THE PAVEMENT MONITORING PROGRAMME IN BOTSWANA.

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Abstract

The project Pavement Monitoring is a part of the Institutional Cooperation between Roads Department, Botswana and the Norwegian Public Roads Administrations.

The three main objectives of the pavement monitoring programme are to:

i. Determine calibration factors for correct prediction of pavement deterioration in any future year by examining different types of pavements under various traffic loading, construction, specifications and environmental conditions.
ii. Provide accurate deterioration models to facilitate a more appropriate and cost-effective maintenance strategy, thereby optimizing the use of allocated funds.
iii. Provide data required for the upgrading of the Botswana Road Design Manual (BRDM).

It can be seen that the new HDM deterioration calibration factors differ significantly from those originally used in the BRMS. As a result of this improvement in the calibration factors, the BRMS pavement condition predictions and budget forecasting will be considerably better than in the past.

The calibration has shown that this is not an exact science, but needs in-depth knowledge of the road network to ensure that “outliers” are not included in the data set used for calibration. This analysis has shown that: in order to decrease the cost of low and medium volume roads, it is essential that optimum use is made of local materials as structural layers. This has been the case in Botswana for more than three decades, where various local materials that would not normally be considered for structural layers in pavements have been used as base and subbase layers. Many of these roads are more than 20 years old and after being closely monitored for more than 15 years, their performance in relation to the traffic has been analysed. The results show that given favourable conditions, their performance can at least equal to that of more costly conventional materials. This paper summarises the basic engineering geological properties of some of these materials and relates these to their performance and the local environmental/external conditions. It is concluded that local materials, as long as their fundamental properties are well understood, can be highly versatile road construction materials.

1. INTRODUCTION

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In order to decrease the life cycle costs of low and medium volume roads (less than about 3 million standard axles (E 80's)), it is essential that optimum use is made of local materials as structural layers. This has been the case in Botswana for more than three decades, where various local materials that would not normally be considered for structural layers in pavements have been used as base and subbase layers. Many of these roads are more than 20 years old and after being closely monitored for the past 15 years, their performance in relation to the traffic has been analysed. The results show that given favourable conditions, their performance can at least equal, if not better, that of more costly conventional materials.

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2. BACKGROUND

Many southern hemisphere countries, especially previously colonial countries and those classified as developing, have inherited the majority of their road material specifications from their previous administrator countries. These countries were mostly situated in the northern hemisphere with cold, moist climates, totally different from the hot and arid climates common to many developing countries. Botswana is a typical case, where the majority of the roads built after independence were based on British specifications, albeit those developed for tropical and sub-tropical countries such as Road Note 31 (TRRL, 1977). In addition, many of the roads were designed by Consultants originating from their previous colonial masters, many of whom were rather conservative, especially when applying the non-conventional considerations such as unsoaked strength design incorporated in Road Note 31, for instance.

At the time of independence in 1966, Botswana had a total of 10 km of paved road which had increased to 80 km by 1972. From the mid 1970s there was acceleration in sealed road construction and by 2009 the sealed road network had increased to 6506 km.

In order for this type of road provision in a developing country such as Botswana, significantly more use had to be made of innovative engineering and local materials. Although the higher trafficked roads were constructed using crushed aggregate or stabilized layers, in order to be affordable, the majority of the roads made use of local gravel materials that would normally be considered as marginal (at that time) for subbase and base course construction.

3. PAVEMENT MONITORING PROGRAMME

Because of the exceptional performance of many of these roads, a monitoring programme was initiated in 1992 and continues today. Figure 3.1 shows the location of the Pavement Monitoring Sections.

