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Untreated aeolian sand base course for low-volume road proven by 50-year old road experiment

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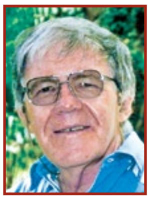
The Hoopstad long-term pavement performance experiments constructed in 1962 between Hoopstad and Bultfontein in the Free State Province of South Africa included a 90 m section of a fine-grained, nonplastic, A-2-4(0), aeolian, Kalahari-type sand as unstabilised base course. After 50 years and approximately 1.0M E80 all the experimental sections are still carrying traffic, none has been rehabilitated and none appears to have ever failed. The results of a pavement evaluation carried out in 2013 indicate that similar sand can be used unstabilised as base course for a Category C or D low-volume road designed to carry up to at least 0.1M E80/lane over 20 years, provided it is compacted to refusal or at least 100% MAASHO on a good support, is well drained, well sealed with at least the equivalent of a double seal, and that the shoulders also offer good lateral support and drainage. The seal must also be sufficiently wide to accommodate the traffic expected. Such a design offers tremendous potential for the construction of relatively inexpensive, all-weather, low-volume roads in the vast area of arid and semi-arid southern Africa in which similar sands and a scarcity of gravels occur.



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INTRODUCTION

Road construction materials other than fine aeolian sands are scarce in the northwestern Free State Province of South Africa, as well as in the vast area of southern Africa covered by Kalahari and similar sands. In their natural state such sands are usually regarded as suitable for use only in the lower pavement layers (e.g. SANRAL 2013). Long-term pavement performance (LTPP) experimental sections were therefore constructed on the Hoopstad–Bultfontein road in the Free State in 1962 to evaluate the use of such sands as base course when stabilised with cement, bitumen and road tar, using sections of unstabilised sand and crusher-run graded crushed stone as control sections (Gregg 1963).

In this paper only the performance of the unstabilised (neat) sand section is reported in comparison with the adjacent 3% and 5% ordinary Portland cement (OPC) stabilised sections, and no attempt is made to review the use of sands in general, for which see Botswana Roads Department (BRD 2010) and Paige-Green *et al* (2011).

Unless stated otherwise, the methods and abbreviations used are shown in the list of references and/or the Appendix.

LOCATION, LAYOUT AND AS-BUILT DATA

Location and layout

The location, layout and as-built test results of the neat sand base Section A and the adjacent cement-stabilised Sections B and C compiled by the authors from Gregg (1963) are shown in Tables 1 and 2. The available test results for the crusher-run section (K), which was not investigated by the authors, are also shown for comparison.

All sections had a similar sand sub-base treated with 3% PBFC on a similar neat sand selected layer, fill, and roadbed. All layers were 150 mm in thickness.

The test methods used were those of the Department of Transport (DOT 1958) for compaction characteristics, and those later published by the National Institute for Road Research (NIRR 1968) for the indicator tests.

In February 1963, eight months after construction, in-situ CBR and Benkelman beam deflection tests were carried out and cores taken for the determination of the cement contents, unconfined compressive strength (UCS) and indirect tensile strength (ITS).

The CBR tests were not dynamic cone penetrometer (DCP) tests, but the traditional in-situ tests as described for example by the Road Research Laboratory (RRL 1952).

Material properties

No laboratory CBR was reported for the neat sand and it was simply stated to be a non-plastic (NP), red, silty, fine sand containing less than 1% organic matter (Table 2).

The compaction characteristics were as follows:

- Maximum dry density (MDD) (kg/m^3): 1 896 (MAASHO); 1 856 (Proctor)

Table 1 Location, layout and as-built test results on some of the Hoopstad sand base course experiments eight months after construction [1]

← To HOOPSTAD					To BULTFONTEIN →
Surfacing: 6.1 m wide 25 mm triple seal; 2.0 m wide neat sand shoulders					
Section	Units	A	B	C	K
Road stake value (100 ft chains)		648 – 651	651 – 654	654 – 657	746 – 749
150 mm base		Neat sand	Sand + 3% OPC	Sand + 5% OPC	Crusher-run
In-situ CBR	%				
Unsoaked [2]	%	(*n* =11)	(*n* = 2)	–	(*n* = 18)
Max	%	122	> 333	–	> 140
80 %-ile	%	101	–	–	132
Mean ($\bar{x}$)	%	81.0	> 153	(> 150)	109.0
20 %-ile	%	61	–	–	86
Min	%	58	153	–	66
SD (*s*)	%	22.7	–	–	26.2
Soaked [3]		(*n* = 6)	(*n* = 3)	–	–
Max	%	36	145	–	–
80 %-ile	%	33	150	–	–
Mean ($\bar{x}$)	%	29.2	121.7	–	–
20 %-ile	%	25	93	–	–
Min	%	26	105	–	–
SD (*s*)	%	4.3	20.8	–	–
Mean UCS [4]					
Dried [5]	kPa	1 200	2 200 [10]	1 800 [12]	–
Cured [6]	kPa	140	1 400 [11]	2 000 [13]	–
Mean ITS [4]					
Dried [5]	kPa	–	55 [10]	345 [12]	–
Cured [6]	kPa	–	165 [11]	228 [13]	–
Mean deflection [7,8]	mm	0.20	0.15	0.25	0.20
Mean deflection [7,8,9]	mm	0.25	–	–	0.30
Mean ROC [8,9]	m	110	–	–	180

New section stake value (m)	0	64	0	64	0	64	–	–
Current log km	20.701	20.765	20.792	20.856	20.883	20.947	23.7	23.8
Lat (S)	28° 00.016'	28° 00.049'				28° 00.141'	–	–
Long (E)	25° 58.964'	25° 58.977'				25° 59.012'	–	–
GPS elevation (m)	1 278.2	1 278.2	1 278.3	1 278.4	1 278.5	1 278.6	–	–

NOTES
[1] Compiled mostly from Gregg (1963) with statistics calculated by authors
[2] Water content not reported; one CBR of 26 on Section A omitted from analysis
[3] After 4–5 hours of soaking
[4] On 102 x 51 mm cylindrical specimens: neat sand compacted in lab at Proctor effort (MDD 1 856 kg/m^3, OMC (11.0%), others on cores from road (density or relative compaction not stated)
[5] After seven days of exposure in the laboratory
[6] After seven days in a humid room
[7] Benkelman beam
[8] 62 kN axle load, 480 kPa tyre pressure
[9] Dehlen (1962), radius of curvature (ROC) by Dehlen curvature meter
[10] Mean OPC content 4.1%, water content 1.3%
[11] Mean OPC content 4.1%, water content 17.2%
[12] Mean OPC content 6.5%, water content 1.6%
[13] Mean OPC content 6.5%, water content 15.8%

Table 2 Grading of the neat sand used

Particle size (mm)	Percentage passing [1]
1.18	100
0.841	100
0.600	99
0.420	(97)
0.250	87
0.150	46
0.074	(9)
0.060	7
0.020	6
0.006	5
0.002	3
Calculated by authors:	
Grading modulus (GM) = 0.94	
Dust ratio [2] = 0.09	
Uniformity coefficient (Cu) ≈ 2.5	
Coefficient of curvature (Cc) ≈ 1.6	
Classification:	
AASHTO: A3/borderline A-2-4(0)	
Unified: SP–SM (poorly graded sand with silt)	
COLTO: potential G7 at best (no CBR)	

NOTES
[1] Figures bracketed estimated by authors
[2] P075 / P425

- Optimum water (moisture) content (OWC) (%) : 9.6 (MAASHO); 11.0 (Proctor)

The untreated (i.e. neat) sand had an unconfined compressive strength (UCS) at Proctor compaction of 140 kPa after seven days of curing in a humid room and 1 200 kPa after seven days of "open curing" (static compaction in 102 × 51 mm moulds).

The sections were compacted using a 50 ton pneumatic roller followed by a steel-wheel roller, and completed in June 1962.

The relative compaction of the neat sand section was not reported, and that of the 3% and 5% OPC sections was only 92% and 90% MAASHO, respectively.

At least until about November 1963 all the sections had performed satisfactorily and no failures had occurred, and it was concluded that the unstabilised sand would have sufficient strength to comply with the usual minimum CBR requirement of 80, provided that it was maintained in a dry condition (Gregg 1963).

Level measurements apparently taken up to June 1963 showed the maximum settlement on any section to be 13 mm. These measurements, as well as visual observations, were apparently continued up to about 1974, but the records could not be found.

According to the surveyor (A Bam 2012, personal communication) no distress had occurred up to that time.

Discussion

Although the neat sand base was nonplastic, the borderline A-2-4(0) classification, the high mean in-situ CBR of 81, and the presence of a significant UCS, especially after drying, indicated that this sand was not the usual cohesionless A3 sand, but did have sufficient strength for base course. At the same time the low mean, soaked, in-situ CBR of 29 indicated the critical necessity of avoiding saturation.

If the lower axle load used to measure the deflection and radius of curvature (ROC) is allowed for, the corrected deflections and radii of curvature of the deflection bowls of about 0.3 mm and 90 m for Section A respectively, were all within the sound range of 0.6 mm and > 80 m for a modern, untreated base on a treated sub-base for a modern Category C road, according to the criteria in TRH 12 (COLTO 1997), by which criteria Section A would be expected to have a structural capacity in excess of 5M E80.

Preliminary investigation in January 2013

Preliminary work confirmed the presence of the neat sand section in a fair condition, a still substantial life, and adequate laboratory soaked and unsoaked CBRs (Table 3 and Figure 1).

The visually identical sand in the road reserve (Photo 1) was also sampled and tested (Tables 4 and 5).

This work was judged sufficiently encouraging to proceed with a full pavement investigation.

PAVEMENT EVALUATION IN OCTOBER 2013

During October/November 2013 a more detailed pavement evaluation was carried out comprising visual evaluation according to TMH 9 (CSRA 1992), measurement of degree and extent of cracking, patching, edge-breaking and rut depths; DCP tests to 800 mm or refusal; profiling; phenolphthalein and acid tests; and sampling of the base at four sites in the outer wheel paths on Section A and two each on the other sections. General views of Sections A, B and C towards Bultfontein are shown in Photos 2, 3 and 4.

Visual assessment

In all cases the 30 mm thick seal was found to consist only of the original triple seal plus one reseal.

