voids in the mineral aggregate (VMA) are the air-void spaces that exist between the aggregate particles in a compacted paving mixture, including spaces filled with asphalt. VMA represents the space that is available to accommodate the asphalt and the volume of air voids necessary in the mixture. The more VMA in the dry aggregate, the more space is available for the film of asphalt.

Based on the fact that thicker the asphalt films on aggregate particles, the more durable the mix, specific minimum requirements for VMA are specified in most specifications. Minimum VMA values should be adhered to, thus a durable asphalt film thickness can be achieved. Increasing the density of gradation of the aggregate to a point where below minimum VMA values are obtained leads to thin films of asphalt and a dry-looking, low durability mix. Therefore, economizing in asphalt content by lowering VMA is actually counter-productive and detrimental to concrete quality. Equation for estimating VMA are shown in Equation 2.9-2.11.

VMA (Voids in Mineral Aggregate)

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$$\text{VMA}=\frac{\text{Vol. of air + effective asphalt}}{\text{Bulk vol. of compacted mix}} \quad \text{……Eq 2.9}$$

$$\text{VMA}=\frac{\text{Va} + \text{Vb} - \text{Vba}}{\text{Vmb}} \text{ X } 100\% \quad \text{……Eq 2.10}$$

$$\text{VMA}=\frac{\text{Va} + \text{Vbe}}{\text{Vmb}} \text{ X } 100\% \quad \text{……Eq 2.11}$$

Adequate VMA ensures sufficient amount of asphalt is added to the mixture without overfilling the voids and result in asphalt bleeding. When VMA is not adequate, two problems may occur:

1. Adding sufficient asphalt to coat the aggregate will result in low air voids and bleeding.

2. Addition of inadequate quantities of asphalt will cause low durability.

2.7 Aggregate Gradation

Gradation is the most significant characteristic of a HMA mixture. Aggregate gradation is directly related to optimum asphalt content. The optimum asphalt content of a mix is highly dependent on aggregate characteristics such as gradation and absorptiveness. The finer the mix gradation, the larger the total surface area of the aggregate, and greater the amount of asphalt required to uniformly coat the particles. Conversely, because coarser mixes have less total aggregate surface area, they demand less asphalt.

Relationship between aggregate surface area and optimum asphalt content is most pronounced where filler material with very fine aggregate fractions, which pass through No. 200(0.075 mm) sieve, is involved. Small increases of the amount of filler in a gradation can literally absorb much of the asphalt binder, resulting in a dry, unstable mix. Small decreases have the opposite effect: too little filler results in too rich (wet) mixture. Variations in filler content will cause changes in mix properties, from dry to wet. If a mix contains too little or too much mineral filler, however, arbitrary adjustments to correct the situation are likely to worsen it. Instead, proper sampling and testing should be performed to determine the cause of the variations and, if necessary, to establish a new mix design. The absorptiveness of the aggregate used in the mix to absorb asphalt is critical in determining optimum asphalt content. Adequate asphalt must be added to the mix to allow for absorption, and yet coat the particles with an adequate film.

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2.8 Binder Content

The binder content of a mixture should be differentiated between two phases: one is determined by the mix design, often known as the optimum binder content (OBC). The other one is the as-constructed binder content, which may deviate from the target value due to production variations.

The optimum binder content is the ultimate outcome of an asphalt mixture design. During mix design, many factors affect the resulting binder content, such as aggregate type, gradation, and the design air voids content.

Another factor that influence OBC is the air voids content specified in mix design, corresponding to which the OBC will be determined. Based on the findings, it appears reasonable to allow design air voids for mixtures to vary within the range from about 3% to 5%; however, engineers should understand how such a change could affect HMA performance. If a larger air voids criterion is adopted in the mix design, the portion of VMA that needs to be filled up by asphalt binder is smaller compared to that of a smaller air voids criterion, resulting in leaner binder content. Therefore, outcome of a design OBC is dependent on the pre-selected air voids criterion. In fact, influence of the air voids criteria on OBC can be simply interpreted as changes in OBC.

These air voids content criteria differences are commonly reflected in the design of surface course and binder course mixes. The mixes for a binder course often use lower air voids criteria, as compared to a surface course.

The binder content variation during plant production is regulated by the quality assurance program. Since contractors should well control this variation and ought to stay in a small range, the binder content tolerances for quality acceptance set forth are small.

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Agencies often prefer these smaller ranges; however, if the tolerance is too small, the contractor may have difficulty complying with it. Conversely, if the tolerance is too wide, it may lose its effectiveness to incentives contractors to do a good job. Therefore, setting up a reasonable tolerance is not an easy task; the decision should be based on how well a contractor can control the deviation when a rigorous quality control method is implemented.

For instance, in the SSCM - ICTAD Publication SCA5(2009), the binder content was set at ± 0.3% design binder content in an attempt to simulate the binder content variation due to construction. However, the true binder content due to construction cannot be permitted to be that high; the binder content treatment level does not represent the binder content variation.

This larger range between the two treatment levels might not adequately examine the possible effect of a smaller binder content variation on mixture performance. In other words, the question pertaining to quality control is whether a small binder deviation from the target value will bring about significant mixture performance change. This question is not answerable by previous research. Furthermore, binder variations would unavoidably be involved in the construction of road test pavement. Therefore, more research needs to be conducted in a realistically smaller range to investigate the effects of binder content variations.

2.9 Mat Density

Density is one of the most important parameters in construction of asphalt mixture. A mixture that is properly designed and compacted will contain enough air voids to prevent rutting due to plastic flow but low enough air voids to prevent permeability of air and water.

Since density of an asphalt mixture varies throughout its life, the low enough initially prevent permeability of air and water and high enough after a few years of traffic to prevent plastic flow. The two methods that have facilitated measuring the bulk density of asphalt mixture are physical measurements of cores and nuclear gage. The nuclear gage is fast and non-destructive but is not as accurate as the core method.

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In-place mat density is a ratio of bulk specific gravity (BSG) of the asphalt concrete to the marshall density (MD), or theoretical maximum specific gravity (MSG) measured immediately after new pavement is constructed. Due to the inverse relationship between mixture density and air voids content, AV is used as the alternative for in-place mat density, throughout the paper.

The stable density is approximated 1.5% less than the mix design density level. If 4% AV is the corresponding density level for laboratory compacted specimens in the mix design process, it can infer that the stable-state field density is roughly about 94.5% with respect to MSG.

2.10 Air Voids

Air voids are small air spaces or pockets of air occur between the coated aggregate particles in the final compacted mix. A certain percentage of air voids is necessary in all dense-graded asphalt concrete mixes to allow some additional concrete compaction under traffic and to provide spaces into which small amounts of asphalt can flow during this subsequent compaction. The allowable percentage of air voids in laboratory specimens is between 3.0 percent and 5.0 percent for most surface course mixes or as required by the engineer.

The durability of an asphalt concrete is a function of the air-void content. This is because the lower the air-voids, the less permeable the mixture becomes. Too high an air-void content provides passageways through the mix for the entrance of damaging air and water. A low air-void content, instead, can lead to flushing, a condition in which excess asphalt squeezes out of the mix to the surface.

Density and void content are directly related; higher the density, lower the percentage of voids in the mix, and vice versa. Design specifications require concrete that allows as low an air void content as is practical, approximately 5.0 percent.

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In the literature, some researchers contended the threshold air void value of asphalt pavement as approximately 8% (Zube, 1962; Brown & Cross, 1989; Vivar & Haddock, 2007). This proposition was made from the standpoint of lowering permeability. Another research study showed that the loss in asphalt concrete penetration increases significantly as the air voids exceed 8 %, indicating rapid oxidation is avoided in the dense-graded asphalt mixtures with an air voids contents below about 8% (Sautucci et al., 1985). Most US agencies use 92-93% as their baseline for as-constructed field density.

Linden et al. (1989) researched on the effect of compaction, which consisted of three parts: literature review about pavement life reduction due to air voids content increase, a survey of state highway agencies (SHAs), and the Washington State Pavement Management System (WSPMS).

Survey results are tabulated in Table 2.1; clearly, from every perspective, increasing the air voids content leads to a reduction in pavement life.

Table 2-1: Effect of compaction of pavement performance (after R.N. Linden et al.)

Air Voids (%)	Pavement Life Reduction (%)		
	Literature[a]	SHASurvey[b]	WSPMS
7	0	7	0
8	10	13	2
9	20	21	6
10	30	27	17
11	40	38	--
12	50	46	36

Inadequate pavement density will leave an excessively high level of air content in it, and the void structure could be interconnected. On top of the poor performance of the pavement itself, the voids allow water and air to permeate into the pavement, causing water damage and binder aging (oxidation/hardening), which exacerbates the process of pavement deterioration, leading to premature failure of the pavement. In a study about the performance of 18 test sections during 11 years of service, Tam et al. (1989) concluded that decreasing the void content from 10% to 5% could yield a 10 percent increase in retained penetration (an indication of age hardening). This reveals the relationship between air voids content and the rate of aging; increasing density will be beneficial to control age hardening.

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A roller pattern needs to be developed to achieve the field mat density. As illustrated in Figure 2.4, too much compactive effort does not necessarily render a denser pavement, as the HMA will move laterally due to insufficient confinement pressure. Establishment of a roller pattern depends on many factors, such as the use of proper compactors for different compaction phases (break-down, intermediate, and finish compaction), the proper pressure, and passes for the compaction etc.

A discussion about achieving field density is available in literature (Kassem et al., 2008; Leiva& West, 2008). Prowell and Dudley (2002) studied an evaluation of field measurement techniques for density and permeability.

