

ASPHALT RUTTING AND ITS PREVENTION

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Abstract

The paper seeks to redress the concerns expressed at the 8th CAPSA that the design of hot-mix asphalt did not adequately address prevention of rutting. It presents a consolidation of best practice in the design and application of asphalt layers pertinent to the mitigation of rutting, while being mindful of other performance requirements in terms of fatigue resistance and durability.

Two stages of rutting are identified – firstly consolidation and secondly shear deformation. Consolidation is categorised into two phases: that which takes place during construction and that which continues to take place under the action of traffic. Compaction methods for the preparation of laboratory samples should replicate the summation of both these effects for realistic assessment of engineering properties. Recommendations are made in this respect. In addition the effect of mix component characteristics such as aggregate interlock and the viscosity and temperature susceptibility of the binder-filler mastic are examined and indications given on how the composition can be optimised to render the mix rut resistant.

Four wheel-tracking test regimens to investigate the propensity of the mix to rutting through shear flow are compared and recommendations made as to the suitability of the methods.

Recommendations are made to ensure that the initial state of compaction of the layer will promote further densification under traffic so as to lessen the potential for oxidative hardening with concomitant adverse effects on fatigue resistance and durability. These recommendations cover the required degree of compaction during construction, attention to environmental factors, the effects of mix components and composition and support layers and construction techniques.

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1. INTRODUCTION

Asphalt rutting is one of the most serious forms of distress associated with hot-mix asphalt (HMA). Over the last few years, South Africa has seen an increase in the occurrence of rutting, particularly on heavily-trafficked roads, in urbanised areas and in situations where heavy vehicles move at slow speeds such as at intersections and on steep grades.

The September-2001 version of the *Interim Guidelines for the Design of Hot-Mix Asphalt in South Africa* (Asphalt Academy, 2001) went some way to address the various aspects influencing HMA's propensity to rut and put forward several recommendations to assess the susceptibility of HMA to permanent deformation both qualitatively and quantitatively. However, at the 8th Conference on Asphalt Pavements for Southern Africa, concern was expressed that, although the Interim Guidelines were seen to be useful, measurement of rutting and, by implication, the design of HMA to prevent rutting were not adequately addressed.

The purpose of the paper is to consolidate best practice for the design and application of HMA to mitigate rutting and to capture lessons learnt.

2. MECHANISMS OF RUTTING

Permanent deformation or rutting of asphalt manifests itself as depressions which are formed in the pavement's wheel path. Water collects in these depressions and cannot drain freely off the pavement surface. This could cause aquaplaning and therefore rutting is also a potential safety hazard.

The development of permanent deformation in asphalt layers has generally been described as a two stage process. The first stage consists primarily of consolidation (densification accompanied by volume change) while the second stage consists primarily of shear deformation (plastic flow not associated with volume change). In extreme cases consolidation and shear deformation may occur concurrently leading to severe distortion of the layer.

3. CONSOLIDATION OF HMA

Initially, some rutting will occur in an HMA layer as a result of the continued densification of HMA under traffic while aggregates are moved into their preferred orientation, resulting in a decrease in air voids. As densification increases with time and traffic, the mix stability usually improves which results in a decrease in the rate of permanent deformation. The improvement in mix stability as a result of densification is attributed to strain-hardening caused by the rearrangement of the aggregate matrix, improved interlock between particles and binder hardening. Typically, the field void content in a well designed and well constructed HMA will reduce from an initial (post-compacted) void content of around 7 per cent to the design void content of between 3 and 5 per cent over a period of approximately 2 years. The mix is typically designed to operate at this design density, hereafter termed "ultimate density", at which shear resistance is expected to be optimal for the demands of traffic.

Typical rut depths deriving from the HMA itself that could develop during the initial traffic densification phase up to a target void contents of 4 per cent, for a given initial layer thickness and initial compaction level, are shown conceptually in Figure 1. The figure clearly demonstrates the *importance of achieving high initial densities during construction*

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of the HMA layer. For a 40-mm thick continuously graded asphalt mix with an initial void content of between 7 and 8 per cent after construction, the void content typically would reduce to approximately 4 to 5 per cent within two years of trafficking, resulting in a negligible amount of rutting (less than 2 mm). On the other hand, if the HMA layer is poorly compacted, and particularly if the layer is thick, significant rutting could develop as a result of excessive consolidation of the layer (e.g. between 9 and 13 mm rut depths for a 150-mm layer with initial void contents of between 10 and 12 per cent). Figure 1 is based on the assumption that the ruts developed during this first stage are only the result of densification associated with volume change, with no shear distortion taking place.