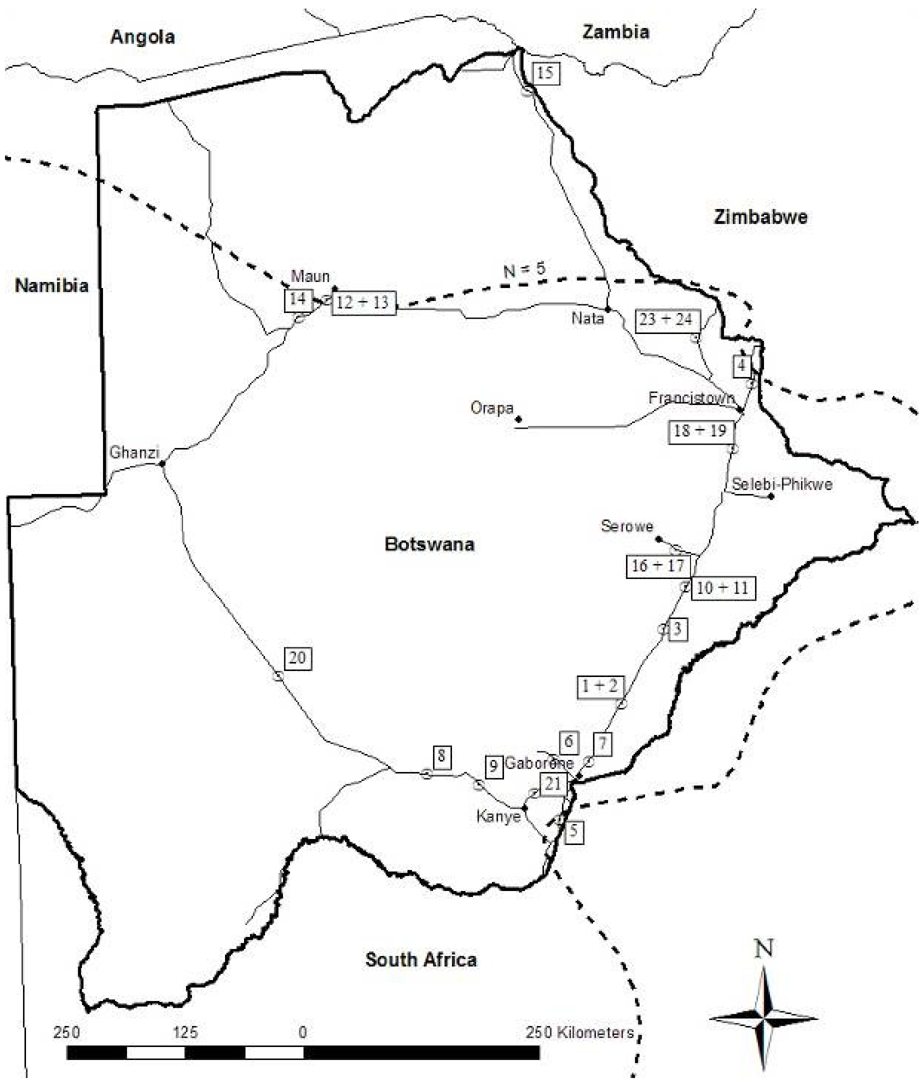


Figure 3.1 Location of the Pavement Monitoring Sections

A partial factorial design with rainfall, traffic and material type was designed, which included 25 roads as summarised in Table 3.1.

Table 3.1: Summary of roads included in the monitoring programme

Base material type	Rainfall (mm)					
	> 500	200 – 500			< 200	
	Traffic (vehicle per day)					
	< 500	> 1000	500 - 1000	< 500	500 - 1000	< 500
Crushed rock	1	1				1
Crushed gravel					1	
Stabilized calcrete		3				
Stabilized gravel		2				
Gravel (non-calcrete)	1	5	4			
Calcrete (high quality		1		1		1
Calcrete (medium quality)		1		1		
Calcrete (low quality)				1		

All sections were one kilometre long and were selected to ensure homogeneity in the horizontal and transverse section. All could be classified as being well drained. Test pits were excavated in the centre of each section as well as 100 m from each end (LHS and RHS), and samples of each layer were collected for laboratory testing. An intensive monitoring programme was developed, which included:

- Rutting (100 profiles /section).
- Cracking (100 profiles/ section).
- Visual condition assessment (entire section length according to HDM III).
- Roughness (entire section length using Merlin).
- Moisture (6 selected sections).
- Dynamic Cone Penetrometer (DCP) (every 100 m in OWP of both lanes).
- Temperature and rainfall (from nearest Meteorological station).
- Traffic (AADT and axle load surveys)
- Field sampling and comprehensive material laboratory testing.

