EVALUATION OF THE PERFORMANCE OF AGGREGATE IN HOT-MIX ASPHALT

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ABSTRACT

The overall performance of an asphalt mix is dependent on, amongst others, the properties of the constituent materials, which include aggregate, binder and filler. The aggregate for production of asphalt mixes is usually sourced from a quarry, which is established through a long and expensive process. A quick evaluation of a new aggregate source will give some indication of its future performance as well as facilitate its introduction into the market place.

This paper presents a study of the comparative performance of two aggregates; a granite aggregate of known good performance and a relatively unknown quartzite aggregate containing up to 30% shale. The basic properties of the quartzite aggregate were assessed; following which, the performance of the aggregate in an asphalt mix was evaluated. The design grading of the asphalt mixes was similar; the only difference being that the coarse granite fractions (6.7 and 9.5 mm) of the asphalt mix of known good field performance were replaced by coarse quartzite fractions. The same binder type and crusher sand was used in both asphalt mixes, allowing for a comparative evaluation of the mixes in the laboratory. The asphalt performance-related tests conducted on the mixes included Repeated Simple Shear Test at Constant Height (rutting indicator), beam fatigue test (cracking indicator), dynamic modulus (stiffness indicator) and modified Lottman test (durability indicator). The test results were analysed statistically, to establish whether the performance of the asphalt mixes differed.

1 INTRODUCTION

Aggregates constitute the larger proportion of material used in the manufacture of Hot-Mix Asphalt (HMA). The aggregate, therefore, plays an important role in determining the overall performance of asphalt mixes in pavements. The aggregate properties required to ensure good performance are described in various pavement design manuals, guidelines and specification documents such as the Standard Specification for Road and Bridge Works for State Road Authorities (CSRA, 1998), the interim guideline for design of HMA (Taute et al., 2001), Draft TRH8: Design and use of hot-mix asphalt in pavements (DOT, 1987) and the US Superior Performing Asphalt Pavement (SUPERPAVE) (Asphalt Institute, 1996).

Aggregates used for the manufacture of HMA are usually sourced from a quarry, which is established through a long and expensive process. The newly established quarry is expected to produce consistent aggregate over a long period of time. The properties of aggregate produced by a specific source may, however, vary over time as different seams in the quarry are operated. A quick evaluation of the properties of aggregate from a new

seam or different potential source will provide an indication of the future performance of the aggregate.
In this paper, a comparative performance of two types of aggregate in asphalt mix is presented; a granite aggregate of known good performance and a relatively unknown quartzite aggregate containing up to 30% shale. The main objective of this study was to investigate whether the relatively unknown quartzite can be used to manufacture asphalt of acceptable performance. This was achieved by evaluating the fundamental properties of the quartzite, followed by evaluation of the performance of the quartzite in asphalt mix. Permanent deformation (rutting), fatigue cracking, stiffness and durability performances of an asphalt mix manufactured using the quartzite aggregate were compared with an asphalt mix manufactured using the granite aggregate.

2 PERFORMANCE OF AGGREGATE IN HOT MIX ASPHALT

Aggregates commonly used for the production of asphalt may be processed aggregates, natural aggregates or manufactured aggregates. Processed aggregates are obtained by quarrying and crushing any of the three primary rock types (igneous, sedimentary and metamorphic). Natural aggregates (i.e. gravel and sand) are naturally occurring deposits found on land, rivers or seabed. Manufactured aggregates are by-products of industrial processes (i.e. steel and chrome slag). Manufactured aggregate can also be obtained by crushing used asphalt to reclaim the aggregate (Reclaimed Asphalt (RA)). The properties of aggregate depend on many factors; these factors include (Taute et al., 2001 and Prowell et al., 2005);

- The mineralogy of the parent rock;
- The extent to which the parent rock has altered (i.e. leaching and oxidation), and
- The process required to produce aggregate particles (i.e. type of crusher being used).

Aggregate properties that are significant to the performance of HMA include; hardness/toughness (strength indicator), durability, shape and surface texture, absorption and cleanliness. Hard and rough textured aggregate results in stable and rut-resistant HMA mixes (Button et al., 1990; Taute et al., 2001). Durability is another key aspect that should be possessed by aggregates used in the production of asphalt. The aggregate should be able to resist breaking down and disintegration under environmental actions.

