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# The identification and treatment of poor durability Karoo dolerite base course aggregate – evidence from case studies

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The Karoo Supergroup covers approximately 57% of South Africa's surface area and the sedimentary rocks therein generally do not yield acceptable pavement aggregates. The Karoo Dolerite Suite (intrusions) present in these sedimentary units have successfully been used as pavement aggregate sources, but numerous cases of premature pavement failure due to alleged rapid degradation of the dolerite have been reported. Durability tests are included in basic or mafic igneous rock aggregate specifications, but rapid pavement failures continue to occur. A study was recently undertaken to identify cases where degradation of Karoo dolerite was the cause of pavement failure. A secondary objective of the study was to determine if any observed degradation could have been identified using currently specified or alternative testing methods. Three such case study sites are presented in this article and the properties of their materials compared to those from five non-problematic dolerite materials.

It is shown that the poor performance of the case study materials was likely due to the poor durability of the materials, manifesting as a reduction in resistance to abrasion and attrition. The identification of the observed poor durability could not have been performed accurately using only the currently specified test specifications. Alternative tests that allow an accurate differentiation to be made were, however, identified and, based on the results, tentative limits set. Additionally it was shown that modification of problematic Karoo dolerite base course materials, by applying lime at a rate less than the initial consumption of lime, can be successful in preventing further rapid pavement failures.

## INTRODUCTION

The main basin of the Karoo Supergroup in South Africa covers approximately 700 000 km$^2$ (57%) of South Africa's surface area and consists predominantly of a flysch-molasse succession which has a maximum cumulative thickness of ~12 km (Johnson *et al* 2006) (Figure 1). The sedimentary rocks of the Karoo Supergroup typically do not yield acceptable pavement aggregates due to the argillaceous nature of the lower (flysch) units and the relatively poor strength arenaceous (molasse) upper units. There is, however, an extensive network of dolerite intrusions which represent the shallow feeder system to the Drakensberg flood basalt eruptions (183 ± 1 Ma) (Duncan & Marsh 2006) which erupted at the end of the Karoo sedimentary succession deposition. These intrusions are collectively called the Karoo Dolerite Suite and have been widely and successfully used as pavement aggregate sources.

There are, however, numerous cases in which dolerite from the Karoo Dolerite Suite has been credited as the cause of premature pavement failure due to alleged rapid degradation of dolerite base course aggregate while in service. Road authorities have therefore included various so-called durability tests in aggregate specifications for basic or mafic igneous rocks in an attempt to prevent such premature failures. Despite this, rapid pavement failures continue to occur and the rapid degradation of Karoo dolerite continues to be blamed for many of the failures. Such failures ultimately result in significant costs related to reconstruction, alternative material investigations, material modification/stabilisation and project delays.

A study with the objective of identifying cases where degradation of Karoo dolerite was the cause of pavement failure was recently undertaken. The secondary objective was to determine if any observed degradation could have been identified using currently specified or alternative testing methods. Three case studies of such sites are presented and the properties of the materials from these sites are compared to those from five other "reference"



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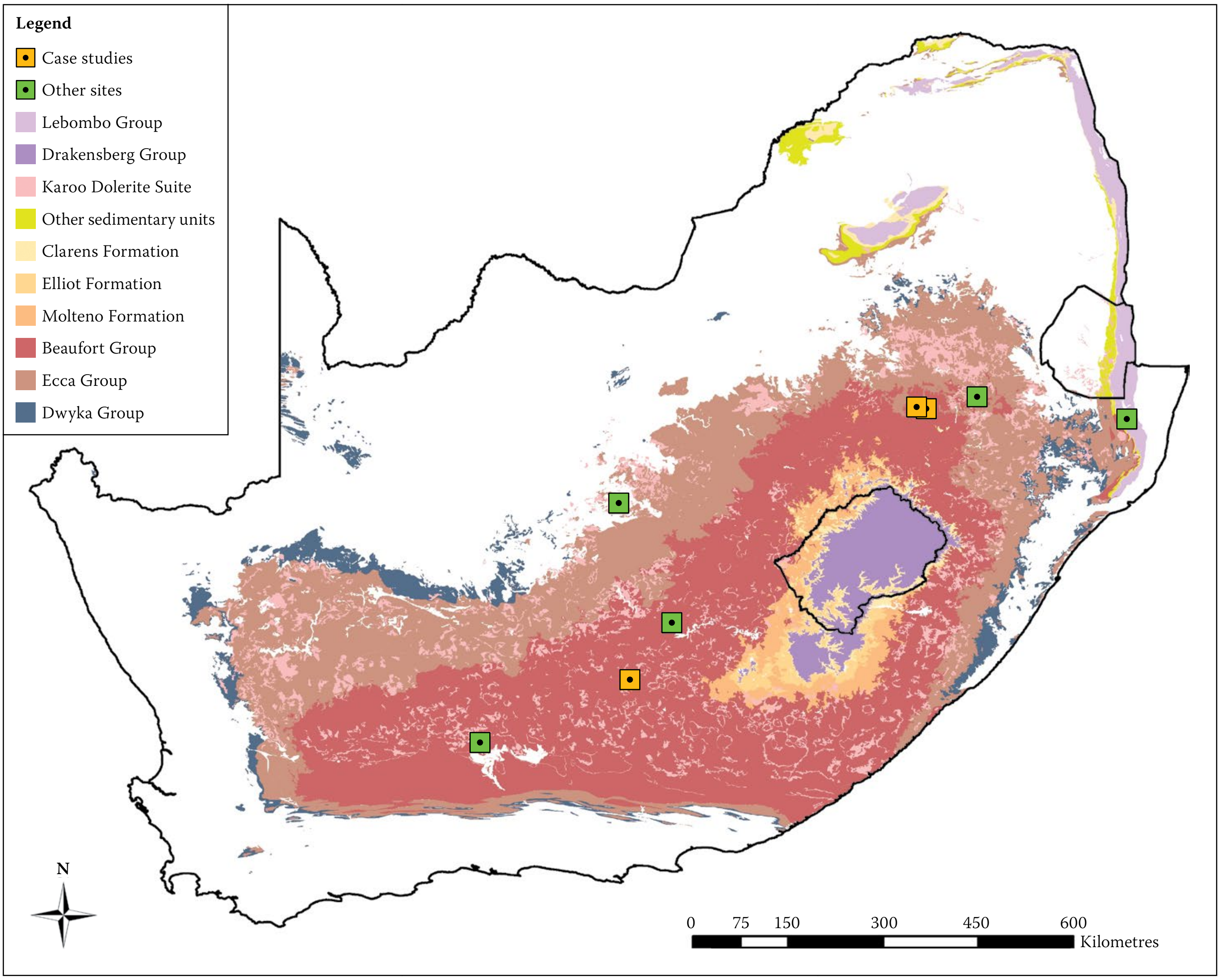