4. CALIBRATION OF THE HDM III PARAMETERS

4.1. Introduction.

Africon, South Africa was in 2003 commissioned to check the various calibration factors for the HDM-III prediction models used in the Botswana Pavement Management System for:

- Crack initiation,
- Crack progression,
- Ravelling initiation,
- Rut progression,
- Roughness progression,
- Pothole progression

The process of calibrating the HDM-III pavement performance prediction models is part of a project undertaken by the Botswana Roads Department to monitor the long term behaviour of pavements under different conditions. This project displays a responsible long term vision on the part of the Botswana Roads Department. The project would help to gain valuable scientific evidence for the modelling of pavement performance prediction. Regular monitoring and measurements have been done on 25 carefully selected calibration sections since 1992. As the traffic loadings differ significantly in each direction, the 25 sections will effectively be doubled to 50 sections in the data analysis.

4.2 Pavement information

General pavement information and data necessary for the calibration exercise included:

1. Pavement Age. The pavement age is calculated based on the original construction date of the pavement.
2. Last Seal Year. The date when the last seal or other surfacing was applied to each section.
3. Traffic Information. Cumulative ESALS until 1996 were provided and then predicted up to 2003. This method is fully explained in the TRH4 (1) manual. For the purpose of this analysis an annual growth factor of 4% was assumed.
4. SNC. The Modified Structural Numbers for each lane of the sections were provided determined on deflection, DCP and laboratory test data.

4.3 The Calibration of deterioration parameters.

The calibration factor for time to crack initiation was calculated to be 0.428 (equal to 5.4 years before cracks shows in the surfacing) whereas the previous data from 1999 gave a value of 0.800, (equal to about 10 years) an improvement of more than 50%. However, the crack progression remains the same with a value of 1.0. The two other parameters, rut depth and roughness progression were improved from 1.0 to 0.1 and from 1.0 to 0.25, respectively. The other parameters such as ravelling and pothole progression will still be set as a 1.0 default value as the dataset did not provide a sample large enough to carry out a meaningful calibration. Table 4.1 shows the calibration factors summarised and in comparison with the factors used previously in the Pavement Management System.

Table 4.1 Calibration factors summarised and in comparison to the factors used previously in the Pavement Management System.

Item	Pre - 1999 Calibration Factor	2003 - New Calibration Factor
Crack Initiation	k_{ci} = 0.8	k_{ci} = 0.428
Crack Progression	k_{cp} = 1.0	k_{cp} = 1.0 T100 = 35,32
Ravelling Initiation	k_{vi} = 1.0	k_{vi} = 1.0 (default)
Ravelling Progression	k_{vp} = 1.0	k_{vp} = 1.0 (default)
Rut Progression	k_{rp} = 1.0	k_{rp} = 0.1
Roughness Progression	k_{gp} = 1.0	k_{gp} = 0.25
Pothole Progression	k_{pp} = 1.0	k_{pp} = 1.0 (default)

The project has helped to gain valuable scientific evidence for the modelling of pavement performance prediction. Regular monitoring and measurements have been done on 25 carefully selected calibration sections since 1992.

It can be seen that the new calibration factors differ significantly from those originally used in the BRMS. As a result of this improvement in the calibration factors, the BRMS pavement condition predictions and budget forecasting will be considerably better than in the past.

The calibration has shown that this is not an exact science, but needs in-depth knowledge of the road network to assure that “outliers” are not included in the data set used for calibration. This analysis has shown that:

- Local calibration of HDM models is important.
- Calibration factors have improved and have added considerable value to date.
- Calibration has significantly improved outputs of strategic (long-term) analysis:

- Improved budget needs forecasting (avoid over/under budgeting).
- Improved performance predictions.
- Improved understanding of local pavement behaviour.

The project has helped to gain valuable scientific evidence for the modelling of pavement performance prediction in Botswana.