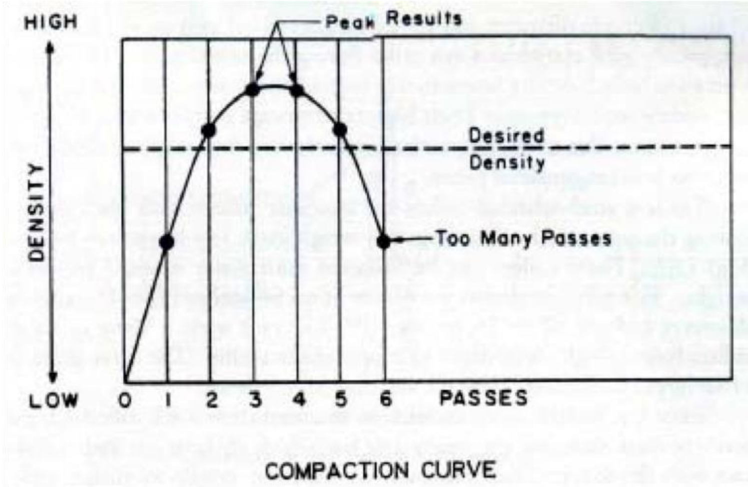

Figure 2-4: Establishing a roller pattern using a test strip

From the above discussion, the consensus that can be reached is that the baseline for high-end air voids content is 7-8%. The next question that needs to resolve is whether a low level of air voids have a detrimental or beneficial effect on the properties of HMA materials. Answering these questions will help to determine whether constructing a higher density pavement should be encouraged by a QA program, when the minimum density requirement is fulfilled.

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The literature provides mixed accounts on rutting performance of high-density pavement. A book on asphalt concrete states that, “there is considerable evidence to show the initial in-place voids for dense-graded mixtures should not be below approximately 3 percent during the life of the pavement” (Roberts et al., 1996).

2.11 HMA Compaction

The purpose of compacting asphalt concrete is to densify the asphalt concrete and thereby improve its mechanical properties as well as to provide a watertight segment for the underlying materials in the concrete structure.

The mixture properties that should be considered when selecting the optimum density compaction include stability, durability, flexibility, fatigue resistance, skid resistance, and fracture strength. By examining the density, requirements for each of these mixture properties, one can make an intelligent judgment as to the degree of compaction necessary to provide a long lasting economical asphalt concrete. Figure 2.5 shows the typical rollers used in intermediate and breakdown rolling.

Figure 2-5: HMA compaction (original in colour)

Five primary variables are under the control of the roller operator during the compaction process:

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1. Roller Speed

The faster a roller moves over a particular point on a new HMA concrete surface, the less time the weight of the roller dwells on that point. This means that less compactive effort is imparted to the mixture at higher roller speeds than at lower speeds. As roller speed increases, the density achieved with each roller passes decreases.

For static steel-wheel rollers and vibratory rollers that have a maximum frequency of 2,400 vibrations per minute (vpm), 2 ½ mph typically is accepted as the maximum speed a roller should travel. For a vibratory roller capable of applying compactive effort at a rate of 3,600vpm, the roller can operate up to 4 mph. For a pneumatic-tire roller, the maximum speed should also be 4 mph. Rollers can move faster or slower than the recommended speed, but compaction varies directly with roller speed.

Roller speed is also governed by the lateral displacement or tenderness of the HMA. If the mixture moves excessively under the rollers, speed of the compaction equipment should be reduced, and the roller speed kept constant. If the paver speeds up and the rollers speed up, the mix will gain less density for the same number of passes of each roller over each point of the concrete surface. It is vital that both the paver and the roller maintain a consistent speed to obtain consistent density.

2. Number of Roller Passes

Actual number of passes needed over a point in the asphalt concrete surface by each of the rollers is a function of many variables. The type of compaction equipment is one primary variable. Three-wheel static steel-wheel rollers apply different compactive effort than tandem static steel-wheel rollers; pneumatic-tire rollers and single or double-drum vibratory rollers all apply different compactive effort.

However, these capabilities vary with layer thickness, mix temperature, mix design (asphalt content and aggregate gradation), and environmental conditions. In addition, the number of roller passes required of a particular roller depends on its position in the roller train-breakdown, intermediate, or finish rolling. It may be possible, for example, to obtain a significant increase in the density of the mat for the same number of roller passes when a pneumatic-tire roller is moved from the intermediate position behind a vibratory roller to the breakdown position in front of that same vibratory roller.

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To determine the optimum number of roller passes needed to achieve the required density level, a test strip should be constructed at the beginning of the project. Typically, only one combination of rollers is tested with one combination of passes of each roller. To pick the most efficient and economical number of roller passes, it is suggested that more than one test strip be constructed, with each test area using different rollers in different positions behind the paver.

Roller passes must distribute uniformly over the width and the length of HMA layer. Most often, center of the paver lane receives more passes from the rollers than do the outside edges of the lane. Number of roller passes applied by each roller must be the same over each point in the concrete surface to obtain consistent density.

To reach maximum density, compaction must be achieved while the mix is still hot enough for the applied compactive effort to reorient the mix's aggregate particles. If the HMA is stable under the rollers, the rollers should operate as closely behind the paver as reasonably possible. Both the breakdown and intermediate rollers should be within 500 ft (152.4m) of the lay down machine.

With a stable mix, fewer roller passes are needed to obtain a given level of density when rolling is accomplished directly behind the paver, where the mix is the hottest. More density is usually obtained with one pass of the roller when the mix temperature is 250 °F (121.1°C) than with a similar pass when the mat is at 220 °F (104.4°C).

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If the mix is tender and moves either longitudinally and/or transversely under the compaction equipment, the breakdown rolling is often delayed to avoid excessive shoving or checking of the mix by the rollers. This, however, is the wrong solution to the problem.

The properties of a mix that cannot be compacted immediately behind the paver need to be modified. It is very difficult to obtain the required degree of density with a tender mix-the rollers cannot compensate for a poor mix design. When a tender mix is encountered, the mix design, not the compaction process, should be changed.

Until the mix design can be changed, often the best way to compact a tender mix is to use a pneumatic-tire roller in the breakdown position, directly behind the paver. A vibratory roller can be operated in the intermediate position, some distance away from the lay down machine. While most of the required density is obtained with the pneumatic-tire roller, care should be taken that the vibratory roller does not operate too close to the paver and cause the mix to move, shove, or check and thereby cause a density reduction instead of the desired increase in density.

3. Rolling Zone

The rolling zone for the static steel-wheel finish roller is the position where marks from other rollers can be removed from the surface of the layer without adding new marks by the finish roller itself. Finish rolling normally happens within a temperature range of 185 °F (85°C) down to 160 °F (71.1°C). Finish rolling for a stable mix is accomplished at higher temperatures than finish rolling for a tender mix.

4. Rolling Pattern

Rollers operate whenever the paver operates. Interestingly enough, when the paver stops, often the breakdown and intermediate rollers also stop. When the paver restarts, the rollers follow suit. While the paver and rollers are stationary, the mix that has not been completely compacted is cooled.

Depending on the length of the shutdown, it may be difficult to obtain the desired level of density if the mix has cooled excessively. It is very important that rollers continue their pattern, regardless of what the paver does, until the required number of roller passes are applied to the concrete surface and the compaction process is over.

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For each roller used on the project, width of the paved lane should be divided by the width of the compaction rolls on each roller to determine the number of passes needed to cover each transverse point of the surface. A tandem static steel-wheel roller, 4 ½ ft (1.372m) wide, for example, would need to make at least four passes across the width of a 12 ft (3.658m) wide lane. This allows for a minimum overlap of 6 inches over each longitudinal edge of the lane and a minimum 6 inches (0.152m) overlap between each roller pass.

However, a 7ft (2.134m) wide double-drum vibratory roller could cover the full 12ft (3.658m) wide lane in only two passes across the width, still allowing for a minimum overlap of 6 inches (0.152m) over each longitudinal edge and between each roller pass. Thus, in terms of a roller pattern, the 7ft (2.134m) wide roller is twice as efficient as the 4 ½ ft (1.372m) wide roller.

A roller that is 5 ½ ft (1.676m) wide would need to make three passes up and down the 12ft (3.658m) wide lane to obtain complete transverse coverage of the surface, allowing for the minimum overlap between passes.

In the longitudinal direction, the rollers should not stop at the same transverse end point with each pass of the roller. The reversal points should be staggered to prevent shoving of the mix. When paving is suspended for a period due to a lack of haul trucks, for example, the roller should not sit on the hot layer. The rollers should be parked either on an adjacent lane, on the shoulder, or at the back of the cold, fully compacted layer.

5. Vibration Frequency and Amplitude for Vibratory Rollers

Vibratory rollers have two additional variables that the operator must control during the compaction process.

1. Frequency of the vibration

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Most vibratory rollers have a range of vibratory frequencies available. With very few exceptions, the maximum possible frequency setting available should be selected. This permits the roller to maximize the amount of compactive effort applied to the mix by minimising the spacing between impacts.

Frequency is measured in terms of vibrations per minute (vpm). At the same roller speed, a vibratory roller operated at a frequency of 2,400 vpm will provide more impacts per foot than will the same roller operated at a frequency of 2,000 vpm.

More impacts per foot provide more compactive effort for each pass of the roller. Frequency of vibration, in conjunction with roller speed, plays a very significant role in the ability of the vibratory roller to obtain density in the HMA material efficiently.

2. Amplitude of the vibration

The amplitude setting (impact height) on a vibratory roller depends on the thickness of the layer being compacted. For the vast majority of mixes placed, the roller should be operated at the lowest amplitude setting. Use of a higher amplitude setting is considered only when the lift thickness is greater than about 3 in.(0.0762m).