**Figure 1** Distribution of the Karoo Supergroup and study sites in South Africa, Lesotho and Swaziland

Karoo dolerite pavements where no material degradation was reported or suspected. Additionally, the treatment of material at one of the study sites is discussed.

## REVIEW

Research on the durability of Karoo dolerite aggregates has been performed by numerous authors since the middle of the previous century (e.g. Bell & Jermy 2000; Dunlevey & Stephens 1996; Hall & Harris 1985; Kleyn *et al* 2009; Orr 1979; Walker & Poldervaart 1949), and based on the findings of such studies additional specifications for basic igneous rock base course aggregates have been added to the current South African standard specifications. The currently accepted published specifications for pavement materials in South Africa (COLTO 1998) include limits for material parameters related to poor durability, and for tests designed to simulate potential degradation and associated changes in material properties (Table 1). Although some South African road authorities do include additional pro forma specifications, these are not

**Table 1** COLTO (1998) specifications relate to basic igneous rock durability

| Parameter | Specification | Motivation |
|---|---|---|
| 10% FACT | 110 kN | Identify materials in which strength has degraded. |
| Wet/dry 10% FACT ratio | ≥ 75% | Accelerate potential degradation of strength. |
| ACV | < 29% | Identify materials in which strength has degraded. |
| PI | Individual ≤ 5 | Identify materials in which fine fractions are of unsuitable nature (possibly due to unfavourable degradation). |
| | Average ≤ 4 | |
| | ≤ 12* | |
| LS | ≤ 2.0 | Identify materials in which fine fractions are of unsuitable nature (possibly due to unfavourable degradation). |
| DMI | ≤ 125 | Accelerate potential degradation of strength and identify materials with poor resistance to attrition and abrasion. |

10% FACT = 10% Fines aggregate crushing test, ACV = Aggregate crushing value, PI = Plasticity index, LS = Linear shrinkage, DMI = Durability mill index
*When PI is determined on –0.075 mm fraction because –0.425 mm fraction is non-plastic

available in published form (e.g. SANRAL's standard pro forma amendments to the COLTO 1998 specifications).

Where materials available in the vicinity of a pavement construction site are considered marginal with respect to these specifications, an alternative to hauling acceptable material long distances is the stabilisation of the materials using lime or cement (Netterberg 1994). The additional cost of stabilising suspected poor durability basic igneous materials is, however, excessive and results in rigid pavement layers which require design adaptations. Alternatively marginal materials may be treated by adding lime at a rate significantly lower than what is typically used in stabilisation (Wienert 1980; Kleyn & Berg 2008; Kleyn *et al* 2009). This process, known as lime modification, does not increase the strength of the material, but is employed to reduce the PI of the material and maintain a high pH to prevent further mineralogical degradation (Clauss 1967; Weinert 1980). The differences between soil modification and stabilisation reactions are discussed in further detail by Ballantine and Rossouw (1989), and the Committee of State Road Authorities (CSRA 1986). The procedures followed to perform lime stabilisation are, however, not standardised and, as shown by Netterberg (2004), if the PI of the lime used is high, the PI of the material may increase, thus negating the desired effect. The permanency of the effects of such modifications to basic igneous rock materials has also not been investigated.