In terms of aggregate shape, equal-dimensional aggregate particles are preferred over flat and elongated aggregates. Flat and elongated aggregate particles tend to lock up (resist re-orientation) resulting in difficulties during compaction (Button et al, 1990; Arasan et al, 2011). Angular aggregate particles are preferred over round-shaped aggregates as they improve mechanical interlock, provide better resistance to permanent deformation (rutting) and improve resilient response of HMA mixes (Pan et al., 2005; Chen et al., 2005). Therefore, the properties of aggregate should be considered during their selection for HMA. Standard tests to evaluate the properties of aggregate include:

- Hardness, i.e. Aggregate Crushing Value (ACV) and Ten Percent Fines Aggregate Crushing Value (10% FACT);
- Durability i.e. Methylene Blue Adsorption;
- Shape properties i.e. Flakiness index, Average Least Dimension (ALD) and Polishing Stone Value (PSV);
- Absorption i.e. Water absorption;
- Apparent Relative Density (ARD), and

- Bulk Relative Density (BRD).

  Recent research work has shown that advanced techniques such as imaging and laser scanning can also be employed for accurate quantification of aggregate shape properties (Tutumluer et al., 2000; Komba et al., 2013 and Komba., 2013). Petrographic examination of thin sections of asphalt samples may also provide useful information regarding the performance of the aggregate.

3 MATERIALS, MIX DESIGN AND LABORATORY TESTING

3.1 Materials

3.1.1 Aggregate

Aggregate sampling was done according to the requirements of TMH 5: "*Sampling Methods for Road Construction Materials*". Granite aggregate was sourced from a commercial asphalt plant in Gauteng. The granite aggregate consisting of 9.5 mm, 6.7 mm and Crusher Sand, is used routinely in the production of a good performing medium continuously graded wearing course. G1 material, 9.5 mm and 6.7 mm fractions of quartzite were sampled from the new aggregate source investigated in this study. Comprehensive testing was undertaken to evaluate the properties of the quartzite aggregate.

3.1.2 Bitumen binder

50/70 penetration-grade binder was used to manufacture the asphalt samples. The bitumen was sourced from a commercial asphalt plant. Standard tests were performed to confirm the specification properties of the binder.

3.2 Mix design

A standard medium continuously graded asphalt mix was used in this study. The aggregate in the original asphalt mix was granite (9.5 mm, 6.7 mm and crusher sand). 9.5 mm and 6.7 mm fractions of quartzite substituted the granite in the second comparative mix. The quartzite factions were blended such that, the grading is similar to the granite asphalt mix containing granite aggregate. A summary of the volumetric properties of the granite asphalt mix is presented in Table 1. Figure 1 plots the grading of both the granite and quartzite aggregates.

Table 1: Summary of volumetric properties of the medium continuous mix

Mix property	Design value
Binder content (%)	4.7
Design air voids (%), saturation surface dry (SSD)	4.9
Volume of voids in mineral aggregate (VMA) (%)	14.9
Volume of voids filled with binder (VFB) (%)	68.0
Mixing temperature (°C)	150 - 160
Compaction temperature (°C)	135