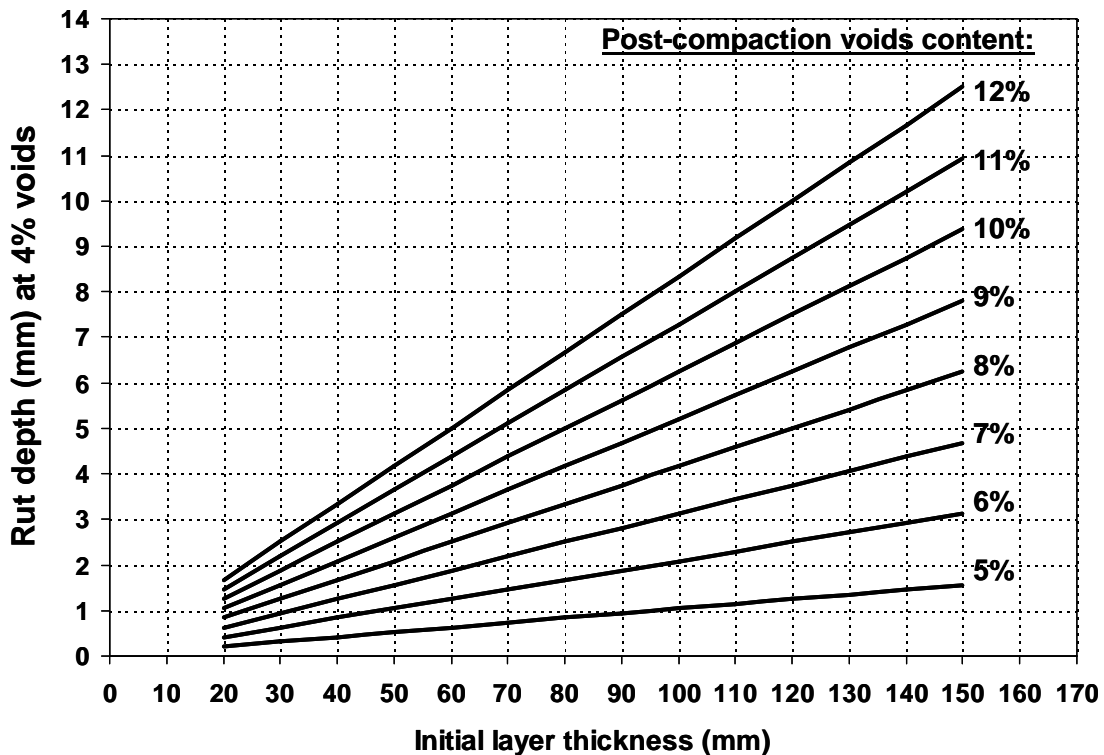


Figure 1: Stage 1 rut depth as a result of consolidation

The general consensus is that HMA usually reaches its ultimate density after the second or third summer, although various case studies (Prowell and Brown, 2007) have indicated that the ultimate densities can be achieved within a couple of months, particularly during a hot summer, or densification can continue up to four and in some cases even ten years after construction. The factors that affect the ultimate HMA density include (Kandhal and Wenger, 1975):

- Degree of initial compaction;
- Material properties:
 - Binder absorption by aggregates;
 - Aggregate surface characteristics;
 - Aggregate grading;
 - Binder/mastic temperature-viscosity relationship;
 - Binder susceptibility to age hardening;
- Mix design:
 - Binder content and film thickness;
 - Voids in the mineral aggregate;

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- Weather conditions:
 - Pavement temperature variations (daily and seasonal);
 - Date of construction;
- Traffic:
 - Amount and type;
 - Distribution during year;
 - Distribution throughout the day;
 - Distribution across lanes;
- HMA layer thickness.

The initial density of the HMA depends on the compactibility of the mix or the ease with which it can be compacted, the type of compaction equipment, the rolling sequence and procedure, the temperature during compaction and the weather conditions at the time of construction.

One of the premises of HMA design is that *the density of laboratory-compacted samples used for the determination of the optimum binder content should approximate the ultimate density of HMA*, i.e. the field density which will be achieved after about two years of trafficking. This is particularly important for sand-skeleton type mixes (i.e. gap-graded and continuously graded mixes) that do not rely on a strong coarse aggregate skeleton and coarse aggregate interlock to carry the traffic loading. If the “ultimate” density after two years of trafficking is too low the air voids may provide an easy passage for air and water which may lead to brittleness, accelerated binder hardening and high permeability causing the mix to be susceptible to moisture damage, all of which will impact adversely on the durability of the mix. On the other hand, if the “ultimate” density is too high (less than 2 per cent actual air voids), the HMA will tend to bleed or rut. Hence, it is *imperative that the laboratory compaction method provides a reasonable replication of the compaction during construction as well as the post construction densification under traffic* so that a mix can be formulated that will produce satisfactory performance.

3.1 Marshall compaction

Since the 1940s, compaction methods and procedures, and response properties such as Marshall stability values, have continuously been adjusted, the driving force being changes in the field density brought about by changes in traffic and, therefore, an attempt to better simulate ultimate densities. In its current version, the Asphalt Institute’s MS-2 specifies three levels of Marshall compactive effort, the selection of which depends on the design traffic (see Table 1).

Table 1: Compaction requirements as a function of traffic

| Traffic Classification | | No. of blows per side |
|------------------------|--|-----------------------|
| Traffic Class | Approximate Pavement Structural Design Capacity* | |
| Light traffic | ESALs < 10 ⁴ | 35 |
| Medium traffic | 10 ⁴ < ESALs < 10 ⁶ | 50 |
| Heavy traffic | ESALs > 10 ⁶ | 75 |

* ESAL = Equivalent Single Axle Load.

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Very few changes have been made to the Marshall compaction method and compactive effort during the last decade, except in South Africa (Asphalt Academy, 2001; Sabita, 2005) where proposals have been made to instrument the Marshall hammer and to introduce control points for initial and final void contents for four design traffic classes (see Table 2).

TABLE 2a: Recommended compaction requirements as a function of traffic

| Traffic Class (see Table 2b) | Allowable Void Content Range after first 75 Marshall blows (to simulate construction densities) | | Allowable Void Content Range after additional compaction to simulate trafficking | | |
|---------------------------------|--|---------|--|--------------|---------|
| | Minimum | Maximum | Additional Blows | Void Content | |
| | | | | Minimum | Maximum |
| Light ¹ | 3.5% | 5.5% | 15 | 3.0% | 4.5% |
| Medium ² | 4.5% | 6.5% | 45 | 3.0% | 5.0% |
| Heavy | 5.5% | 7.0% | 75 | 4.0% | 5.0% |
| | Min. voids content of 1.5% after 300 gyrations with Gyratory compactor, according to the Superpave testing protocol Permeability of the mix within acceptable norms | | | | |
| Very Heavy | 6.0% | 7.0% | 75 | 4.5% | 5.5% |
| | Min. voids content of 2.5% after 300 gyrations with Gyratory compactor, according to the Superpave testing protocol Permeability of the mix within acceptable norms | | | | |

¹After 2x75 blows, the void content should be between 2 and 3 per cent

²After 2x75 blows, the void content should be between 3 and 4 per cent

TABLE 2b: Traffic Classification

| Measure of Traffic Intensity | | Traffic Class |
|------------------------------------|--|---------------|
| Number of Heavy Vehicles/ Lane/Day | Approximate Pavement Structural Design Capacity* | |
| <80 | < 1 million ESALs | Light |
| 80 to 200 | 1 to 3 million ESALs | Medium |
| 200 to 700 | 3 to 10 million ESALs | Heavy |
| > 700 | > 10 million ESALs | Very Heavy |

* ESAL = Equivalent Single Axle Load.

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Several field studies have indicated that even 75 blows per side with a manual hammer, which tends to yield higher densities than a mechanical hammer, does not always produce the densities ultimately achieved in the field under traffic (Roberts et al, 1996). This suggests that for high traffic conditions, the laboratory compactive effort may have to be increased well beyond 75 blows per side. However, since there have been no definitive studies that produced recommendations on appropriate levels of additional compaction, and since the aggregate might be at risk of being crushed as a result of further increases in the number of compaction blows, it is recommended that for high traffic conditions gyratory compaction be used instead.