The amplitude setting is dependent, in part, on the characteristics of the mix. If the mix is tender, only the lowest amplitude setting should be used. If the HMA is stiff and stable, and the lift thickness is at least 2 ½ in.(0.064m), use of a higher amplitude may be possible. A high amplitude setting on a thin lift (less than 2 in.(0.051m)) will typically cause the vibratory roller to bounce, making it very difficult to obtain the desired density level.

For very thin lifts, i.e. 1 in.(0.025m) or less in thickness, the vibratory roller should not be used in the vibratory mode. Instead, the unit should operate in the static mode.

Compaction of an asphalt-concrete mix is really common sense. Because density, or its inverse air-void content, is the single most important variable affecting the long-term durability of an HMA material, it is crucial that proper attention is applied to those primary factors affecting the time available to compact the mix-air temperature, base temperature, mix lay down temperature, layer thickness, and wind velocity.

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2.12 Effect on the Compaction Process

The initial density of the concrete is dependent upon;

1. The compactability of the mix or the ease with which it can be compacted

   The compactability of a mix is dependent on material properties, mix design, sub grade support, thickness of lift, temperature of mix, weather conditions during placement, and moisture in the mix to be determined.

Both the aggregate and the asphalt influence the compactability of a mix. The gradation, shape, surface texture, and mineralogical composition of the aggregate as well as the type and amount of asphalt, influence the resulting density of a particular mix for a certain compactive effort. Since both aggregate and asphalt characteristics widely vary, it becomes difficult to predict the compactability of a given mix before it is actually placed on the roadway under the prevailing environmental conditions.

Lift Thickness is important that compaction equipment follow the lay down machine as closely as possible for thin lifts. The application speed of compaction equipment is especially critical for thin lift.

Effect of weather on compaction is primarily manifested in its effect on cooling rate of the asphalt concrete. This includes air temperature, wind velocity, and solar radiation. As the temperature drop increases the resistance to compaction by increasing the viscosity of the asphalt, thin lifts placed on cool base materials will cool rapidly and thus becomes difficult to compact. High wind velocities, especially if the air temperature is low, will decrease the asphalt concrete temperature very rapidly. Humidity also has an effect on concrete cooling rate.

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2. The type of compaction equipment

Various types of compaction equipment provide different load range to obtain different compaction on the road:

- Hand-guided compaction equipment such as tamper, vibratory plates, and hand-guided tandem rollers
- Light tandem rollers
- Combination rollers
- Pneumatic tired rollers
- Articulated tandem rollers
- Pivot steered tandem rollers
- Steering systems

Various types of rollers have been used to compact asphalt concretes. It is an advantage of using intermediate pneumatic rolling in the sequence of rolling operation. Pneumatic tire rolling does not increase density for high bearing capacity mixes. Instead, low pressure pneumatic rolling contributes a great deal to compaction if the mix has a low bearing capacity. Steel-wheeled rollers with various types of roller configurations have been used and the equipment used for breakdown rolling is predominantly steel wheel rollers while pneumatic tired rollers are used in the majority of cases when intermediate rolling is specified. Tire pressures range from 60-90 psi (0.414-0.621 N/mm^2). Steel wheel rollers smooth the concrete during final rolling whereas Rubber tired rollers have been specified over a wide range of tire pressures.

Steel wheel diameters are also important in large wheel diameters, and allow higher pressures to be used. Thus, higher densities obtained before excessive shear deformation occurs. Wheels with small diameters cause excessive shear stresses at rather low loadings and will give low maximum densities.

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3. The rolling sequence and procedure

For various compactions, equipment can be used in different rolling sequences and procedures to obtain different compaction on the road. Typical rolling procedures are shown in figure 2-6, 2-7, 2-8 and 2-9. Rolling procedures are fairly well established and the optimum number of coverage depends on the individual mix and the type of rolling used.

Figure 2-6: Initial compaction with the finisher

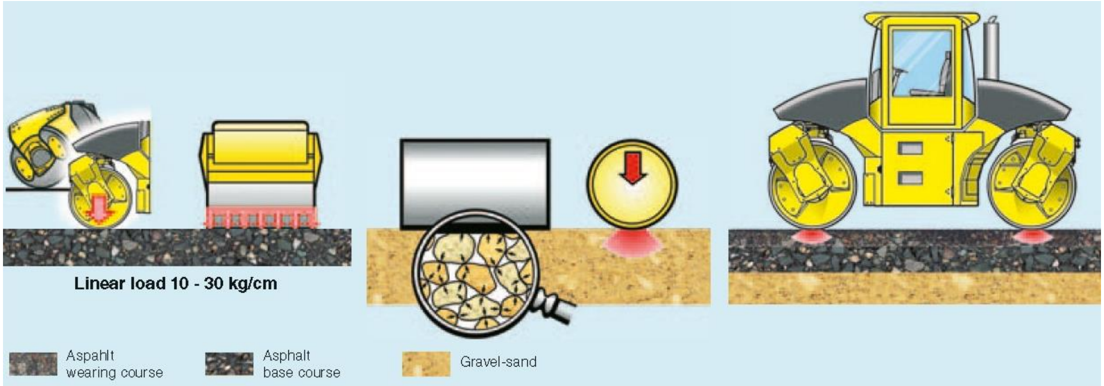

Figure 2-7: Static compaction

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Figure 2-8: Compacting by pneumatic tired roller

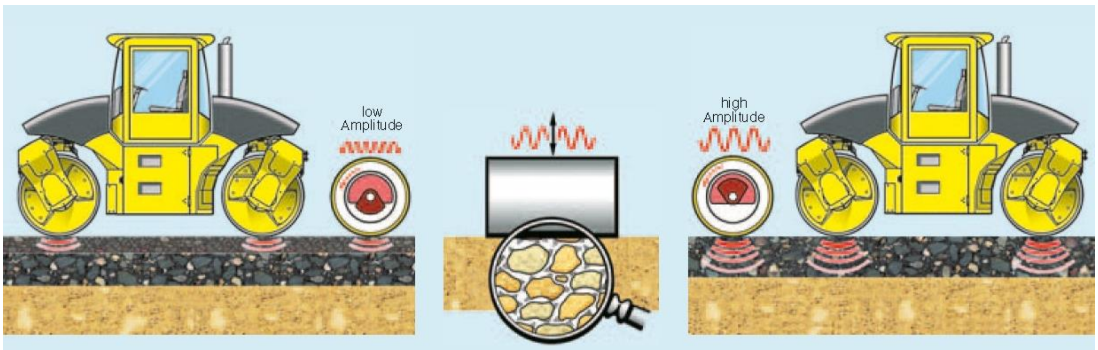

Figure 2-9: Vibratory compaction

4. Timing of compaction processes

Temperature changes after asphalt layering. Temperature of the mix during rolling affects the asphalt viscosity, which affects mixture compactability. Compaction is different for different temperature levels. Best time for compaction is at the instant of asphalt layering.

Compaction on road structure has been separated into initial compaction that occurs during concrete construction; while the asphalt concrete is at an elevated temperature, and long-term compaction that is considered to be due to the action of traffic and environment, and takes place after initial compaction occurs.

A properly designed paving mixture compacted to the optimum degree will, for selected types of aggregates, provide a smooth, skid-resistant concrete at minimum costs for its design life while being subject to traffic and environmental loading conditions.

2.13 Common Failures in HMA Roads

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This section identifies and picture specific failures commonly observed in HMA roads in Sri Lanka.

2.13.1 Alligator Cracking

Alligator cracking is a load associated structural failure. The failure can be due to weakness on the surface, base or sub grade; a surface or base that is too thin; poor drainage, or the combination of all three. It often starts in the wheel path as longitudinal cracking and ends up as alligator cracking. It is shown in figure 2-10.

Figure 2-10: Alligator cracking (original in colour)

2.13.2 Block Cracking

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Figure 2-11: Block cracking (original in colour)

Block cracks appears large interconnected rectangles (roughly), as illustrated in figure 2-11. Block cracking is not load-associated, but generally caused by shrinkage of the asphalt pavement due to an inability of asphalt binder to expand and contract with temperature cycles. The reasons may be the mix was mixed and placed too dry, fine aggregate mix with low penetration asphalt and absorptive aggregates, poor choice of asphalt binder in the mix design, or aging dried out asphalt.

2.13.3 Longitudinal (Linear) Cracking

Figure 2-12: Longitudinal (linear) cracking (original in colour)

Longitudinal cracking are cracks parallel to the pavements centerline or lay down direction. It is shown in figure 2-12. These can be a result of both pavement fatigue, reflective cracking, and/or poor joint construction. Joints are generally the least dense areas of a pavement.

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2.13.4 Transverse Cracking

Figure 2-13: Transverse cracking (original in colour)

Transverse cracks are single cracks perpendicular to the pavement centerline or lay down direction, as illustrated in figure 2-13. Transverse cracks may be due to reflective cracks from an underlying layer, daily temperature cycles, and poor construction due to improper operation of the paver.

2.13.5 Edge Cracks

Figure 2-14: Edge cracks (original in colour)

Edge Cracks travel along the inside edge of a pavement surface within one or two feet, as illustrated in figure 2-14. The most common cause for this type of crack is poor drainage conditions and lack of support at the pavement edge. As a result, underlying base materials settle and become weakened. Heavy vegetation along the pavement edge and heavy traffic can also be the instigator of edge cracking.

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2.13.6 Joint Reflection Cracks

Figure 2-15: Joint reflection cracks (original in colour)

These are cracks in a flexible pavement overlay of a rigid pavement, as illustrated in figure 2-15 (i.e. asphalt over concrete). They occur directly over the underlying rigid pavement joints.

