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INNOVATIONS ON THE ASPHALT MIX DESIGN FOR THE REHABILITATION OF NATIONAL ROUTE 3 BETWEEN MARIANNHILL AND KEY RIDGE

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Abstract

In 2005, National Route 3 between Mariannhill Toll Plaza and Key Ridge was due for rehabilitation after about 20 years of service. The unique type of loading on the road required a rut resistant asphalt mix without loss of fatigue resistance properties. A comprehensive asphalt mix design process was embarked upon, with the performance prediction of the mix of key importance. Mixes with an A-P1 and A-E2 binders were pre-selected for evaluation. Three trial sections, of varying binder contents were constructed for each binder type. Extensive performance tests, which included Model Mobile Load Simulator (MMLS), Hamburg wheel tracking and flexural beam fatigue testing were performed on cores and beams extracted from the trial sections. This paper presents the results from this mix design procedure in detail and also how the decision was made on binder type selection. The A-P1 mix was found to have superior performance over the SBS mix in this case and almost 60 000 t of asphalt base was produced and paved during the construction.

1 INTRODUCTION

Asphalt pavements on high volume roads are often subjected to extreme operating conditions with respect to loading and environmental conditions. The structural design of these pavements requires a high degree of care, and has to be done in conjunction with the mix design of individual structural supporting pavement layers. In a few extreme cases, the mix design of individual pavement layers is of considerable importance to the structural design in order for the structural design to live up to the expectations. National Route 3 between Durban and Gauteng in South Africa is a very good example of such a road. Sections of this route experience extreme loading conditions due to the topography and/or environmental conditions.

Thus, when the section of the N3 between the Mariannhill Toll Plaza and Key Ridge was due for rehabilitation, a high degree of care and attention was required during the structural pavement design, pavement layer mix design and construction for this intervention to have a good chance of success.

The objective of this paper is to illustrate the innovative mix design process followed, which considered a large number of parameters within the pavement structural design restrictions experienced on the project.

2 BACKGROUND

The N3 freeway is the major economic arterial between the port city of Durban and the economic hub of Africa 600 km inland in Gauteng. A large quantity of import and export freight is transported over this route daily.

The section of the N3 between Mariannhill and Key Ridge was initially constructed in 1985/1986. The original pavement consisted of a 40 mm semi-gap graded asphalt surfacing, 125 mm continuously graded asphalt base layer, two cement stabilised crushed gravel subbase layers of 150 mm each and a 150 mm natural gravel selected layer. The subgrade was generally poor material with an in-situ CBR of generally less than 3. During construction a large quantity of earthworks was required, which somewhat dilute the effect of the subgrade due to high fill and deep rock cuttings.

The pavement received some maintenance activities between 1994 and 1999 (9 to 14 years after construction), which mainly consisted of mill and replacement of rutted asphalt in the slow lane, crack sealing and a 40 mm continuously graded asphalt overlay. A short section also received a 13 mm bitumen rubber seal. However no major rehabilitation was carried out, until the road was identified for rehabilitation and possible pavement strengthening in 2006.

3 TRAFFIC LOADING

Traffic volumes on the N3 reached 16 000 vehicles per direction in 2006 with up to 17 % heavy vehicles. Total number of heavy vehicles were in the order of 3 000 to 4 000 trucks per day per direction, the majority of this being in the slow lanes. Individual traffic counting loops in each lane made it possible to determine the design traffic loading in each traffic lane, and in addition to data from an adjacent Weigh in Motion (WIM) site, could be used to accurately determine the current traffic loading as well as estimating the future traffic loading for each traffic lane. Vehicle loading in terms of Equivalent Standard Axle Loads (E80's) was found to be between 2.8 E80 per heavy vehicle southbound and 3.1 E80's per heavy vehicle northbound. This is lower than expected, but is due to the high percentage of small commercial vehicles in the total heavy vehicle stream as well as effective overload control measures in place along this portion of the route.

The cumulative E80'straffic loadings over a period of 15, 20 and 30 years are presented in Table 1.

Table 1: Cumulative traffic loading

		15 years	20 years	30 years
South of Key Ridge (M13 i/c)	Slow lane	57 million	83 million	146 million
	Middle lane	18 million	24 million	44 million
North of Key Ridge (M13 i/c)	Slow lane	68 million	105 million	185 million
	Middle lane	22 million	32 million	54 million

4 PAVEMENT REHABILITATION SELECTION AND DESIGN

The existing pavement could be considered too thin for the future traffic loading in the slow lane, while the middle and fast lane pavements was considered adequate. Any overlay option to increase the strength of the slow lane, would be required over the full width of all lanes (including middle and fast lanes). This will result in considerable additional pavement layers on two lanes of the pavement that does not require any strengthening. Asphalt and concrete overlays were therefore not considered to be economical alternatives at the time.