The material at one of the case study sites was modified by adding 1% lime after the initial failures were noticed, and following this no similar failures were observed at that site.

## CASE STUDY SITES AND METHODOLOGY

The study sites were widespread across South Africa and the Karoo Main Basin,


**Figure 2** Examples of premature failures at case study sites: (a) general surface failure, (b) rutting and aggregate loss in wheel tracks, (c) bleeding in wheel tracks, (d) bleeding in isolated areas

**Table 2** Case study pavement properties

| Study site number | Surface | Base | Subbase | Observed defects |
|---|---|---|---|---|
| 1 | 19/6.7 mm double seal | 150 mm dolerite G2 | 275 mm weathered dolerite C4 | Localised bleeding and stone loss with minor rutting |
| 2 | Slurry seal | 150 mm dolerite G1 | 300 mm weathered dolerite C3 | Crocodile cracking spreading rapidly and causing widespread ravelling |
| 3 | 19/9.5 double seal | 150 mm dolerite G1 | 2.5% lime-stabilised weathered dolerite | Widespread bleeding with associated rutting |

**Table 3** Tests performed on samples

| Test | Method | Notes |
|---|---|---|
| Petrographic examinations | N/A | Thin section analysis including point counting using a PETROG digital stepping stage and PetrogLite™ software |
| Aggregate impact value (AIV) | British Standard BS812: Part 3 (BS 1975) | Performed on dry material and material soaked in ethylene glycol (not water) for 24 hours |
| Durability mill index (DMI) | South African National Standard SANS 3001-AG16 (SANS 2013a) | Performed on dry material and material soaked in ethylene glycol for five days |
| Compressive and shear wave velocity | ISRM (1978) standard method | Performed on quarry sample cores only |
| Point load index (PLI) | ISRM (1985) method | – |
| Modified ethylene glycol durability index (mEGDI) | South African National Standard SANS 3001-AG14:2013 (SANS 2013b) | – |
| Water absorption | ASTM C 97–02 method (ASTM 2002) | Performed on quarry sample cores only |

**Table 4** Results summary

| Sample | AIV (%) | | 10% FACT* | | | DMI | | | Velocity (m/s) | | PLI (MPa) | WA (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Std | Gly | Std (kN) | Gly (kN) | Glycol/ dry ratio (%) | Std | Gly | Mod Gly | P wave | S wave | | |
| 1.1-B | 10 | 14 | 425 | 305 | 72 | 66 | 87 | 5.2 | | | 14.1 | |
| 1.2-B | 10 | 16 | 409 | 272 | 66 | 89 | 123 | 6.8 | | | 12.7 | |
| 1.3-Q | 9 | 11 | 430 | 377 | 88 | 28 | 49 | 7.7 | 5 963 | 3 699 | | 0.14 |
| 2.1-B | 14 | 24 | 312 | 154 | 49 | 144 | 227 | 8.9 | | | 10.5 | |
| 2.2-B | 13 | 25 | 342 | 145 | 42 | 157 | 265 | 24.5 | | | 11.9 | |
| 2.3-Q | 14 | 30 | 314 | 105 | 33 | 43 | 103 | 24.8 | 5 426 | 3 047 | 8.9 | 0.31 |
| 2.4-Q | 12 | 12 | 370 | 361 | 98 | 63 | 74 | 10.8 | 5 450 | 3 284 | 10.3 | 0.24 |
| 3.1-B | 14 | 29 | 321 | 111 | 35 | 166 | 263 | 48.9 | | | 11.8 | |
| 3.2-Q | 12 | 18 | 369 | 242 | 66 | 39 | 111 | 37.2 | 6 091 | 3 518 | 11.6 | 0.19 |
| 3.3-Q | 11 | 16 | 372 | 270 | 72 | 41 | 77 | 12.9 | 6 439 | 3 611 | 10.1 | 0.18 |
| 4.1-B | 10 | 10 | 409 | 422 | 103 | 37 | 51 | 9.0 | | | 12.6 | |
| 4.2-B | 9 | 9 | 454 | 432 | 95 | 32 | 47 | 3.9 | | | 12.2 | |
| 4.3-Q | 8 | 11 | 469 | 378 | 81 | 31 | 37 | 1.8 | 6 530 | 3 893 | 10.3 | 0.09 |
| 4.4-Q | 7 | 9 | 494 | 456 | 92 | 35 | 39 | 1.5 | 6 757 | 3 897 | 14.6 | 0.07 |
| 5.1-B | 12 | 9 | 371 | 449 | 121 | 56 | 79 | 5.3 | | | 11.4 | |
| 5.2-B | 9 | 10 | 428 | 425 | 99 | 70 | 80 | 2.3 | | | 13.7 | |
| 5.3-Q | 8 | 7 | 473 | 491 | 104 | 40 | 43 | 2.1 | 6 581 | 3 996 | 15.8 | 0.16 |
| 6.1-B | 7 | 7 | 493 | 492 | 100 | 74 | 82 | 2.5 | | | 15.5 | |
| 6.2-B | 8 | 7 | 471 | 494 | 105 | 68 | 98 | 5.3 | | | 17.3 | |
| 6.3-Q | 10 | 9 | 414 | 434 | 105 | 59 | 73 | 2.0 | 6 565 | 4 086 | 15.9 | |
| 6.4-Q | 9 | 9 | 431 | 432 | 100 | 47 | 70 | 5.7 | 6 582 | 3 961 | 14.6 | 0.10 |
| 7.1-B | 8 | 7 | 481 | 510 | 106 | 51 | 49 | 1.9 | | | 12.9 | |
| 7.2-B | 7 | 7 | 494 | 523 | 106 | 56 | 56 | 0.1 | | | 12.8 | |
| 7.3-Q | 11 | 10 | 397 | 417 | 105 | 28 | 41 | 3.9 | 5 960 | 3 879 | 13.7 | 0.11 |
| 7.4-Q | 8 | 8 | 477 | 464 | 97 | 28 | 38 | 1.6 | 6 440 | 4 113 | 15.8 | 0.07 |
| 8.1-B | 14 | 23 | 323 | 168 | 52 | 37 | 54 | 5.8 | | | 9.3 | |
| 8.2-B | 13 | 25 | 338 | 151 | 45 | 42 | 72 | 5.7 | | | 8.5 | |
| 8.3-Q | 10 | 12 | 398 | 350 | 88 | 37 | 46 | 7.0 | 6 540 | 4 129 | 11.4 | 0.08 |
| 8.4-Q | 11 | 12 | 378 | 362 | 96 | 45 | 54 | 4.1 | 6 639 | 4 217 | 11.3 | 0.06 |