2.13.7 Slippage Cracks

Figure 2-16: Slippage cracks (original in colour)

Slippage cracks are crescent-shaped cracks or tears on the surface layer(s) of asphalt where the new material has slipped over the underlying course, as shown in figure 2-16. Lack of bonding between layers cause this problem and is often because a tack coat was not used to develop a bond between the asphalt layers or because a prime coat was not used to bond the asphalt to the underlying stone base course. Lack of bond can also cause by dirt, oil, or other contaminants preventing adhesion between the layers.

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2.13.8 Pot Holes

Figure 2-17: Pot holes (original in colour)

Small, bowl-shaped depressions in the pavement surface penetrate all the way through the asphalt layer down to the base course. They generally have sharp edges and vertical sides near top of the hole. Potholes are the result of moisture infiltration and usually the result of untreated alligator cracking. As alligator cracking becomes severe, and the interconnected cracks create small chunks of pavement, which can be dislodged as vehicles drive over them. The remaining hole after the pavement chunk is dislodged is called a pothole. It is shown in figure 2-17.

2.13.9 Depressions

Depressions are localized pavement surface areas with slightly lower elevations than the surrounding pavement, as illustrated in figure 2-18. Depressions are very noticeable after a rain when they fill with water.


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Figure 2-18: Depression (original in colour)

2.13.10 Rutting

Ruts in asphalt pavements are channelized depressions in the wheel-tracks, as shown in figure 2-19. Rutting results from consolidation or lateral movement of any of the pavement layers or the sub grade under traffic. It is caused by insufficient pavement thickness, lack of compaction of the asphalt, stone base or soil, weak asphalt mixes, or moisture infiltration.

Figure 2-19: Rutting (original in colour)

2.13.11 Shoving

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Figure 2-20: Shoving (original in colour)

Shoving is the formation of ripples across a pavement, as shown in figure 2-20. Its characteristic shape sometimes names this type of distress as wash boarding. Shoving occurs at locations with severe horizontal stresses, such as intersections. It is typically caused by excess asphalt, too much fine aggregate, rounded aggregate, too soft an asphalt, or a weak granular base.

2.13.12 Raveling

Figure 2-21: Raveling (original in colour)

Raveling is the on-going separation of aggregate particles in a pavement from the surface downward or from the edges inward. Usually, the fine aggregate wears away first and then leaves little "pock marks" on the pavement surface, as shown in figure 2-21. As the erosion continues, larger and larger particles are broken free and the pavement soon has the rough and jagged appearance, typical of surface erosion. There are many reasons for raveling, but one common cause is high permeability of asphalt.

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2.13.13 Bleeding

Figure 2-22: Bleeding (original in colour)

Bleeding occurs when asphalt binder fills the aggregate voids during hot weather and then expands onto the pavement surface, as shown in figure 2-22. Since bleeding is not reversible during cold weather, asphalt binder will accumulate on the pavement surface over time. This can happen due to one or a combination of the following:

Excessive asphalt binder in the HMA (either due to mix designing or manufacturing)

- Excessive application of bitumen for tack coat
- Low HMA air void content (e.g. not enough space for the asphalt to expand during hot weather

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CHAPTER 3: EXPERIMENTAL PROCEDURE

3.1 Methodology

3.1.1 Literature Review

A comprehensive literature review was conducted regarding the HMA, mixture parameters of HMA such as aggregate gradation, binder content, bulk density, theoretical maximum density, Marshall Density, degree of compaction, and air void content. The Impotency of air void content to the HMA performance and evaluation characteristics of HMA roads reviewed.

3.1.2 QA/ QC Practices in HMA Projects

In-depth field investigation was performed regarding the current QA/QC practices in HMA projects in Sri Lankan road construction industry. Majority of the contractors and Engineers control following parameters under Sri Lankan practice:

- Aggregate Properties (LAAV, AIV, FI, Water Absorption, Bitumen Coating, Soundness and, Gradation)
- Binder Content
- Temperature
- Mixture properties of laboratory samples
- Theoretical Maximum Density
- In place Density
- Smoothness
- Elevation

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In table 3.1, aggregate properties are specified in the SSCM - SCA/5, which is the most commonly used standard specification for Sri Lankan roads.

Table 3-1: Aggregate properties

Aggregate Property	Test Method	Limit Specified
Los Angeles Abrasion Value (LAAV)	AASHTO T-96	<40%
Aggregate Impact Value (AIV)	BS - 812	<30%
Flakiness Index (FI)	BS - 812	<35%
Water Absorption		<2%
Bitumen Coating and Stripping	AASHTO T-182	Coated Area >95%
Loss on Sodium Sulphate Soundness	AASHTO T-104	<12%

Aggregate gradation is a routing test during construction. The contactor shall control the production of coarse aggregate, fine aggregate, and filler for asphalt concrete at the crushing and screening plant. Hence, the grading of aggregates in stockpiles shall be uniform and consistent throughout the period of asphalt production and paving operations.

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The contractor shall do regular sampling of stockpiles to demonstrate the uniformity and consistency of grading the aggregate production to the satisfaction of the Engineer.

The grading requirements for the binder course and wearing course is presented in table 3.2 according to SSCM-SCA/5.

Table 3-2: Aggregate grading, binder content, and thickness requirement

STANDARD SPECIFICATIONS FOR CONSTRUCTION AND MAINTENANCE OF ROADS AND BRIDGES (SCA/5) — 500 – Surface Applications, Surface Dressings and Surfacings

Table 506-1 - Aggregate grading, binder content and thickness requirements for Binder course and wearing courses

Mix classification	Binder Course	Wearing Course Type-1	Wearing Course Type-2	Wearing Course Type-3	Wearing Course Type-4
Compacted Thickness mm -					
Max.	75	75	75	75	75
Min.	35	35	35	40	40
Sieve Size Mm µm					
28	100	100	-	100	100
20	90 – 100	85 - 100	100	93 – 100	95 100
14	-	-	82 - 92	-	-
10	56 – 82	66 - 94	61 - 81	59 – 94	58 - 84
5	36 – 58	46 - 74	41 - 66	38 – 69	36 - 66
2.36	21 – 38	35 - 58	27 48	25 – 48	23- 49
1.18	15 – 32	26 - 48	20 - 40	20 – 40	
600	10 – 26	18 - 38	15 - 35	15 – 32	-
300	6 – 20	11 - 28	10 25	10 – 23	5 - 19
150	3 – 13	7 - 20	7 - 17	4 – 15	-
75	1 – 7	3 - 12	5 - 9	3 – 12	2 - 8
Percentage binder content by total weight of mix	3.5 – 5.5	4.0-6.5	4.0-6.0	4.0 – 6.5	4.0-6.0

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The contractor shall submit the job mix formula proposed to be used based on the mix design, to engineer, which include;

- A single % of aggregate passing each specified test sieve
- A single % of binder content
- A single temperature at which is emptied from the mixture
- A single temperature at the mix is to be delivered to the paver on the road

According to SSCM-SCA/5, the HMA produced shall confirm to the job mix formula approved by the engineer within the ranges of tolerances given in table 3-3.

Table 3-3: Permissible variations from job mix formula

Table 506-3 – Permissible Variations from job Mix Formula

Aggregate Passing 14 mm and larger	± 8%
Aggregate Passing 10 mm and 5 mm sieves	± 7%
Aggregate Passing 2.36 mm and 1.18 mm sieves	± 6%
Aggregate Passing 600 µm and 300 µm sieves	± 5%
Aggregate Passing 150 µm sieves	± 4%
Aggregate Passing 75 µm sieves	± 1.5%
Binder content percent by weight of total mix	± 0.3%
Temperature of mixture when emptied from mixer	± 10 ^{0}C
Temperature of mixture when delivered on road	± 10 ^{0}C

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3.2 Flow Chart of the Experimental Procedure

Figure 3.1 shows the flow chart of the experimental procedure.