A concrete inlay approach would require considerable reconstruction of the slow lane in order to accommodate a concrete pavement, while a pavement design life of less than 30 years was also not considered to be appropriate for a concrete inlay alternative. The high traffic loading over 30 years in the slow lane (150 to 200 million standard axles), would also require careful consideration of the concrete/asphalt longitudinal joint between the middle and slow lanes. In order to maintain structural integrity of the slow lane at that high traffic loading, a concrete shoulder and middle lane would also need to be considered.

An economic analysis was carried out on the various alternatives and a asphalt inlay of the slow lane and partial inlay of the middle lane was proven to be the most economical alternative at the time. A structural design period of 15 years was adopted.

4.1 Slow lane

The 15 year cumulative traffic loading in the slow lane was between 57 and 68 million standard axles E80's, and according to the TRH4 pavement design guide, a 450 mm stabilised subbase and 130 mm asphalt base would be required for a ES100 pavement. However, the existing pavement only has a 300 mm stabilised subbase and a total of 160 mm available for an asphalt base and surfacing. Reconstruction and stabilisation of the subbase to a depth of 450 mm over the entire length on the project would not be cost effective. The subbase reconstruction measure to a thickness of 300 mm was implemented over a 2 km road section at Key Ridge where the existing subbase was damaged.

Over the remainder of the project the existing pavement structure was retained. Additional attention to the design of the asphalt base layer was required in order to accommodate the traffic loading. Mechanistic-empirical analysis, using available mechanistic-empirical models at the time, indicated that if the required stiffness of the asphalt base should be around 4 000 MPa for the pavement to perform well over a 15 year design life. It is however important to note that the mechanistic-empirical models available at the time only consider the fatigue of asphalt and not asphalt rutting, which is more common in pavements with very high traffic loadings over long periods of time.

The stabilised subbase has performed remarkably well since construction, with very little reflective cracking and weakened areas, as indicated through a number of intact cores extracted from the subbase. Weakened subbase areas were identified by analysing deflection measurements taken by the Lacroix Deflectograph at very close intervals.

The slow lane rehabilitation option was the replacement of the 160 mm asphalt base with a stiff asphalt base. Selective in-situ stabilisation of the slow lane subbase 300 mm thick was carried out where weakened subbase areas were identified.

4.2 Middle lane

From a structural point of view the middle lane was still structurally sound and did not require any strengthening. The upper sections of the asphalt base showed signs of bleeding and rutting and the rehabilitation of the middle lane was to remove the upper 80 mm of the asphalt and replace it with an asphalt mix similar to that for the slow lane.

Only isolated repairs were done on the fast lane, while all the lanes were surfaced with an ultra thin friction course (UTFC) in order to meet functional requirements (texture depth and skid resistance).

5 ASPHALT MIX DESIGN REQUIREMENTS

The pavement design option selected required the design and construction of an asphalt base mix that is rut resistant. In addition, however, the type of loading the pavement will experience, as well as the supporting pavement layers required the asphalt mix to also have fatigue resistance properties in order to recover from strains induced by the expected traffic loading, thereby limiting the development of fatigue cracking. The asphalt mix design would therefore require the assessment of various mix properties not normally associated with standard selection and asphalt mix design processes.

In terms of the American binder performance grading (PG) system, a PG70-10 binder was required for the traffic, environmental and road geometry conditions.

6 ASPHALT MIX DESIGN PROCESS

In order to satisfy the pavement design requirements, a mix design process, which is more intensive than the normal mix design processes, was adopted. The mix design included the parallel design of two mixes with two binder types, followed by the construction of trial sections of both binders and detailed performance tests on cores extracted from the trial sections.

The normal mix design process is shown in Figure 1.

Interim Guidelines for the Design of Hot-Mix Asphalt in South Africa (HMA, 2001) was prepared in 2001 as part of the Hot-Mix Asphalt Design Project launched in 1998 by SANRAL (South African National Road Agency Ltd), CSIR Transportek and Sabita (Southern Africa Bitumen Association). The aim of this project was to develop a new HMA design method for South Africa.