*Determined using Equation 1
Sample number index: B = base sample, Q = quarry sample, Std = Standard test procedure, Gly = Samples soaked in ethylene glycol, Mod = Modified, WA = Water absorption

and one site was located in the Lebombo Group (Figure 1). The climatic N-value (Weinert 1980) of the sites varied from 3 to 9. The pavement layer materials are summarised in Table 2, and the observed defects at these sites included rutting and aggregate loss in wheel tracks, bleeding in wheel tracks over extended areas, bleeding in isolated areas and total surface failure (Figure 2). Material was sampled from the base layer of each pavement and from the original material source quarry. Since the three sites at which rapid pavement failures were observed experienced failure while sections of the road were still under construction, the respective material sources were easily accessible for sampling. The reference sites were of various ages and the source quarry of each had to be identified from construction records.

Currently specified (COLTO 1998) aggregate characterisation tests, as well as additional tests commonly included in pro forma amendments to the COLTO (1998) specifications, were then performed on all samples and the resultant data analysed. The aim was to determine if the rapid failure site materials had undergone any, or unusually high amounts of, degradation compared with materials from the reference sites. The tests performed are listed in Table 3.

Although petrographic examinations are not formally incorporated in the quantitative specifications, a qualitative classification of "shall not contain deleterious material such as weathered rock" is required by COLTO (1998), and the examinations were therefore included. The results and interpretations of the modified Ethylene Glycol Durability Index (mEGDI) tests and petrographic investigations are presented in Leyland *et al* (2013), Leyland (2014a) and Leyland (2014b) and are not discussed further here. The problematic quantification of expansive clay minerals in dolerite aggregates is discussed by Leyland *et al* (2014), and for this reason smectite clay minerals are not included in this discussion.

Because the limited sample size of materials obtained from the base layers was not sufficient for 10% FACT testing, the AIV result was used to estimate the 10% FACT using the relationship published by Sampson and Roux (1982) (Equation 1). The PI and LS values were determined during Durability Mill Index (DMI) testing procedures.

$$10\%\ FACT = 10^{(2.915-(0.03.AIV))} \quad (1)$$

At the site where lime modification was performed, samples were obtained from the base layer directly after construction and at irregular intervals over a period of two years. The pH and PI of the samples were determined to observe the immediate and long-term effects of the modification process.

# RESULTS AND ANALYSIS

## Material parameters

The results of testing performed on quarry and untreated base samples from all sites are summarised in Table 4 on page 29. Samples labelled 1.x, 2.x or 3.x are from the case study sites, while all other samples are from reference sites. What are not included in Table 4 are the PI and LS values obtained during DMI testing. For all quarry samples these parameters were within the required specifications, while the base samples from the case studies, and one reference site, had LS values above the current limit of 2% (maximum value of 5%). The PI value of one case study site base material was one percentage point above the specified limit of 4%.