Apart from the concern that Marshall compaction may not be the ideal method to simulate ultimate densities, another concern is the reliability of the method, as was recently demonstrated by Denneman and Marais (2007) who investigated inter-laboratory variability of Marshall compaction on the Marshall design parameters using data produced by seven participating laboratories. They reported significant variability which was attributed to operator dependent variables, mix preparation techniques, mix characteristics and, most notably, equipment related variability. For instance, the standard deviation for void contents between laboratories was shown to be close to 1 per cent. Such variability could potentially lead to erroneous decisions on the optimum binder content to meet specific requirements. The authors called for (1) proper calibration of the Marshall hammers amongst the parties involved in a roads project to prevent conflict over product acceptance, as well as calibration as required by ASTM D6926-04, (2) the implementation of a national proficiency testing scheme, and (3) a revision and tightening-up of the South African standard test methods.

3.2 Superpave gyratory compaction

Superpave, a product of the asphalt research undertaken as part of the Strategic Highway Research Programme (SHRP) in the USA, integrates performance-based specifications, test methods, equipment, testing protocols and a mix design system. It represents a fundamentally new system for designing asphalt mixes, moving away from the established design systems such as the Marshall and Hveem mix design methods. Forming an integral part of Superpave, a mix optimisation procedure was developed, based on volumetric principles using the gyratory tester. One of the outcomes of the original study was a N_{design} table which consisted of seven traffic classes and four climatic levels which allowed the designer to optimise the grading and binder content to yield 8 per cent initial (post-compaction) voids, 4 per cent design voids and 2 per cent terminal voids at specified numbers of gyrations (N_{initial} , N_{design} and N_{max} , respectively), depending on traffic class and climatic conditions.

The N_{design} table has since been modified following the outcomes of several verification studies. In 1999, the climatic regions were eliminated because (1) it was observed that volumetric properties displayed low sensitivity to the number of gyrations between temperature columns, and (2) the original N_{design} research did not recognise that stiffer binders are used in hotter climates, thereby making the need for separate temperature columns redundant (C-SHRP, 1999).

In one of the larger studies, NCHRP 9-9, 40 field projects were assessed over a period of four years after construction. Analysis of the HMA densification data produced the following findings (Prowell and Brown, 2007):

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- 55 per cent of the pavements tested had as-constructed densities less than 92 per cent of maximum theoretical relative density;
- The majority of the HMA densification occurred in the first three months after construction;
- The month of construction significantly affected HMA densification. HMA constructed in late spring or early summer tended to densify more than average, and pavements constructed in early spring or in autumn tended to densify less than average;
- The ultimate densities of the pavements evaluated in this study were obtained after two years of traffic;
- Based on data collected at the National Centre for Asphalt Technology (NCAT) Test Track, Superpave mixes containing modified binders (PG 76-22) densified 25 per cent less and rutted 60 per cent less than mixes containing unmodified binder (PG 67-22);
- All 40 mixes investigated in the study were found to be very rut resistant. The maximum observed rutting for a field project was 7.4 mm, with an average rut depth for all of the projects of 2.7 mm after four years of traffic.

The NCHRP 9-9 study also found that the N_{initial} and N_{max} requirements were not indicative of rutting potential and should be eliminated. The study also recommended that the dynamic internal angle (DIA) of the Superpave gyratory compactor (SGC) be reduced from 1.25 degrees to 1.16 ± 0.02 degrees. In Table 3, the recommended new N_{design} levels emanating from the study are given (Prowell and Brown, 2007):

Table 3: Proposed N_{design} levels for an SGC DIA of 1.16 ± 0.02 degrees

| 20-year design traffic, ESALs | 2-year design traffic, ESALs | N_{design} for binders < PG 76-XX | N_{design} for binders \geq PG 76-XX or mixes placed > 100 mm from surface |
|-------------------------------|------------------------------|--|---|
| < 300 000 | < 30 000 | 50 | N/A |
| 300 000 to 3 000 000 | 30 000 to 230 000 | 65 | 50 |
| 3 000 000 to 30 000 000 | 230 000 to 2 500 000 | 80 | 65 |
| > 30 000 000 | > 2 500 000 | 100 | 80 |

It is noted from the above table that two-year design traffic classes have been specified, which is a sensible approach in view of the fact that the ultimate densities are expected to be achieved within this time window.

The proposed N_{design} levels are a relaxation from previous recommendations, which would enable designers to increase the binder content in mixes slightly, which would contribute to better initial compaction and also improve the durability of the mix.

3.3 Mix optimisation

As was noted in Section 2.1.1, there are several factors that will impact on the ultimate density. In this section, only those aspects that the mix designer has control over will be discussed.

HMA design should not be done in isolation. The mix designer should have a clear understanding of the particular conditions at the site where the mix is to be applied before selecting the most appropriate mix type to suit the site requirements and initiating the mix design process. The type of information that the designer would typically require upfront includes: the layout of the site (e.g. gradients, intersections, lane width); traffic information (e.g. traffic volumes and composition, operating speeds, directional/lane distribution); climatic conditions (e.g. air/pavement temperature data, expected month of construction, rainfall), proposed HMA layer thickness, pavement composition and particular requirements imposed on the mix (e.g. skid resistance, noise reduction).

The above information should enable the designer to identify the required level of design reliability that would need to be achieved during design, and once the design requirements have been established, the designer would then proceed with the selection of the most appropriate mix design strategy and mix design method that will satisfy all requirements. For instance, if a continuously graded asphalt mix needs to be designed for a road that will be subjected to heavy or very heavy traffic, the designer would be advised not to rely on the Marshall mix design method since there is doubt as to the ability of the compaction procedure associated with this method to adequately simulate the ultimate field densities. The designer should rather use a gyratory-based design method in an attempt to gain better confidence that the design density used for the optimisation of the mix approximates the ultimate densities that will be achieved in the field after approximately two years of trafficking, where the recommendations in Table 3 could possibly be used as a guideline.

Having decided on an overall design strategy, the most appropriate mix type would then be selected, whether it is a mix type with a strong coarse aggregate skeleton, such as porous, semi-open graded or stone-mastic asphalt, hereafter referred to as stone-skeleton mixes, or a mix type that will carry the traffic load by a combination of both coarse and fine aggregate or only fine aggregate, such as continuously-, semi-gap- and gap-graded asphalt, hereafter referred to as sand-skeleton mixes, following which the mix optimisation procedure can be initiated.

In general terms, it can be said that the behaviour of HMA is analogous with that of a Coulomb material (Tan et al, 1994), where resistance to permanent deformation is derived from cohesion and internal friction due to the interlocking resistance of particles. For HMA, the cohesive component is clearly derived from *the mastic* (binder and filler), while the shear component is derived from *aggregate interlock*. Since binder tends to soften at high temperatures and low rates of loading, the cohesion will also tend to be lower under these conditions. Aggregate interlock, on the other hand, is not dependent on temperature or rate of loading, but only on the extent to which the stress state tends to push the particles together or apart (i.e. on the magnitude of the volumetric stress component).