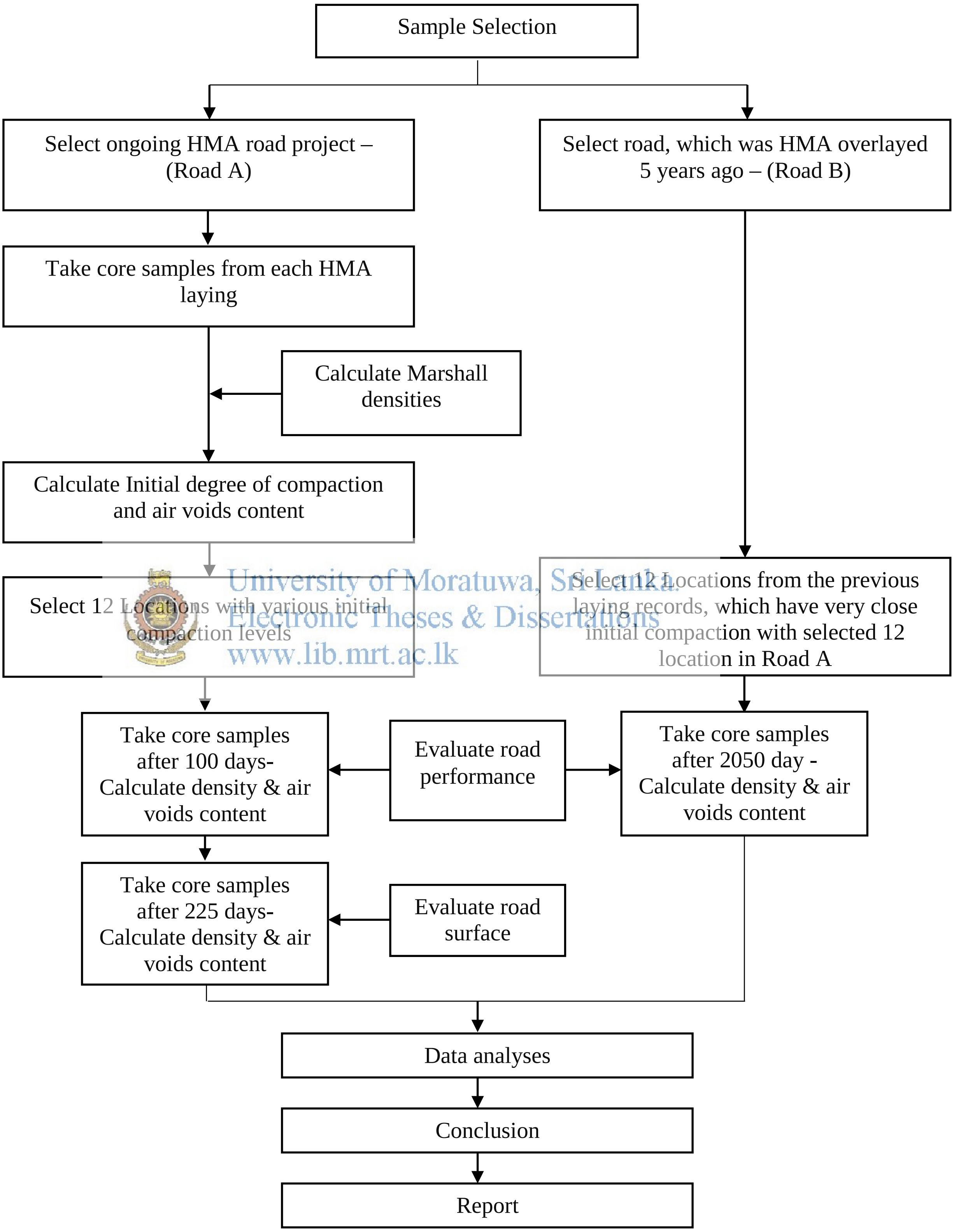

Figure 3-1: Flow chart of the experimental procedure

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1. On-going road project was selected for the investigations, which is a rehabilitation road doing 50mm thick HMA overlay on top of the complete base course overlay (Road A). Figure 3.2 shows asphalt laying on top of the complete base course.

All permissions were obtained from the Road Development Authority, contractor, and the engineer.

Figure 3-2: Asphalt laying on top of the complete base course (original in colour)

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2. Several Core samples were obtained from each HMA laying after compaction and before open to the traffic. Core samples are taking before open to traffic is shown in figure 3-3.

Figure 3-3: Core sample taking before open to traffic (original in colour)

3. Marshall density and maximum theoretical density of the mixture were calculated for each laying day. Bulk density of the HMA, thickness, and Degree of compaction of HMA were calculated. Figure 3-4, 3-5 and 3-6 show the obtaining of sample mass and volume to calculate above parameters.

Figure 3-4: Obtain mass of dry completed sample (original in colour)

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Figure 3-5: Place the core sample here (original in colour)

Figure 3-6: Obtain mass of the specimen in SSD condition (original in colour)

4. Initial degree of compaction and the initial air void content were calculated and tabulated for all core samples.

5. Twelve locations were selected with various initial compaction levels from the data collected.

6. Core samples of above 12 locations were collected after 100 days and again after 225 days, Density and air void content calculated as shown in figure 3-7.

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Figure 3-7: Core sample taking after 100 days and 225 days (original in colour)

The core samples from the same location in three stages (0 days, 100 days, and 225 days) in road A were collected as illustrated in the figure 3.8 in such away, to ensure the previous core cutting does not affect the next core sample and all three samples comply with same initial compaction and same traffic densification.

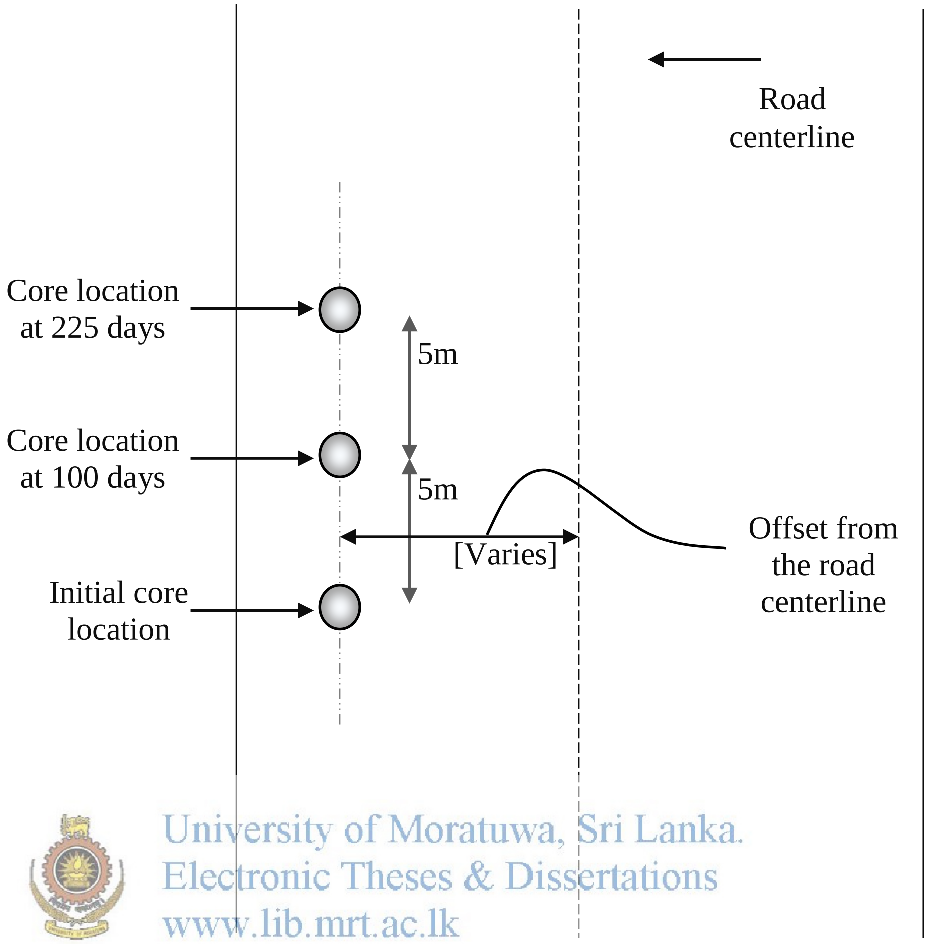

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Figure 3-8: Core locations of road A

7. Performance of the road surface has been evaluated in each stage (after 100 and 225 days) with respect to following performance evaluation characteristics:

- Cracking
- Rutting
- Shoving
- Bleeding
- Pot Holes
- Depressions
- Raveling

8. Another road was selected, which already has HMA over lay five years before. Figure 3-9 show the laying of asphalt on a road (Road B) which has HMA over lay 5 years before. The selection was based on the facts that the roads A and B used HMA from the same asphalt plant, aggregate for HMA from the same quarry, and the mix design of HMA virtually same.

Figure 3-9: Laying of asphalt on a road which has HMA over lay 5 years before (original in colour)

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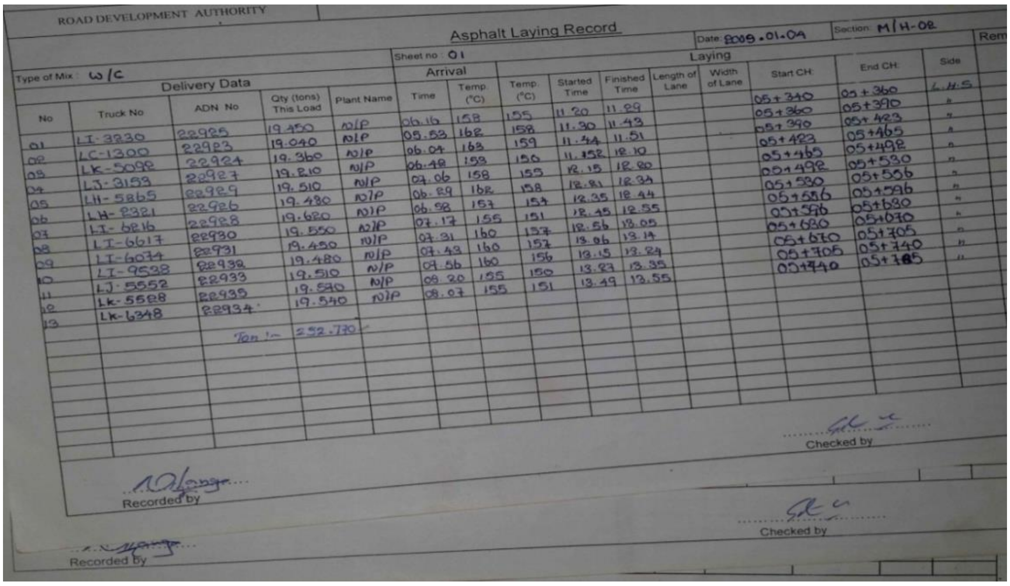
9. Previous laying records were checked and 12 locations were selected from road B, with same degree of compaction with the 12 locations of road A. Asphalt laying record of road B is shown in figure 3-10.