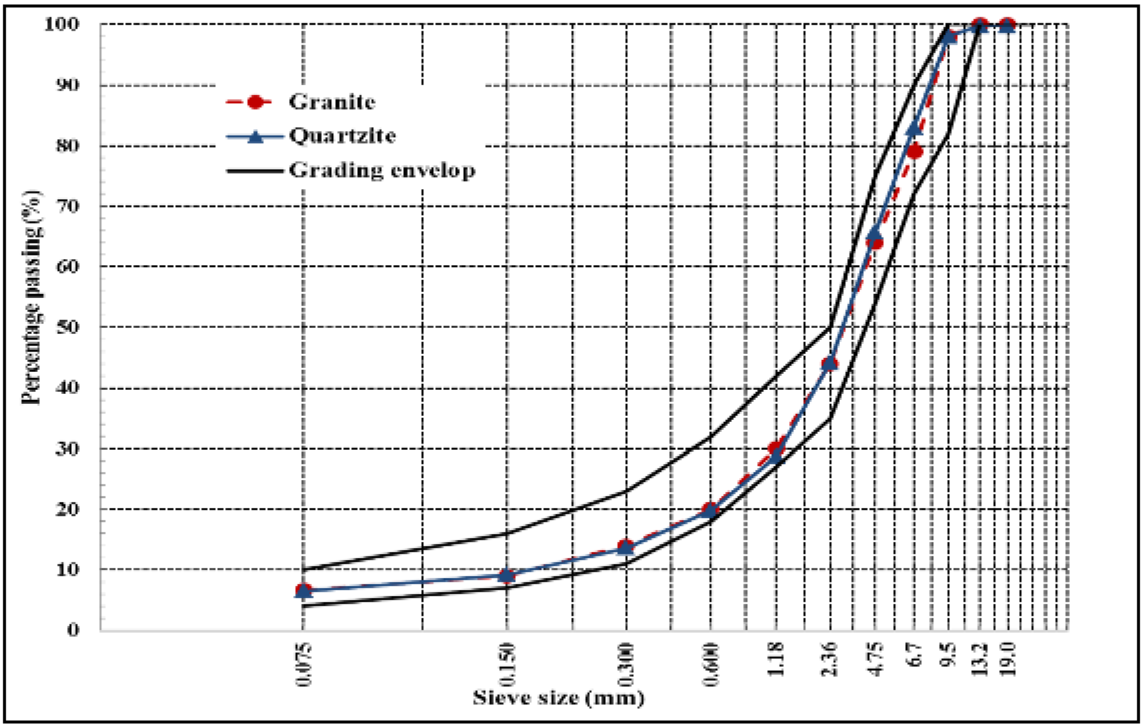

Figure 1: Aggregate grading

3.3 <u>Mechanical mixing and compaction of asphalt samples</u>

The mechanical mixing and compaction of the asphalt samples were done in accordance with CSIR's test protocols for testing asphalt mixes in South Africa (Anochie-Boateng et al., 2010). Calculated masses of aggregates were blended in accordance with the design grading and pre-heated to the required mixing temperature. A calculated mass of the bitumen binder and the pre-heated aggregate were placed into a pre-heated mechanical mixer. The materials were mixed until a uniform mixture was obtained (approximately 15 minutes). After mixing, the loose asphalt material was aged to simulate ageing that takes place during the normal production process in an asphalt plant and transport to site. The ageing of the loose asphalt material was done in accordance with Superpave short-term ageing procedures as described by Von Quintus et al., (1991), but slightly modified in the CSIR's test protocol (Anochie-Boateng et al., 2010). The ageing procedures require placing loose asphalt material into an oven set at compaction temperature for four hours before compaction.

Following the simulation of short-term ageing, asphalt samples were compacted by using a Transport Research Laboratory (TRL) slab compactor, Superpave gyratory compactor and Marshall compactor. The quantities of loose asphalt materials to be placed into the compaction moulds were calculated by using the maximum theoretical relative density (MTRD) of the mix, the volume of the mould and the voids required in the mix.

Compacted slabs were used to prepare specimens for fatigue testing and Repeated Simple Shear Test at Constant Height (RSST-CH). Gyratory compacted samples were used to prepare specimens for dynamic modulus testing. Marshall compacted briquettes were used for Modified Lottman testing. Samples for dynamic modulus, RSST-CH and Modified Lottman tests were compacted to field voids (approximately 7 %); whereas fatigue samples were compacted to design voids (approximately 4.9 %). The performances of the asphalt samples manufactured by using the two aggregate types (granite and quartzite) were then evaluated side by side.

4 RESULTS AND DISCUSSION

4.1 Aggregate test results

4.1.1 Basic properties of aggregate

The analysis performed at the CSIR's laboratory indicated that the quartzite aggregate contains 71% quartzite, 9% green shale and 20% grey shale (metavocanic). The percentages were determined based on the separation of 15.8 kg of material. Figure 2 shows examples of quartzite, green shale and grey shale materials. Depending on the quantity of the sample of each material type (quartzite, green shale and grey shale) obtained from separation of 15.8 kg of the material, various test were performed on bulk and individual samples. The tests included Average Least Dimension (ALD), Flakiness index, Aggregate Crushing Value (ACV), Ten percent Fines Aggregate Crushing Test (10% FACT), Aggregate Impact Value (AIV) (soaked in water, ethylene glycol and dry), apparent and bulk relative densities, water absorption and Polishing Stone Value (PSV). The results are presented in Table 2. Apart from flakiness index (the shale materials were more flaky), the properties of individual material types do not differ significantly, and the materials conform to the requirements of aggregates for asphalt.

Figure 2: Examples of quartzite aggregate (left), green shale (centre) and grey shale (right)

Table 2: Basic properties of quartzite aggregate

Property	Bulk sample	Quartzite	Grey shale	Green metavolcanic
Average Least Dimension (mm)	7.921	8.571	7.01	6.85
Flakiness index (%)	18.2	17.9	34.6	36.5
ACV (%)	14.2	N/D	N/D	N/D
10% FACT (kN)	255	N/D	N/D	N/D
AIV - Dry (%)	12.4	13.2	9.9	10.8
AIV – Soaked (%)	N/D	15.9	10.4	16.4
AIV soaked to dry ratio	N/D	1.2	1.05	1.52
ARD +4.75 mm	2.741	2.710	2.810	2.776
BRD +4.75 mm	2.717	2.690	2.833	2.802
Water Absorption +4.75 mm	0.3	0.3	0.3	0.3
ARD -4.75 mm	2.724	N/D	N/D	N/D
BRD -4.75 mm	2.642	N/D	N/D	N/D
Water Absorption -4.75 mm	1.1	N/D	N/D	N/D
Polished Stone Value	62.2	65.5	62.5	62.2

N/D: test not done due to insufficient material.