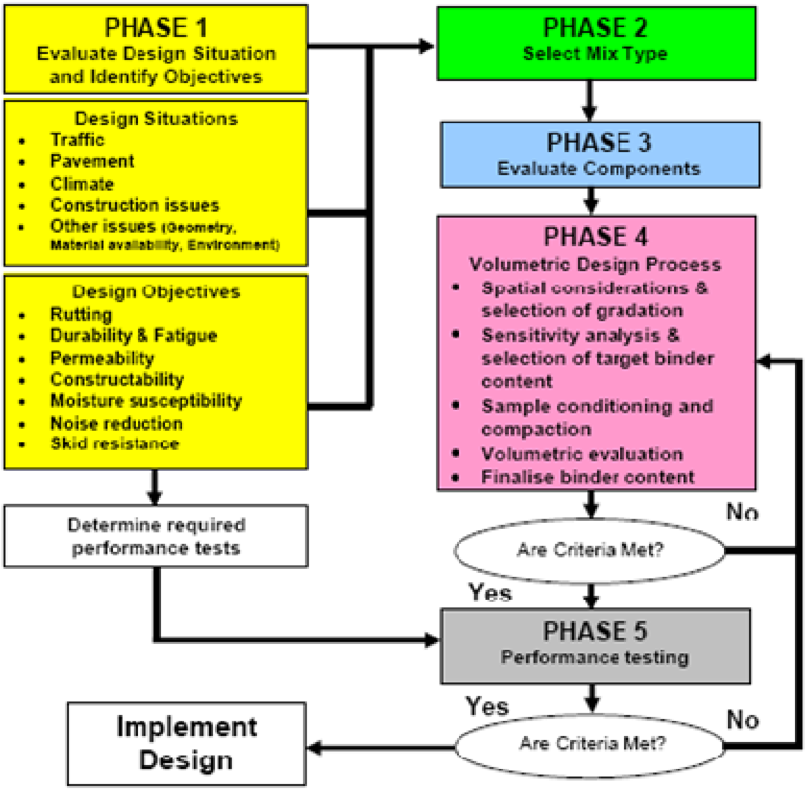

Figure 1: Standard Hot Mix Asphalt Design Process (HMA, 2001)

The most commonly utilised bitumen modifiers in South Africa are SBS, SBR and EVA. The manufacture, use and relative properties for these binders are contained in TG1: Gudelines for the use of Modified Bitumens (Asphalt Academy, 2007). Although generic in nature the relevant properties specified for use in hot mix asphalt are divided into elastomer (SBS) and plastomer (EVA) classes. Both classes of modifier were thus selected to be included in the mix selection and design process

A mix design process was developed for the project that followed the familiar mix design process in the initial stages, but includes more detailed performance testing towards the advanced stages of the mix design (Phase 5 in Figure 1). The mix design process further included the parallel design of two mixes with the same grading to assess the most fit for purpose mix for the required design parameters. The mix design process consisted of the following steps:

1. Pre-selection of binders to be used in the mix design process, in this case an A-P1 binder (using EVA) and an A-E2 binder (using SBS). This was mainly done based on experience of the behaviour of particular binder types and the required properties that a production mix is expected to have.

2. Pre-selection of aggregate sources, including the required tests to confirm compliance with specification as per the normal mix design process.

3. Determine the mix grading by using the COLTO (1998) specified grading envelopes as an initial guideline followed by the determining the optimum

grading using the Bailey method. It was also noted at this stage that the optimum grading may fall slightly outside the COLTO grading.

4. Perform normal Marshall and Gyratory mix design methods and determine mix volumetrics and optimum binder content on both the A-P1 and A-E2 binders, as normally done in the standard mix design procedure.

5. Perform plant trial mixes for each binder at optimum binder content, and evaluating consistency with laboratory prepared mixes by using Marshall briquettes as normally done in the standard mix design procedure.

6. Construct six trial sections; three for each binder at various binder contents. The trial sections were constructed in the slow lane directly north of the Toll Plaza and were removed at the end of the project. The trial sections were constructed at the optimum binder content and 0.3 % above and 0.3 % below the optimum binder content for each of the binders.

7. Perform laboratory evaluation of each trial section mix using Marshall briquettes and confirm consistency with plant trials and laboratory mixes as per the standard mix design procedure.

8. Extract cores and beams from trial sections and evaluate for: the following.

   a. Resistance to stripping using modified Lottmann and wet MMLS tests.

   b. Rutting resistance using MMLS and Hamburg wheel tracking tests

   c. Fatigue using bending beam fatigue tests.

   The performance tests were done on cores from the actual paved trial mixes in order to take into account possible differences the various binders may have on the manufacturing, placement and compaction of the mixes

9. Compare performance of mixes and compliance with specifications and recommend the most appropriate mix. This was done holistically taking into account all test results to date and compare them with the asphalt mix requirements.

10. Confirm production mix and determine quality control and assurance parameters in line with specification, such as target binder, target voids, and target compaction with acceptable and upper and lower limits.

The contract document was written such as to allow for the intensive mix design process and to encourage cooperation between the client, the contractor and the consulting engineer. The mix design process took approximately 6 to 8 weeks until an approved production mix was agreed and approved.

7 ASPHALT MIX DESIGNS

7.1 Aggregates

The coarse aggregates used were quartzite, while the fine aggregates were a mix of quartzite and tillite. Between 10 and 15% Reclaimed Asphalt (RA), that was fractionised and screened into coarse and fine fractions were included in the mix. During asphalt mix production, the RA was added into the mixing drum just before the bitumen and active filler injection point in order to avoid additional binder aging of the RA aggregates.