ROAD DEVELOPMENT AUTHORITY

Asphalt Laying Record

Date: 2009.01.04 Section: M/H-02

Sheet no: 01

Type of Mix: w/c

	Delivery Data				Arrival		Laying								Rem
No	Truck No	ADN No	Qty (tons) This Load	Plant Name	Time	Temp (°C)	Temp (°C)	Started Time	Finished Time	Length of Lane	Width of Lane	Start CH	End CH	Side	
01	LI-3230	22925	19.450	N/P	06.16	158	155	11.20	11.29			05+340	05+360	L.H.S	
02	LC-1300	22923	19.040	N/P	05.53	162	158	11.30	11.43			05+360	05+390	"	
03	LK-5092	22924	19.360	N/P	06.04	163	159	11.44	11.51			05+390	05+423	"	
04	LJ-3159	22927	19.210	N/P	06.48	159	156	11.52	12.10			05+423	05+465	"	
05	LH-5865	22929	19.510	N/P	07.06	158	155	12.15	12.20			05+465	05+498	"	
06	LH-2321	22926	19.480	N/P	06.29	162	158	12.21	12.34			05+498	05+530	"	
07	LI-6216	22928	19.620	N/P	06.58	157	154	12.35	12.44			05+530	05+556	"	
08	LI-6617	22930	19.550	N/P	07.17	155	151	12.45	12.55			05+556	05+596	"	
09	LI-6074	22931	19.450	N/P	07.31	160	157	12.56	13.05			05+596	05+630	"	
10	LI-9538	22932	19.480	N/P	07.43	160	157	13.06	13.14			05+630	05+670	"	
11	LJ-5552	22933	19.510	N/P	07.56	160	156	13.15	13.24			05+670	05+705	"	
12	LK-5528	22935	19.590	N/P	08.20	155	150	13.23	13.35			05+705	05+740	"	
13	LK-6348	22934	19.540	N/P	08.07	155	151	13.49	13.55			05+740	05+785	"	
		Ton :-	252.770												

Checked by

Recorded by

Checked by

Recorded by

Figure 3-10: Asphalt laying records

10. Core samples were collected from the 12 locations of road B and air void content was calculated. Figure 3-11, 3-12

Performance of the road surface was evaluated with same evaluation criteria of road A.

Figure 3-11: Obtaining core samples from road B (original in colour)

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Figure 3-12: Measuring the thickness of core sample (original in colour)

CHAPTER 4: DATA ANALYSIS

4.1 Degree of Compaction vs. Air Initial Void Content

Core sample data of road A are tabulated in Appendix A.

Core sample data of road B are tabulated in Appendix B.

Relationship between air void contents and the degree of compaction is presented in Figure 4 -1.

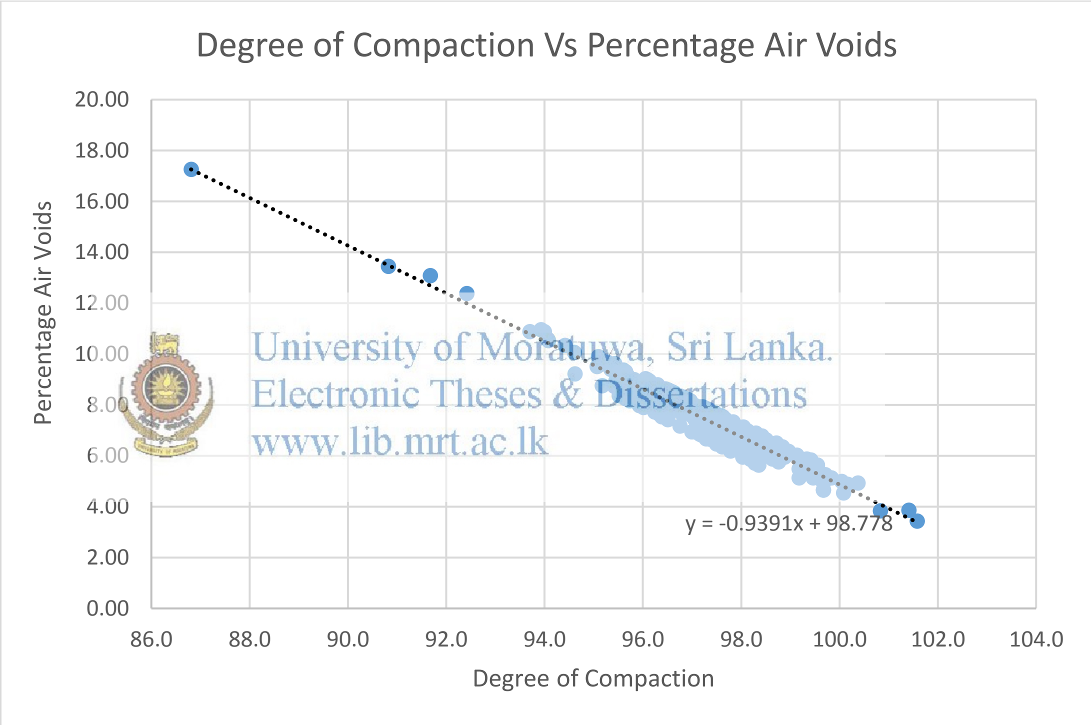

Figure 4-1: Degree of compaction vs. percentage air voids – Data of both roads A& B

The air void content and the degree of compaction shows inverse linear relationship.

y = -0.9391x +98.778

Variation of degree of compaction of road A with time is tabulated in table 4-1 and graphically presented in figure 4-2.

Table 4-1: Degree of compaction - Road A

Location No.	Location	Initial Compaction	Compaction 100 Days	Compaction 225 Days
1	17+246	97.1	97.2	98.3
2	16+990	94.6	97.7	98.5
3	16+870	95.6	98.1	98.4
4	16+543	94.9	97.1	97.7
5	16+240	97.5	99.1	101.2
6	16+015	94.0	100.5	101.1
7	16+630	98.8	99.7	100.2
8	16+510	96.4	97.3	98.0
9	16+267	100.8	102.1	103.2
10	16+150	98.1	101.8	102.6
11	16+085	99.7	101.1	101.2
12	16+032	96.1	100.4	100.6

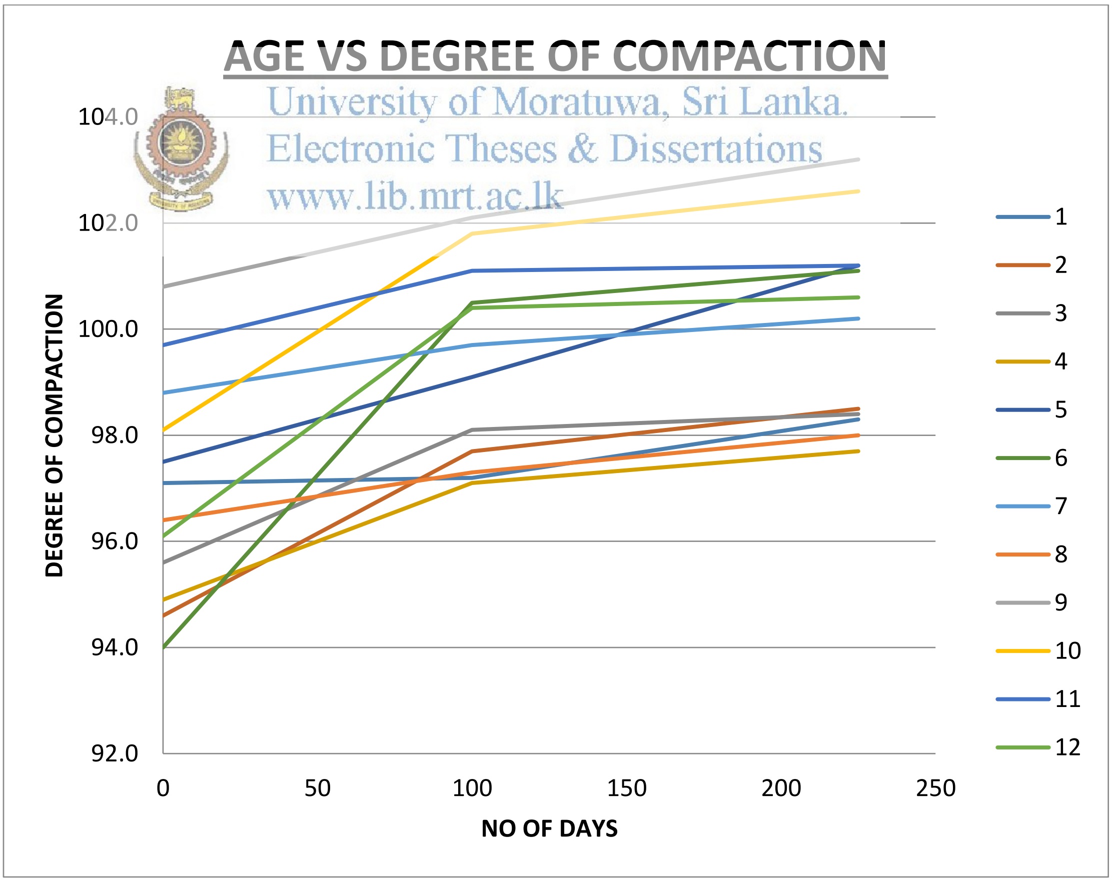

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Figure 4-2: Degree of compaction vary with the time - Road A

Variation of degree of compaction of road A and B with time is tabulated in table 4-2 and it is graphically presented in figure 4-3.