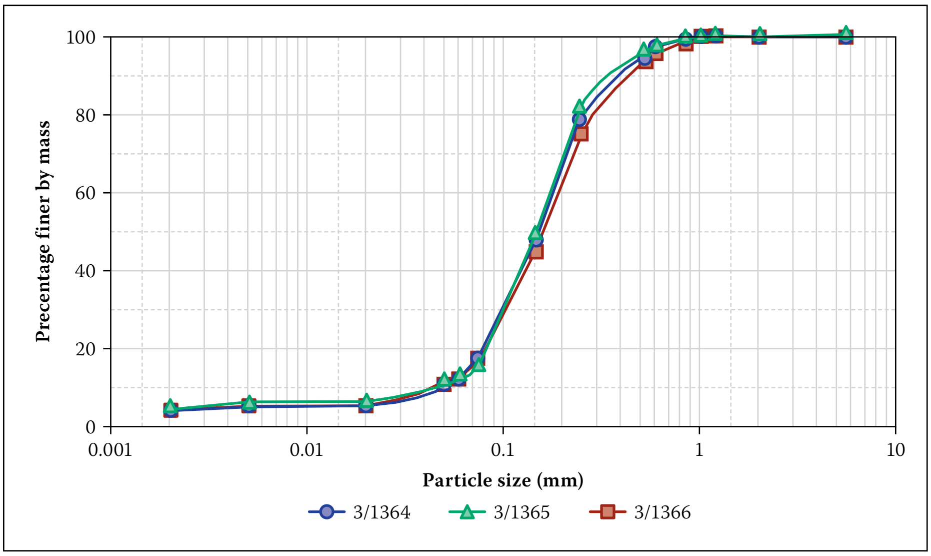

Figure 1 Grading of January 2013 base course samples

Photo 1 Yellowish-brown sand in road reserve similar to base course and presumed borrow material

Photo 2 General view of Section A (neat sand base) towards Bultfontein, and Sections B and C from just before the start (marked A and O) of the 64 m length evaluated, showing also the geotextile plus emulsion-treated edge of the left lane, and the edge-breaking and patching extending into the midlane in places in the right lane; the safety posts marking the culvert are at 50 m; the patches at 17 m left midlane, 35 m centre line and 55 m right midlane represent the January 2013 sampling sites; general pavement condition: poor, because of edge-breaking, patching and cracking

Table 3 Results of January 2013 preliminary investigation of neat sand Section A

Log km	"Chainage"[1]		Units	20.720	20.740	20.760
Section			m	19 LM	35 CL	58 RM
DCP [2]			Units			
150 mm	DN		mm/blow	11.0	11.8	12.2
Base [3]	CBR	[5]	%	20	18	17
	CBR	[6]	%	90	80	80
	FWC		%	5.1	6.0	5.5
	FWC/OWC		–	0.74	0.91	0.83
Redefined	DN		mm/blow	8.2	7.7	10.8
Base [4]	CBR	[5]	%	28	31	20
	CBR	[6]	%	> 100	> 100	90
	Thickness		mm	395	376	185
Pavement						
Balance No (A)		[7]		2 946	5 970	1 288
Balance No (B)		[7]		– 9	– 1	– 7
DSN_{800}		[7]	Blows	169	178	139
Structural capacity		[7]	M E80	1.9	2.3	0.9
DSN_{800}		[8]	Blows	164	173	135
Structural capacity		[8]	M E80	1.7	2.0	0.9
Base [9]	**Sample No**			**3/1364**	**3/1365**	**3/1366**
Passing (mm)						
1.18			%	100	100	100
0.600			%	97	97	96
0.425			%	95	96	94
0.250			%	79	81	74
0.150			%	48	49	45
0.075			%	18	16	17
0.002			%	4	4	4
Soil constants		[10]				
LL / PI / LS		[11]	%	NP/0.0	NP/0.0	NP/0.0
MDD / OWC (MAASHO)			kg/m^3/%	1 898 / 6.9	1 891 / 6.6	1 949 / 6.6
CBR @ 100% MAASHO						
Soaked at 2.54 / 5.08 mm			%	79 / 32	77 / 44	58 / 17
At OWC at 2.54 / 5.08 mm			%	140 / 70	97 / 34	98 / 31
Swell at 100% MAASHO			%	0.0	0.0	0.0
Derived data						
GM			–	0.87	0.88	0.89
Dust ratio			–	0.19	0.17	0.18
Uniformity coefficient			–	4.0	4.0	4.3
Coefficient of curvature			–	1.3	1.4	1.3
Classification						
AASHTO			–	A-2-4(0)	A-2-4(0)	A-2-4(0)
UNIFIED			–	SM	SM	SM
COLTO		[12]	–	G7	G7	G7

NOTES
[1] LM = left midlane, CL = centre line, RM = right midlane
[2] Processed by P Paige-Green using EasyDCP program (J Lea, personal communication) for Kleyn model (Kleyn & Savage 1982; De Beer *et al* 1989)
[3] Surfacing (15–30 mm) removed by inspection of penetration curve during processing
[4] Surfacing removed as above and base redefined as uniform by computer
[5] From mean Kleyn (1984) relationship: CBR = 410 $DN^{-1.27}$ for DN < 2 in program
[6] From mean sand relationship (this work): CBR = 3 000 $DN^{-1.46}$ for DN > 10
[7] Including surfacing (i.e. DCP zero taken at top of surfacing)
[8] Excluding surfacing (i.e. zero taken at top of base)
[9] Laboratory testing by Geostrada, Pretoria
[10] On P425 fraction unless stated otherwise
[11] On both P425 and P075 fractions
[12] G6 on CBR of ≥ 25 at 95% MAASHO, but fails GM requirement of 1.2–2.6

Table 4 Summary of laboratory indicator test results on initially presumed sand borrow [1]

Log km	Units	19.771 L	19.796 L	19.821 L
Sample	No	3 / 573	3 / 574	3 / 575
Test [2]				
Passing (mm)				
4.75	%	100	100	97
2.00	%	100	99	96
1.18	%	100	98	96
0.600	%	98	96	93
0.425	%	97	95	92
0.250	%	83	86	83
0.150	%	54	48	50
0.075	%	18	17	20
0.002	%	4	4	4
Soil constants [3]				
LL / PI / LS [4]	%	NP / 0.0	NP / 0.0	NP / 0.0
ARD	–	2.643	2.637	2.641
Derived data				
GM	–	0.85	0.89	0.92
FM	–	0.65	0.74	0.85
Dust ratio	–	0.19	0.18	0.22
Uniformity coefficient	–	2.6	2.7	2.9
Coefficient of curvature	–	0.9	1.0	0.9
Classification				
AASHTO		A-2-4(0)	A-2-4(0)	A-2-4(0)
UNIFIED		SM	SM	SM

NOTES
[1] Sampled in road reserve from depth of 0.3–0.8 m; colour yellowish-brown (10 YR 5/6) dry, dark yellowish-brown (10 YR 4/4) wet
[2] Testing by Geostrada, Pretoria
[3] On P425 fraction unless stated otherwise
[4] On both P425 and P075 fractions

Table 5 Comparison between soaked and unsoaked CBRs on composite sample [1]

CBR at penetration [2]	mm	2.54				5.08
Compaction (MAASHO)	%	102	100	98	95	100
Soaked	%	80	69	57	43	36
At OWC	%	110	90	71	47	60
At 75% OWC	%	150	123	96	56	140
At 50% OWC	%	220	171	130	86	180
At 25% OWC	%	270	238	183	141	270
Classification						
COLTO [3]					G7	

NOTES
[1] Approximately equal masses of Samples 3/573–3/575 (MDD 1 854 kg/m^3, OWC 6.2%)
[2] Testing by Geostrada, Pretoria
[3] G6 on CBR of ≥ 25 at 95%, but fails GM requirement of 1.2– 2.6

Photo 3 General view of Section B (sand plus 3% of OPC base) towards Bultfontein; general pavement condition: fair, mostly because of cracking

Photo 4 General view of Section C (sand plus 5% OPC base) towards Bultfontein; general pavement condition: fair, mostly because of cracking

Photo 5 The typical six- and seven-axle trucks are too wide for the seal; the strip on the left is the geotextile plus emulsion repair, the large patches on the centre line and the right midlane represent sample sites, and the full-width patches in the foreground under the truck (across the culvert) were apparently to strengthen the seal across some patched potholes

Photo 6 Right-hand sand shoulder on Section A being ridden out by large trucks

On all sections the binder of the reseal was still good, i.e. Degree 1 / Extent 5 (D1/E5), and there was little bleeding (D2/E5), but severe and extensive map to hexagonal to block (D5/E5) cracking at an average spacing of about 1 m over the full width, and crocodile (D4-5/E 3-5) cracking occurred in all wheel paths and extended across the centre line.

The cracking was confined to the seal, was usually only 10–15 mm deep, and did not penetrate into the base course at any of the sites excavated or at the additional cracks tested. Although often spalling to a width of 10 mm, the actual width of the cracks was only 1 mm or less.

Longitudinal cracking and pumping were absent, rutting was minimal (D2-3/E5), there were no undulations, and patching was almost entirely confined to the edge-breaking and edge-cracking, which was severe (D 3-5) and extensive (E 4-5) on all three sections.

Some secondary cracking was evident on Sections B and C only, especially in the left lane, but was less extensive on Section C.

There was no evidence of shear failures (the patches appeared to be all due to large edge breaks) or even excessive rutting (except on the sand shoulders), and the main problem was edge-breaking due to the seal being now too narrow for the large trucks currently using it (Photos 5 and 6). This was worst on Section A, where the value of the geotextile and emulsion holding the seal in the left lane was evident.

The riding quality was assessed as only fair because of the edge breaks and patching.

The surface drainage was assessed as adequate and the grassed sand shoulders as safe. The road had been built with a camber and the edges of the seal were about 80–130 mm below the centre line, giving an effective total crossfall of about 3–4%.

The centre line of the road was about 500 mm and 300 mm above the side drains on the left- and right-hand sides, respectively. The fall of the natural ground level in the 30 m wide road reserve was about 0.5 m from right to left.

The alignment was straight and practically level, with a fall of only about 0.2% from the end of Section C to the start of Section A.

Further details of the cracking and patching and the rut depths are shown in Table 6.