7.2 Grading

The asphalt base was to be a continuously graded mix, but the grading was determined by using the Bailey method and only using the specified COLTO grading envelopes as a guide. The main objective of using the Baily method was to achieve an optimum and dense as possible grading. Figure 2 presents a comparison between the adopted grading as determined by the Bailey method and the specified grading as per COLTO. The objective of the grading was to create an optimum grading matrix with good interlock.

Table 2. Bailey design parameters

%CA LUW	74
Primary Control Sieve (PCS) CA	4.75 0.68
FA.c	0.55
FA.f	0.47
Nominal Maximum Particle Size (NMPS)	13.2 mm(2)

note. (1) The above properties are based on the "old" ratios
(2) 13.2 mm at 75% passing the 13.2 mm sieve, However 19mm would probably be more representative of the mix at 94% passing 19 mm sieve

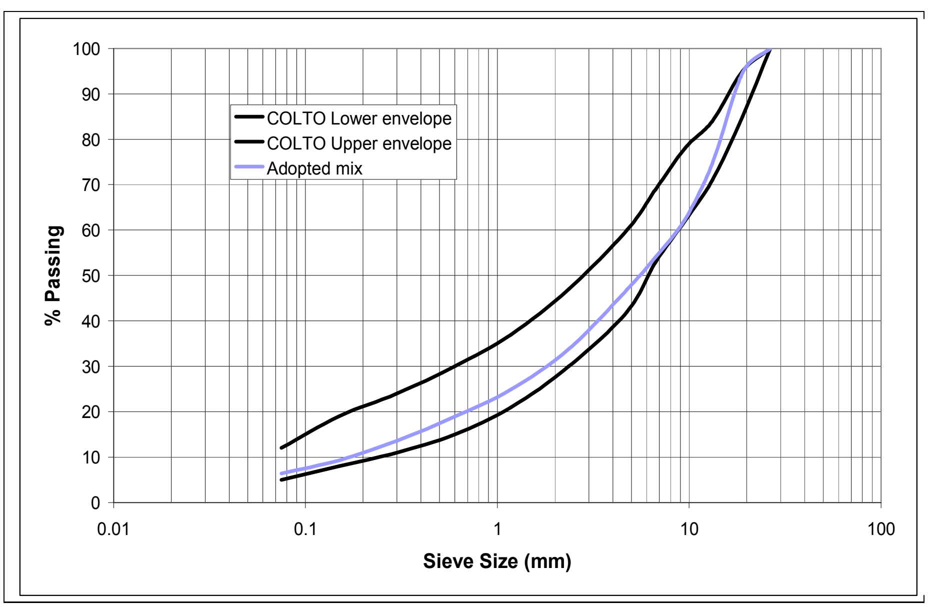

Figure 2: Adopted mix grading vs. standard COLTO continuous grading

7.3 Bitumen binders

The two mixes evaluated were an A-P1 mix with 4% EVA and an A-E2 mix with 3.5% SBS. The major properties of the two binders are presented in Table 2.

Table 3: Binder properties

Property	A-P1 (4% EVA) binder	A-E2 (3.5% SBS) binder
Softening point	68 °C	92 °C
Dynamic viscosity @ 165 °C	0.26 Pa.s	0.31 Pa.s
Dynamic viscosity @ 150 °C	0.44 Pa.s	0.56 Pa.s
Ductility @ 15 °C	32.7 cm	101 cm
Elastic recovery @ 15 °C	15 %	81 %
Torsional recovery @ 25 °C	25 %	38 %
PG Grading	70-28	64-22

Note. The PG grading was determined by determining the G*Sin γ for each binder at various temperatures.

The binder tests clearly indicate the increased elasticity and ability to recover from strain of the A-E2 (3.5% SBS) binder, which is generally beneficial for fatigue resistance. The ability of an A-P1 to stiffen the binders and assists in rut resistance is also apparent in the decrease in viscosity and increase in PG grading (Asphalt Academy, 2007).

7.4 Volumetric properties

The volumetric properties of the A-P1 and A-E2 mixes were similar, although the Marshall design resulted in a 0.2 % difference in optimum binder content for the two binders. The marginal differences in the gyratory compaction results and marshall air voids between the mixes can therefore be explained by the difference in binder content. The volumetric properties of the optimum binder content mixes of each binder are presented in Table 4.