4.1.2 Results of thin sections

Visual observation of the laboratory prepared asphalt samples showed no defects. The aggregate (quartzite, green shale and grey shale) is well distributed with strong contact between the aggregate and binder. Thin sections (30 µm thick) of laboratory manufactured asphalt core were prepared by the Council for Geoscience (CGS) for further examination under a petrographic microscope. Figures 2a and 2b are thin sections showing shale (bottom of figure) and quartzite (top and right of figure). The black in the central area (Figure 2a) is bitumen showing good distribution along edges of the aggregate and within the fine matrix and clean contacts with particles of shale and quartzite. Figures 2c and 2d are thin sections showing a shale particle (left of figure) and quartzite (top right of figure). Black in the central area (Figure 2c) is bitumen showing a more dispersed nature among the fines but a less even distribution along the shale and quartzite particles. There is no evidence of absorption into any of the particles. Figures 2e and 2f are thin sections showing two shale particles ("grey shale" on left and "green shale" on right of figure) and quartzite (top centre of figure). Black in the central area (Figure 2e) is bitumen showing almost totally binder with good adhesion to shale and quartzite. No absorption into any of the particles is evident.

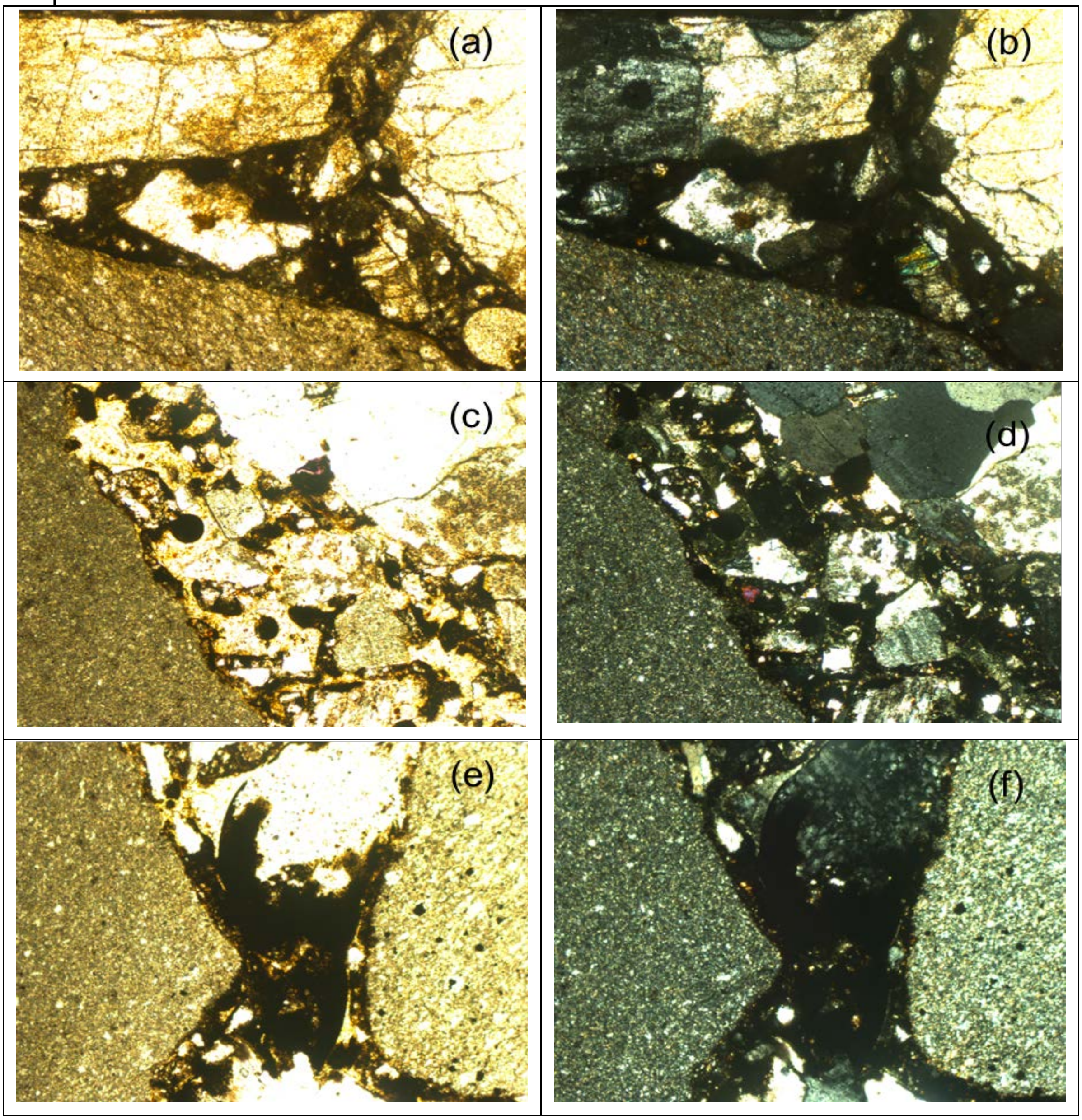

Figure 2: Results of thin sections

4.2 Binder test results

Standard specification tests were performed on the 50/70 penetration grade bitumen. The results are presented in Table 3. The binder recovered from short-term aged asphalt was also tested to determine binder content and softening point. This was necessary to ensure that the properties of the in-situ binder are similar for both mixes, so that bitumen does not introduce variability in the performance of the two asphalt samples. The test results of recovered binder are presented in Table 4. The results indicate that there is no significant difference in softening point (stiffness indicator) of binder recovered from the two asphalt mixes. Binder stiffness would, therefore play an insignificant role in the difference between the asphalt properties.