As the road had been built with a camber the rut depths were measured in two ways. The end of the straight edge was first placed in the normal way on the edge of the seal or on the centre line, and the deviations under the ends recorded as 'edge' and CLL or CLR, respectively, while the maximum deviation

Table 6 Summary of visual evaluation in October 2013 [1]

Parameter		Section A (neat sand)	Section B (sand + 3% OPC)	Section C (sand + 5% OPC)
Cracking [2]	**Block**	D5 / E4 (80% of area)	D4 / E5 (100% of area)	D5 / E5 (100% of area)
	Croc			
	LOWP	24 m D4 – D5 up to 1.0 m in from edge	40 m T4 – CC4	D4 / E3
	LIWP	D5 / E5	64 m D5 to centre line	40 m D4 to centre line
	RIWP	D5 / E5	30 m D4 – 5 to centre line	40 m D4 to centre line
	ROWP	31 m D4 – D5 up to 1.0 m in from edge	60 m T4 – CC3	Nil
Patching	**Full width**	5 m (near culvert) (30 m^2)	Nil	Nil
	Edge			
	LOWP	40 x 1.0 m geotextile and emulsion + 8 × 1.0 m = 48 m (48 m^2)	17 × 0.5 m = 8.5 m^2	15 × 0.5m = 7.5 m^2
	ROWP	62 m 0 – 0.5 – 1.5 m = 33 m^2	1 × 0.5 m = 0.5 m^2	2 × 0.5 = 1.0 m^2

Deviations [3]		A Max	A 90 %-ile	A 80 %-ile	A 50 %-ile	A Min	A Patches	B Max	B 90 %-ile	B 80 %-ile	B 50 %-ile	B Min	B Patches	C Max	C 90 %-ile	C 80 %-ile	C 50 %-ile	C Min	C Patches
		mm	mm	mm	mm	mm	No	mm	mm	mm	mm	mm	No	mm	mm	mm	mm	mm	No
LHS Bultfontein-bound	Edge [3]	13	12	10	6	0	1	8	5	4	2	0	3	10	7	6	3	0P	2
	OWP [3]	12	11	10	7	0	0	14P	9	8	5	0	2	16P	12	10	7	3	1
	OWPH	6	4	3	2	0	0	14P	9	7	4	0	2	–	–	–	–	–	–
	IWP	15	15	13	10	0	1	8	10	9	7	2	0	16	13	11	9	5	0
	IWPH	7	6	5	4	2	1	7	7	6	4	2	0	–	–	–	–	–	–
	CLL	41	35	30	20	3	0	19	13	10	6	0	0	22	20	17	12	0	0
RHS Hoopstad-bound	CLR	22	17	14	8	0	0	24	16	13	9	2	0	26	21	17	11	0	0
	IWPH	4	4	3	1	0	1	6	5	5	4	2	0	–	–	–	–	–	–
	IWP	9	9	8	6	4	0	18	14	13	10	6	0	13	12	11	8	5	0
	OWPH	11P	8	7	4	0	2	8	7	6	4	0	0	–	–	–	–	–	–
	OWP	15P	12	10	7	3	2	8	7	6	4	2	0	11	8	7	5	3	0
	Edge [4]	26P	23	19	13	0	4	6	5	4	2	0P	1	10P	8	6	3	0P	2

NOTES
[1] Only the central 64 m length of each section was assessed
[2] Mean spacing: block-hexagonal: 1 m, crocodile (CC): 140–150 mm, transverse (T) mostly short (≤ 1 m) on Section C edges only; degree (D) and extent (E) according to TMH 9 (1992)
[3] Deviations under a 2.0 m straight edge (n = 14) measured with a 20 mm wide wedge at approx 5 m intervals including on patches (number shown); P = patch; Seal edge, OWP, IWP, CLL, CLR: straight edge at rest; OWPH, IWPH: straight edge held down at seal edge and centre line respectively; including nine on geotextile plus emulsion from 0 to 40 m, but no separate patches in OWP
[4] Including four deviations (13–26 mm) on patches on Section A) (80 %-ile: 16 mm without patches; n = 10)
[5] According to TRH12: 1992 all three sections were in a severe condition with respect to block cracking (≥ 50% of length), crocodile cracking (≥ 25%) and Section A with respect to patching (≥ 50%)

in the wheel paths was recorded as the rut depth (OWP or IWP respectively). Then the straight edge was held down on the edge or the centre line and the deviations in the wheel paths re-measured and recorded as OWPH or IWPH. The latter procedure usually yielded the smaller deviations and may be a closer representation of the true traffic-induced rut depths.

The general pavement condition was assessed as poor on Section A and fair on Sections B and C, with C slightly better than B.

The inspection panel recommended that the edge-breaking should be patched at an A priority and the shoulders gravelled (or at least bladed), the seal softened and rolled in hot weather to seal the cracks, and the road resealed at a B priority. However, damage to the edges must be expected to continue, as the road is simply too narrow for the large trucks currently using it.

DCP survey

The results of the rut depth and DCP surveys in the outer wheel paths are summarised in Table 7. As the DCP results for both wheel paths were similar, they have been combined to total six points on Section A and B, and five on Section C (one was invalid).

The analysis was carried out by means of the EasyDCP program (J Lea 2013, personal communication) using the Kleyn granular base model (De Beer *et al* 1989) for Category C and D roads (i.e. 80/20 and 50 percentile (%-ile) basis respectively) and – conservatively assuming an optimum moisture condition – considering that the survey was carried out at the end of the dry season when the field

Table 7 Summary of rut depth and DCP test results in outer wheel paths in October 2013

Layer	Test	Units	Section A (neat sand)					Section B (sand + 3% OPC)					SectionC (sand + 5% OPC)				
			Max	80 %-ile	50 %-ile	20 %-ile	Min	Max	80 %-ile	50 %-ile	20 %-ile	Min	Max	80 %-ile	50 %-ile	20 %-ile	Min
OWPs [1]	**Rut depth**																
	LHS	mm	12	10	7	–	0	14	8	5	–	0	16	10	7	–	3
	RHS	mm	15	10	7	–	3	8	6	4	–	2	11	7	5	–	3
	DCP [2,3]																
150 mm base [4]	DN	mm/blow	7.4	7.0	6.3	5.6	4.7	6.4	5.7	4.5	2.6	2.2	7.5	5.4	4.2	3.5	2.5
	CBR [6]	%	58	46	41	34	32	155	120	67	45	39	131	88	67	50	32
	CBR [7]	%	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100
	FWC [8]	%	4.1	–	3.5	–	3.3	8.3	–	5.2	–	6.0	9.8	–	7.3	–	6.2
	FWC/OWC [9]	–	0.55	–	0.47	–	0.44	–	–	–	–	–	–	–	–	–	–
150 mm shoulders	DN	mm/blow	22	–	19	–	15	–	–	–	–	–	–	–	–	–	–
	CBR [6]	%	14	–	11	–	8	–	–	–	–	–	–	–	–	–	–
	CBR [7]	%	60	–	48	–	35	–	–	–	–	–	–	–	–	–	–
	FWC	%	8.5	–	8.5	–	8.4	–	–	–	–	–	–	–	–	–	–
Redefined upper layer [5]	DN	mm/blow	7.0	6.9	5.6	5.3	4.7	6.2	5.5	4.0	2.7	2.3	6.4	5.2	4.1	3.1	2.1
	CBR [6]	%	58	49	46	35	35	144	118	79	47	41	131	88	67	50	32
	CBR [7]	%	> 100	> 100	> 100	> 100	> 100	> 100	>100	> 100	> 100	> 100	> 100	> 100	> 100	> 100	> 100
	Thickness	mm	595	358	275	171	170	662	595	236	123	85	471	334	290	195	217
Redefined shoulders	DN	mm/blow	14	–	14	–	13	–	–	–	–	–	–	–	–	–	–
	CBR [6]	%	17	–	16	–	15	–	–	–	–	–	–	–	–	–	–
	CBR [7]	%	70	–	60	–	60	–	–	–	–	–	–	–	–	–	–
	Thickness	mm	710	–	522	–	333	–	–	–	–	–	–	–	–	–	–
Pavement																	
Balance No (A)	[10]	–	2 821	2 478	1 902	1 707	1 657	2 635	2 149	1 893	1 605	1 000	3 067	2 596	2 193	1 391	538
Balance No (B)	[10]	–	10	5	1	– 1	– 9	12	11	– 0.5	– 3	– 8	20	16	11	5	– 1
DSN_{800}	[10]	Blows	245	205	196	185	170	320	280	255	190	170	195	187	175	168	160
Structural capacity	[10]	M E80	6.9	3.7	3.3	2.6	1.9	>10	7.8	4.8	2.5	1.9	3.1	2.7	2.1	1.8	1.6
DSN_{800}	[11]	Blows	235	181	175	170	161	290	260	237	175	166	176	174	152	148	145
Structural capacity	[11]	M E80	6.0	2.4	2.1	1.9	1.6	12.4	8.5	6.1	2.1	1.8	2.2	2.1	1.3	1.2	1.1

NOTES

[1] Outer wheel paths (n = 14 in each), including those on patches
[2] Three in each outer wheel path on Sections A and B, three in left and two in right on Section C
[3] Processed by P Paige-Green using EasyDCP program (J Lea, personal communication) using Kleyn granular base model and Cm = 30
[4] Surfacing (15–30 mm) removed by inspection of penetration curve during processing (i.e. zero taken at top of base)
[5] Surfacing removed as above and uniform layers redefined by computer; water contents of sub-base 8.0, 8.3%; selected 5.2, 5.3%, fill 7.8%
[6] Mean CBR of layer from mean Kleyn (1984) relationship: $CBR = 410\ DN^{-1.27}$ for DN > 2 in program
[7] From mean sand relationship (this work): $CBR = 3\ 000\ DN^{-1.46}$ for DN > 10
[8] Field water content (n = 4 on A, 2 on B, 3 on C)
[9] Mean OWC = 7.5% (n = 4)
[10] Including surfacing (i.e. zero taken at top of surfacing)
[11] Excluding surfacing (i.e. zero taken at top of base); (20 %-ile capacity of 1.9 for Section A remains unchanged if outlier of 6.9M E80 removed)

water content (FWC) was about 50% of the MAASHO optimum (OWC).

An example of the output of this program for a 20 %-ile DN of 7.0, an 80 %-ile DSN_{800} of 185 and a predicted 20 %-ile structural capacity of 2.6M E80 for Section A (in the 10 m right outer wheel path) is shown in Figure 2. For comparison, the required DCP profile for a Class II rural road pavement designed for 0.10–0.30 M MISA (i.e. actual 80 kN standard axles) of the former Transvaal Roads Department (1994) is also shown.

For simplicity and ease of comparison the granular base model has been used on all three pavements. Although it tends to over-predict the structural capacity of cemented pavements (De Beer *et al* 1989) it can be used on them (Kleyn & Savage 1982; COLTO 1997).