5. FEEDBACK FOR THE REVISION OF THE BOTSWANA ROAD DESIGN MANUAL.

5.1 Material properties

Laboratory testing covered a large range of standard and non-standard properties, but concentrated on classification tests (Atterberg limits and grading), compaction characteristics and California Bearing Ratio (CBR). All testing was carried out according to the standard South African methods (NITRR, 1986).

The mean of the three tests for each layer on each section was used to classify the base, subbase and subgrade materials according to the South African G-class system (based on soaked California Bearing Ratio (CBR) at specified compaction densities (COLTO, 1998)). Occasionally, some of the samples had large variations, in which case, the results of the two samples that were most similar were averaged. A brief summary of the results is given in Table 5.1.

Table 5.1Summary of test results

Material	Grading modulus	Plasticity Index (%)	CBR @ 98% MDD	G-class
Calcrete base	1.37 -2.12	0 - 12	50 - 86	G4 – G6
Gravel base	1.80 -2.49	0 - 16	19 - 136	G4 – G9
Subbase (gravel)	1.05 -2.56	0 - 16	13 - 64	G5 – G9
Kgalagadi sand subgrade	0.90 -1.07	0 - 8	28 - 45	G5 – G7
Gravel subgrade	0.99 -2.18	0 - 15	14 - 75	G5 – G9

The natural gravel bases investigated consisted primarily of quartzitic gravels derived from the weathering of granite and gneiss, ferricretes and calcrete, an almost ubiquitous material in the central and western regions of Botswana. Both ferricrete and calcrete are pedogenic materials and thus can have highly variable properties, depending on their source materials and the environment under which they formed. Both of these materials have the potential to self-cement in a moist environment as temperature and moisture fluctuates, with the dissolution and re-crystallization/deposition of soluble components in their composition. The subbases were all natural gravels and no differentiation was made between calcretes and other gravels. During the investigation it became clear that there was a significant difference between the natural gravel subgrades and the Kgalagadi sand (a widespread Aeolian sand in the western and south western parts of Africa) subgrades and a differentiation between these two subgrade materials was thus made.

It is clear that the calcrete base materials are generally of better quality than the natural gravels, although they tend to be finer and slightly less plastic (but higher than normally recommended). It is also clear that the Kgalagadi sands generally provide subgrades that easily comply with the minimum specification for subgrades in southern Africa of a CBR of 15%.

6. FINDINGS AND DISCUSSION

6.1 Traffic and Performance

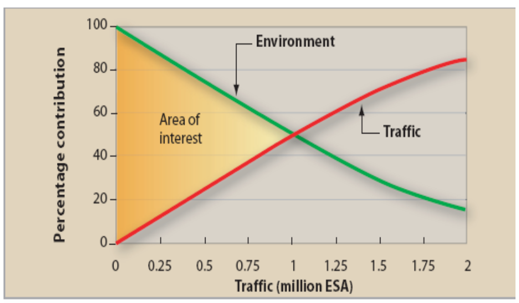
The cumulative traffic in terms of standard axles was determined from various counting and weighing stations around Botswana. The traffic on the gravel base roads investigated varied between 145,000 and 2.3 million standard axles.

Nearly all of the roads investigated during this monitoring exercise can be considered to have performed satisfactorily and in most cases exceptionally, considering their standard. None of the roads had 90 percentile rut depths in excess of the normally accepted terminal depth of 20 mm after between 9 and 29 years of service with normal routine maintenance.

Nine of the roads had performed for longer than their 20 year design periods, often with minimal maintenance and had carried traffic significantly more than their design traffic or what would reasonably be expected from the pavement structures.

Although some of this can performance be attributed to the so-called favourable conditions (good drainage, relatively dry climatic zone, strong subgrades, good compaction during construction, etc), it is clear that the materials have contributed significantly as well. The concept of "environmentally optimised design" needs to be considered in the performance of roads such as these. Work carried out by TRL Ltd (Gourley and Greening, 1999; Bradbury, 2005) indicated that the performance of bases of roads carrying up to about 500,000 standard axles is affected more by climatic (environmental) factors than by traffic loadings as shown in figure 6.1.