Table 3: Results of the standard binder tests

Property	Test result	Test method
Original binder		
Penetration @25°C (10^{-1}mm)	56	EN 1426
Softening Point, R&B (°C)	49.2	ASTM D36
Viscosity @ 135°C (Pa.s)	0.403	ASTM D4402
After RTFOT (Rolling thin film oven treatment) Ageing		
Mass Change (% m/m)	+0.08	ASTM D2872
Viscosity @ 135°C (Pa.s)	0.528	ASTM D4402
Softening Point, R&B (°C)	53.8	ASTM D36
Softening Point Increase (°C)	4.6	

Table 4: Results of the binder recovery from short-term aged laboratory mix

Property	Granite mix	Quartzite mix	Test method
Binder Content (%)	4.7	4.9	BE-TM-BINDER-1
Softening Point,	56.6	55.4	ASTM D36

4.3 Asphalt performance test results

4.3.1 Permanent deformation test results

The Repeated Simple Shear Test at Constant Height (RSST-CH) gives an indication of the resistance of an asphalt mix to permanent deformation (rutting). The RSST-CH tests were performed on specimens prepared from laboratory compacted slabs (150 mm diameter x 60 mm high) at three different temperatures (25, 40 and 55°C). For each of these temperatures, three replicate specimens were tested. The tests were performed in accordance with procedures contained in AASHTO 320-03 (2007) standard test method with certain alterations and improvements by Denneman (2009).

Figure 3 shows the average RSST-CH test results for the most repeatable specimens (permanent deformation plotted against number of load cycles). At 25 and 40°C the asphalt mix manufactured using granite aggregate had better resistance against permanent deformation, whereas at 55°C the asphalt mix manufactured using the quartzite aggregate had better permanent deformation. The student t-Test was applied to the permanent strain data set to assess whether there is a significant difference between the permanent deformation behaviour of the two asphalt mixes. The statistical analysis results are presented in Table 5. Although the plots of average RSST-CH test results in Figure 2 show differences in the behaviour of the two asphalt mixes, the statistical analysis results indicates that the difference in the behaviour of the two mixes is not statistically significant.