Table 4: Volumetric properties of mixes

Property	A-P1 (4% EVA) binder	A-E2 (3.5% SBS) binder
Optimum binder content	4.20%	4.40%
Marshall Stability	22.2	17.5
Marshall flow	4.9	3.5
VMA	14.3	13.5
VFB	61	64
Air Voids	5	4.9

7.5 Performance testing

The testing of the performance of the asphalt mixes under laboratory and accelerated testing was the main component of the mix selection process. The performance testing would reveal whether minor adjustments would be necessary in the grading, binder selection and mix volumetrics in order to achieve the required performance.

Six trial sections of approximately 120 tonnes each were constructed in the northbound slow lane, approximately 1 km north of the Mariannhill Toll plaza. The trial sections were constructed 160 mm thick, using the equipment and team that would be used during construction. The A-P1 binder mixes were placed at 3,9 %, 4,2 % and 4,5 % binder contents, while the A-E2 mixes were placed at 4,0 %, 4,3% and 4.6% binder contents.

A total of 228 cores were extracted from the trial sections and tested for rut resistance, moisture susceptibility and permeability. In addition, 16 beams were extracted and subjected to flexural beam fatigue testing.

7.5.1 Resistance against deformation or rutting

Dry MMLS tests at 60 °C and 7 200 repetitions per hour (standard speed) were performed on the 4,2 % and 4,5 % A-P1 mix as well as the 4,3 % and 4,5 % A-E2 mix. To determine the performance of the mixes under slow loading conditions, further MMLS tests were done at 2 400 repetitions per hour (3rd of standard speed) on the 4,2% A-P1 and 4,3% A-E2 mixes. Wet MMLS tests at standard speed were performed on the 4,2% A-P1 and 4,3% A-E2 mixes to determine moisture susceptibility and potential for stripping of the mixes.

Figure 3 presents the results of the MMLS tests at standard speed dry, 33% speed dry and standard speed wet.

For high volume roads with steep inclines, a dry MMLS rut at standard speed after 100 000 repetitions of less than 2,5 mm is normally considered to be acceptable (Hugo, 2008). The A-P1 binder mix had a rut of approximately 1,19 mm after 100 000 repetitions, while the A-E2 binder mix had a rut of approximately 1,9 mm after 100 000 repetitions; an approximate 60% difference.

On completion of the dry MMLS test at 2 400 repetitions per hour, the A-P1 binder mix had a terminal rut of 1.66 mm after 100 000 repetitions, while the A-E2 binder mix had a rut depth of 2.49 mm after 100 000 repetitions.

The rut depth after 100 000 repetitions of the wet MMLS test at 7 200 repetitions per hour for the A-P1 mix was 1.81 mm and no signs of stripping was observed when the sample was visually inspected. The rut depth after 100 000 repetitions of the A-E2 mix was 3,21 mm and some early signs of stripping was observed at the end of the test.

On all three the MMLS tests, the A-P1 binder exhibit better rut resistance than the A-E2 mix, and on all three tests the A-P1 binder was below the maximum threshold of 2,5 mm after 100 000 load repetitions.

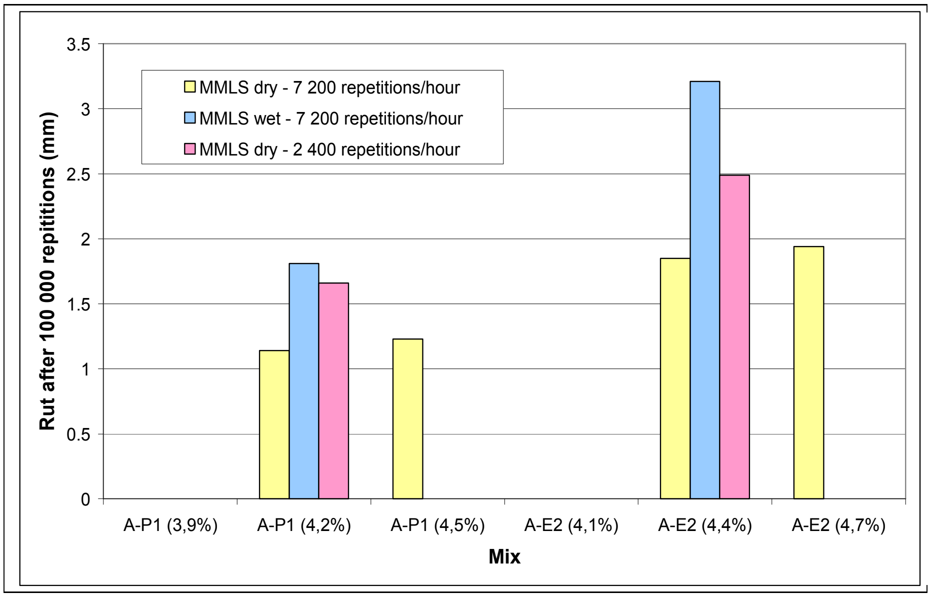

Figure 3: Results from MMLS tests

Hamburg wheel tracking tests on the 4,2 % and 4,5 % A-P1 mix as well as the 4,3 % and 4,5 % A-E2 mix were also performed to confirm the MMLS results and to gain further confidence in the potential rut resistance of the mixes.