Figure 6.1 Traffic loading versus dominant mode of distress (After Gourley and Greening, 1999)



This concept certainly appears to contribute to the favourable conditions described above. The importance and effects of the good drainage of the monitored sections can thus not be ignored.

6.2 Base course

Design catalogues such as TRH 4 (DOT, 1996) and Road Note 31 (TRRL, 1977) identify typical base course materials as having a minimum CBR strength (soaked by definition) for the majority of roads falling into reasonable rural roads of 80% at 98% Maximum Dry Density (MDD). A G4 material is the usual minimum base standard for a rural road with a G5 subbase (minimum CBR of 45 at 95% MDD). This is the minimum design and is appropriate for roads carrying up to 1 x 10^6 standard axles. All of these structures assume a minimum layer thickness of 150 mm.

For lower category roads, the minimum base standard is G5 provided it is underlain by a lightly cemented subbase, which is not the case in any of the roads investigated. For unstabilized subbases, the minimum base quality is a G4 material, which over a G6 subbase will carry up to 300,000 standard axles. A G5 base is only likely to be successful for roads defined as rural access or experimental and of low importance with a design reliability of only 50% and carrying less than 10,000 standard axles.

Of note in this investigation is that only 6 of the roads had base courses of G4 quality or better, 3 were of G5 quality and the remainder of G6 quality or worse. The latter pavements should not have carried more than 3,000 standard axles at best.

High measured insitu strengths (supported by field observations) of some of the pedocrete base materials were strongly indicative of the presence of self cementation. The potential for this to take place in road layers can be assessed by using cycled CBR testing, although it should be borne in mind that this is only likely to occur in the pavement layers slowly with time. Relying on the high strengths generated by self-cementation could thus lead to premature failure of the road early in its life.

There is no doubt, however, that higher plasticity index values and finer gradings can be tolerated, particularly for pedogenic materials.

6.3 Subbases

The majority of subbases were of G5 or G6 quality although 6 were of G7 or worse (i.e., a CBR of < 15%). Five of the subbases were actually of better quality than the overlying base and two were of similar quality.

Where the subbase is stronger than the base, the pavement balance can be affected, but this does not appear to have had any severe effects on the performance of the roads in this study. Of course, a G6 quality material compacted to 98% MDD as a base could in some cases have a higher in situ strength than an underlying G5 material compacted to 95% MDD as a subbase.

6.4 Subgrades

In general the subgrade qualities were relatively high with more than half of the sections having subgrade materials of G6 quality or better. TRH 4 (DOT, 1996) assumes that all subgrades will be brought up to at least G7 quality. Only 5 of the sections were constructed on subgrades worse than G7. Seven of the subgrades were Kgalagadi sands which, when well confined and compacted, are known to have high insitu strengths.

6.5 Pavement structure

It is not only the pavement material quality but also the individual layer thicknesses and order of superimposition that affect the pavement performance. Calculation of the conventional structural number (SN) based on the nominal G class CBR of each layer as well as the modified structural number (SNC) incorporating the subgrade component (Paterson, 1987) was thus done for each pavement. The conventional structural numbers varied between 0.58 and 1.79 (average 1.25) but when the subgrade contribution was included these increased to 1.12 and 3.77 (average 2.80). It is clear that the strong subgrades generally contribute significantly to the overall pavement strengths.

It should be noted that the structural number concept does not take into account the relative positions of the stronger layers and the implications of this on the overall pavement balance. The case of two pavements, one with a G5 base and a G6 subbase and the other of equal thicknesses of G6 base and G5 subbase would have similar structural numbers, although the pavement balance (and probably their performances) would be very different.

6.6 Compaction/Density

No information on the densities of the layers was available. The degree of compaction of the material in each layer can affect the performance of the layer significantly, particularly in terms of rutting, but obviously mostly in terms of the mobilised shear strength within the pavement. In addition, higher density materials would generally have lower permeability to water, making their strengths less moisture sensitive.