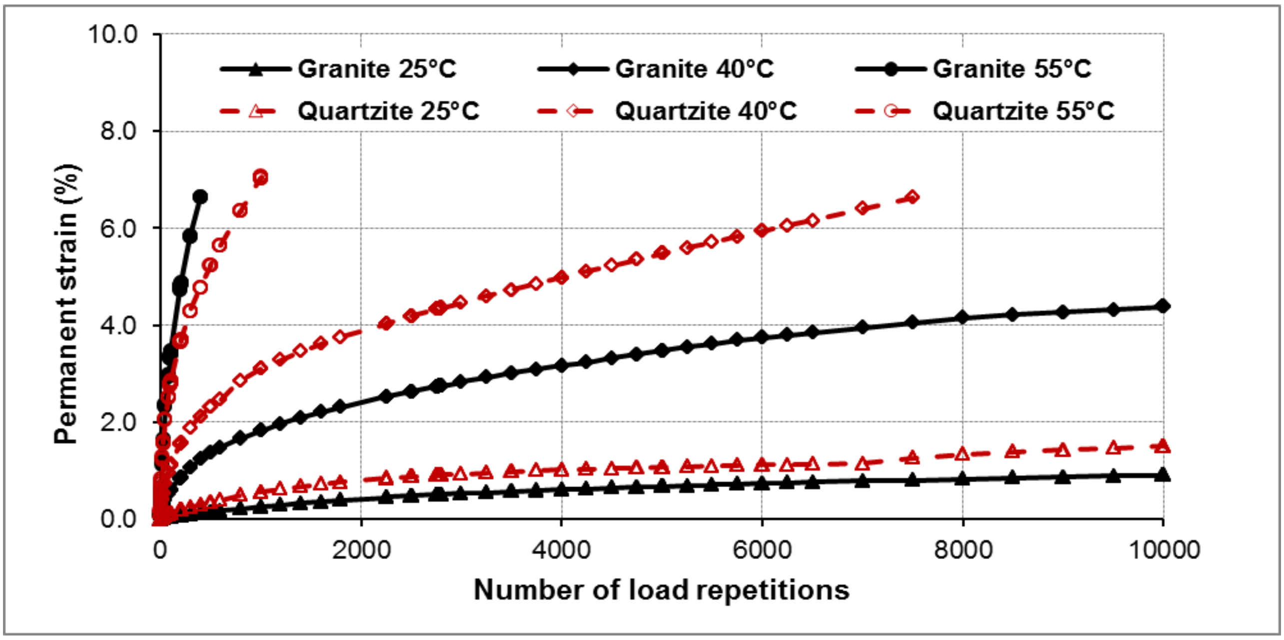

Figure 3: RSST-CH test results

Table 5: Statistical analysis of RSST-CH test results

Temperature (°C)	No. of Cycles	Permanent strain (%): Granite mix		Permanent strain (%): Quartzite mix		t-Test	
		Mean	STDEV	Mean	STDEV	P(T<=t) one-tail	Significant difference
25	2250	0.45	0.20	0.85	0.52	0.246	no
	4000	0.61	0.26	1.02	0.70	0.288	no
	6000	0.73	0.31	1.12	0.82	0.320	no
	8000	0.83	0.35	1.34	0.69	0.257	no
	10000	0.91	0.38	1.51	0.58	0.162	no
40	1600	2.21	1.10	3.61	0.54	0.077	no
	3000	2.83	1.23	4.46	0.71	0.078	no
	4500	3.32	1.41	5.23	0.91	0.081	no
	6000	3.74	1.68	5.94	1.01	0.083	no
	7500	4.05	1.89	6.63	1.13	0.076	no
55	400	6.64	1.22	4.78	0.56	0.052	no

4.3.2 Fatigue test results

Four-point beam fatigue tests were carried out on prismatic beam specimens of dimensions 400 x 63 x 50mm prepared from laboratory compacted slabs. The four- point beam fatigue test gives an indication of the resistance of an asphalt mix to fatigue cracking. The tests were conducted at three strain levels (200, 300 and 400 µε), and at a frequency of 10 Hz at 10°C. The tests were performed in accordance with the protocol developed by Anochie-Boateng et al., (2010), which follows procedures in AASHTO T 321(2007), with some modifications. For each strain level, three replicate specimens were tested. The conventional failure criterion which is defined as the number of load cycles to reach 50% reduction in the initial stiffness was adopted.

Figure 4 shows plots of the strain versus number of load cycles to failure. At the lower strain amplitude level (200 με), the fatigue life of the asphalt mix manufactured using granite aggregate is slightly higher than that of asphalt mix manufactured using quartzite aggregate. The student t-Test was applied to the fatigue data set to further assess whether there is a significant difference between the fatigue results of the two asphalt mixes. The statistical analysis results are presented in Table 6. The statistical analysis results indicated that there is no significant difference between the fatigue results of the two mixes.