Results from the Hamburg wheel tracking tests, presented in Figure 4, indicate a significantly higher rut rate for the A-E2 binder. For high volume roads, a Hamburg Wheel Tracking rut rate of less than 0.0005 mm/pass is generally considered to be acceptable (Williams and Prowell, 1999). The Hamburg wheel tracking test generally confirms the trend observed in the MMLS tests, that the A-E2 binder mixes are more rut susceptible than the A-P1 binder mixes. A comparison (figure 5) of the results from this project with other mixes tested under the Hamburg wheel tracking device, however showed a much increased rut resistance of both mixes when compared to other mixes. (Denneman, 2007).

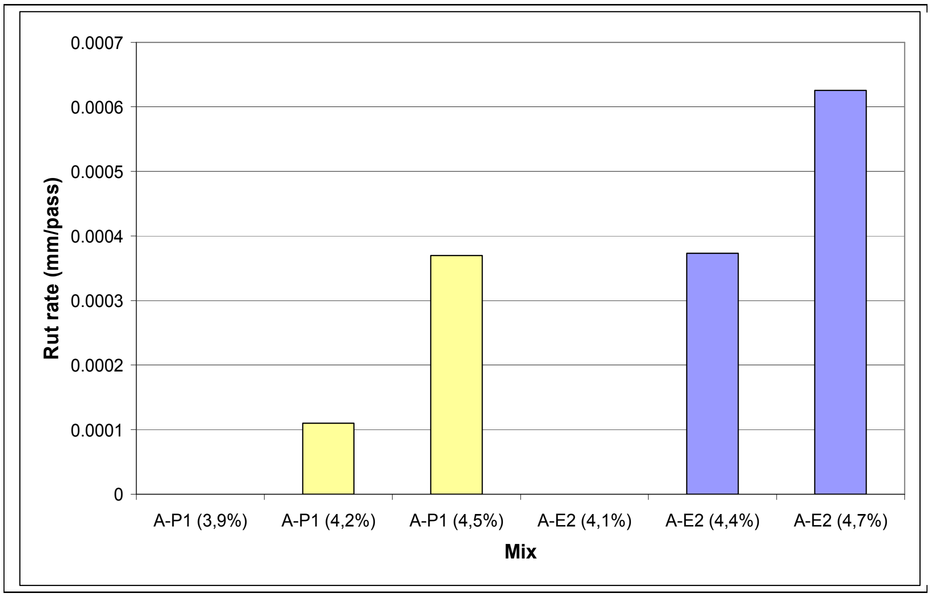

Figure 4: Result from Hamburg wheel tracking test

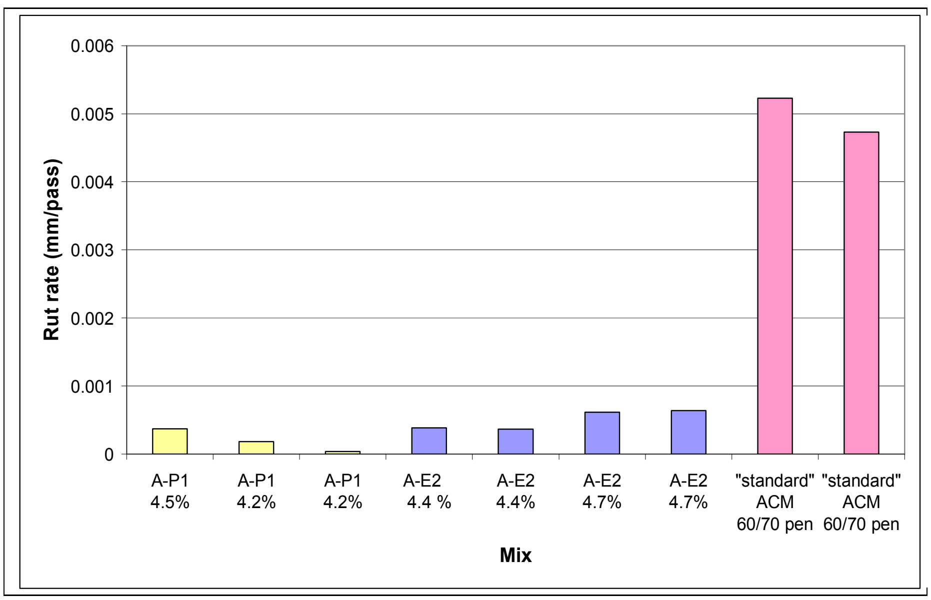

Figure 5: Comparison of Hamburg wheel tracking tests with other tested asphalt mixes.