Discussion

As a check on the published relationships between DN and CBR, DCP tests were carried out on most of the specimens after determination of the CBR. Plotting of these results led to a very rough but very different relationship from that of Kleyn (1984) for materials in general (and used in the program) and closer to those of Sampson and

DCP summary								
Area	Free State			Moisture	Optimum	Category		2
Road	P21/3 A	Distance	0.01 km	Position	8	Test date		10/10/2013
Structure number (DSN_{800})			185	Base type	Granular	Structural capacity (E80s)		2.584×10^6
B = –1		A = 1 657		Category VIII	Averagely balanced inverted structure (ABI)			

User defined layer summary

From–to (mm)	Average DN (mm/blow)	Standard deviation (mm/blow)	CBR (%)	Range 20%–80%	UCS (kPa)	Range 20%–80%	E-Mod (MPa)	Range 20%–80%
0–30	2.39	1.00	135	78–244	1 128	695–1 893	442	279–832
30–180	7.00	0.62	35	30–40	339	301–387	141	126–160
180–330	4.48	0.69	61	48–80	559	455–706	227	187–283
330–480	4.63	0.17	59	55–62	539	513–568	219	209–231
480–630	3.51	0.54	83	66–108	734	597–926	294	242–367
630–800	5.03	0.88	53	40–71	491	390–641	201	161–259

Redefined layer summary

From–to (mm)	Average DN (mm/blow)	Standard Deviation (mm/blow)	CBR (%)	Range 20%–80%	UCS (kPa)	Range 20%–80%	E-Mod (MPa)	Range 20%–80%
0–24	1.69	0.30	196	162–242	1 561	1 318–1 880	639	513–824
24–195	6.92	0.59	35	31–41	344	306–390	143	128–161
195–545	4.45	0.45	62	53–73	563	491–652	228	201–263
545–685	3.17	0.33	95	81–113	824	715–960	328	287–380
685–800	5.83	0.54	44	38–51	417	367–477	172	152–195

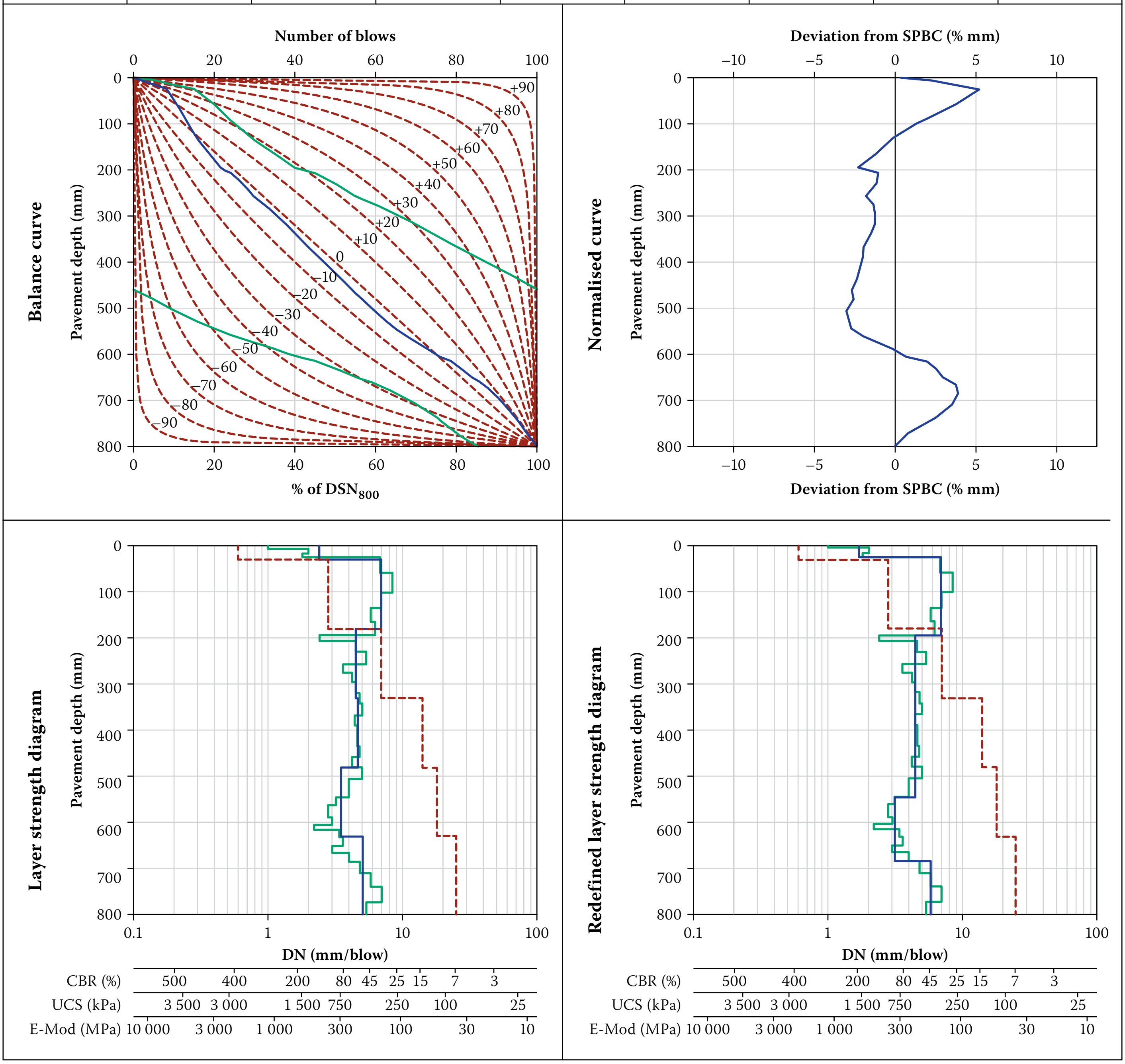

Figure 2 Example of DCP program output for a single point

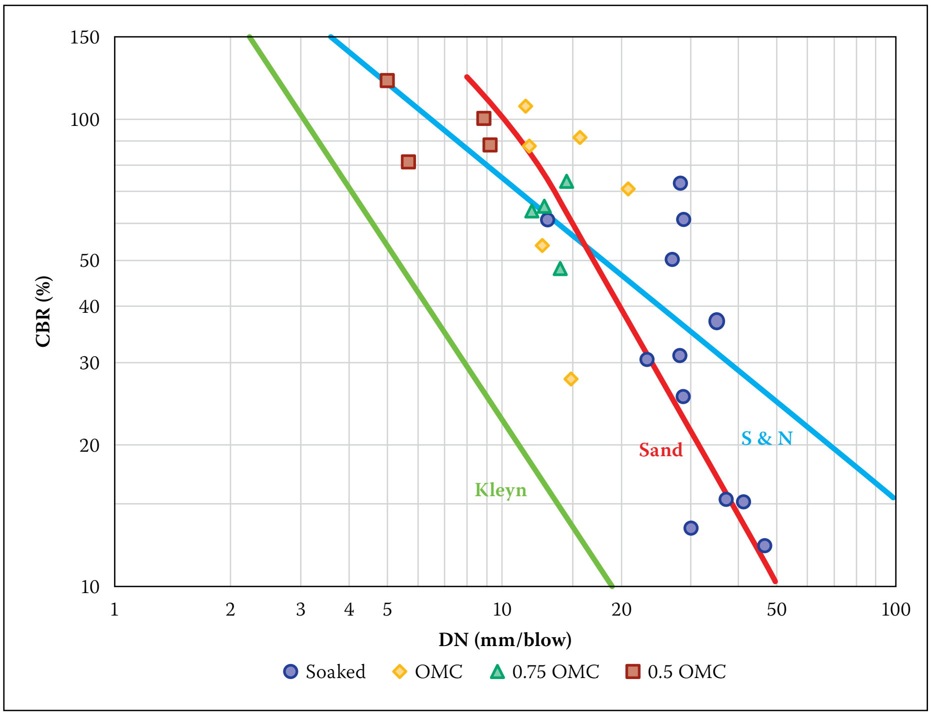

Figure 3 Relationship between DN and CBR for Hoopstad sand in comparison with those of Kleyn (1984) for materials in general and Sampson & Netterberg (1990) for nonplastic material

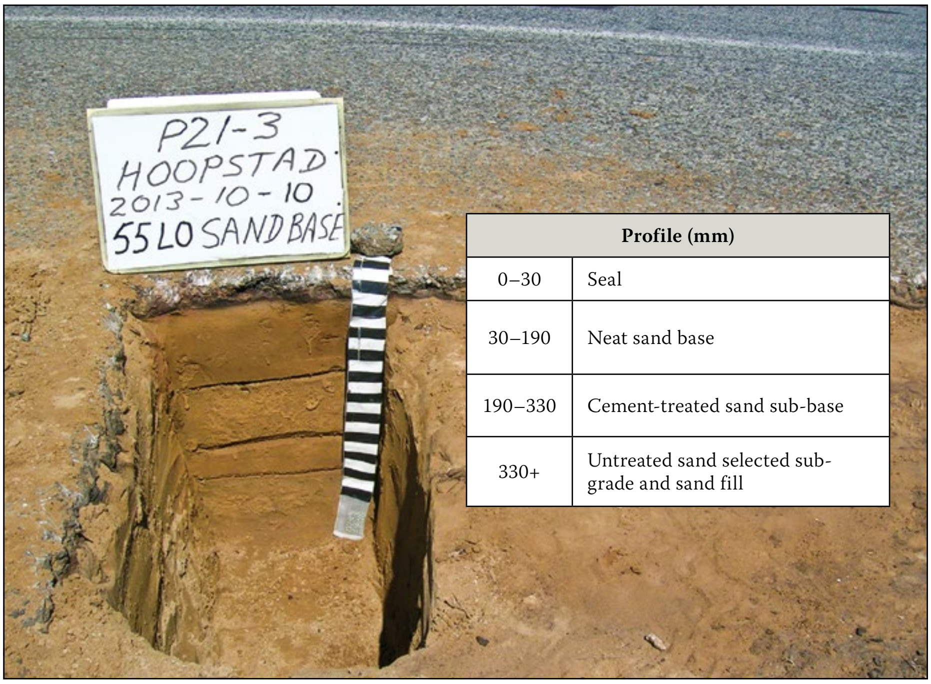

Photo 7 Profile in left outer wheel path at 55 m on neat sand base Section A (scale in 20 mm intervals)

Netterberg (1990) for nonplastic materials, although none of them were sands (Figure 3). The relationship for the Hoopstad sand was:

- $CBR \approx 3\ 000\ DN^{-1.46}$ for DN > 10 (1)

in comparison with that of Kleyn (1984):

- $CBR = 410\ DN^{-1.27}$ for DN > 2 (2)

and Sampson and Netterberg (1990):

- $CBR = 354\ DN^{-0.69}$ for DN > 3 (3)

The analysis showed that the 80 %-ile DN of the base of the neat sand section was about 7.0 mm/blow, indicating that 80% of the assumed 150 mm – and the redefined 175 mm – base had an in-situ DCP CBR of more than about 34% according to the Kleyn model, or over 100 according to the Hoopstad model (Figure 3 in this paper). With similar 80 %-ile DNs of about 5.5 and 20 %-ile CBRs of about 45 (or over 100), the two cemented bases were not greatly stronger than the uncemented base, both in the assumed 150 mm and in the redefined cases.