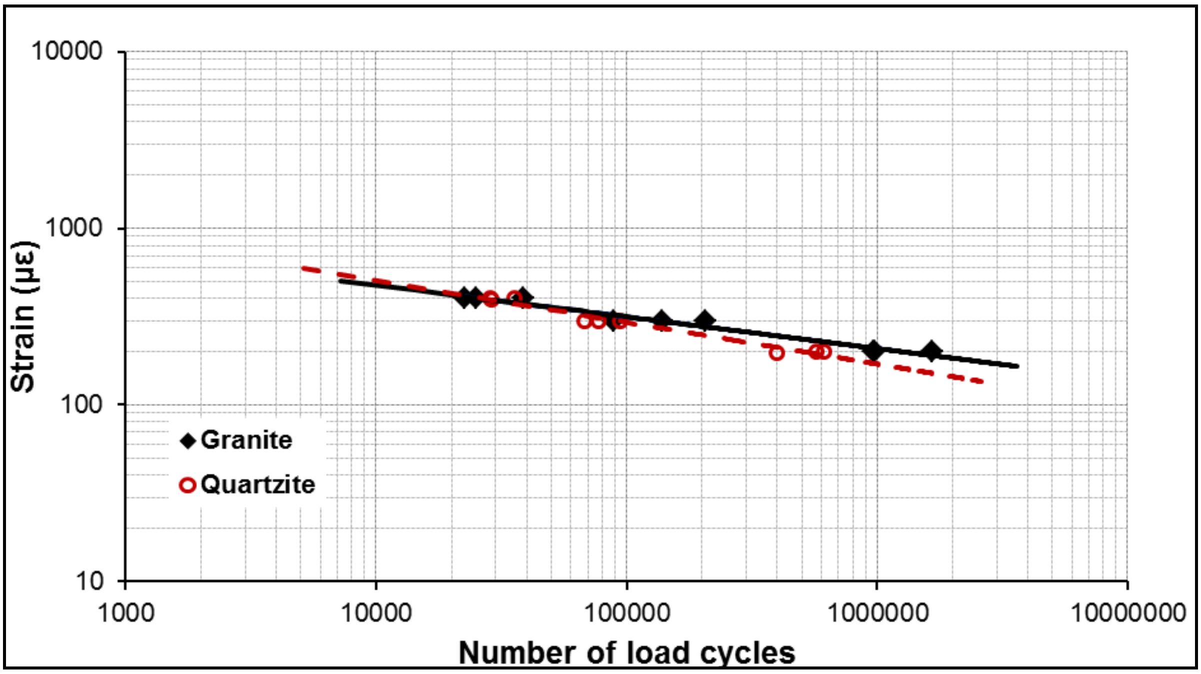

Figure 4: Strain versus number of load cycles at 10°C

Table 6: Statistical analysis of fatigue results

Strain level (με)	Number of load cycles				P(T<=t) one-tail	Significant difference
	Granite mix		Quartzite mix			
	Mean	STDEV	Mean	STDEV		
200	1 201	389 077	528 013	115 656	0.073	no
300	144 813	58 840	79 767	13 526	0.119	no
400	28 827	8 637	31 000	3 886	0.386	no

4.3.3 Dynamic modulus test results

The dynamic modulus gives an indication of the resilient response of asphalt mixes. A Universal Testing Machine (UTM-25) device available at the CSIR's pavement material laboratory was used to conduct the dynamic modulus tests. The tests were performed on specimens prepared from gyratory compacted asphalt samples (100 mm diameter x 150 mm high). The dynamic modulus tests were performed in accordance with protocols developed by Anochie-Boateng et al., (2010), and presented by Maina and Anochie-Boateng, (2010). The tests were performed at five temperatures (-5, 5, 20, 40 and 55°C) and six loading frequencies (25, 10, 5, 1, 0.5 and 0.1 Hz). For each asphalt mix, three specimens were tested.

Table 7 presents the summary of the dynamic modulus test results. The Student t-Test was applied to the two sets of dynamic modulus results to determine if there is a significant difference between the mean values. The statistical analysis results are presented in Table 7. The statistical analysis results indicate that the only statistically significant difference between the mean values is at the higher frequencies at 20°C and at the lower frequencies at 5°C. The majority of instances are found to be not statistically different. Overall, the resilience response of the two asphalt mixes does not differ significantly.