The first analysis performed on the data was the determination of the maximum and minimum differences between the quarry and base AIV and DMI values. Since these parameters are a measure of a material's resistance to impact loads and attrition/abrasion forces respectively, an increase in these values, and therefore a positive difference in results, would indicate that the material has undergone degradation in the time between quarrying and sampling from the pavement. The precision associated with both tests will always result in minor variations in results, thus small changes and even small negative changes are to be expected, even if no actual change in the material occurred. Similarly, minor variations in the material will result in a range of results which may also result in minor

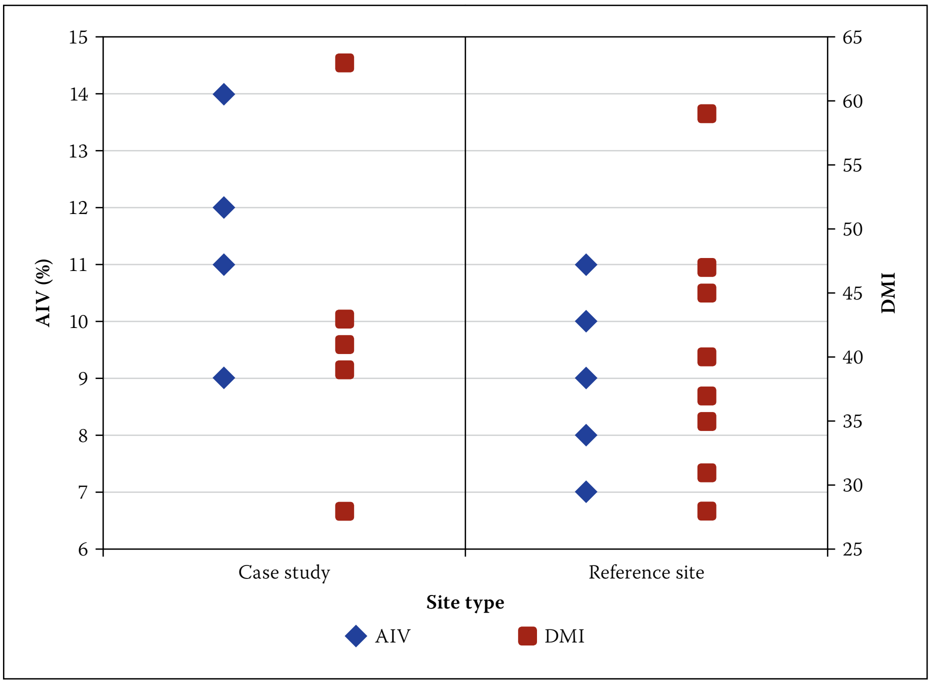


**Figure 3** Comparison of case study (left) and reference site (right) quarry sample AIV and DMI results

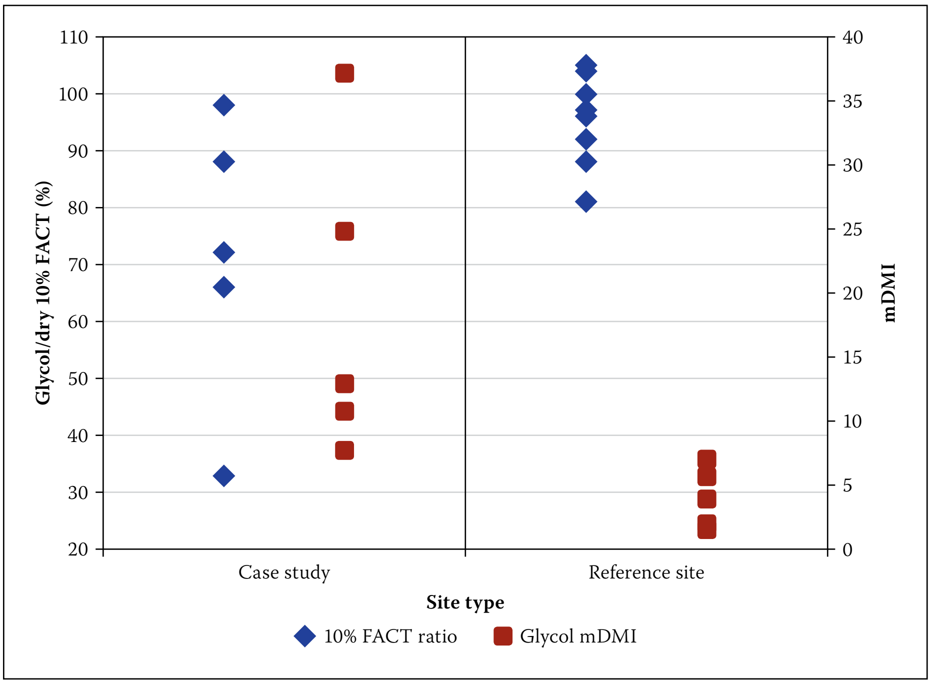


**Figure 4** Comparison of case study (left) and reference site (right) quarry sample glycol/dry 10% FACT ratios and glycol-soaked mDMI results

**Table 5** Maximum and minimum changes in AIV and DMI from quarry to pavement

| Site | Site type | ΔAIV | | ΔDMI | |
|---|---|---|---|---|---|
| | | Max | Min | Max | Min |
| 1 | Rapid failure case study | 1 | 1 | 61 | 38 |
| 2 | Rapid failure case study | 2 | –1 | 114 | 81 |
| 3 | Rapid failure case study | 3 | 2 | 127 | 125 |
| 4 | Reference site | 3 | 1 | 6 | –3 |
| 5 | Reference site | 4 | 1 | 30 | 16 |
| 6 | Reference site | –1 | –3 | 27 | 9 |
| 7 | Reference site | 0 | –4 | 28 | 23 |
| 8 | Reference site | 4 | 2 | 5 | –8 |