The importance of good compaction cannot be overestimated. The insitu stiffness of almost any material can be improved by compacting it to a higher degree, and compaction to refusal using the heaviest compaction equipment available on site should be attempted as far as possible. The higher the density achieved, the less potential there is for traffic moulding to take place.

The contribution of the subgrades to the total structural number (SNC) was very high as shown in the following section. The standard specification for the compaction of sand is 100% of the BS vibrating hammer MDD and this certainly appears to have paid dividends in the performance of the roads.

6.7 Rainfall

For the experimental design, rainfall was taken as one of the factors and the road sections were incorporated in the factorial experiment on the basis of their long term average rainfall. The actual rainfalls during the monitoring period were obtained from rainfall stations closest to the road sections. The long term mean for all of the sections was 429 mm while the actual mean recorded was 418 mm. There was little difference between these means although one of the sections received more than 300% higher than the mean rainfall during one of the monitoring years. This had little influence on the sections' performance.

The rainfall in the more arid areas of Botswana generally occurs as short duration, high intensity thunderstorms and persistent rainfall over a number of days is infrequent. The higher potential for greater runoff than deep penetration of water (subject to effective side drainage) is therefore more likely. In addition, the rainfall is usually followed by hot-dry weather during which time evaporation from the road and adjacent areas is rapid. By ensuring that extensive cracking of bituminous seals is not allowed to develop, the temporary detrimental effects of water in the upper pavement structure can be eliminated.

6.8 Material strength and moisture/strength sensitivity testing

The G-class material classification system is based on the CBR, a notoriously variable property which is difficult to test repeatably, even under the best controlled laboratory conditions. As a result of this it is always difficult to classify materials based on a limited number of test results. This was the case in this work, where for example, the three base samples from a single monitoring section with each tested in duplicate had an average CBR strength of 53% at 100% MDD although the individual results ranged from 31 to 95%. This always results in classification problems when a material is border line e.g., should an average CBR of 79% be accepted when a G4 material with a CBR of 80% is specified?

In addition to conventional CBR testing, nearly all of the samples were compacted and their strengths tested after being dried back to various percentages of optimum moisture content. This testing certainly indicated that drying back has significant strength benefits in all materials, but the degree of strength gain on drying back varied considerably among the materials. In addition, some of the standard testing deficiencies resulted in strengths at 50% and 25% of Optimum Moisture Content (OMC) being less than those at 75% of OMC as well as large differences between the three samples that were tested in duplicate (e.g. CBR results at 75% OMC varied between 35 and 172 for one material with a mean of 92% (Standard deviation = 72.5)).

The results of the change in CBR strength on drying back of some of the materials are summarised in Figure 6.2. The soaked moisture content has been assumed as 125% of OMC for this diagram (Emery, 1985).