Table 4-2: Degree of compaction - Roads A and B together

Location No.	Initial Compaction	Compaction 100 Days	Compaction 225 Days	Compaction 2050 Days
1	97.1	97.2	98.3	102.8
2	94.6	97.7	98.5	102.4
3	95.6	98.1	98.4	102.7
4	94.9	97.1	97.7	102.7
5	97.5	99.1	101.2	103.4
6	94.0	100.5	101.1	102.6
7	98.8	99.7	100.2	103.5
8	96.4	97.3	98.0	102.8
9	100.8	102.1	103.2	103.9
10	98.1	101.8	102.6	103.4
11	99.7	101.1	101.2	103.9
12	96.1	100.4	100.6	103.3

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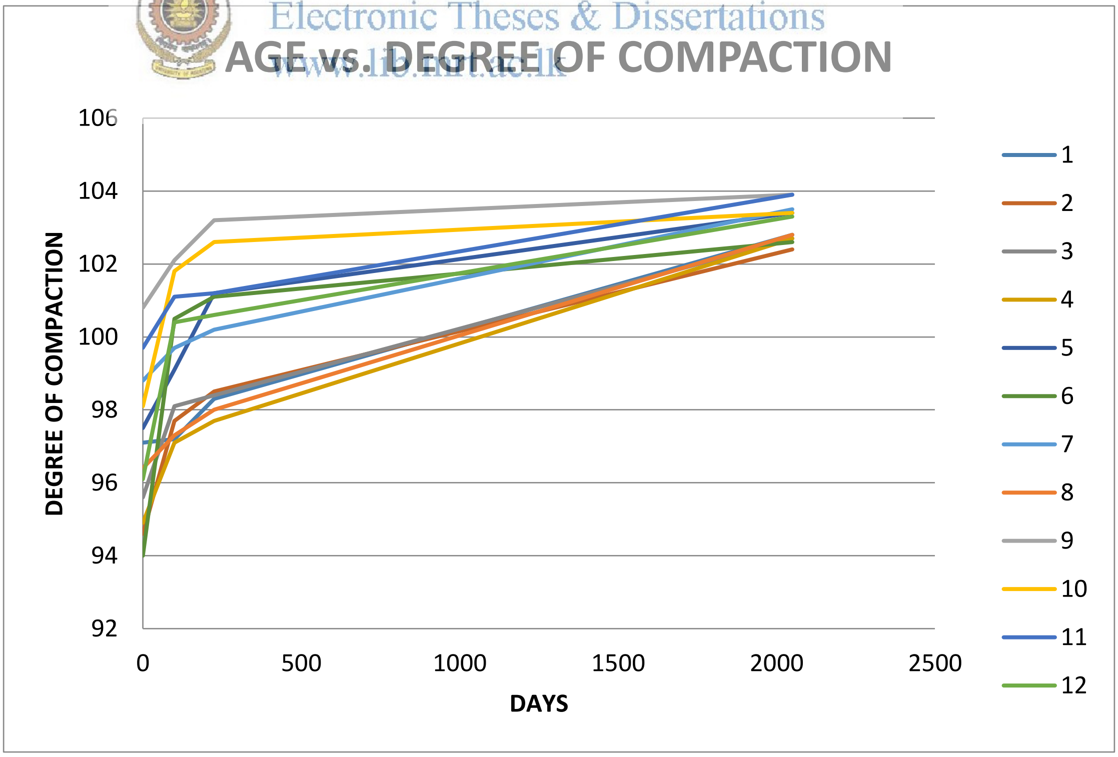

Figure 4-3: Degree of compaction vary with the time - Roads A and B together

The figure 4-2 and figure 4-3 shows that the degree of compaction increases in a faster rate within a short period. Variation of air void content of road A with time is tabulated in table 4-3 and it is graphically presented in figure 4-4.

Table 4-3: Air void content - Road A

Location No.	Location	Initial Air Voids Content	Air Voids Content 100 Days	Air Voids Content 225 Days
1	17+246	7.78	7.30	6.25
2	16+990	10.11	6.78	6.09
3	16+870	9.20	6.45	6.25
4	16+543	9.19	7.41	6.83
5	16+240	6.65	5.14	3.56
6	16+015	9.98	4.96	3.97
7	16+630	5.80	4.92	4.45
8	16+510	8.01	7.20	6.59
9	16+267	3.83	2.30	1.63
10	16+150	6.40	2.54	2.25
11	16+085	4.93	4.12	3.81
12	16+032	8.37	5.09	4.31

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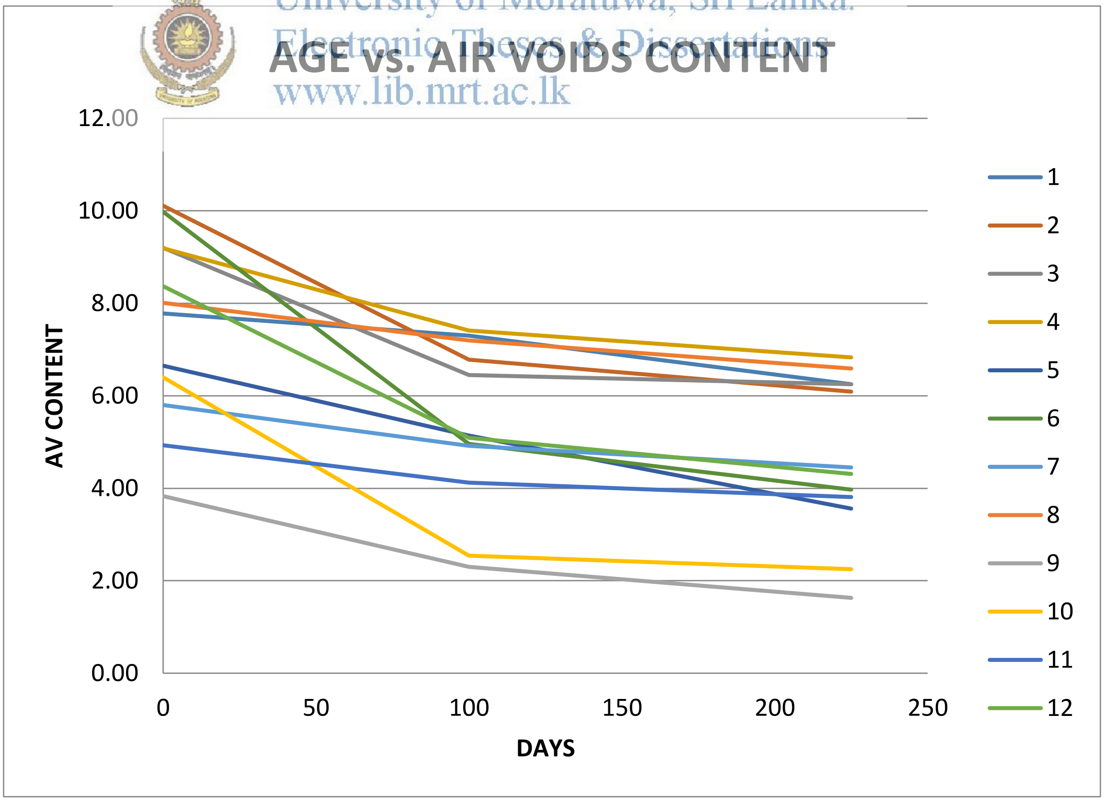

Figure 4-4: Air void content vary with the time - Road A

Variation of air void content of road A and B with the time is tabulated in table 4-4 and graphically presented in figure 4-5.

Table 4-4: Air void content - Roads A and B together

Location No.	Location	Initial Air Voids Content	Air Voids Content 100 Days	Air Voids Content 225 Days	Air Voids Content 2050 Days
1	17+246	7.78	7.30	6.25	2.21
2	16+990	10.11	6.78	6.09	2.65
3	16+870	9.20	6.45	6.25	2.34
4	16+543	9.19	7.41	6.83	2.30
5	16+240	6.65	5.14	3.56	1.69
6	16+015	9.98	4.96	3.97	2.45
7	16+630	5.80	4.92	4.45	1.60
8	16+510	8.01	7.20	6.59	2.26
9	16+267	3.83	2.30	1.63	1.14
10	16+150	6.40	2.54	2.25	1.66
11	16+085	4.93	4.12	3.81	1.22
12	16+032	8.37	5.09	4.31	1.73

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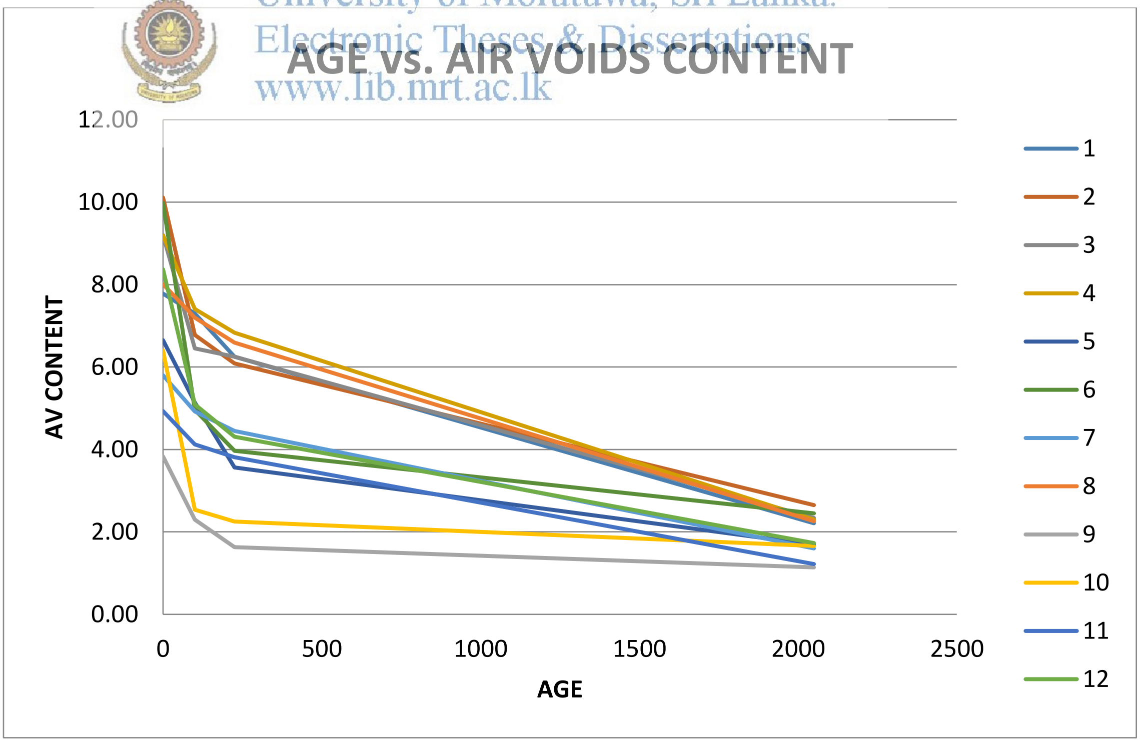

Figure 4-5: Air void content vary with the time - Roads A and B together

According to figure 4-5, the initial air voids content significantly decrease in a very short period and the rate of decrease of air voids content decreases with the time and become almost stable in few years.