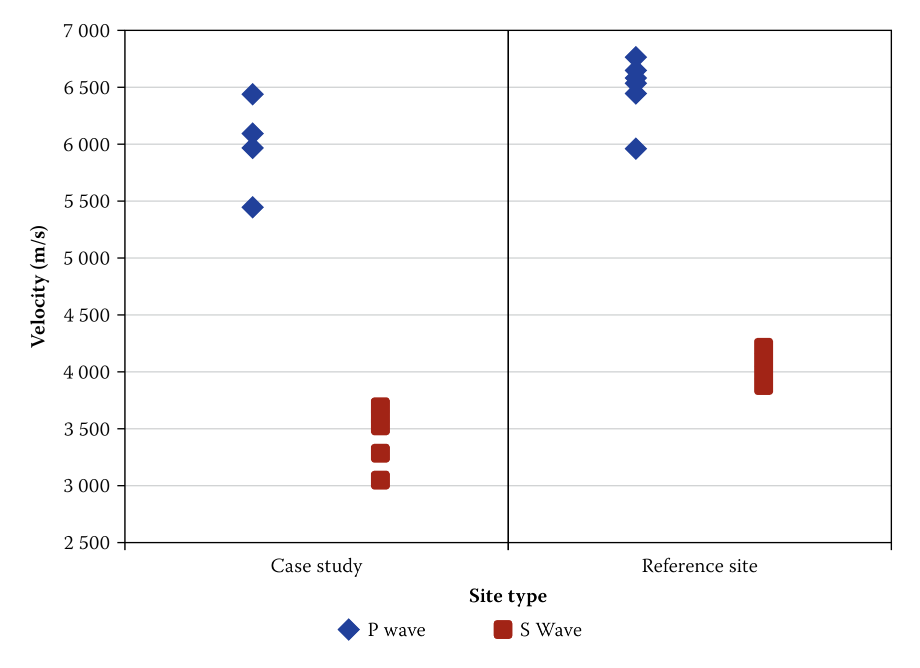


**Figure 5** Comparison of case study (left) and reference site (right) quarry sample compressive (P) and shear (S) wave velocity results

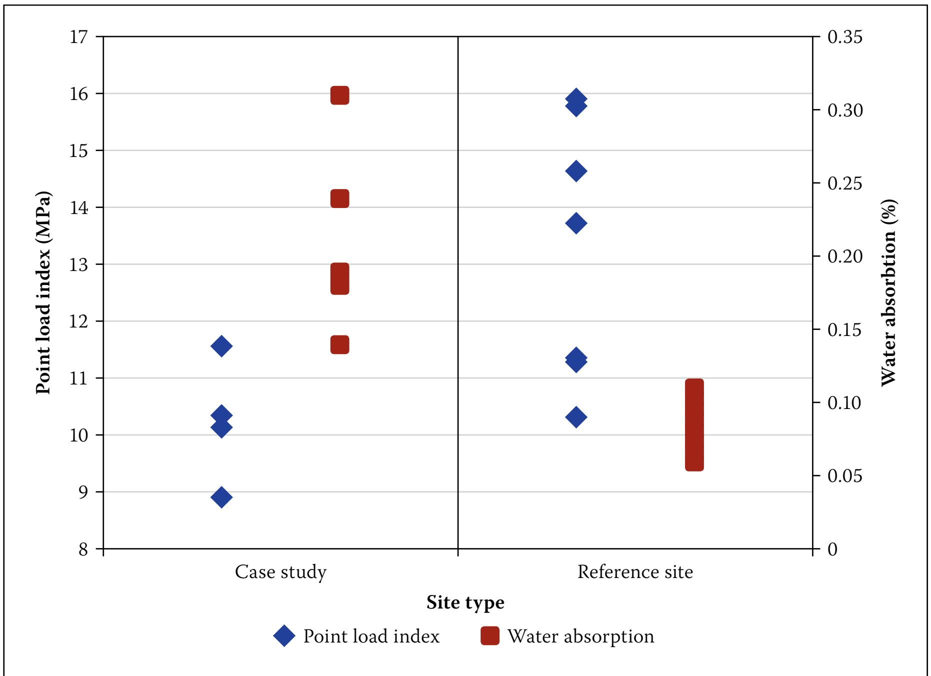


**Figure 6** Comparison of case study (left) and reference site (right) quarry sample point load index and water absorption results

positive and negative differences that are not representative of actual changes due to material degradation.

The results (Table 5) revealed that the changes in AIV were minimal at most sites and not significantly different for the case studies. The range of AIV increases after quarrying also included negative results typically similar to the positive results indicating that most of the results are due to material variation and not due to degradation. A very different observation was made for DMI changes, as all of the case studies revealed significant increases in DMI values, while the reference sites revealed either irrelevant changes (due to material variation) or very low increases in DMI values. This analysis proves that the materials used in the case study sites did undergo a change in resistance to attrition and abrasion (as measured by DMI test) and are therefore, compared with the reference sites, of poor durability. It is interesting that the observed degradation did not cause similar trends in the resistance in impact loads (as measured by AIV results).