3.3.1 Aggregate interlock and aggregate properties

To maximise resistance to permanent deformation, *HMA, when compacted to design (ultimate) density, should have developed a dense and stable aggregate skeleton with optimal aggregate interlock which, ideally, should resist further plastic deformation of the*

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mix. Various analytical and laboratory-based techniques have been developed in recent years (BRRC, 2002; Semmelink, 1999) which assist designers in optimising the aggregate grading in line with the above. Studies have shown that important performance parameters such as resistance against permanent deformation and fatigue resistance (Roque et al, 2006) as well as mix permeability (Khosla and Sadasivam, 2005) and compactability (Vavrik et al, 2002) are strongly related to the grading characteristics of the coarse aggregate.

A recent technique that is gaining popularity in South Africa is the method originally developed by Robert Bailey, which gives the designer tools to develop and adjust aggregate blends so as to ensure aggregate interlock and good aggregate packing, providing resistance to permanent deformation, while maintaining volumetric properties that enhance constructability and provide resistance to environmental distress (Vavrik et al, 2002). The Bailey method can be used to optimise the gradings of sand- and stone-skeleton mixes, including stone-mastic asphalt (SMA) and mixes that incorporate Recycled Asphalt Pavement (RAP).

Once the grading and binder content have been optimised using methods such as the Bailey method and the gyratory design method, and given that the volumetric requirements have been met, the resulting mix should be submitted to validation checks. One such method is proposed by Pine (1997), who developed the concept of the “3-2-2 locking point” in gyratory testing, which is defined as the gyration at which the aggregate skeleton “locks” together and further compaction results in aggregate degradation and very little additional compaction. In order to identify the 3-2-2 locking point, the method uses the first gyration in a series of three successive gyrations of equal sample height that immediately followed two sets of two successive gyrations of equal heights, as illustrated in Figure 2. In NCHRP 9-9 (Prowell and Brown, 2007), the specimen density determined at this 3-2-2 locking point was correlated to the ultimate HMA density. It was shown that for 90 per cent of the mixes evaluated, the locking point densities exceeded the ultimate HMA density. Thus, conservatively, the optimum binder content of an HMA could be selected to produce 4 per cent design air voids at the 3-2-2 locking point as an alternative to the N_{design} values given in Table 3.

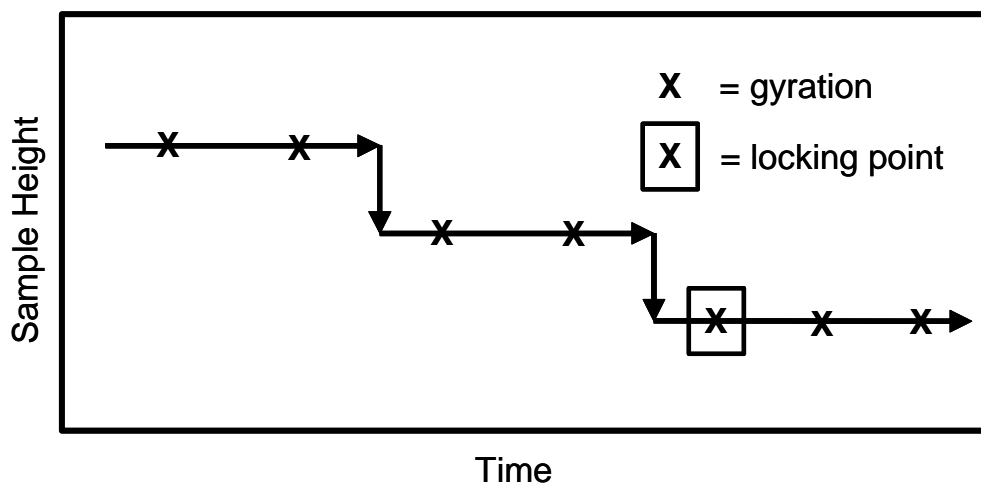


Figure 2: Locking Point 3-2-2 (Pine, 1997)

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It is recommended that both the Bailey method as well as the concept of the locking point be further explored for possible wider implementation in South Africa, and that the latter be considered as a replacement of the 300-gradation proposals currently contained in the South African Interim Guidelines (see Table 2a).

Optimisation of the grading presupposes that all aggregate and volumetric requirements have met the necessary requirements a-priori. To achieve optimal resistance to permanent deformation, particularly for HMA wearing courses subjected to heavy traffic, this would include the following:

- Ensure that the maximum particle size in the mix is not too small (aim to select a maximum particle size that is one-half of the layer thickness, or a nominal maximum particle size that is one-third of the layer thickness);
- Ensure that aggregate particles are angular-shaped so that greater interlock and internal friction, and hence greater mechanical stability, are achieved;
- Avoid flat, elongated particles because these would impede compaction and may prevent the development of satisfactory strength in HMA;
- Maximise the use of crushed aggregates (in the COLTO Specifications (1998) it is specified that for surfacings, at least 95 per cent of all particles shall have at least three fractured faces);
- Avoid the use of single size rounded natural sands in HMA to improve workability and compactibility (continuously graded mixes that are easy to compact during construction will tend to densify beyond the design (ultimate) density under traffic, ultimately leading to rutting due to low voids and plastic flow);
- Select durable, hard and tough aggregates with a high resistance to abrasion and crushing, particularly for stone-skeleton type mixes;
- Assuming that the laboratory compaction procedure adequately simulates the ultimate field densities, optimise the binder content for sand-skeleton mixes to yield 4 per cent air voids, and not 5 per cent, as the latter could expose the layer for a longer period of time to oxidative hardening of the binder and potential moisture damage. (Note: for stone-mastic asphalt, the design void content could be selected between 3 and 4 per cent);
- Ensure that the minimum requirements for Voids in the Mineral Aggregate (VMA) are met;
- Ensure that the optimum binder content is selected on the dry side of the VMA curve;
- Ensure that for stone-mastic asphalt the Voids in Coarse Aggregate (VCA) are not overfilled with fine aggregate, filler and binder;
- Ensure that the criteria for Voids Filled with Binder (VFB) are met.