Table 4-5 shows performance of road A after 100 days open to traffic.

Table 4-5: Performance of road A after 100 days open to traffic

Location No.	Initial Air Voids content	100 Days Air Voids Content	Cracking	Rutting	Shoving	Raveling	Bleeding	Pot Holes	Depressions
1	7.78	7.30	No	No	No	No	No	No	No
2	10.11	6.78	No	No	No	No	No	No	No
3	9.20	6.45	No	No	No	No	No	No	No
4	9.19	7.41	No	No	No	No	No	No	No
5	6.65	5.14	No	No	No	No	No	No	No
6	9.98	4.96	No	No	No	No	No	No	No
7	5.80	4.92	No	No	No	No	No	No	No
8	8.01	7.20	No	No	No	No	No	No	No
9	3.83	2.3	No	No	No	No	No	No	No
10	6.40	2.54	No	No	No	No	No	No	No
11	4.93	4.12	No	No	No	No	No	No	No
12	8.37	5.09	No	No	No	No	No	No	No

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There was no any failure can observe within 100 days according to the evaluation criteria.

Table 4-6 shows performance of road A after 225 days open to traffic.

Table 4-6: Performance of the road 'A' after 225 days open to traffic

Location No.	Initial Air Voids content	225 Days Air Voids Content	Cracking	Rutting	Shoving	Raveling	Bleeding	Pot Holes	Depressions
1	7.78	6.25	No	No	No	No	No	No	No
2	10.11	6.09	No	No	No	No	No	No	No
3	9.20	6.25	No	No	No	No	No	No	No
4	9.19	6.83	No	No	No	No	No	No	No
5	6.65	3.56	No	No	No	No	No	No	No
6	9.98	3.97	No	No	No	No	No	No	No
7	5.80	4.45	No	No	No	No	No	No	No
8	8.01	6.59	No	No	No	No	No	No	No
9	3.83	1.63	No	No	No	No	No	No	No
10	6.40	2.25	No	No	No	No	No	No	No
11	4.93	3.81	No	No	No	No	No	No	No
12	8.37	4.31	No	No	No	No	No	No	No

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There was no any failure can observe even after 225 days open to traffic, according to the evaluation criteria.

Table 4-7 shows performance of road B after 2050 days open to traffic.

Table 4-7: Performance of road 'B' after 2050 days open to traffic

Location No.	Initial Air Voids content	2050 Days Air Voids Content	Cracking	Rutting	Shoving	Raveling	Bleeding	Pot Holes	Depressions
1	7.78	2.21	Yes	Yes	No	No	No	No	No
2	10.11	2.65	No	Yes	No	No	No	No	No
3	9.20	2.34	No	No	No	No	No	No	No
4	9.19	2.30	No	No	No	No	No	No	No
5	6.65	1.69	No	No	No	No	Yes	No	No
6	9.98	2.45	No	No	No	No	No	No	No
7	5.80	4.45	No	No	No	No	No	No	No
8	8.01	2.26	No	No	No	No	No	No	No
9	3.83	1.14	No	No	No	No	No	No	No
10	6.40	1.66	No	No	No	No	No	No	No
11	4.93	1.22	No	No	No	No	No	No	No
12	8.37	1.73	Yes	No	No	No	No	No	No

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There are some failures can be observed after 2050 days open to traffic. Location no 1 failed by both cracking and rutting, location no 2 by rutting, location no 5 by bleeding and location no 12 by cracking.

The initial air voids content of the above failure locations are 7.78, 10.11, 6.65 and 8.37 respectively. But there are some other locations exist without any failure even with higher initial air voids contents. It revealed in long-term performance analysis, 4 out of 6 high initial air voids sections (low degree of compaction) did not fail. Long-term performance of HMA roads cannot be evaluated by initial Air Voids content alone.

CHAPTER 5: CONCLUSION

Conclusions and findings reached in this research are as follows:

- Air Voids Content has an inverse linear relationship with Degree of Compaction.
- Degree of Compaction increases in a faster rate within a short period in the road considered for the study.
- Initial Air Voids content significantly decrease under traffic in a short period.
- The rate of decrease of Air Voids content decreases with time and become almost stable in a few years.
- The Air Voids variation with traffic does not depend solely on the Initial AV content; it may be affected by other properties of HMA mix and environmental factors since the reduction of air void was not in a uniform rate.
- It revealed in long-term performance analysis, 4 out of 6 high initial air voids sections (low degree of compaction) did not fail.
- Long-term performance of HMA roads cannot be evaluated by initial Air Voids content alone.
- Construction quality control documentation was not adequate on many paving projects.

Samples of asphalt mixtures from the mixing plant should be compacted in the laboratory during construction to verify that the air voids are within an acceptable range. If the air voids are not within an acceptable range, adjustments to the mix are needed.

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CHAPTER 6: DISCUSSION

Road structure consists of sub base, base, and wearing course layers. The asphalt layers are asphalt wearing course, asphalt bearing course, and asphalt base course. The asphalt-wearing course has main impact on the behaviour of the road. The asphalt layers are compacted to obtain an optimum air void percentage. Besides the mix composition and the paving, the compaction of the mix is of utmost importance with respect to the quality and the service life of the road. On the construction site, successful compaction mainly depends on the compaction technique, and the knowledge and experience of the contractor.

Apart from the initial compaction in road, one should consider the compaction due to traffic on the road. The asphalt layers of a road should be able to reliably carry various traffic loads and to discharge these to the substructure or sub base, in order to prevent harmful deformation of the road.

In lab compaction, the air void content, which is the reverse of the density, is mainly a function of compactive effort. The mixture is confined in a rigid mould and more blows will result in less air voids in the mix. Density in the field depends on a proper compaction pattern, such as combination of rollers and number of passes. Aggregate gradation and binder content are the other factors, which influence mix compactability.

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The air voids content in mix design is widely adopted at 4%, but in the field, the pavement is not always compacted to the mix design level, so the density is often much smaller compared to the corresponding 4% V_a level.

Therefore the present study was concerned on the in place initial compaction and air void contents and their variation with time/traffic. Simultaneously, performance of the road surfaces with various initial air void contents was evaluated. In this research, most data were collected from field investigations. Bulk densities and theoretical maximum densities of core samples were investigated through laboratory tests. During field investigations, potential factors that may influence the test results were carefully controlled and retained constant.

The core samples from the same location in three stages (0 days, 100 days, and 225 days) in road A were collected, in such away, to ensure the previous core cutting does not affect the next core sample and all three samples comply with same initial compaction and same traffic densification.

Core samples were collected until 225 days from road, 'A' which is a newly constructed road. However, it was not possible to obtain core samples from the same road for the investigation of behaviour in the long run. Therefore, for the investigation of long run, road 'B' was selected considering following factors:

Road age

To obtain a reasonable curve, graph values that are neither close, nor too far from 225 days need to be plotted.

Traffic load

Each road has various traffic loads due to vehicles in different categories. Hence considering an alternative road with similar traffic will provide an accurate curve.

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Environment conditions

Environment conditions such as weather and atmosphere temperature influence road properties. Considering an alternative road with similar environment condition will provide an accurate curve.

Asphalt mixture

Asphalt mixtures used on roads differ from each other according to the mix designs and material sources. Considering an alternative road with similar Asphalt mixture will produce an accurate curve.

Initial compaction level

Core locations in alternative roads were selected with same initial compaction, approximately equal to the selected core locations in newly constructed road.

Considering these factors, the researchers in this study discovered a specific road with HMA laid over 5 years ago in which HMA from the same plant, same aggregate source, and same mix design was applied as with the newly constructed road. To find the properties of 2050 days old road, the cores were cut with same offset, considering those laid on the same wheel path.

After analyzing all data, it was observed that,

- Air voids content has an inverse linear relationship with degree of compaction
- Initial air voids content significantly decrease under traffic in a short period
- The air voids content decrease rate reduce with time and become almost stable in few years
- It is rationalized that three or four years after a HMA being open to traffic, the pavement will be brought to 1.5% to 3% air void level from the initial as constructed in-place air voids content due to traffic densification. After four to five years of traffic, the traffic no longer has a significant effect on the air voids content and pavement density appears to reach a stable state.
- It was not possible to observe any direct relationship between initial air voids content and the long-term performance of the road surface.
- Due to variations during the production and placement, and compaction pattern and efforts, a pavement can end up with a number of combinations of values regarding mixture parameters. Because several mixture parameters such as gradation, binder content, filler content, and air voids content intertwine with each other, effects of them on the pavement performance are confounding. Thus, it is difficult to discern the effect of initial air voids content to the road performance without controlling other parameters in a real road surface.
- It is proposed to assign pay factors to evaluate the ability of a contractor to comply with parameters such as binder content, gradation, and air voids content, compared to other developed countries.

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APPENDICES

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APPENDIX - A. Core Sample Data

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APPENDIX - B. Core Sample Data in 12 Selected Locations - Road A

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APPENDIX - C. Core Sample Data - After 100 Days - Road A

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APPENDIX - D. Core Sample Data - After 225 Days - Road A

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APPENDIX - E. Core Sample Data - After 2050 Days - Road B

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APPENDIX - F. HMA Mix Design Report of Road A

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APPENDIX - G. HMA Mix Design Report of Road B

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