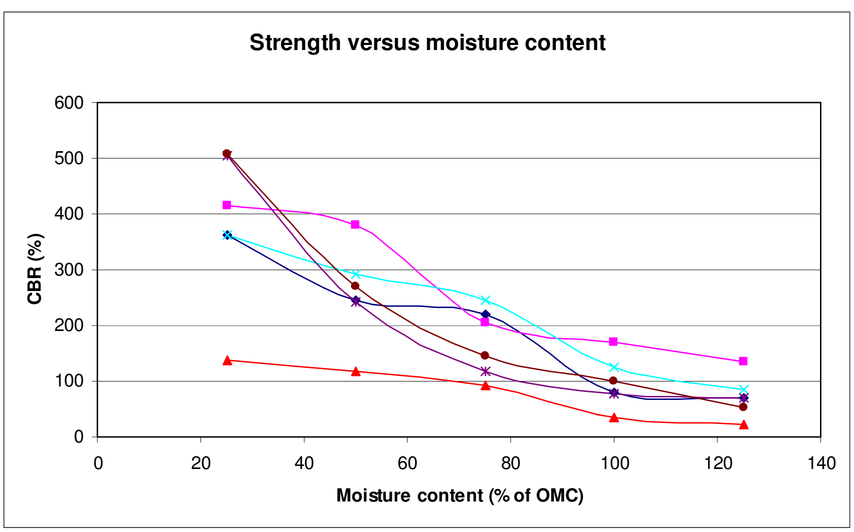


Figure 6.2 Illustration of dried back strength

Experience has also shown that the material strength is probably more important than material properties that in the past have been used to assess strength without actual strength testing (i.e. grading and Atterberg limits). Many materials have CBR strengths easily complying with the requirements for the specific layer but fail (and are rejected) on the basis of high PI's or incorrect grading moduli. Although the importance of these other properties has been noted (and included in the requirements) in the TRL study (Bradbury, 2005) it is proposed that the CBR test (assessed at different moisture contents in relation to the expected values in the pavement structure) should take precedence over these other "indicator" properties. This requires a good understanding of the material properties and performance combined with good engineering judgement of the local micro-climate and expected moisture conditions of the road.

6.9 Structural analysis

During the investigation of the test data, assessment of the relationships between the relative damage exponent (n) used to calculate the load equivalency factor and the pavement structure was investigated. The value of 4.2, which was based on the results of the 1952 AASHO Road Test, is usually used in Botswana. Kleyn and Savage (1982) proposed that the value of "n" is affected by the pavement structure, with deep structures having values lower than 4.2, while for shallow structures, a higher value should be utilised. This has been included in TRH 4 (DOT, 1996) with values of 3 to 6 being suggested for roads with granular bases and a proposed value of 4. Kleyn and Savage (1982) related the damage exponent to the Pavement Balance number (BN or DN_{100}) obtained from DCP testing through the following model:

$$N = 0.044(BN)^{1.24}$$

The BN value (number of blows to penetrate the top 12.5% of the pavement as a percentage of the number required to penetrate 800 mm) of each pavement was calculated from the available DCP data. These included up to 11 tests in each direction per section as well as up to four series of tests at various times. Only the results of DCP tests that penetrated to 800 mm were used in the analysis and the BN was calculated for each DCP test. The mean, maximum and minimum values for each monitoring section are shown in Figure 6.3.

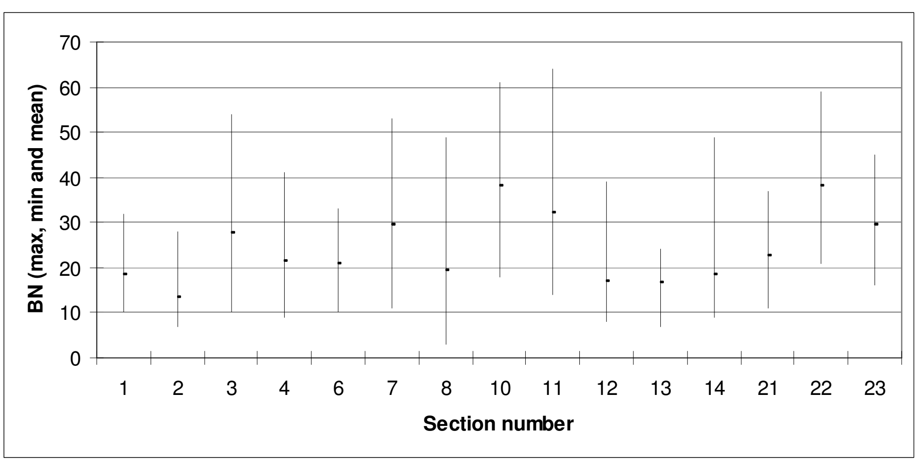


Figure 6.3 Plots of minimum, maximum and mean BN for each section

It is clear that there is a wide range of values for each section but all granular base sections had mean values between 14 and 38 with an overall average of 24. Based on Kleyn's model, the recommended n value for granular base roads in Botswana would then be 2.4 (Figure 6.4).