3.3.2 Binder-filler mastic

As was pointed out earlier, both the aggregate interlock and binder-filler mastic contribute to HMA's resistance to permanent deformation. For stone-skeleton mixes, the resistance to permanent deformation is predominantly derived from the coarse aggregate skeleton. However, for sand-skeleton mixes, both the fine aggregate structure and the binder-filler mastic are important contributors, the relative importance of the latter being dependent on the strength of the skeleton structure and the effectiveness of the frictional resistance offered by the grading and the aggregate characteristics.

Since bituminous binders are thermoplastic and viscoelastic in nature, they soften at high temperatures and at low rates of loading. Therefore the two characteristics of the mastic

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that play a role in determining rut resistance are (1) the viscosity of the mastic and (2) the temperature sensitivity of the mastic. Because of these characteristics, design of sand-skeleton mixes that have to operate at high service temperatures (up to say 60°C) require more effort to ensure that the resistance to permanent deformation is mainly provided by the aggregate components, and that mastic stiffness is only of secondary importance. However, increased resistance to rutting can be obtained by selecting stiffer mastics. This can be achieved by the addition of filler (in moderation), the selection of higher viscosity binders, modification of the binder or a combination of these.

Mineral filler characteristics

Mineral filler characteristics and their impact on permanent deformation of HMA vary with the grading and shape of the filler as well the amount of filler added to HMA:

- If the filler contains a high proportion of superfines (i.e. particles typically smaller than 10 microns), these fine particles will act as binder extenders, resulting in an over-rich HMA. This could result in a situation where the voids would be overfilled with binder, resulting in a potentially unstable mix. The fineness of the filler can be determined from particle size analysis and the Methylene Blue test, where high values of methylene blue would indicate finer filler material.
- Some fillers have a considerable effect on the binder, making it to act as a much stiffer grade of binder compared to the neat binder grade, which will positively impact on resistance to permanent deformation, but could have adverse effects on workability and compactibility, as well as on fatigue and fracture behaviour.

Whereas a reasonable amount of filler in HMA will increase the cohesive strength and stiffness of the mastic, thus providing additional resistance to permanent deformation in sand-skeleton mixes, designers should steer away from using excessive amounts of filler. For this reason, filler-binder ratios are usually limited to a maximum of 1.5, based on mass.

Binder selection

The higher the viscosity at 60°C and the higher the softening point of the binder both after rolling thin-film oven ageing and in situ ageing, the greater the binder's contribution will be to increasing the stiffness of the mastic and reducing HMA's temperature sensitivity. Whereas the selection of harder binders will contribute to improving resistance to permanent deformation, the designer should always assess the implication of this selection on other properties essential to well-performing HMA, such as resistance to fatigue and thermal cracking, as well as durability.

With CSIR ThermalPADS™, South African designers now have at their disposal a tool by which the average 7-day maximum pavement temperature can be estimated for a particular site (Denneman, 2007). Dynamic Shear Rheometer (DSR) testing is also available locally and can be used to determine the necessary rutting parameters and classify bituminous binders in terms of Performance Grade (PG). With that, all the necessary tools are in place to provide better information to mix designers, enabling design risks associated with binder-related uncertainties to be reduced considerably. However, for this system to be fully functional, it is recommended that South Africa moves towards the adoption of performance graded specifications for bituminous binders.

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The use of the average 7-day maximum pavement temperature as a design parameter allows a departure from design methods relying on monthly mean pavement temperatures (MMPT) for rut prediction, such as those included in the earlier South African Mechanistic Pavement Rehabilitation Design Method (Jordaan, 1994). The MMPT in South Africa is generally below 30°C, while the average 7-day maximum pavement temperature can exceed 60°C. Preliminary results from Heavy Vehicle Simulator (HVS) tests (see Section 4.2) have indicated that the rut rate of a mix evaluated under a standard axle load at a pavement surface temperature of 60°C can be 20 times higher than at 40°C. Hence, the highest increase in permanent deformation is expected to occur during the hot hours of the days, and not necessarily at the mean temperature.

The addition of certain binder modifiers and additives, such as plastomers, some elastomers, carbon black and naturally occurring hydrocarbons, can increase the mastic stiffness and the toughness of the mix, and hence contribute to improving rut resistance and reducing the risk for bleeding at high service temperatures. The decision whether to incorporate a modifier or additive in HMA, and if so, what class, should always be based on the service conditions and/or fundamental requirements such as (Asphalt Academy, 2007):

- Predicted traffic loading;
- Predicted in-service pavement temperatures (high: increased risk of rutting; low: increased risk for premature cracking);
- Stiffness/resilience of the support structure;
- Degree and extent of existing distresses prior to rehabilitation;
- Type of mix selected.

It should be noted that although each modifier or additive may offer certain benefits over conventional HMA, none is a panacea. Sometimes one property is improved at the expense of another. Hence, the designer should always first ensure that HMA, and particularly the aggregate structure, is optimised using conventional wisdom, and only then should the binder or mix be engineered with the right type of modifier or additive to enhance specific properties.

Evaluation of the mastic

One way to evaluate the cohesive strength of the mastic is by means of the Indirect Tensile Strength (ITS) test. ITS is a measure of the tensile strength required to pull apart the combined aggregate skeleton and mastic structure of HMA by the application of tensile forces. However, since the specimens are unconfined at the sides and the contribution of the coarse aggregate structure to ITS is limited to the adhesive strength at the contact points between coarse aggregate particles, ITS is predominantly a measure of the cohesive strength of the mortar, i.e. the combination of the mastic and fine aggregate portion of the grading.

The above explains the vast differences in ITS responses obtained between stone-skeleton and sand-skeleton mixes. In stone-skeleton mixes, which typically tend to produce low ITS values, there is either insufficient mortar available (i.e. porous asphalt), or the mortar is only being added to fill the voids created by the coarse aggregate skeleton (i.e. stone-mastic asphalt). In the latter case, at the point of coarse aggregate interlock, mortar is left in a fairly uncompacted state. In both cases, the ITS values will be fairly low (values of between 300 and 600 kPa are not uncommon). In fact, if fairly high ITS values of approximately 800 kPa were to be obtained for SMA mixes, this could imply that the

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voids in the coarse aggregate are overfilled and the mortar has been compacted and, hence, that coarse aggregate interlock has not been achieved. This may result in the HMA mix functioning as a sand skeleton mix instead with reduced resistance against permanent deformation.

Conversely, in sand-skeleton mixes, the mortar is compacted and hence contributes to the structural strength of HMA, where high ITS values would be indicative of high cohesive strength, which for these types of mixes is an essential contributing factor for the provision of adequate resistance to permanent deformation. ITS tests conducted over a range of temperatures would also provide information on the temperature sensitivity of the mastic.