7.5.2 Fatigue resistance

Coarse graded rut resistant mixes generally have poor fatigue properties (HMA, 2001), due to lower binder content and binder film thickness. Other factors influencing the fatigue cracking in asphalt include:

• Layer support

- Temperature
- Aging of binder
- Type of binder
- Wander
- Mix gradation

In order to assess the fatigue properties of the two mixes, two slabs of 1 m by 1m were cut and removed from the pavement and then cut up in beams. Flexural beam fatigue tests at low, intermediate and high strain levels were performed on the 4,2% A-P1 and 4,3% A-E2 mixes and the results are presented in Figure 6.

The fatigue requirements were evaluated against the interim guidelines for the design of Hot Mix Asphalt in South Africa. (HMA, 2001).

Both the mixes had very good fatigue properties at high (400 με) and intermediate (300 με) strain levels. At low stain (200 με) levels, the A-P1 binder mix slightly outperformed the A-E2 binder mix, but the fatigue resistance for both the mixes can still be regarded as acceptable.

It is however important to note that the fatigue results were based on a limited number of tests and that some scatter in the data were observed. Considering these limitations, the fatigue properties of both mixes were considered to be acceptable.

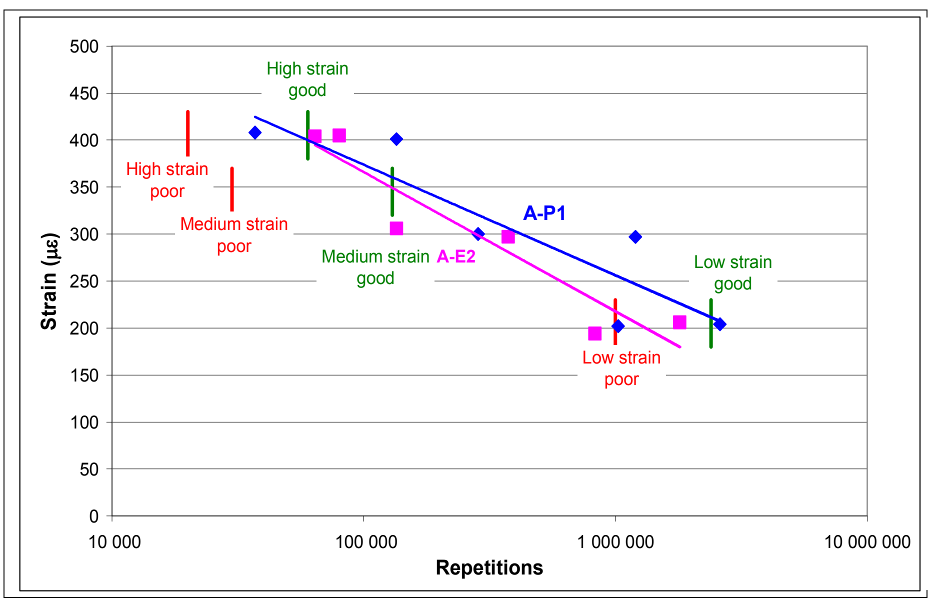

Figure 6: Result from repeated bending beam fatigue tests

By following this mix design and selection process, the final mix selected for this project was an A-P1 mix with a target binder content of 4.3% and target marshall void content of 4.8 %.

8 CONSTRUCTION

Specialised asphalt mixes often require particular attention to construction techniques in order to ensure that the structural and functional requirements of the asphalt mix, as considered in the design process, are met. It is often not possible to foresee specific construction techniques, that arise from the mix design process, when the project is priced or tendered. However an experienced contractor should be aware of this in the case of a heavy duty pavement.

On this particular project, specific requirements in terms of density and construction techniques were shared in a pre-construction workshop with the contractor. During this session the contractor accepted a voluntary higher lower compaction limit of 93.5% of Theoretical Maximum Relative Density (RICE) as well as to the continuous monitoring of the asphalt density during construction to achieve this target density. This was done by a mobile nuclear density meter, calibrated by the results from the daily core densities. By adopting this technique, no densities below the voluntary target was achieved and also no mixes were rejected based on compaction.

Deleted:

The daily production of the asphalt mix was monitored closely by daily marshall testing. All the results from the marshall testing were plotted and analysed daily to address any change in the mix volumetrics as soon as it was discovered. Parameters monitored included all sieve sizes, binder content, voids, stability, flow, film thickness, VMA, VFB, BRD and core densities. Figure 7 illustrates a typical daily monitoring graph of Binder content.

A total of almost 60 000 tonnes of asphalt was placed over a period of 12 months, with most of the work done at night. Peak production was in March 2008 with more than 1 000 tonnes placed in one shift.