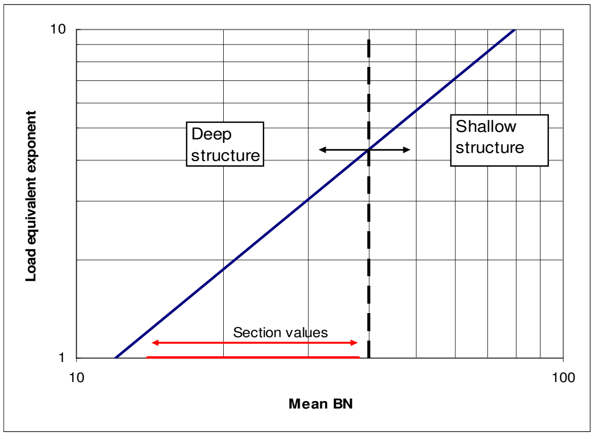


Figure 6.4 Plot of BN values for monitored sections on Kleyn and Savage diagram

Calculation of the load equivalent exponents for each road using the Kleyn and Savage (1982) model produced n values between 1.1 and 4.0. The natural gravel bases generally ranged between 1.9 and 4.0 (mean of 2.9), while the calcrete bases varied between 1.1 and 2.9 (mean of 1.7). The use of n-exponents less than 4.2 in the individual traffic calculations for each section would result in only small changes to the estimated cumulative standard axles for each section. A plot of the n-exponent against the calculated traffic used in the investigation is shown in Figure 6.5.

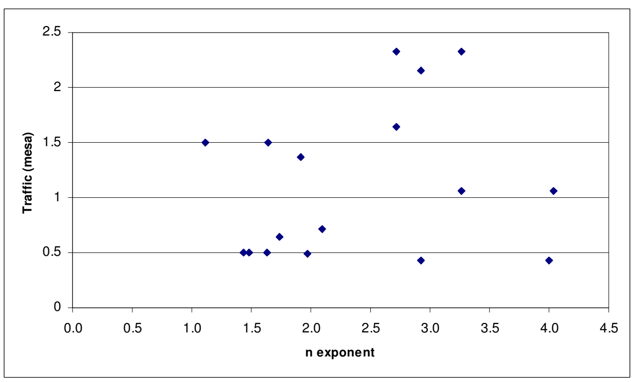


Figure 5: Plot of calculated cumulative traffic (million equivalent standard axles (mesa)) versus n-exponent

Of major significance was the fact that all of the roads with Kgalagadi sand subgrades had n-exponents of less than 2 and the majority of these had calcrete bases. This is indicative of the deep pavements and strong subgrades (particularly when the Kgalagadi sands are confined and compacted to high densities) prevailing in many parts of Botswana.

7.0 CONCLUSIONS

The 18 sections of road in Botswana constructed from natural gravel bases have been monitored for a period of up to 16 years. It is clear from the analysis of the data collected that materials conventionally regarded as marginal or sub-standard according to existing standards, can perform very well in roads carrying up to more than 2 million standard axles. Many of the successful base courses were constructed from materials of G6 quality, and other conditions being favourable, it appears that more use should be made of such materials in the appropriate situations. Although the "favourable conditions" are not all totally clear from the testing and monitoring results, they appear to relate to the good subgrade support, good drainage and high standards of construction (there are no specific test results to support this, but the insitu material strengths appear to indicate this) – it is of course assumed that such standards are constantly achieved today, although the assessment of a number of as-built records indicates that this may not always be the case.

The importance of the strong subgrades and good drainage conditions cannot be overemphasised. It has been clearly shown from the results that the well-compacted and strong subgrades (particularly the Kgalagadi sands) have a major influence on making the pavements deeper and thus reducing their susceptibility to damage resulting from overloading.

Experience with such materials, a sound understanding of their characteristics and behaviour (e.g., moisture sensitivity, self cementation characteristics, durability, etc) and good engineering judgement can allow relaxation of the standard specifications for base materials. The achievement of higher compactions (to refusal) will also assist in mobilising the currently unexploited inherent strengths in many of the materials

ACKNOWLEDGEMENTS

The authors are grateful for the assistance provided by the Director of Roads, Roads Department, Botswana and being allowed to present this paper. Part of this paper was extracted from a paper presented at IAEG 2010 in Auckland, New Zealand in September 2010.

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