Although, from the perspective of rutting resistance, high ITS values for continuously graded sand-skeleton mixes would be preferred, excessively high values of ITS (say in excess of 1 400 kPa) could be indicative of a stiff and potentially brittle asphalt mix which, when applied as a thin-layer wearing course, could be susceptible to premature cracking.

The above suggests that for sand-skeleton mixes, both minimum and maximum values for ITS should be specified, whereas for stone-skeleton mixes, and particularly for stone-mastic asphalt, only maximum values for ITS should be recommended.

4. SHEAR DEFORMATION OF HMA

4.1 Manifestation

The adoption of sound design principles in line with those presented in Section 3 will go a long way in ensuring that the mix will be sufficiently resistant to permanent deformation. However, there are cases where additional precautions need to be taken to ensure that the mix is sufficiently stable to resist shear forces induced by traffic. This would typically be the case for HMA applied on high-stress areas such as steep inclines, intersections and sharp curves, particularly when the completed HMA layer is to be opened to traffic shortly after construction (especially when the mix has not yet fully cooled down), where heavy traffic moves at slow speeds, traffic lanes are narrow and/or when service temperatures are high.

In such situations, permanent deformation is normally the result of shear deformation that takes place when the combined resistance to deformation afforded by friction and cohesion is overcome by the applied shear stress, as previously discussed. This will result in small flow movements associated with repetitive traffic loads. Aggregates and mastic are gradually pushed downwards, sideways, upwards and, in zones of acceleration or deceleration, forwards and backwards in small increments, eventually resulting in rutting, shoving, corrugations and/or tearing of the HMA.

The highest risk for the above is when the mix is opened to traffic shortly after completion of construction, and thus when the in situ HMA density is at its lowest and the frictional resistance, which has been optimised for conditions at ultimate density, has therefore not yet been fully mobilised, and where the binder may not yet have set adequately, particularly if the HMA is placed during hot summer months.

In the worst case, the early distresses observed on the road will be analogous with those observed on field sections subjected to accelerated pavement testing (see Figure 3), or on slabs or cores subjected to wheel-tracking tests in the laboratory, when performed in channelised mode on recently compacted HMA at high pavement temperatures. Hence, in

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order to simulate the worst conditions, it is recommended that wheel-tracking tests be conducted at the compacted density of HMA, at slow speeds and at the average 7-day maximum pavement temperature. This said, wheel-tracking tests are also ideally suited to assess whether HMA that has consolidated to its ultimate density (see Section 3) is likely to further deform under traffic as a consequence of further consolidation (if the mix is not properly designed) and/or shear deformation.



(a) Uphill section of R103 between Villiers and Warden



(b) Heavy Vehicle Simulator test section on R80 outside Pretoria (tested at an equivalent 80 kN axle load at 60°C)

Figure 3: Examples of premature permanent deformation

4.2 Evaluation procedures

There are a variety of procedures and devices that have been used to evaluate the rutting potential of HMA. In a study by Zhang et al (2003), these were classified into three groups as listed below:

Fundamental tests:

- Unconfined and confined cylindrical specimens in creep;
- Repeated dynamic loading;
- Cylindrical specimens in diametral creep or repeated loading;
- Superpave Shear Test;
- Repeated shear at constant height test;
- Shear modulus test;
- Quasi-direct shear test and shear strength test.

Empirical mix design related tests:

- Marshall Stability and Flow;
- Hveem Stabilometer and Cohesionmeter.

Simulation tests:

- Wheel-tracking tests.

The equipment required for the fundamental tests is complex, expensive and in some cases rarely available, which makes these test methods unsuitable for evaluating rutting

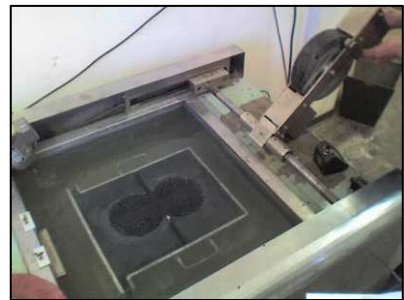
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potential on a routine basis. Some of the test methods, such as the unconfined dynamic creep test that has gained acceptance in South Africa, have been found to be unable to properly evaluate aggregate interlock and correctly rank mixes for permanent deformation. Empirical tests, such as Marshall Stability and Flow, have outgrown their usefulness for assessing rut potential of HMA placed on high-stress areas and heavily trafficked roads.

On the other hand, wheel-tracking tests have been shown to be able to adequately simulate field traffic and loading conditions and, generally, good relationships have been obtained between wheel-tracking results, rutting performance measured in the field and data collected during full scale loading tests. Wheel-tracking tests are also sufficiently easy to conduct, which make them ideally suited for evaluating rutting potential on a routine basis.

There are three types of wheel-tracking testing devices currently available in South Africa (see Figure 4). These are:

- (a) the Model Mobile Load Simulator (MMLS Mk3) which is able to perform tests in both dry and wet conditions at predetermined temperatures and loading conditions on cores in the laboratory or on actual pavements in the field;
- (b) the Transportek Wheel-Tracking Tester (TWTT), a fixed laboratory facility that has the ability to both produce and test asphalt slabs at a range of loading regimes and temperatures, and
- (c) the Hamburg Wheel-Tracking Device (HWTDD), a fixed laboratory facility that is able to perform tests in both dry and wet conditions at predetermined temperatures on both cores and slabs. Both the MMLS Mk3 and the Hamburg Wheel-Tracking Device are able to assess permanent deformation resistance and moisture susceptibility of HMA.



(a) MMLS Mk3

(b) TWTT

(c) HWTDD

Figure 4: Examples of Wheel-Tracking Devices

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In a recent study conducted for the Gauteng Department of Public Transport, Roads and Works, the permanent deformation characteristics of a standard asphalt mix, i.e. a mix used for general applications, were assessed in a comprehensive study involving accelerated pavement tests conducted on test sections constructed near Road R80 in Pretoria, and laboratory tests. As part of the test programme, the deformation rates produced by the TWTT and HWTD on laboratory specimens were compared against the creep rates produced by the Heavy Vehicle Simulator (HVS) and MMLS on test sections in the field over a range of temperatures. The aim was to develop reliable deformation prediction protocols for the different tests, which would address the concerns raised by Lorio et al (2004), who stated, after investigation of the TWTT and MMLS, that these two wheel-tracking test device could not be used confidently to assess the rut susceptibility of HMA.