Table 7: Dynamic modulus results

Temperature (°C)	Frequency (Hz)	Granite				Quartzite				t-Test	
		Dynamic modulus of specimens (MPa)			Mean (MPa)	Dynamic modulus of specimens (MPa)			Mean (MPa)	P(T<=t) one-tail	Significant difference
		C1	C2	C3		C1	C2	C3			
-5	25	25515	30255	27385	27718	26805	26908	28230	27314	0.405	no
	10	24852	28994	26361	26736	25518	25728	27153	26133	0.348	no
	5	24208	27925	25539	25891	24436	24758	26239	25144	0.301	no
	1	22157	25121	23274	23517	21690	22247	23832	22590	0.229	no
	0.5	21321	23711	22171	22401	20365	21033	22645	21348	0.184	no
	0.1	18832	20202	19405	19480	17234	18100	19674	18336	0.127	no
5	25	23227	25398	23841	24155	23034	22718	23309	23020	0.141	no
	10	22251	23616	22569	22812	21439	21155	21870	21488	0.073	no
	5	21421	22149	21441	21670	20126	19846	20636	20203	0.040	yes
	1	18800	18588	18550	18646	16892	16692	17551	17045	0.017	yes
	0.5	17632	16858	17160	17217	15406	15228	16086	15573	0.019	yes
	0.1	14336	12955	13836	13709	12018	11837	12701	12185	0.031	yes
20	25	14463	14260	14936	14553	13766	13144	13663	13524	0.013	yes
	10	12794	11962	12880	12545	11634	11146	11635	11472	0.007	yes
	5	11384	10286	11317	10996	10116	9667	10137	9973	0.019	yes
	1	8091	6728	8005	7608	6843	6555	6942	6780	0.065	no
	0.5	6820	5394	6739	6318	5628	5369	5741	5579	0.089	no
	0.1	4044	2835	4151	3677	3177	3013	3301	3164	0.138	no
40	25	3534	2643	3444	3207	2673	2796	2507	2659	0.129	no
	10	2408	1636	2369	2138	1715	1788	1600	1701	0.139	no
	5	1717	1083	1702	1501	1178	1232	1093	1168	0.151	no
	1	698	386	699	594	450	485	415	450	0.179	no
	0.5	478	258	478	405	309	345	286	313	0.207	no
	0.1	209	117	206	177	135	162	132	143	0.236	no
55	25	1096	661	1000	919	919	747	717	794	0.187	no
	10	643	386	589	539	574	474	431	493	0.291	no
	5	448	278	407	377	416	355	318	363	0.397	no
	1	207	141	179	176	231	194	152	192	0.276	no
	0.5	179	137	154	157	212	190	149	184	0.126	no
	0.1	147	131	122	133	180	183	148	170	0.020	yes

4.3.4 Modified Lottman test results

The Modified Lottman test gives an indication of the durability of an asphalt mix in terms of resistance to moisture damage. Moisture resistance of the asphalt samples was tested in accordance with ASTM D 4867M. The test relies on Indirect Tensile Strength (ITS) measurements taken before and after conditioning asphalt samples by freeze-thaw cycles. The ratio of the indirect tensile strengths of the conditioned and unconditioned specimens which is referred to as the tensile strength ratio (TSR) is used to get an indication of the resistance of the asphalt to moisture damage.

Table 7 shows the Modified Lottman test results. Both mixes appear to have good resistance to moisture damage (i.e. TSR greater than 0.8). However, the ITS values of the asphalt samples manufactured using the quartzite are relatively higher than those manufacture using the granite aggregate. This may be due to the slightly more flaky shale particles in the former.

Table 7: Modified Lottman results

Granite samples					
Treated Briquettes			**Dry Subset**		
Void (%)/Saturation level (%)			Void (%)		
6.8/69.6	6.8/68.3	6.7/69.6	7.5	6.3	6.2
ITS (kN)			ITS (kN)		
1113	1178	1024	1252	1146	1290
Average ITS = 1105			Average ITS = 1230		
TSR = 0.90					
Quartzite samples					
Void (%)/Saturation level (%)			Void (%)		
6.1/62.1	6.6/62.5	7.3/65.3	7.2	6.5	6.5
ITS (kN)			ITS (kN)		
1434	1451	1395	1591	1440	1465
Average ITS = 1427			Average ITS = 1499		
TSR = 0.95					

5 CONCLUSIONS AND RECOMMENDATIONS

This paper presents the results of the comparative performance of two aggregates in a standard asphalt mix; a granite aggregate of known good performance and a relatively unknown quartzite aggregate containing up to 30% shale. Based on the results contained in this paper, the following conclusions and recommendations can be drawn:

- The properties of the quartzite aggregate are satisfactory;
- Statistical analysis applied to the set of asphalt performance test results indicates that the performance of the asphalt mixes manufactured using the granite and quartzite aggregates do not differ significantly, and
- It is recommended that further testing including traffic loading and field performance monitoring be carried out to confirm these findings and to increase the confidence with which the future performance of the quartzite aggregate can be predicted.

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SESSION 1C: TRAFFIC MANAGEMENT, SAFETY AND SECURITY

SESSION 1D: RURAL TRANSPORT/ CAPACITY BUILDING

SESSION 2A: PUBLIC TRANSPORT PLANNING AND REGULATION

SESSION 2B: INFRASTRUCTURE

SESSION 2C: TRAFFIC MANAGEMENT, SAFETY AND SECURITY

SESSION 2D: FREIGHT AND LOGISTICS

SESSION 3A: TRAFFIC ENGINEERING

SESSION 3B: TRANSPORT SYSTEMS AND OPERATIONS

SESSION 4A: IPTN (IMPLEMENTING PUBLIC TRANSPORT NETWORKS) WORKSHOP

STUDENT ESSAY