This suggests that the in-situ CBR of the neat sand base was far higher than that indicated by the Kleyn model, and thereby offers a plausible explanation for its unexpectedly good performance.

The 20 and 50 %-ile redefined thicknesses of similar strength to the base of 171 mm and 275 mm respectively indicate that most of the originally cemented sub-base on Section A had reverted to a granular state and was by then operating at the same strength as the base. Experience with respect to the usual rates of carbonation (Netterberg 1991) suggests that, regardless of traffic, this would actually have been the case from about year five to ten.

The upper 150 mm of the shoulders on Section A had a 50 %-ile (in this case a mean) DN of 19, indicating an in-situ DCP CBR of about 11 according to the Kleyn model, and 48 according to the Hoopstad model. The redefined case indicates that the whole 522 mm had a mean DN of 14, indicating a CBR of about 16 (or 60) and therefore good lateral support both to the base and lower layers. This conclusion is also supported by the minimal rutting of the sand shoulder shown in Photo 6, which is far less than would be expected from a nonplastic sand.

The 80 and 20 %-ile pavement balance numbers A of about 2 500 and 2 100, and B of -1 and -5 for Sections A and B respectively, indicate that both pavements were averagely balanced and marginal to inverted deep pavements, and that the Kleyn model requirements of a good (or apparently at least an average balance) have been at least marginally met. With respective A and B balance numbers of 2 600 and 5 Section C was an averagely balanced deep pavement.

The predictions of the remaining structural capacities of the pavements depend somewhat on whether the zero point of the DCP is taken at the top of the seal according to TMH 6 (NITRR 1984) or at the top of the base, which is where it was taken during the testing from which the Kleyn model was developed (E G Kleyn 2010, personal communication).

For example, the 20 %-ile predictions are:

Zero point	Section number		
	A	B	C
Top of seal (M E80)	2.6	2.5	1.8
Top of base (M E80)	1.9	2.1	1.2

As the 80 %-ile rut depths of all three sections were also only in the 6–10 mm range they were all sound according to TRH 12 (COLTO 1997). Sections A and B have a similar residual capacity of about 2M and C about 1M E80 to an additional rut depth of 20 mm. Assuming proportionality, this indicates that the remaining capacities to a total rut depth of 20 mm of the three sections are about 1.0 for Sections A and B and about 0.5M E80 for Section C.

If all of the 13 available DCP results are considered, the lowest predicted 20 %-ile capacity of all 10 of the DCPs carried out in

Table 8 Results of in-situ and laboratory tests on October 2013 samples of neat sand base

Location	OWP	10 m L	30 m L	55 m L	55 m R
In-situ	Units				
Rut depth	mm	9	12	6	7
DCP base (150 mm)	DN	5.6	4.7	7.0	5.6
CBR [1]	%	46	58	34	46
CBR [2]	%	> 100	> 100	> 100	> 100
Wet density [3]	kg/m³	1 933	1 933	1 935	1 933
FWC [4]	%	5.5 / –	4.4 / –	4.5 / 3.3	5.3 / 4.1
Compaction [4]	%	99.6 / –	101.3 / –	100.0 / 101.2	98.2 / 99.3
FWC / OWC [4]	–	0.82 / –	0.49 / –	0.59 / 0.43	0.77 / 0.59
Lab sample, passing (mm)	**No**	**3/11650**	**3/11651**	**3/11652**	**3/11653**
1.18	%	100	100	100	100
0.850	%	99	99	99	99
0.600	%	97	97	97	98
0.425	%	96	96	96	97
0.250	%	80	81	82	84
0.150	%	47	50	50	53
0.075	%	20	21	19	25
0.038	%	19	21	18	24
0.020	%	14	14	14	14
0.002	%	13	14	12	13
Soil constants (P425)					
LL/PI/LS	%	NP	NP	NP	NP
SE	–	13	20	38	40
Soil constants (P075)					
LL/PI/LS	%	21 / 6 / 2.0 [6]	SP / 1.0	SP / 1.0	SP / 0.5
MDD/OWC	kg/m³ / %	1 840 / 6.7	1 828 / 8.9	1 850 / 7.6	1 870 / 6.9
Soaked CBR at % MAASHO					
104 / 102	%	90 / 76	90 / 69	90 / 69	110 / 84
100 / 98	%	62 / 51	50 / 38	51 / 39	63 / 48
95 / 93	%	37 / 23	26 / 20	26 / 19	32 / 24
Swell at 100%	%	0.0	0.0	0.0	0.0
Derived data					
GM	–	0.84	0.83	0.85	0.78
FM	–	0.71	0.68	0.67	0.61
Dust ratio	–	0.21	0.22	0.20	0.26
Cup IF075	–	120	42	38	25
Classification					
AASHTO	–	A-2-4(0)	A-2-4(0)	A-2-4(0)	A-2-4(0)
UNIFIED	–	SM	SM	SM	SM
COLTO	–	G7	G7	G7	G7
Compactability [5]	–	Marginal	Good	Good	Good

NOTES
[1] From mean Kleyn (1984) relationship: CBR = 410 $DN^{-1.27}$ for DN > 2
[2] From mean sand relationship (this work): CBR = 3 000 $DN^{-1.46}$ for DN > 10
[3] CPN MC-30 nuclear, upper 20 mm of primed base removed and 0–100 mm results used; 0–150 and 0–250 mm results generally similar
[4] Using nuclear/lab water content
[5] Assuming upper limit of IF075 (using TMH 1 cup for LL) is 120 for P075 of 20%
[6] Repeat test: 21 / 7 / 1.5

October of 1.7M E80, and the three single points in January 2013 of 0.9M E80, indicate a residual capacity for Section A of at least about 0.5M E80.

As the Kleyn model is an average prediction the most conservative view would be to halve this again. On this basis it can be concluded with a high degree of certainty that the residual structural capacity of Section A to a total rut depth of 20 mm (i.e. an additional 10 mm) at an in-situ water content of OWC is at least about 0.3M E80. If it were to become saturated this would be halved to about 0.1–0.2M E80.

The capacities of the cemented base sections must be regarded as more approximate, as they have not been checked by the De Beer model (De Beer *et al* 1989) for cemented pavements.

The low degree and extent of rutting, the absence of shear failures (at least at the time of the two site visits), the absence of base course patching found in any of the six holes and 13 DCP points, the reasonable DN (and apparently very good in-situ operating CBR of over 100) and the predicted residual structural capacity of at least about 0.3M E80 all indicate that the further use of such a sand as untreated base course for a low-volume road is viable under similar conditions.

As most of the sub-base was operating at about the same strength as the base it should not be necessary to treat it with cement, provided that it is compacted to a similar strength and that this support is sufficient to achieve the necessary high degree of compaction of the base.

Comparison with the Transvaal Roads Department (1994) design (the dashed red strenth line shown in Figure 2) shows that the 20 %-ile example on Section A met all the requirements of all layers for both the 150 mm layer and the redefined cases, except that of the base for which a DN of 2.8 was required, whereas only 7.0 was found.

MATERIAL TEST RESULTS

Soil engineering testing

Results

The pavement profile was confirmed to be approximately as reported (Photo 7) and was sampled and tested (Table 8, Figure 4 and Table 9).

Discussion

The laboratory test results confirmed and extended those of the January work, the following being of particular importance:

Table 9 Comparison between soaked and unsoaked CBR and DN on neat sand base

Location	(OWP)	10 m L			30 m L			55 m L			55 m R		
Sample	**No**	**3/11650**			**3/11651**			**3/11652**			**3/11653**		
Test [1,2]	Units	CBR %	DN mm/blow	Comp. %	CBR %	DN mm/blow	Comp. %	CBR %	DN mm/blow	Comp. %	CBR %	DN mm/blow	Comp. %
MAASHO 2.54 mm CBR [3]													
Soaked [4]	%	74	28	102	50	27	100	62	29	101	62	13	100
OWC	%	89	12	100	78	16	99	71	21	99	92	16	99
0.75 OWC [5]	%	62	12	98	71	14	100	64	12	100	47	14	96
0.5 OWC [5]	%	119	5.0	99	101	8.6	100	89	9.3	101	80	5.6	95

NOTES
[1] All testing by Geostrada, Pretoria
[2] Mean DN in mould to maximum depth of 95–100 mm before striking base plate
[3] Soaked and OWC 5.08 mm CBRs all < 2.54 mm CBRs
[4] Water content of top 25 mm of specimen after DN test 12–13%
[5] Dried back after compaction at OWC to approx condition shown

- Much higher apparent in-situ CBRs (of over 100) than the 34–58 indicated by the Kleyn model.
- High relative compactions in the wheel paths of about 99–100%.
- No plasticity on the P425, but SP-6 on the P075 using the Casagrande cup method.
- Good laboratory soaked CBRs of about 50–60 at 100% MAASHO and 70–90 at higher compactions.
- Very good laboratory unsoaked 100% MAASHO CBRs of over 80% at water contents of OWC and less – the apparent in-situ condition.
- AASHTO and Unified classifications of A-2-4(0) and SM, respectively, but only COLTO G7 (because of the fine grading).

A substantial depth of prime penetration of some 10–20 mm was noted, suggesting that a higher than normal rate of application may have been used.

The sub-base and lower layers were subjected to minimal testing as (except for the cement treatment of the sub-base) they were visually similar to the untreated sand base (Table 10). The main differences were that the sub-base had been rendered nonplastic even on the P075 fraction and that its SE was much higher than the other layers (as well as the base).

Table 10 Results of indicator tests on other layers on Section A [1]

Location	Layer	GM –	FM –	P075 %	PI (P425) %	PI (P075) %	SE
55 m LOWP	Sub-base	0.89	0.83	20	NP	NP	59
	Selected	0.79	0.55	24	NP	6	22
	Fill	0.86	0.71	19	NP	5	26
30 m R	Shoulder	0.81	0.67	26	NP	8	22

NOTES
[1] Testing by Geostrada, Pretoria
[2] All four layers classified as A-2-4(0) and SM

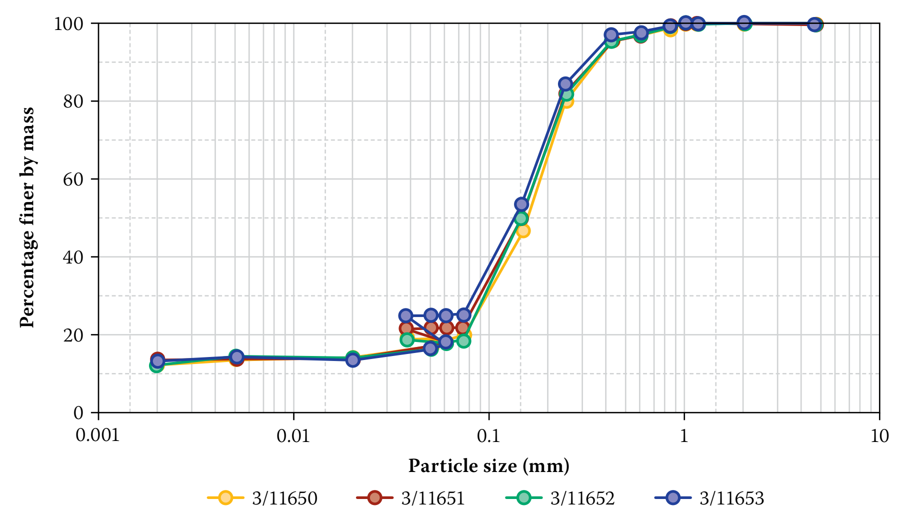

Figure 4 Gradings of October 2013 base course samples

Check testing

Results

Check testing of the most important soil properties of the four samples of base course taken in October 2013 was carried out by the CSIR in Pretoria on material that had already been used at least once for compaction-related tests and subsequently air-dried (Tables 11 and 12, and Figure 5).