Having established that the materials from case study sites had undergone some form of degradation, the differences in all test results between quarry samples from case study sites and reference sites were considered. Detailed descriptions of petrographic features are presented in Leyland (2014a; 2014b) and only summarised here. The petrographic properties of the case study sites were different from the reference sites, due to higher percentages of secondary minerals, evidence of secondary mineral alteration to clays and large degrees of myrmekitic alteration. Although many reference materials had secondary minerals present, they lacked evidence of significant or pervasive further alteration to clays or other deleterious minerals. The occurrence of olivine and the degree of olivine alteration was similar in all materials, and was therefore proven to not be related to the observed durability problems.

The AIV values obtained for the case study quarry samples were generally higher than the reference sample values, but the ranges did overlap (Figure 3). According to Sampson and Roux (1982), crushed rock AIVs can be equated to aggregate crushing values (ACVs), and when this was considered all samples were seen to be well below the current ACV limit of 29%. The DMI result ranges overlapped even more and were all well below the limit of 125 (Figure 3).

When the AIV results are converted to 10% FACT values and the ratio of the glycol and dry results are considered, it is seen that all reference values are above the current wet/dry ratio minimum limit of 75%, but that the case study material results cover a wide range on either side of the limit (Figure 4). The modified DMI (mDMI), proposed by Leyland (2014b), is calculated from the DMI test data using the changes in PI and P0.425 mm values during testing. These values showed a significant differentiation, as all case study sites produced higher values than the reference sites, which had a limited range of results (Figure 4).

The seismic wave velocity results (Figure 5) revealed a significant overlap between compressive (P) wave velocities, while the shear (S) wave velocities were noticeably lower in case study materials. The P wave velocities measured are within the ranges reported by Kilic (1995) for diabase, Tuğral and Zarif (1999) for granites, and Bell and Jermy (2000) for Karoo dolerites, but well above those for weathered basalts measured by Sharma and Singh (2008). Similarly to the S wave velocities, the PLI results for different site types overlapped a significant amount (although those for reference sites were on average much higher (Figure 6)), while the water absorption results for different site types fell into two distinctly different ranges.