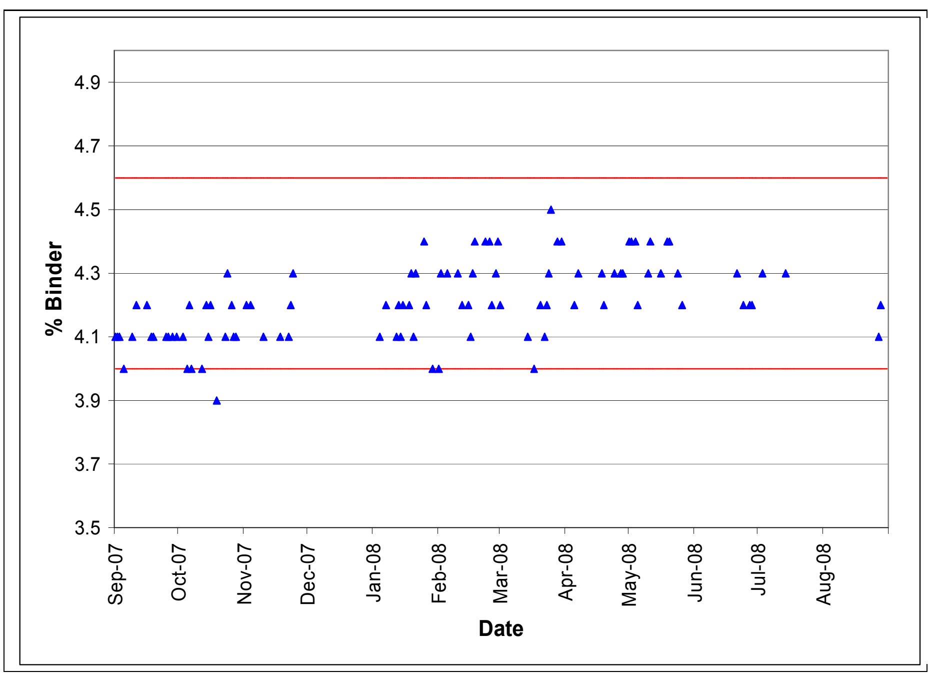

Figure 7: Typical daily Binder Content monitoring of production mixes during construction

The asphalt base was surfaced with an Ultra Thin Friction course across the width of the pavement which provided superior riding quality and adequate macro texture.

9 CONCLUSIONS

The rehabilitation of National Road 3 between Mariannhill and Key Ridge required an asphalt base layer that has been designed with a great degree of care and attention in order to achieve the design objectives defined in the structural pavement design. The normal single iteration asphalt mix design process was found not to be sufficient in addressing all the performance requirements of a heavy duty mix, and a detailed parallel mix design which considered two types of binders were performed. A detailed mix design process to optimally achieve the structural design requirements was developed for this project and the final mix selection relied substantially on the outcome of performance testing.

The developed mix design process resulted in the selection of an A-P1 binder (with 4% EVA) as the most appropriate mix for the environmental and traffic conditions of the pavement.

Carefully planned and executed quality assurance procedures, as well as a contractor that placed a high premium on delivering quality, resulted in no reworks or any substandard asphalt works

The most important aspect of the mix design process described in this paper is that it was essential to close the loop of the pavement design in the case of high performance pavements. The asphalt mix design process described above focussed on making sure the mix characteristics achieved in the field match the characteristics assumed during the pavement design process.

REFERENCES

Hot Mix Asphalt Design project (HMA), 2001, **Interim guidelines for the design of Hot-Mix Asphalt in South Africa,** Sanral, CSIR and Sabita, Pretoria, South Africa.

Asphalt Academy, 2007, **Technical Guideline TG1: The use of Modified Bituminous Binders in Road Construction, 2nd edition,** Asphalt Academy, Pretoria, South Africa.

Committee of Land Transport Officials (COLTO), 1998, **Standard Specifications for Road and Bridge Work**, South African Institute of Civil Engineers (SAICE), Midrand, South Africa.

Hugo, F., 2005, **DPG1 – Method for evaluation of permanent deformation and susceptibility to moisture damage of bituminous road paving mixtures using the Model Mobile Load Simulator (MMLS3)**, SANRAL Chair, Stellenbosch University, Stellenbosch, South Africa.

Pretorius F.J., Grobler J.E., Onraet J., 2004, **Development of a fit-for-purpose product performance specification system for asphalt mixes**, Proceedings of the 8th Conference on Asphalt Pavements for Southern Africa (CAPSA'04), 12 – 16 September 2004, Sun City, South Africa.

Williams R.C and and Prowell B.D., 1999, **Comparison of Laboratory Wheel Tracking Test Results with Wes Track Performance**, Transportation Research Board of the National Academies, Washington, US.

Denneman E, 2007, **Personal correspondence and communication**, Pretoria, South Africa.