Here the liquid limits were determined by the more accurate TMH 1 flow curve method in the case of the Casagrande cup method, and the BS 1377 penetration curve method in the case of the cone method.

As no cup liquid limit could be obtained on the P425 fraction, the LS was determined from the FME instead.

The soaked CBR was determined in duplicate on two samples.

As an additional measure of the compacted strength of the material, the shear strength was determined by the TMH 6 (NITRR 1984) vane method after determination of the CBR, with the top of the vane at least 50 mm below the top of the specimen.

Discussion

Both the soaked and unsoaked vane shear strengths show a reasonable correlation with their respective CBRs, although they lie on different curves (not shown), suggesting that this is a viable alternative test for fine sands. Strengths of 94–224 kPa

Table 11 Results of check tests for soil constants and CBR on neat sand base from Section A

Location (OWP)	m, L or R	10 L	10 L	30 L	55 L	55 R	55 R
Sample	No	11650 a	11650 b [2]	11651	11652	11653 a	11653 b [2]
Test [1]	Units						
Soil constants (P425)							
LL / PI / LS [3]	%	NP	–	NP	NP	NP	–
BS cone LL [3]	%	21	–	21	19	21	–
FME	%	18	–	19	20	20	–
LS from FME	%	0.1		0.1	0.1	0.1	
Soil constants (P075)							
LL [3]	%	28	–	28	26	26	–
PI	%	2	–	4	4	2	–
LS	%	2.9	–	3.3	3.0	1.3	
BS cone LL [3]	%	31	–	31	33	31	–
BS cone PI [4]	%	5	–	6	10	7	
MDD [5]	kg/m^3	1 840	1 840	1 828	1 850	1 870	1 870
OWC [5]	%	6.7	6.7	8.9	7.6	6.9	6.9
CBR (MAASHO)							
Soaked							
2.54 mm	%	47	39	42	41	54	58
5.08 mm	%	36	47	39	34	42	43
Swell	%	0.08	0.08	0.08	0.08	0.08	0.08
Compacted dry density	kg/m^3	1 809	1 805	1 827	1 860	1 834	1 851
Relative compaction	%	98	98	100	101	98	99
Water content							
Compacted	%	6.6	6.6	8.6	7.2	6.5	6.6
Whole specimen after soak	%	13.3	13.4	13.5	12.1	12.7	12.8
Top 25 mm	%	13.5	13.1	13.5	12.7	13.2	12.8
At OWC							
2.54 mm	%	58	–	53	63	46	–
5.08 mm	%	56	–	47	31	47	–
Swell	%	0.02	–	0.01	0.01	0.01	–
WC compacted	%	6.7	–	8.6	7.0	6.8	–
WC top 25 mm	%	6.6	–	8.6	7.1	6.7	–
Compacted dry density	kg/m^3	1 831	–	1 828	1 866	1 845	–
Relative compaction	%	100	–	100	101	99	–
Vane shear strength [6]							
Soaked	kPa	118	94	94	106	224	212
At OWC	kPa	106	–	71	189	177	–

NOTES
[1] Testing by CSIR
[2] Repeat test on same sample
[3] Three or more flow/penetration curve method
[4] Cone LL – TMH 1 PL
[5] Determined by Geostrada
[6] Vane 50 mm high x 30 mm diameter

Table 12 Results of check gradings and classifications of neat sand base [1]

Location (OWP, m)	10 L	30 L	55 L	55 R
Sample No	11650	11651	11652	11653
Sieve size (mm)	Cumulative percentage passing			
4.75	100	100	100	100
2.00	94	98	100	95
1.18	92	97	99	94
0.850	92	97	99	94
0.600	91	96	99	93
0.425	88	93	96	90
0.300	81	86	87	82
0.250	74	79	78	74
0.150	43	45	43	42
0.075	15	14	15	15
Derived data				
GM	1.03	0.95	0.89	1.00
Dust ratio	0.17	0.15	0.16	0.17
Cone IF075	75	84	150	105
Cup IF075	30	56	60	30
Classification				
AASHTO	A-2-4(0)	A-2-4(0)	A-2-4(0)	A-2-4(0)
UNIFIED	SM	SM	SM	SM
COLTO [2]	G7 ?	G7 ?	G7 ?	G7 ?
Compactability [3]	Good	Good	Good	Good

NOTES
[1] Testing by CSIR
[2] Probably meets CBR requirement of ≥ 25 at 95% for G6, but fails GM requirement of 1.2–1.6
[3] Good when BS cup [= BS cone] IF075 = 0–200 (Mainwaring 1968)

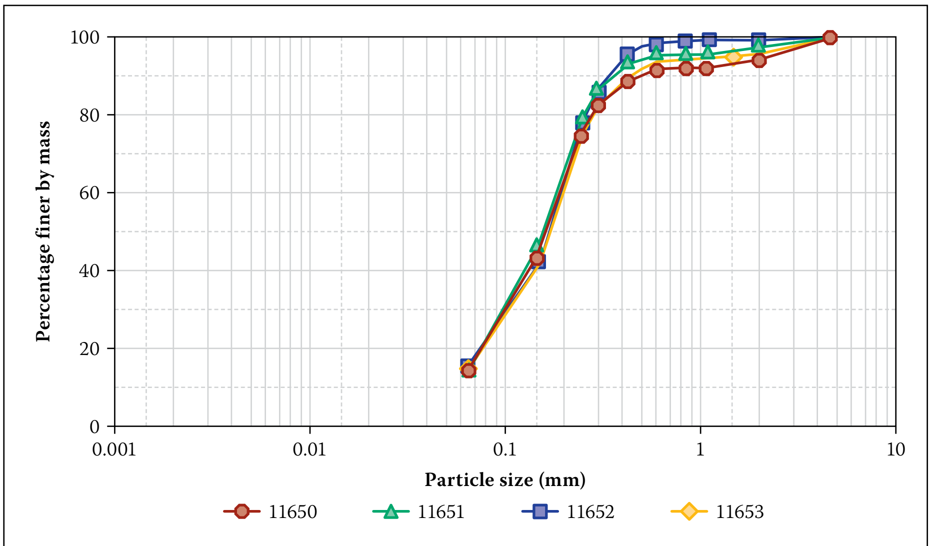

Figure 5 Results of check gradings by the CSIR

were achieved at 97–101% compaction after soaking, but only 71–189 kPa at 98–101% at OWC, apparently due to disturbance on inserting the vane.

For comparison, the recommended minimum vane shear strength of sand-asphalt at 40 °C is 200 kPa at 100% MAASHO density (NITRR 1985). However, minimum in-situ strengths of only 100 kPa at 40 °C at only 90% proved adequate for up to 0.1M E80, 120 kPa for up to 0.2M E80, and 200 kPa at 93% for up to an extrapolated

Photo 8 Photomicrograph of washed neat sand base showing angularity and partial goethite coating of grains; scale in bottom RHS is 200 µm long (Photo: S Verryn)

0.5M E80 on a seven-year-old Kalahari sand-asphalt road experiment in Botswana (Netterberg 1989).

The results of the check tests by the CSIR on the gradings and CBRs show that, although somewhat coarser and lower respectively, they agree reasonably with those by Geostrada, the samples remained NP on the P425 and SP on the P075, and their AASHTO and Unified classifications were unaltered.

It was therefore concluded that the reuse of this particular material resulted in no significant change in its properties, and that the results of the Geostrada and CSIR tests on reused material were valid.

Particle shape

It has been known for many years that the shear strength of a non-cohesive sand consists of two parts, the internal frictional resistance between grains (a combination of rolling and sliding friction) and interlocking, which is particularly important in dense sands (Taylor 1948).

Results

A photograph of the particles of a composite sample of the neat sand base as seen under a stereo microscope (Photo 8) showed that they were angular in shape.

In an attempt to quantify this the ASTM C 1253-93 particle angularity test used in the concrete and asphalt industries was carried out (Table 13).

Discussion

The results on the whole grading and the plus 075 µm fraction are not significantly different. In terms of the criteria used by the South African asphalt industry (TCAM van Rijckevorsel 2013, personal communication) 35% would be regarded as round, 45% as average and 65% as very angular – the results of 47–50% found would only be regarded as slightly more angular than average, but

Table 13 Results of particle angularity tests on neat sand base [1]

Location (OWP)	Units	10 L	30 L	55 L
Sample	No	3/11650	3/11651	3/11652
Uncompacted voids [2]				
Whole grading [3]	%	50.1	48.7	49.3
Plus 075 µm [4]	%	47.9	47.4	47.9

NOTES
[1] Testing on oven-dried material by Much Asphalt Gauteng Regional Lab, means of three tests on each sample
[2] BRD of 2.650 assumed for all tests
[3] Method C
[4] Dry-sieved

adequate for a sand for use in normal asphalt mixes.

Chemical and mineralogical composition

Physicochemical properties

In order to ascertain whether the outstanding performance of the pure sand base was due to some unusual composition it was also characterised by chemical and mineralogical analysis. The results of tests for organic carbon, pH, paste resistance and sulphate content (not shown) did not indicate any unusual properties.

The P002s of 5–6% found on the base using the more accurate pipette method were much lower than the 13–14% reported by Geostrada on the same samples (Table 9), but were comparable with those of the January samples also tested by them (Tables 3 and 4).

The results (not shown) showed that the neat sand was composed mostly of about 90% SiO_2 occurring as quartz (SiO_2), 4% total Al_2O_3 as kaolinite clay and felspars, with about 1.5% total Fe_2O_3 as goethite, FeO(OH), and that all of these minerals (and some muscovite mica), except quartz, were more concentrated in the P075 fraction. Little or no $CaCO_3$ was present.