The preliminary results shown in Figure 5 indicate that the slopes of the deformation rate versus temperature curves of the TWTT and HWTD are similar to that of the HVS. Hence, the rutting propensity of HMA as a function of temperature as determined in a laboratory environment seems to correlate well with that observed under full-scale testing, although the magnitude of the creep rates differs. It is speculated that the latter could partly be attributed to temperature gradients in the field, which were absent in the laboratory, as well as scale and load geometry/confinement differences between laboratory and field. The lower MMLS deformation rates arising from MMLS testing at 60°C requires further investigation.

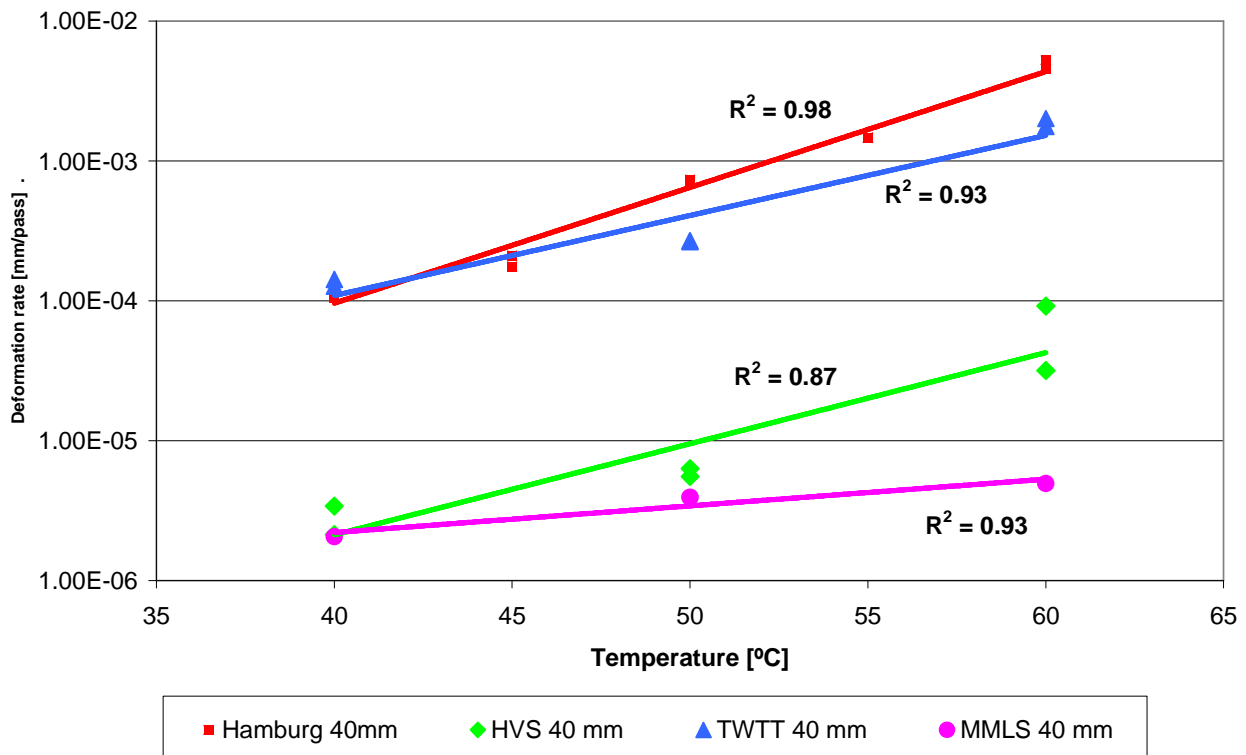


Figure 5: HVS, MMLS, TWTT and HWTD deformation rates as a function of temperature

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Since the Hamburg Wheel-Tracking Device is a fairly simple, easy to operate and affordable device, it is recommended that it be adopted by testing laboratories for routine evaluation of not only rutting resistance but also moisture susceptibility of HMA.

5. CONSTRUCTION CONSIDERATIONS

As was pointed out in Sections 3 and 4, one of the main causes of accelerated rut development, especially when the asphalt mat is still fresh, is insufficient compaction at the time of construction, which not only would result in higher levels of consolidation under traffic, but also could render the mix more susceptible to shear deformation in the early life of the HMA layer. Not achieving adequate levels of compaction will also result in higher degrees of permeability and binder hardening which in turn may lead to premature cracking, stripping and ravelling, shortening the life of the mix. Additionally, if left untrafficked for an extended period, further consolidation may be significantly inhibited which will aggravate the onset of the distresses referred to above. Generally, for every one per cent decrease in density below the minimum required density, the life of HMA could reduce by 10 per cent.

Hence, during construction every attempt should be made to meet the density specifications of HMA. For continuously graded mixes, minimum density specifications are usually set at 93 per cent of maximum theoretical relative density (MTRD), and for stone-mastic asphalt mixes at between 94 and 95 per cent. Also maximum density requirements should be specified to prevent over-compaction of HMA, which may result in bleeding as well as permanent deformation in the form of rutting, shoving and corrugations. Maximum density specifications are often set at 96 per cent of MTRD.

There are a number of factors which impact on achieving the required level of compaction. These include:

- HMA temperature during the various stages of construction, particularly during paving and compaction, which strongly relates to air temperatures, wind velocity, humidity, layer thicknesses and binder viscosity;
- High filler-binder ratios which may increase the viscosity of the mastic;
- High volume concentrations of coarse aggregates, particularly with respect to the ratio between the maximum (nominal) stone size and the layer thickness;
- Temperature and firmness (stiffness) of the layer on which the HMA layer is to be placed.

When HMA is paved in thin layers, there is usually only a small time window available during which minimum densities can be achieved, especially if the mix is paved in fairly cold and windy conditions. Sabita Manual 22 (2006) provides recommendations for paving in adverse weather conditions and also enables the risks of operating in such adverse conditions to be assessed.

The layer on which the HMA is to be placed must be clean, hard, firm, fully compacted, sufficiently dry and level. The supporting layer should act as an anvil in that, ideally, it should not absorb any compaction energy, and it should be level so as to prevent differential compaction in the HMA layer. Also, the use of a tack coat should be mandatory for all situations as the tack coat acts as a glue preventing sideways movement of the mix during compaction. In the absence of a tack coat, the HMA may separate (“debond”) from the supporting layer, resulting in a reduction of pavement life.