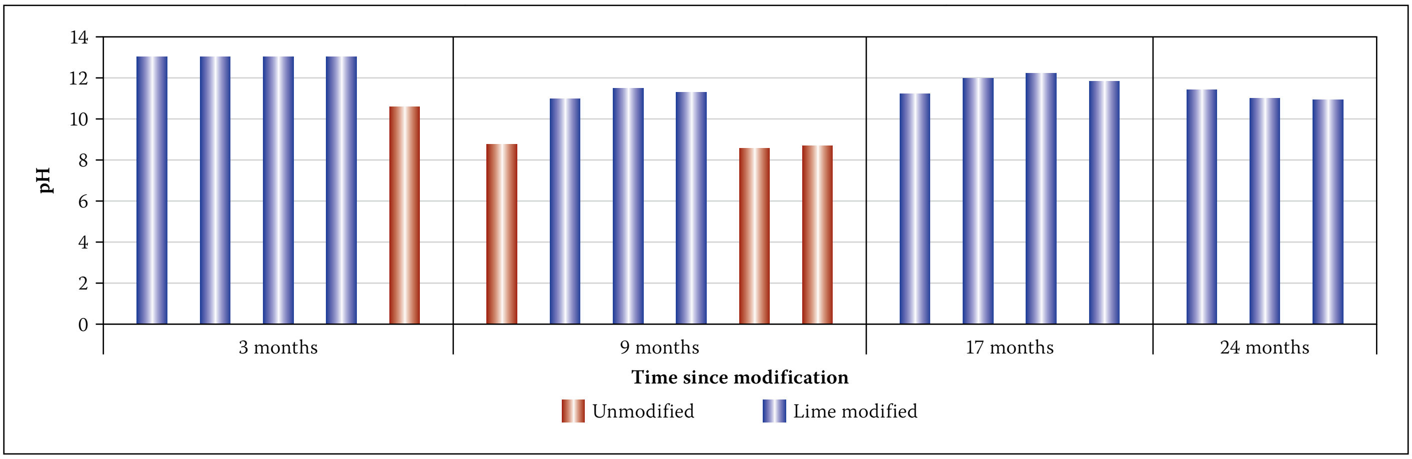


**Figure 7** pH results for unmodified and lime modified base course materials

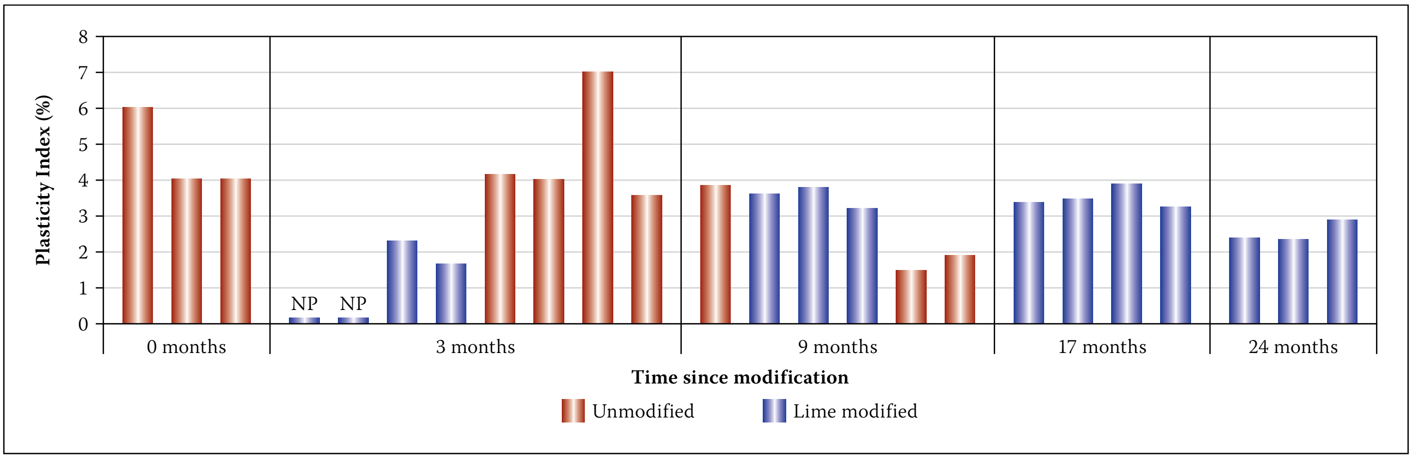


**Figure 8** Plasticity index results for unmodified and lime modified base course materials

## Material treatment

Despite the initial consumption of lime of the material being 2.0–2.5%, the addition of 1% lime resulted in a general increase in the pH of the material to a level of at least 11, and this change was sustained throughout the monitoring period (24 months) (Figure 7). The PI results indicated that all materials had a PI of 4–6% before modification, while three months after modification the PI treated materials had a PI ranging from non-plastic (NP) to 2% (Figure 8). At the sampling time, samples that were not modified were collected as control specimens and these had a PI of 3.5–7.0% (similar to initial samples).

The addition of lime therefore appeared to have lowered the PI of the materials by similar amounts, resulting in materials with relatively low unmodified PI values becoming NP, and PI values slightly above the currently specified limit being lowered to within the limit. Continued sampling and testing of both treated and untreated materials after 9, 17 and 24 months revealed that all materials had PI values of less than 4, thus providing evidence that the modification effects had been sustained. The untreated materials sampled after nine months that had PI values of < 2% are believed to represent the lower extreme of natural PI variation in the source material.

## DISCUSSION

It has been shown that the poor performance of the three case study sites was likely linked to the poor durability of the materials, which resulted in a reduction in the material resistance to abrasion and attrition forces. The identification of the poor durability of the material from all three quarries could not have been performed accurately using only the currently specified test specifications and, due to the significant overlap of results obtained, it appears that even an adjustment in the limits will not allow suitable differentiation between materials of varying durability. The water absorption, when performed on core samples, is an exception to this, but this parameter is not specified exclusively for basic igneous rocks and the observed results would require an adjustment of specified limits (currently determined on crushed samples).

The results did, however, prove that the interpretation of alternative tests can allow an accurate differentiation to be made. The modified interpretation of DMI results obtained from ethylene glycol soaked samples and the S wave velocity measured on core samples both produced promising results in this regard. Tentative limits for Karoo dolerite mDMI and S wave velocities can therefore be set as 7 (maximum) and 3 850 m/s (minimum) to ensure suitable durability. These, along with the promising specifications proposed for mEGDI (Leyland *et al* 2013), provide a method to identify poor durability materials.

It may also be possible for cross-hole seismic tomography surveys to be used to identify areas of poor durability rock within a prospective source. However, rock with a low rock quality designation (RQD) will provide low seismic velocities (as discussed by Wadhwa *et al* 2009), but may not necessarily be poor durability rocks. In the study by Wadhwa *et al* (2009) cross-hole seismic surveys showed the Deccan traps bedrock basalts to have primary wave velocities lower (± 5 500 m/s) than those presented in this research.

The modification of Karoo dolerite base course materials that have slightly elevated PI values has been shown to be effective when performed with lime applied at a rate less than the initial consumption of lime. The effects of the modification are an increase in pH and a decrease in PI. Although the PI of the materials was not consistently reduced to an NP level, it was reduced to within the current specifications, and this appeared to be sufficient, based on the observed cessation of further rapid pavement failures. However, as mentioned, the longevity of the effects are not known as of yet.

## CONCLUSIONS

The objectives of the study were reached because cases of rapid pavement failure were positively linked to degradation of Karoo dolerite, and it was shown that the currently specified tests, and their proposed result limits, were unable to identify the materials from these case studies as poor durability materials. Alternative testing methods were, however, identified as potentially more favourable solutions to identify Karoo dolerite sources that are susceptible to degradation when used as crushed rock pavement aggregates.

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