The citrate-bicarbonate-dithionite (CBD)-extractable Al, Fe and Mn (i.e. present as free oxides/hydroxides) in one composite sample of neat sand base were 0.09%, 0.51% and 0.12% calculated as Al_2O_3, Fe_2O_3 and MnO respectively.

It seems that these small amounts of kaolinite and goethite were sufficient to account for the cohesion and to contribute towards the high strengths of the sand, particularly when unsaturated, and the good performance of the neat sand base, as well as the neat sand shoulders.

TRAFFIC HISTORY

Traffic

An estimate of the traffic history of that part of the road, including the experimental sections, is shown in Table 14.

Only eight counts were available for the total of 50 years, and none at all for the first

Table 14 Traffic history

Year	Month	Average annual daily and yearly traffic [1]								Cumulative		Age
	No/ Month	ADT	ADTT	AADT	AADTT [2]		LEF [3]	AADE	AAE	In both directions		
										Calculated	Rounded	
Units		v/d	hv/d	v/d	%	hv/d	E80/hv	E80/d	E80/y	E80	M E80	Years
1962	6	–	–	(150)	(10)	(15)	(0.5)	8	1 460	1 460	–	–
1963	12	–	–	150	(10)	(15)	(0.5)	8	2 920	4 380	–	1
1968	12	–	–	–	–	(20)	(0.6)	12	4 380	22 000	–	**5**
1973	12	–	–	–	–	(27)	(0.6)	16	5 840	48 000	0.05	**10**
1978	12	–	–	–	–	(36)	(0.6)	22	8 030	83 000	0.1	**15**
1983	12	–	–	–	–	(48)	(1.2)	58	21 170	150 000	0.15	**20**
1988	12	–	–	–	–	(64)	(1.2)	77	28 105	275 000	0.3	**25**
1993	12	–	–	–	–	(86)	(1.2)	103	37 595	405 000	0.4	**30**
1994	12	–	–	–	–	(91)	(1.2)	109	39 785	485 000	–	31
1995	Nov	–	–	1 172	9	100	(1.2)	120	43 800	528 523	0.5	33
1997	March	–	–	765	12	89	**1.2**	107	39 055	611 378	–	34
1998	12	–	–	–	–	(100)	(1.7)	170	62 060	673 428	0.6	**35**
2003	12	–	–	–	–	(100)	(1.7)	170	62 060	983 728	1.0	**40**
2005	May	–	–	862	12	103	(2.5)	258	94 170	1 139 948	–	42
2007	–	667	37	(500)	(6)	(28)	**3.8**	106	38 690	1 272 818	–	44
2008	–	780	–	(580)	(15)	(87)	(3.8)	331	120 815	1 393 633	1.4	**45**
2011	Nov	800	–	(600)	(15)	(90)	(3.8)	342	124 830	1 760 093	–	48
2013	Sept	738	145	848	18	151	(3.8)	574	209 510	2 094 433	2.0	**50**

NOTES
[1] Total of both directions; opened to traffic in June 1962; legal axle loads increased in 1996, e.g. for single dual wheel axle from 8 200 kg to 9 000 kg; based mostly on seven-day counts; figures bracketed are estimates, others are actual counts
[2] Six percent heavy vehicle growth rate assumed between 1963 and 1994; actuals subsequently and assumed same between subsequent counts
[3] Heavy vehicle load equivalency factors based on historical data for similar rural roads, partly in TRH 16 (CSRA 1991) and TRH 4 (COLTO 1996) assuming a load equivalency exponent of 4; those in bold derived from WIM surveys in those years on similar road in similar farming area; however DCP tests suggested a possible exponent of only about 1.2

20 years between 1963 and 1995. A heavy vehicle growth rate of 6% per annum from an initial 10% of the AADT (i.e. 15 hv/d) as recommended by the former Transvaal Roads Department (1994) for this class of road was therefore assumed, which also agreed well with the count of 120 hv/d in 1995.

Discussion

Although the calculations are accurate, they nevertheless must be regarded as only giving an approximate indication of the cumulative number of standard axles (E80) carried since construction.

As practically no information is available on the early buildup of traffic, or on the traffic split, it is safest to assume that the sections had carried about 0.1M E80/lane in the first 20 years, about 0.2 in 30 years, about 0.5 in 40 years, and about 1.0 M in 50 years.

During both site visits a significant number of six- and seven-axle multi-trailer vehicles travelling in both directions were noticed. A two-day count of these alone in June 2014 averaged 35/day travelling towards Bultfontein and 30 towards Hoopstad, all approximately 75% loaded (J Nkabinde 2014, personal communication). There is no overloading control on this road and no other split counts were carried out. The use of such vehicles apparently commenced in about 1996 and runs throughout the year, with the greatest number during the months of July to December, and with the degree of loading approximately equal in both directions.

Assuming an average load equivalency factor (LEF) of 6.0 per vehicle for these trucks obtained from an HSWIM count in 2007 on a similar road in a similar farming area, it was estimated that these vehicles alone contributed about 180 E80/lane/day or about 50 000 E80/lane/year, and at least one half of the total cumulative E80 carried up to 2013.

MAINTENANCE

The original surfacing was a triple seal. As far as can be ascertained the road was only resealed once with a single reseal in 1986, and received one rejuvenation spray at an unknown date.

The presence of only the triple seal and a single 13 mm reseal was confirmed in all the many trial holes, DCP points and edge breaks examined on all three sections.

All of the sections, especially the neat sand section, had also been extensively patched (Photos 2–5), although this had largely been confined to the outer 0.5–1.0 m in both lanes due to the severe and extensive edge-breaking. No evidence of base course patching was found.

Table 15 Climatic data for the area [1]

Station / Parameter	Units	Plessisdraai	Hoopstad
Station Ref No		0363/239 5	0362/710 9
Source		[2]	[3]
Lat; Long	° ′	27° 59′ S 26° 08′ E	27° 50′ S 25° 54′ E
Altitude	m	1249	1239
Rainfall			
Period	Years	1974–1990	1951–1974
Mean annual	mm/y	503	501
Highest annual	mm	836	664
Lowest annual	mm	378	286
Max in 24 h	mm	74	196
Days with ≥ 1.0 mm (mean)	no	57	54
Days with ≥ 10 mm (mean)	no	16	17
Evaporation [4]		–	
Period	Years	–	3
Mean annual Class A pan	mm	(2500)	2522
Temperature		–	
Period	Years	1974–1990	1951–1974
Mean annual daily	°C	16.9	17.5
Mean daily max	°C	25.8	26.0
Mean daily min	°C	8.0	8.9
Mean daily range	°C	17.8	17.2
Mean days with max ≥ 35°C	no	≥ 12	18
Mean days with max ≥ 30°C	no	≥ 96	105
Mean days with min < 5°C	no	129	117
Mean days with min < 0°C	no	67	56
Mean days with min < –2.5°C	no	10 (< –5°C)	25
Relative humidity			
Period	Years	1974–1990	1951–1974
Mean annual at 08 h, 14 h, 20 h	%	74, 32, 48	71, 32, 50

NOTES
[1] Site location: S 28° 00′, E 25° 59′, altitude 1 278 m
[2] Approximately 14.5 km east of site; data source: Dawn.Mahlobo@weathersa.co.za (2014, personal communication)
[3] Approximately 20 km north of site; date source: Weather Bureau (1986)
[4] (Weather Bureau 1980), figures in brackets estimated by authors

The edge-breaking had apparently been worse on the Bultfontein-bound lane of the neat sand section where a 40 m length of the outer 1.0 m had been repaired with a geotextile and a bituminous emulsion in about 2007 (Photos 2 and 5). This was holding up well and appeared to be a viable repair option.

CLIMATE AND WEATHER

Climate

According to the map of macroclimatic regions of southern Africa (Figure 4 in TRH 4 – COLTO 1996) and in the South African pavement engineering manual (SANRAL 2013), the experiment lies within the "dry" macroclimatic region for pavement design purposes. However it lies close – about 50 km – to the boundary between the dry and moderate macroclimatic regions.

The climatic indices and classifications are as follows:

- Weinert's (1980) N-value: 5.5.
- Thornthwaite: Dry semiarid warm, moisture deficient in all seasons (Schulze 1947), with a moisture index (I_m) of minus 21–22 (Schulze 1958, confirmed from Emery 1992, and Council for

Table 16 Mean rainfall near the site for the period 1997 to 2013 and actual rainfall from 2012 to April 2014 [1]

Statistic	Units	Jan	Feb	Mar [2]	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Total
2012	mm	41	49	43	14	0	14	7	3	15	21	59	66	332
2013	mm	25	25	98	54	0	0	3	2	0	35	38	75	355
2014	mm	50	144	62	4	–	–	–	–	–	–	–	–	–
Mean	mm	100.7	62.1	75.7	42.1	23.5	14.6	3.5	8.3	14.8	33.4	56.6	85.2	520.2
SD	mm	76.0	47.3	55.4	42.6	24.8	21.3	6.0	15.8	19.1	34.8	36.1	47.1	181.8
COV	%	75.4	76.2	73.2	101	101	146	171	190	129	104	63.8	55.3	35.0
Min	mm	16	1	12	4	0	0	0	0	0	0	4	0	238
Max	mm	284	175	180	170	75	75	17	58	58	123	116	178	880
Years	no	17	17	17	17	17	17	17	17	17	17	17	17	17

NOTES
[1] At nearby farm house on Vesuvius 316; 3.5 km southeast of site at altitude of 1 291 m (BD Naudé 2014, personal communication, with statistics by authors)
[2] 52 mm on or before 11 March 2014
[3] 4 mm on 1 April 2014

Scientific and Industrial Research (CSIR 2009)).

- Köppen: BSk, (semiarid steppe), with the dry, cold season in winter (Schulze & McGee 1978) and calculated from Table 16; borderline BSh (semiarid hot).

Hail and fog are rare and snow almost unknown.

Further details of the climate of the area and the rainfall received during almost the full 50 years of the experiment are shown in Tables 15 and 16.

Because the site lies at approximately the wet limit in terms of rainfall, Im and N-value of the occurrence of most Kalahari-type sands, the results of the experiment should be conservative as far as the climatic factor is concerned. They should therefore be conservatively applicable to Kalahari-type sands almost everywhere in southern Africa.

TOPOGRAPHY, GEOLOGY, SOILS AND DRAINAGE

The topography of the area is flat and at an altitude of about 1 280 m. No rock outcrops are shown on the 1:250 000 geological maps (Council for Geoscience 1993, 1994), and the surface material is shown simply as quaternary aeolian sand of unspecified depth, and the underlying geology as shale and subordinate sandstone of the Ecca Group of the Karoo Supergroup, intruded in places by Jurassic dolerite.