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During compaction, the contractor may experience excessive shoving of the mix under steel wheel rollers which could result in difficulty in achieving minimum densities, as well as encounter mixes that are tender to any type of distortion. Such mixes are often defined as “slow setting”. Given that compaction equipment is used appropriately, there are various causes that may contribute to this phenomenon, including (Tarrer and Wagh, 1994):

- Incorrect mix design, including the selection of binder contents that are too high;
- Too much middle-sized sand in the mix, particularly if a high percentage of natural sands is used, as well as the presence of smooth, rounded aggregates;
- Small maximum size aggregate with respect to layer thickness;
- Insufficient amount of filler in the mix;
- The use of low-viscosity binders, or binders that do not set properly;
- The ambient temperature is too high;
- Density of the supporting layer (i.e. a high percentage of the compaction energy is absorbed by the supporting structure);
- The presence of moisture in the mix.

To prevent the above from occurring, the recommendations of Sections 3.3.1 and 3.3.2 should be adhered to, especially with respect to the selection of mix components and the mix design processes that should be adopted. It is also recommended that:

- *The binder absorption characteristics of the aggregates be assessed.* Often, the use of highly absorptive aggregates increases the propensity of the mix to be tender, especially if the rate of absorption is slow. Since additional binder may have to be used to allow for eventual binder absorption, the mix may be temporarily too rich in binder, which may result in a tender mix at the time of construction;
- *The binder hardening characteristics be assessed.* The binder is expected to undergo ageing during the manufacturing process. This is simulated in the laboratory by means of the Rolling Thin-Film Oven Test (RTFOT). After RTFOT, it is expected that the softening point and viscosity of the binder would increase, and often minimum values for these properties after RTFOT have been set, so as to ensure that the binder will have adequate setting characteristics. Asphaltene content of the binder before and after RTFOT could also be used to assess whether sufficient polymerisation (i.e. joining together of smaller molecules to form larger molecules) has occurred.

Finally, trial sections should be constructed to assess and validate the final mix design and to hone the contractor’s operations. Trial sections should be of sufficient length to achieve this. The following should be determined from the trial sections:

- Optimum asphalt paver settings;
- Optimum rolling techniques to ensure optimal compaction to achieve density specifications;
- Mix properties and fine tuning of laboratory-based mix design to cater for shifts between laboratory and construction operations (including the asphalt plant);
- A precise recording of the process between laboratory and plant to be used for quality control;
- Performance-related properties of as-built HMA (e.g. wheel-tracking tests and permeability).

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The end product of the construction process should be a fit-for-purpose asphalt mix of adequate density with not only adequate frictional resistance and cohesion to resist shear deformation under traffic, but also of sufficient durability to resist fatigue cracking and moisture-induced damage.

6. CONCLUSIONS AND RECOMMENDATIONS

The paper has attempted to redress the concerns expressed at the 8th CAPSA that the design of hot mix asphalt did not adequately address prevention of rutting. It presents a consolidation of best practice in the design and application of asphalt layers pertinent to the mitigation of rutting, while being mindful of other performance requirements in terms of fatigue strength and durability.

The following conclusions and recommendations are made:

- HMA design should always start with a review of the site conditions which should show the potential for rutting, which then would enable the designer to identify the required level of design reliability that would need to be achieved, as well as to select the most appropriate mix design strategy (including mix type selection) and design method that will satisfy all requirements.
- There are several techniques and procedures available to aid the designer to mitigate the potential for rutting in asphalt layers while ensuring that fatigue strength and durability are not adversely affected.
- Two stages in the development of permanent deformation in HMA – consolidation and shear flow – leading to ruts in the wheel tracks should be taken into account in the design and specification of density to be achieved after construction.
- Care should be taken to ensure that the density of laboratory-compacted samples, used for the determination of compositional parameters and performance characteristics, approximate the ultimate density of the layer, i.e. the density that will be reached after about two years under traffic. Hence it is imperative that the method adopted for compaction in the laboratory provides a reasonable replication of the compaction during construction and further densification under traffic.
- The Marshall mix design method can be used for lower traffic or rut potential situations, but for high traffic or rut potential situations, gyratory-based design methods should be used, possibly coupled with performance tests.
- A clear understanding of the packing and interlocking characteristics of the mix components as they affect the behaviour and performance of HMA is a prerequisite for achieving an optimal design. The method proposed by Robert Bailey should be considered for general use in cases where rutting is a key design parameter.
- To avoid excessive laboratory compaction that may lead to aggregate degradation with little gain in density, such as may arise by adopting a 300 gyration cycle, it is suggested that the method known as the “3-2-2 locking point”, being the gyration at which the aggregate skeleton locks together, be considered. The resultant density should be used as an additional criterion to establish the optimum binder content that will result in four per cent design air voids.

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- In the preparation of asphalt specimens using the Marshall compaction apparatus, it is imperative that the apparatus used by various parties in the quality management and acceptance control process of a specific road project be calibrated as suggested in ASTM D6926-04 to improve the accuracy of results and hence avoiding conflict over product acceptance. The implementation of a national laboratory proficiency scheme would advance the achievement of this goal.
- The Indirect Tensile Strength (ITS), considered to be measure of the strength of the mastic of a mix, is likely to yield higher values for sand skeleton mixes than for stone skeleton mixes, particularly SMA. High ITS values of SMA mixes could imply that the voids in the coarse aggregate are overfilled, with the mortar compacted as a consequence and coarse aggregate interlock not having been achieved. Excessively high ITS values for SMA (say > 800 kPa) should be avoided. Conversely, for continuously graded sand skeleton mixes, excessively high ITS values, (say, > 1400 kPa) could be indicative of a stiff and potentially brittle asphalt mix, which may lead to premature cracking in thin layers. Hence it is suggested that for continuously graded sand skeleton mixes both a minimum and maximum values for ITS should be adopted and that an upper limit of ITS be applied to SMA.
- International experience as well as the (tentative) outcomes of South African studies suggest that wheel-tracking test are able to appropriately rank asphalt mixes in terms of their resistance to permanent deformation. It is suggested that the Hamburg Wheel-Tracking Device, a fairly simple, easily operated and affordable device, be considered for routine laboratory investigations into the rutting potential of HMA.
- The achievement of adequate compaction during construction of an HMA layer is of paramount importance to its satisfactory performance and requires particular attention to mat temperatures and rolling techniques.
- The use of an average 7-day maximum pavement temperature as a design parameter allows a more realistic assessment of rut development than other methods in use, such as a monthly mean pavement temperature. If Performance Grade binder specifications were to be adopted in South Africa, this would facilitate the selection of the most appropriate binder for a given situation.

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KEYWORDS

Hot-Mix Asphalt, Permanent Deformation, Design, Construction