The soil maps of the area (Land Type Survey Staff 1986, 2012) show the soils of the area to be red-yellow in colour, apedal, freely drained, with a high base status, and with a "clay" (i.e. passing 2 µm) content of usually less than 15%. These soils are deep, sandy, aeolian soils of the Hutton soil form in the Kalahari vegetation unit and mostly of the Avalon soil form in the Grassland unit, which latter was confirmed by the three pits dug in the road reserve between km 19.7 and 19.8 (Photo 1), together with the laboratory test results on samples from both these pits (Table 4). Such soils are characterised by their yellow-brown colour, well-drained, fine sandy nature with a low clay content (< 6%), but with seasonal wetness.

Local information indicates that the permanent water table lies at a depth of about 30 m, but that a perched water table can develop at a depth of about 2 m due to an underlying layer of clay – presumably the residual shale.

SUGGESTED SAND BASE COURSE SPECIFICATION

Material

Based only upon the results of this investigation, the following is suggested as a specification for a sand base course material for sealed, low-volume roads designed to carry up to about 0.1M E80 over 20 years:

Essential:

- Colour: yellowish-brown or reddish-brown (**not** white or grey)
- AASHTO classification: A-2-4(0)
- Unified classification: SM
- GM: 0.75–1.10
- P075: 10–25%
- TMH 1 PI on P425 fraction: NP-SP
- TMH 1 PI on P075 fraction: SP-6
- TMH 1 IF075: 20–120
- Minimum soaked 2.54 mm CBR at 100% MAASHO: 50
- Minimum unsoaked 2.54 mm CBR at OWC at 100% MAASHO: 60
- Maximum MAASHO CBR swell: 0.1%
- Minimum CBD-extractable Fe: 0.3% or, less reliably, minimum Fe_2O_3 content by XRF analysis: 1.2% Fe_2O_3.

Probably desirable:

- Sand equivalent: 13–40
- Particle angularity:
  - Minimum uncompacted voids (ASTM C1252) on the plus 075 µm fraction: 45%, or
  - Mostly angular particles visible under stereo microscope
  - Dominant clay mineral: kaolinite.

Terrain:

- Relatively flat
- Drainage: good
- Permanent or perched water table: at least 1.0 m below top of roadbed

Construction:

- Cross-section: surface camber or adequate (≥ 3% ?) crossfall
- Seal: at least a double seal
- Prime: required
- Compaction:
  - Base to refusal, or at least 100% MAASHO, whichever is the lesser.
  - Shoulders, sub-base (if not cemented), selected subgrade and fill of similar sand to at least 100% MAASHO
  - Roadbed to at least 95% MAASHO, preferably with deep compaction by impact, vibrating or heavy pneumatic roller if collapsing

Seal maintenance:

- Good

Discussion

As there were no failures, the material specifications simply attempt to circumscribe the apparently desirable properties of the material tested, and it is uncertain which can be safely omitted – or which perhaps still need to be added.

The purpose of specifying colour is to ensure that there are some suitable iron oxide/hydroxide minerals present and to act as a field proxy for chemical analysis, which is difficult to get done by the more desirable CBD method, and which cannot be done on site.

The most essential requirements would appear to be to have at least about 10% P075 with just sufficient plasticity to provide some cohesion to allow good compaction, stability during construction, and an adequate CBR, both soaked and unsoaked, as well as a sufficiently stiff platform for compaction.

The use of a more sophisticated grading specification, such as that of the Botswana Roads Department (BRD 2010), involving phi grading units does not appear to be necessary.

The limitation of the design traffic to about 0.1M E80 at this stage of our knowledge is deliberately conservative in view of the absence of traffic counts over the first 20 years, the well-known beneficial effects of slow remoulding by traffic (Van Niekerk 1953; Kleyn & Savage 1982; De Beer *et al* 1989), and the uncertainty both regarding the correct load equivalency exponent, and the degree to which the original triple seal and the later reseal – currently a total of 30 mm – may have acted as a structural layer.

COMPARISON WITH SOME EXISTING SAND BASE SPECIFICATIONS

Sand bases with a capillary-soaked triaxial class of 3.3 at intermediate compaction have been used in Zimbabwe for roads designed to carry up to 0.1 M E80 in both directions over 10 years (Mitchell *et al* 1975; Mitchell 1982). However, although the Hoopstad samples tested would meet the equivalent soaked CBR requirement of about 35–55, their gradings are too fine.

A novel approach to the selection of sands in West Australia using sedimentological phi (Φ) units instead of mm for particle size characterisation in terms of the mean and standard deviation, is that of Metcalf and Wylde (1984). Plots of the Hoopstad samples (not shown) showed them to all fall outside of their Zone B (which included most successful sealed bases) due to insufficient fines.

A preliminary specification for Kalahari sands for base course in Botswana (BRD 2010) is as follows :

- $\frac{LS_{075}}{\Phi_{\bar{x}}} = 5 - 10$
- Soaked British Standard vibrating hammer (BSVH) CBR: ≥ 60%
- Total $Al_2O_3 + Fe_2O_3$ content: > 8%
- Field compaction (BSVH): ≥ 100%

Although the Hoopstad sand might have met the CBR requirement, it would have failed this specification on account of its unsatisfactory $LS/\Phi_{\bar{x}}$ ratios of less than 5 (LSs all too low) and an $Al_2O_3 + Fe_2O_3$ content of less than 8%.

CONCLUSIONS

This investigation has shown that, as predicted by Gregg (1963), provided that it does not become saturated, a fine, A-2-4(0) aeolian sand with sufficient fines of low plasticity and a good CBR can be used as untreated base course for a low-volume road with an expected life of more than 20 years.

This finding should go some way towards the provision of low-volume sealed roads in the vast area of arid and semiarid southern Africa covered with similar sands and devoid of conventional gravels.

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The site work was carried out by a team from the Free State Department of Police, Roads and Transport under the authors' supervision.

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The experimental sections were constructed as a joint project between the Department and the CSIR.

Most of the laboratory engineering testing was carried out by Geostrada Engineering International (Pty) Ltd, with some check testing at the CSIR Division of Built Environment, and the Institute for Soil, Climate and Water, all in Pretoria.

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APPENDIX
NOTATION AND METHODS

Soil engineering tests

- Bar Linear Shrinkage (LS): TMH 1: 1979 (National Institute for Transport and Road Research) (NITRR 1979).
- British Standard Vibrating Hammer (BSVH): BS 1377: 1990: Part 4 (British Standards Institution) (BSI 1990).
- California Bearing Ratio (CBR): TMH 1: 1986 – in accordance with normal South African practice, unless otherwise stated, at a specified percentage of compaction relative to the MAASHO MDD, at a penetration depth of 2.54 mm after soaking for at least four days (NITRR 1986).
- Carbonation and presence/absence of cement using 0.5% phenolphthalein

solution and diluted hydrochloric acid (HCl) (Netterberg 1984), except that 1.2 N HCl was used instead of the previously recommended 5 N, as it has subsequently been found to be more sensitive.

- Compactive effort: TMH 1: 1986: Modified American Association of State Highway Officials (MAASHO), i.e. 2 413 kJ/m^3 (which is less than the current heavy American Association of State Highway and Transportation Officials (AASHTO) T180 effort of 2 695 kJ/m^3); National Road Board (NRB, i.e. Intermediate), i.e. 1 096 kJ/m^3; and Proctor, i.e. 531 kJ/m^3 (Department of Transport) (DOT 1970).
- Cone Liquid Limit: BS 1377: 1990: Part 2.
- DCP tests on CBR specimens: Average DN through specimen after CBR test, with annular weight in place as used by EG Kleyn (1984), (2013, personal communication), and Sampson and Netterberg (1990), with the cone zero at the bottom of the CBR indentation and with heave measurements during test (which were usually zero, rarely up to 4 mm).
- Dynamic Cone Penetrometer (DCP): TMH 6: 1984 (NITRR 1984), with the cone zero at the top of the seal.
- Field Moisture Equivalent (FME): AASHTO T93-86 (1936), (AASHTO 1998a).
- Fineness Index (FI_{075}, FI075): P075 × $PI_{075,}$ (Mainwaring 1968). (Note: When PI_{075} = NP or 0, then FI075 = P075).
- Fineness Modulus (FM): [600 – (P5000 + P2000 + P1000 + P600 + P300 + P150)] / 100: South African National Standard (SANS) 3001-PR5: 2009 South African Bureau of Standards (SABS 2010).
- Grading modulus (GM): (R2000 + R425 + R075) / 100 (Kleyn 1955); **or**
- [300 – (P2000 + P425 + P075)] / 100 (SANS 3001-PR5: 2009) (SABS 2010).
- Laboratory test methods in general: TMH 1: 1986 (NITRR 1986).
- Particle angularity: ASTM C 1252 – 93 (ASTM 1995).
- Particle size distribution (“grading”):
  - Sieve analysis: TMH 1: 1986 Method A-1(a).
  - Hydrometer analysis: Geostrada Pretoria Method MT 1.
- P425, P075, etc: Cumulative percentage passing 425, 075 µm, sieves, etc.
- R425, R075, etc: Cumulative percentage retained on 425, 075 µm, sieves, etc.
- Sand equivalent (SE) (on whole grading): SANS 3001-AG5: 2013 (SABS 2013).
- Soil classification:
  - AASHTO M 145-91 (1995) (AASHTO 1998b).
  - COLTO: 1998: Section 3400 (Committee of Land Transportation Officials 1998) (COLTO 1998).
  - Unified: ASTM D2487-11 (ASTM International 2013).
- Soil preparation:
  - Passing 0.425 mm fraction (P425) for soil constants: TMH 1: 1986 Method A–1(a).
  - Passing 0.075 mm fraction (P075) for soil constants: SANS 3001 – GR1: 2008 (SABS 2008).
- Unsoaked CBR: At optimum water content (OWC) for MAASHO effort after four days equilibration in sealed plastic bags; at less than OWC after drying in the sun or oven at ≤ 60 °C to the approximate percentage OWC specified and then sealing in plastic bags for at least four days.
- Visual assessment of the road: TMH 9: 1992 (Committee of State Road Authorities) (CSRA 1992).

Soil science tests

- pH, paste resistance, dithionite-citrate-bicarbonate (CBD) – extractable free iron (Fe), aluminium (Al) and manganese (Mn) oxides/hydroxides; organic carbon (by Walkley-Black dichromate oxidation), and particle size distribution (washed sieve plus pipette method): The Non-Affiliated Soil Analysis Work Committee (1990).
- Soil colour: Standard Munsell colours (Soil Colour Chart compiled by Soils Research Institute, Pretoria, undated; Munsell Color Co, Inc, Baltimore.